ARR
Australian Rainfall & Runoff

A GUIDE TO FLOOD ESTIMATION

Version 4.2

Australian Government

ENGINEERS AUSTRALIA

Acknowledgement of Country
We acknowledge the Traditional Owners of Country throughout Australia and recognise their continuing connection to land, waters and culture. We pay our respects to their Elders past and present.

If you have any questions about the copyright of the ARR, please contact:
hazards@ga.gov.au or admin@arr-software.org
c/o 11 National Circuit,
Barton, ACT

ISBN 978-1-925848-36-6

How to reference:
Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M, Testoni I, (Editors) Australian Rainfall and Runoff: A Guide to Flood Estimation, © Commonwealth of Australia (Geoscience Australia), Version 4.2, 2019.

How to reference Book 9: Runoff in Urban Areas:
Coombes, P., and Roso, S. (Editors), 2019 Runoff in Urban Areas, Book 9 in Australian Rainfall and Runoff - A Guide to Flood Estimation, Commonwealth of Australia, © Commonwealth of Australia (Geoscience Australia), Version 4.2, 2019.

PREFACE

Since its first publication in 1958, Australian Rainfall and Runoff (ARR) has remained one of the most influential and widely used guidelines published by Engineers Australia (EA). The 3rd edition, published in 1987, retained the same level of national and international acclaim as its predecessors.

With nationwide applicability, balancing the varied climates of Australia, the information and the approaches presented in Australian Rainfall and Runoff are essential for policy decisions and projects involving:

- infrastructure such as roads, rail, airports, bridges, dams, stormwater and sewer systems;
- town planning;
- mining;
- developing flood management plans for urban and rural communities;
- flood warnings and flood emergency management;
- operation of regulated river systems; and
- prediction of extreme flood levels.

However, many of the practices recommended in the 1987 edition of ARR have become outdated, and no longer represent industry best practice. This fact, coupled with the greater understanding of climate and flood hydrology derived from the larger data sets now available to us, has provided the primary impetus for revising these guidelines. It is hoped that this revision will lead to improved design practice, which will allow better management, policy and planning decisions to be made.

One of the major responsibilities of the National Committee on Water Engineering of Engineers Australia is the periodic revision of ARR. While the NCWE had long identified the need to update ARR it had become apparent by 2002 that even with a piecemeal approach the task could not be carried out without significant financial support. In 2008 the revision of ARR was identified as a priority in the National Adaptation Framework for Climate Change which was endorsed by the Council of Australian Governments.

In addition to the update, 21 projects were identified with the aim of filling knowledge gaps. Funding for Stages 1 and 2 of the ARR revision projects were provided by the now Department of the Environment. Stage 3 was funded by Geoscience Australia. Funding for Stages 2 and 3 of Project 1 (Development of Intensity-Frequency-Duration information across Australia) has been provided by the Bureau of Meteorology. The outcomes of the projects assisted the ARR Editorial Team with the compiling and writing of chapters in the revised ARR. Steering and Technical Committees were established to assist the ARR Editorial Team in guiding the projects to achieve desired outcomes.

Assoc Prof James Ball
ARR Editor

Mark Babister
Chair Technical Committee for
ARR Revision Projects

ARR Technical Committee:

Chair: Mark Babister
Members:
Associate Professor James Ball
Professor George Kuczera
Professor Martin Lambert
Associate Professor Rory Nathan
Dr Bill Weeks
Associate Professor Ashish Sharma
Dr Bryson Bates
Steve Finlay

Related Appointments:

ARR Project Engineer:	Monique Retallick
ARR Admin Support:	Isabelle Testoni
Assisting TC on Technical Matters:	Erwin Weinmann, Dr Michael Leonard

ARR Editorial Team:

Editors: James Ball
Mark Babister
Rory Nathan
Bill Weeks
Erwin Weinmann
Monique Retallick
Isabelle Testoni

Associate Editors for Book 9 - Runoff in Urban Areas

Peter Coombes
Steve Roso

Editorial assistance: Mikayla Ward

Status of this document

This document is a living document and will be regularly updated in the future.

In development of this guidance, and discussed in Book 1 of ARR 1987, it was recognised that knowledge and information availability is not fixed and that future research and applications will develop new techniques and information. This is particularly relevant in applications where techniques have been extrapolated from the region of their development to other regions and where efforts should be made to reduce large uncertainties in current estimates of design flood characteristics.

Therefore, where circumstances warrant, designers have a duty to use other procedures and design information more appropriate for their design flood problem. The Editorial team of this edition of Australian Rainfall and Runoff believe that the use of new or improved procedures should be encouraged, especially where these are more appropriate than the methods described in this publication.

Care should be taken when combining inputs derived using ARR 1987 and methods described in this document.

Change Log

Version 4.2 - Climate Change Chapter Update

In late 2022 the Australian Government Department of Climate Change, Energy, the Environment and Water in partnership with Engineers Australia commenced an 18 month project to update the climate change considerations chapter of the Australian Rainfall and Runoff guidelines (Chapter 6, Book 1) to incorporate the most recent and relevant climate science and projections. The project involved the undertaking of a rigorous literature review of hydroclimatology under climate change relevant to design flood estimation, which was peer reviewed and published in a leading international journal. The findings were used to draft practical flood guidance which was finalised after an extensive process of review and feedback by industry. Funding for this project was received from National Emergency Management Agency under the Disaster Risk Reduction Package. The project report was adapted to replace Book 1 chapter 6.

Climate Change Update Project Control Group:

Leanne Haupt
Simon Koger
Andrew Dyer
Karl Braganza
Duncan McLuckie
Monique Retallick
Euan Brown
Andrew Gissing
Martyn Hazelwood
Professor Rory Nathan

Climate Change Update Technical Working Group:

Dr Conrad Wasko
Professor Seth Westra
Dr Dörte Jakob
Chris Nielsen
Professor Jason Evans
Simon Rodgers
Mark Babister
Dr Andrew Dowdy
Dr Wendy Sharples
Dr Ramona Dalla Pozza
Dr Michelle Ho

This version updates Book 1 Chapter 6 to reflect updates in climate science as discussed above. While no other chapters have been updated some minor amendments were made to remove inconsistencies with the new chapter. FAQs relating to the update are available https://arr.ga.gov.au/contact-us.

Key updates in Version 4.2

Update	Version 4.2
Book 1	Book 1 Chapter 6 Climate change updated
Guideline formats	PDF Web-based version Epub version
User experience	FAQs added to Geoscience Australia Website
Climate change	Reflected best practice as of 2024 and IPCC 6
Other Minor Changes	List the minor changes to the following chapters for consistency Book 1 Chapter 4 Section 15.1 Book 1 Chapter 4 Section 16.1 Book 1 Chapter 5 Section 10.4 Book 2 Chapter 1 Section 3 Book 2 Chapter 3 Section 3 Book 6 Chapter 5 Section 5 Book 8 Chapter 7 Section 7 Book 9 Chapter 6 Section 4.2 Book 9 Chapter 6 Section 4.6

ARR 2019 (now Version 4.1)

Geoscience Australia, on behalf of the Australian Government, asked the National Committee on Water Engineers (NCWE) - a specialist committee of Engineers Australia - to continue overseeing the technical direction of ARR. ARR's success comes from practitioners and researchers driving its development; and the NCWE is the appropriate organisation to oversee this work. The NCWE has formed a sub-committee to lead the ongoing management and development of ARR for the benefit of the Australian community and the profession. The current membership of the ARR management subcommittee includes Mark Babister, Robin Connolly, Rory Nathan and Bill Weeks.

The ARR team have been working hard on finalising ARR since it was released in 2016. The team has received a lot of feedback from industry and practitioners, ranging from substantial feedback to minor typographical errors. Much of this feedback has now been addressed. Where a decision has been made not to address the feedback, advice has been provided as to why this was the case.

A new version of ARR is now available. ARR 2019 is a result of extensive consultation and feedback from practitioners. Noteworthy updates include the completion of Book 9, reflection of current climate change practice and improvements to user experience, including the availability of the document as a PDF.

Key updates in ARR 2019

Update	ARR 2016	ARR 2019
Book 9	Available as “rough” draft	Peer reviewed and completed
Guideline formats	Epub version Web-based version	Following practitioner feedback, a pdf version of ARR 2019 is now available
User experience	Limited functionality in web-based version	Additional pdf format available
Climate change	Reflected best practice as of 2016 Climate Change policies	Updated to reflect current practice
PMF chapter	Updated from the guidance provided in 1998 to include current best practice	Minor edits and reflects differences required for use in dam studies and floodplain management
Examples		Examples included for Book 9
Figures		Updated reflecting practitioner feedback

As of May 2019, this version was considered to be final.

ARR 2016 (now Version 4.0)
Released July 2016

Table of Contents

Scope and Philosophy
Rainfall Estimation
Peak Flow Estimation
Catchment Simulation for Design Flood Estimation
Flood Hydrograph Estimation
Flood Hydraulics
Application of Catchment Modelling Systems
Estimation of Very Rare to Extreme Floods
Runoff in Urban Areas

BOOK 1

Scope and Philosophy

Scope and Philosophy

Table of Contents

1. Introduction 1
    1.1. General 1
    1.2. Contents 3
    1.3. References 5
2. Fundamental Issues 6
    2.1. Introduction 6
    2.2. Terminology 6
        2.2.1. Background 6
        2.2.2. Clarity of Meaning 6
        2.2.3. Technical Correctness 7
        2.2.4. Practicality and Acceptability 8
        2.2.5. Adopted Terminology 8
    2.3. Difference Between Design Events and Actual Events 10
    2.4. Probability Concepts 11
        2.4.1. Probability Relationship Between Design Rainfall and Design Flood Characteristics 11
        2.4.2. Choosing a Quantile Estimator 12
        2.4.3. Avoiding Inconsistencies in Procedures and Resolution 12
    2.5. Risk-Based Design 12
        2.5.1. Route Serviceability 12
    2.6. The Importance of Data 13
    2.7. Climate Change 13
        2.7.1. Climate Change Impacts on Flooding 14
    2.8. Dealing with Uncertainty 16
        2.8.1. Introduction 16
        2.8.2. Types of Uncertainty in Design Flood Estimation 16
        2.8.3. Motivation for Incorporating Uncertainty Into Design Flood Estimates 18
        2.8.4. Sources of Uncertainty in Context of Design Flood Estimation 21
        2.8.5. Raising Awareness of the Sources of Uncertainty in Techniques Used for Design Flood Estimation 23
        2.8.6. Summary 28
    2.9. References 28
3. Approaches to Flood Estimation 31
    3.1. Introduction 31
    3.2. Flood Data Based Procedures 33
        3.2.1. Overview 33
        3.2.2. Flood Frequency Techniques 34
        3.2.3. Regional Flood Methods 35
    3.3. Rainfall-Based Procedures 36
        3.3.1. General 36
        3.3.2. Event-Based Simulation 37
        3.3.3. Continuous Simulation 39
    3.4. Selection of Approach 39
        3.4.1. Overview 39
        3.4.2. Advantages and Limitations of Flood Data Based Procedures 40
        3.4.3. Advantages and Limitations of Rainfall-Based Procedures 42
        3.4.4. Relative Applicability of Different Approaches 44
    3.5. References 47
4. Data 50
    4.1. Introduction 50

4.2. Background ... 50
4.3. Risks From Inadequate Data ... 51
4.4. Stationarity ... 52
4.5. Hydroinformatics ... 52
    4.5.1. Introduction to Hydroinformatics ... 52
    4.5.2. Components of a Hydroinformatics System ... 53
4.6. Data Categories and Issues ... 53
    4.6.1. General ... 53
    4.6.2. Data Source Organisations ... 54
    4.6.3. Data on Historical Events ... 54
4.7. Discussion on Hydrologic Data Issues ... 55
    4.7.1. Data Types ... 55
    4.7.2. The Data Cycle ... 57
4.8. Hydrologic Data ... 60
4.9. Rainfall Data ... 61
    4.9.1. Overview ... 61
    4.9.2. Rainfall Observations ... 61
    4.9.3. Review of Rainfall Data ... 64
    4.9.4. Rainfall Databases ... 64
    4.9.5. Application of Rainfall Data for Flood Estimation ... 66
4.10. Other Precipitation Types ... 66
4.11. Water Levels ... 66
    4.11.1. Overview ... 66
    4.11.2. Historical Flood Level Data ... 67
    4.11.3. Application of Water Level Data in analysis ... 69
4.12. Streamflow Data ... 69
    4.12.1. Introduction to Streamflow Records ... 69
    4.12.2. General Stream Gauging Procedures ... 71
4.13. Catchment Data ... 77
    4.13.1. General ... 77
    4.13.2. Types of Catchment Data ... 78
    4.13.3. Topographic and Infrastructure Data ... 78
4.14. Other Data Considerations ... 94
    4.14.1. Storage of Data and Meta-data ... 94
    4.14.2. Co-ordinate Systems and Datums ... 94
4.15. Other Hydrological Data ... 95
    4.15.1. Tidal Data ... 95
    4.15.2. Meteorological Data ... 95
    4.15.3. Sediment Movement and Deposition ... 95
    4.15.4. Water Quality ... 96
4.16. Climate Change Data ... 96
    4.16.1. Types of Climate Change Data ... 96
4.17. References ... 102
5. Risk Based Design ... 106
5.1. Introduction ... 106
5.2. Flood Risk ... 107
5.3. Risk Analysis ... 109
5.4. Managing Flood Risk ... 110
    5.4.1. Managing Changes to Flood Behaviour ... 110
    5.4.2. Managing Flood Risk by Limiting Likelihood and Consequences ... 111
5.5. Managing Flood Risks to Communities ... 115
    5.5.1. Using Flood Estimation to Inform Flood Risk Management ... 116

5.5.2. Using Design Flood Estimation to Support Management of Future Development ... 119
5.5.3. Understanding and Treating Risk to the Existing Community ... 124
5.6. Managing Flood Risks to Mining, Agricultural and Infrastructure Projects ... 131
5.6.1. Mines ... 131
5.6.2. Agriculture ... 132
5.6.3. Road and Rail Projects ... 133
5.6.4. Short Term Projects ... 133
5.7. Managing Flood Risks in Relation to Dams ... 134
5.8. Managing Flood Risks using Basins ... 134
5.9. Effective Service Life of Infrastructure ... 137
5.9.1. Estimating Effective Service Life ... 138
5.10. Estimating Change in Risk over Time ... 139
5.10.1. Changes to Likelihood ... 139
5.10.2. Changes to Consequence ... 140
5.10.3. Changes to Risk Preference ... 140
5.10.4. Literature ... 141
5.10.5. Non-Stationary Risk Assessment ... 143
5.10.6. Economic Assessment ... 146
5.11. Further Reading ... 148
5.12. Examples ... 149
5.12.1. Calculation of Average Annual Damages for a Community ... 149
5.12.2. Calculating Average Annual Benefits of a Treatment Measure ... 151
5.12.3. Calculating Net Present Value of Benefits ... 152
5.12.4. Calculating Net Present Value (NPV) of Lifecycle Costs ... 154
5.12.5. Calculating Benefit Cost Ratio (BCR) ... 155
5.13. References ... 156
6. Climate Change Considerations ... 159
6.1. Introduction ... 159
6.2. Decison Contexts for Flood Guidance ... 160
6.3. Selection of a Design Flood Estimation Method Under Climate Change ... 162
6.4. Incorporating Climate Change into Event-Based Design Flood Estimates ... 163
6.4.1. Intensity-Frequency-Duration Curves ... 164
6.4.2. Temporal Patterns ... 167
6.4.3. Spatial Patterns ... 167
6.4.4. Loss Parameters ... 167
6.4.5. Sea Levels and Sea Level Interaction ... 169
6.4.6. Increased Uncertainty Due to Climate Change ... 169
6.5. Future Updates to the Climate Change Considerations ... 170
6.6. Worked Examples ... 170
6.7. Appendix ... 173
6.8. References ... 174

List of Figures

1.2.1. Australian Rainfall and Runoff Preferred Terminology 9

1.2.2. Different Types of Uncertainty, Aleatory and Epistemic, in the Context of Design Flood Estimation 18

1.2.3. Impact of Uncertainty on a Design Flood Estimate for Two Design Cases 19

1.3.1. Illustration of Stochastic Influence of Hydrologic Factors on Flood Peaks and the Uncertainty in Flood Risk Estimates Associated with Observed Flood Data 32

1.3.2. Illustration of Relative Efficacy of Different Approaches for the Estimation of Design Floods 45

1.4.1. The Data Cycle 58

1.4.2. Standard Rain Gauge (Source: Bureau of Meteorology) 62

1.4.3. Tipping Bucket Rain Gauge (Source: Bureau of Meteorology) 63

1.4.4. Typical Rating Curve 73

1.4.5. Loop in Rating Curve 74

1.4.6. Stage-Discharge Relationship Zones 75

1.4.7. Annual Maximum Series 76

1.4.8. Concept of RTK GPS Technique of Field Survey 82

1.4.9. Aerial Photograph Example (Region A) 84

1.4.10. Sample of Processed Photogrammetry data set (Region A) 85

1.4.11. Sample of Processed Photogrammetry data set (Region A detail) 86

1.4.12. Sample of Raw ALS data set (Region A) 87

1.4.13. Sample of Processed ALS data set (Region A) 89

1.4.14. Global Greenhouse Gas Emissions Scenarios for the 21st Century (from IPCC, 2007) 99

1.4.15. From (IPCC, 2001) 100

1.5.1. Components of Flood Risk (After McLuckie (2012) 108

1.5.2. Map Showing Different AEP Flood Extents Including an Extreme Event 109

1.5.3. Map of Flood Extents 121

1.5.4. Map of Flood Extents and Flood Function 122

1.5.5. Map of Flood Extents and Flood Hazard 122

1.5.6. Map of Flood Extents and Flood Emergency Response Classification 123

1.5.7. Map of Variation in Constraints Across the Floodplain 123

1.5.8. Example of Estimated Average Risk to a Community Due to Flooding 125

1.5.9. Indicative Stage Damage Curve for some Residential House Types 126

1.5.10. Example of Flood Damage Curve for a Range of AEP Flood Events 127

1.5.11. Example of Flood Damage With and Without Treatment Options for a Range of AEP Flood Events 128

1.5.12. Example of Annual Lifecycle Benefits and Costs 129

1.5.13. Example of Lifecycle Benefits and Costs Adjusted to Todays $ Using a 7% Discount Rate 130

1.5.14. Example of Estimating Changing Average Risk to a Community Due to Flooding with Instigation of a Treatment Option 131

1.5.15. Example of Impacts of a Basin on Flood Flows in a Design Event 135

1.5.16. Example of Difference in Impacts of a Basin on Flood Flows in a Design Event Compared to an Extreme Event 137

1.5.17. Design Service Life versus Effective Service Life (derived from United States Environment Protection Agency – 2007) 138

1.5.18. Flood Risk Plot Versus Constant Risk Plot (derived from Rootzen et al 2012) 141

1.5.19. Schematic of a Design Flood with Exceeding (P_t) and Non-Exceeding ($q_t = 1\text{-}P_t$) Probabilities Varying with Time (Salas & Obeysekera, 2014) 142

1.5.20. Change in Realised Risk Through Adopting a T(x) Design Approach. 145

1.6.1. Climate change is shifting our best estimate of the relationship between event magnitude and frequency and increasing the inherent uncertainty in such estimates 161

1.6.2. Projected temperature increases associated with AR6 socioeconomic pathways relative to 1961-1990 and their associated uncertainty 164

List of Tables

1.2.1. Sources of Uncertainty in Design Flood Estimation 27

1.3.1. Summary of Common Procedures used to Directly Analyse Flood Data 34

1.3.2. Summary of Recommended Rainfall-Based Procedures 37

1.3.3. Summary of Advantages and Limitations of Common Procedures used to Directly Analyse Flood Data 41

1.3.4. Summary of Advantages and Limitations of Common Rainfall-Based Procedures 43

1.4.1. Typical Accuracies of Field Survey 81

1.4.2. Typical Swath Values 87

1.5.1. Example Qualitative Risk Matrix 110

1.5.2. Infrastructure types and potential Effective Service Life 139

1.6.1. Recommended rates of change (α) and associated uncertainty derived in Wasko et al. (2024), presented per degree global temperature change (%/°C). The factors in this table are applicable for exceedance probabilities from 1EY up to and including the PMP and are designed for application across mainland Australia and Tasmania. 165

1.6.2. Global mean surface temperature projections (∆T) for four socio-economic pathways relative to 1961-1990. The 90% uncertainty interval is provided in parentheses .. 166

1.6.3. Rates of change for initial loss (IL) and continuous loss (CL) parameters per degree global temperature change (%/°C) for Natural Resource Management Regions clusters (CSIRO and Bureau of Meteorology, 2015), adapted from Ho et al. (2023). The 'likely' range (corresponding to ~66% range) is presented in parenthesis. These rates of change should be applied relative to a 1961-1990 baseline global temperature date unless a reasonable alternative is justified 168

1.6.4. Design inputs factor for climate change for Example 2. Design inputs assume an event critical duration of 24 hours duration and historical design rainfall of 125 mm, with initial loss of 15.0 mm and continuing loss of 2.5 mm/hr 173

1.6.5. Interpolated rate of change for rainfall depth with associated uncertainty range. Values have been interpolated from the values provided in Table 1 which was derived in Wasko et al. (2024). Rates of change are presented per degree global temperature change (%/°C). The factors in this table are applicable for exceedance probabilities from 1EY up to and including the PMP and are designed for application across mainland Australia and Tasmania. If applied to the 2016 IFD curves these rates of change should be applied relative to a 1961-1990 baseline global temperature date unless a reasonable alternative is justified. Less information is available for rates of change for storm bursts between 1 and 24 hours, and hence estimates for these durations are obtained by a simple non-linear interpolation that represents the pragmatic interpretation of results obtained from Visser et al. (2021) 173

Chapter 1. Introduction

James Ball, Mark Babister, Monique Retallick, Erwin Weinmann

Chapter Status	Final
Date last updated	14/5/2019

1.1. General

While previous editions of Australian Rainfall and Runoff have served the engineering profession and the general community well, in the period since the release of the previous edition, a number of developments have arisen that necessitate the production of a new edition. These developments include the many recent advances in knowledge regarding flood processes, the increased computational capacity available to engineering hydrologists, expanding knowledge and application of hydroinformatics, improved information about climate change and the use of stochastic inputs and Monte Carlo methods.

The intention during the development of this new edition has been to provide appropriate guidance addressing these issues. In many situations, the guidance provided in this edition of Australian Rainfall and Runoff requires an enhanced knowledge of flood generation and the design process. The guidance developed has maintained the aim of Australian Rainfall and Runoff which is to provide the best available information on design flood estimation in a manner suitable for use by Australian practitioners with varying levels of knowledge about the design flood problem, flood processes, and engineering hydrology.

Development of guidance for inclusion in Australian Rainfall and Runoff consistent with the aims previously stated poses the question of a definition for the design flood problem. Design flood estimation remains a problem for many engineering projects. Advice is required regarding design flood characteristics for the:

- design of culverts and bridges for cross drainage of transport routes;
- floodplain management and planning;
- design of urban drainage systems;
- design of flood mitigation levees and other flood mitigation structures;
- setting of flood planning levels; and
- design of dam spillways.

The flood characteristic of most importance depends on the nature of the problem under consideration, but typically it is one of the following:

- *Flow rate* - commonly the peak but other flood flows may be needed for particular projects;
- *Level* - commonly the peak but other flood levels may be needed for particular projects;
- *Volume* - the volume of flood hydrographs is required for the design of many hydraulic structures designed to retain part of the flood hydrograph for flood mitigation purposes;
- *Rate of rise* - needed for the planning associated with operation flood management such as preparation of evacuation routes; or

- *System failure* - this may be failure of a dendritic network within a catchment, the failure of a transport route crossing multiple catchments, or the failure of some other system due to the occurrence of one or more flood events.

While all of these flood characteristics have been noted as being of interest to flood practitioners, the dominant characteristic of concern, historically, has been the peak flood flow. The peak flood flow was also the main focus of the previous edition of Australian Rainfall and Runoff (Pilgrim, 1987).

In this edition of Australian Rainfall and Runoff, many of the recommended practices focus on the prediction of peak design flows and prediction of full hydrographs. Since publication of the last edition of Australian Rainfall and Runoff, it has been recognised that this focus on flows provided insufficient guidance on other flood characteristics. For the holistic planning, design and operation of flood management systems, flood characteristics other than peak flow will also be relevant. For example, the design flood storage for the many retarding basins located in urban areas is usually a flood volume issue rather than a peak flow issue. As a result, other recommendations in this edition of Australian Rainfall and Runoff focus on all flood characteristics that may be of interest in design flood estimation.

This approach is consistent with the aims of Engineers Australia's National Committee on Water Engineering when they resolved that a revision of Australian Rainfall and Runoff was needed by the profession and the wider community. These aims can be stated broadly as being:

- to collect, review and evaluate available design procedures, and to update the document to include the best available methods and design data Australia wide;

- to provide guidance to designers on procedures and design values to be used in design flood estimation;

- to provide guidance on the concepts involved in the recommended procedures and their application;

- to provide separate design information for individual regions where necessary;

- to provide guidance on design flood estimation under changing climatic conditions;

- to provide guidance on the likely accuracies, or uncertainty, in the application of the recommended techniques; and

- to carry out those research activities necessary to meet the above objectives.

In development of this guidance, it was recognised that knowledge and information availability is not fixed and that future research and applications will develop new techniques and information. This is particularly relevant in applications where techniques have been extrapolated from the region of their development to other regions and where efforts should be made to reduce large uncertainties in current estimates of design flood characteristics.

Therefore, where circumstances warrant, designers have a duty to use other procedures and design information more appropriate for their design flood problem. The authorship team of this edition of Australian Rainfall and Runoff believe that the use of new or improved procedures should be encouraged, especially where these are more appropriate than the methods described in this publication. Assessment of the relative merits of new procedures and design information should be based on the following desirable attributes:

- based on observed data relevant to the specific application;
- consistent with current knowledge of flood processes;
- able to reproduce observed flood behaviour in the area of interest; and
- where possible, endorsed by a peer review process

While most of the procedures presented in the guidelines require software for their implementation, the role of Australian Rainfall and Runoff is not to endorse particular software packages but rather to provide details of the procedures to be incorporated in flood estimation software packages. However, enabling software is provided to allow site-specific design data to be extracted from databases (e.g. for the design rainfall database and the regional flood frequency estimation). These databases will be updated when warranted by the availability of significant amounts of new or revised information.

1.2. Contents

While the presentation and formats of Australian Rainfall and Runoff have varied between the editions, the focal aim has remained one of providing information relevant to design flood estimation in a form readily accessible to practitioners.

This edition of Australian Rainfall and Runoff has followed the same philosophy and has grouped information on different aspects of design flood estimation into separate books. The aim of this is to allow easy updating of components in the future. A total of 9 Books has been prepared for this edition of Australian Rainfall and Runoff with the following contents:

<u>Book 1</u> - SCOPE AND PHILOSOPHY

This book provides a general introduction to Australian Rainfall and Runoff with an emphasis on the need for the revision and the basic philosophy for the application of the guidelines. It gives a brief introduction to terminology used within the document, discusses fundamental issues and basic approaches to flood estimation, data related aspects inclusive of its management and data uncertainty, risk based design and dealing with climate change.

<u>Book 2</u> - RAINFALL ESTIMATION

This book discusses the importance of design rainfall for flood estimation, and includes discussion of differences between historical and design rainfalls, issues associated with development of rainfall models for design flood estimation in Australian Rainfall and Runoff. It provides the basis for the recommended Intensity Frequency Duration relationships, design spatial patterns of rainfall and design temporal patterns of rainfall. Also considered in this book are continuous sequences rainfall inclusive of the stochastic generation of alternative design storm sequences.

<u>Book 3</u> - PEAK FLOW ESTIMATION

This book provides a general introduction to peak flow estimation based on flood frequency analysis, as well as covering specific technical aspects of this topic area. The first of the technical chapters provides guidelines for Flood Frequency Analysis at a specific site, illustrated by a range of examples. The second deals with Regional Flood Frequency Estimation techniques and describes the application of a tool developed to readily provide peak flow frequency estimates for any location in Australia.

Book 4 - CATCHMENT SIMULATION FOR DESIGN FLOOD ESTIMATION

This book deals with general concepts and issues in catchment modelling for design flood estimation. The first chapter discusses the need for catchment simulation and introduces general catchment simulation concepts. The next chapter discusses key hydrologic processes contributing to floods and how they are represented in modelling systems. This chapter is followed by a discussion of the types of catchment modelling systems (event and continuous) and the need for integrating hydrologic, and hydraulic components of the system. The final chapters deal with the treatment of joint probability issues and uncertainty in the outputs of simulation models.

Book 5 - FLOOD HYDROGRAPH ESTIMATION

The focus of this book is the hydrologic models necessary for prediction of design flood hydrographs. The first chapter gives a general introduction to concepts presented in this book while the remaining chapters deal with the modelling of particular components of the flood formation process. The first of the technical chapters deals with the different types of hydrologic models used to represent the runoff generation and runoff routing phases of the flood formation process. The final two chapters deal with baseflow and losses for design flood estimation and provide design data for these important inputs to flood hydrograph estimation.

Book 6 - FLOOD HYDRAULICS

This book is concerned with the basic aspects of hydraulics. It is worth noting that the material presented in this chapter is not a replacement for the many textbooks in this area or that it will cover all the information necessary for the application of hydraulic principles in design flood estimation. The chapters in this book present information relevant to the hydraulic modelling of river reaches, floodplains and structures for design flood estimation, the application of software for numerical modelling of flood hydrographs, blockage of hydraulic structures and interaction of coastal and catchment flooding. A tool has been developed to assist practitioners in assessing the interation of coastal and catchment flooding. Also included in this book is guidance on designing for the safety of people and vehicles. The people safety information presented includes a discussion of the importance of the demographics in assessing safety.

Book 7 - APPLICATION OF CATCHMENT MODELLING SYSTEMS

This book provides discussion of major issues in the practical application of catchment modelling systems to different flood estimation problems, including establishment of catchment modelling systems, calibration and validation of model parameters and dealing with uncertainty in model outputs.

Book 8 - VERY RARE TO EXTREME FLOOD ESTIMATION

This book provides information and guidelines for the special design applications where floods of low Annual Exceedance Probabilities need to be estimated. Examples of these design applications include the sizing of spillways for large dams, design of major structures located in the floodplain and flood risk management in situations where very large flood damages or significant risk to life from flooding could be expected. Floods in the range of very rare to extreme events are generally estimated by the methods described in Book 8, Chapter 2 to Book 8, Chapter 7 but a number of special considerations and additional design data are required, as described in Book 8. This book includes an overview of the procedures available for estimating very rare to extreme floods, estimation of design rainfall and rainfall excess for rarer events, and special requirements for the models used to generate flood hydrographs for very rare to extreme flood events. The application of these special procedures is illustrated by a number of examples.

Book 9 - RUNOFF IN URBAN AREAS

This book first provides a general introduction to urban drainage systems and the philosophy adopted in Australian Rainfall and Runoff. It then discusses urban drainage approaches, changes to the natural hydrologic cycle resulting from urbanisation and how these changes impact on design flood estimation in urban environments, and use of storage facilities from on-site storage to detention (retention) basins to large flood mitigation dams. An important aspect of this discussion relates to limitations of the Rational method and the changes in approach necessary for consideration of volume-based problems rather than peak flow based problems.

1.3. References

Pilgrim, DH (ed) (1987) Australian Rainfall and Runoff - A Guide to Flood Estimation, Institution of Engineers, Australia, Barton, ACT, 1987.

Chapter 2. Fundamental Issues

James Ball, Mark Babister, Monique Retallick, Fiona Ling, Mark Thyer

Chapter Status	Final
Date last updated	14/5/2019

2.1. Introduction

This chapter introduces important concepts of probability and statistics with respect to flood estimation, and defines the recommended terminology for these probability concepts. The chapter also discusses the difference between design and actual events, conversion of rainfall of a given probability to a flood of the same probability, risk-based design and dealing with uncertainty in flood estimates. Much of the text from the 1987 edition of Australian Rainfall and Runoff is still relevant and has formed the basis for the information provided in some of following sections.

2.2. Terminology

2.2.1. Background

Probability concepts are fundamental to design flood estimation and appropriate terminology is important for effective communication of design flood estimates. Terms commonly used in the past have included "*recurrence interval*", "*return period*", and various terms involving "*probability*". It is common for these terms to be used in a loose manner, and sometimes quite incorrectly. This has resulted in misinterpretation by the profession, the general community impacted by floods, and other stakeholders.

In considering the terminology that should be used in this edition of Australian Rainfall and Runoff, the National Committee on Water Engineering's three major concerns were:

- Clarity of meaning;
- Technical correctness; and
- Practicality and acceptability.

2.2.2. Clarity of Meaning

Use of the terms "*recurrence interval*" and "*return period*" has been criticised as leading to confusion in the minds of some decision-makers and members of the public. Although the terms are simple superficially, they are misinterpreted regularly as implying that the associated event magnitude is only exceeded at regular intervals, and that they are referring to the expected elapsed time till the next exceedance. This misinterpretation of the terms used for expressing probabilities of flood magnitudes can be misleading and result in poor decisions.

It is believed that irrespective of the terms used, it is critical that all stakeholders have a common interpretation of the terms. Furthermore, it is important that stakeholders understand that the terms refer to long term averages. This means, for a given climatic environment, that the probability of an event of a given magnitude being equalled or exceeded in a given period of time (for example, one year) is unchanged throughout the life of the structure or the drainage network. Furthermore, it is not uncommon for an event to occur more than once in a single year.

Additionally, given the wet and dry phases that occur in many regions of Australia, these events are likely to be clustered in time. The occurrence of these wet and dry climatic phases highlight the misleading and inappropriate interpretation that flood events occur at regular intervals as implied by "*recurrence interval*" and "*return period*".

Flood events generally are random occurrences, and the period between exceedances of a given event magnitude usually is a random variate, the properties of which are assumed to be constant in time for a given location and climatic environment. The adopted terminology reflects this fundamental concept and is intended to convey a clear and precise interpretation.

2.2.3. Technical Correctness

In view of the loose and frequently incorrect manner in which probability terms are often used, it was considered that Australian Rainfall and Runoff should adopt terminology that is technically correct, as far as this is possible and in harmony with other objectives. Additionally, even if this is not entirely popular with all practitioners, Engineers Australia has a responsibility to encourage and educate engineers regarding correct terminology.

The two approaches used when describing probabilities of flood events in previous editions of Australian Rainfall and Runoff were:

- *Annual Exceedance Probability (AEP)* - the probability of an event being equalled or exceeded within a year. Typically the AEP is estimated by extracting the annual maximum in each year to produce an Annual Maxima Series (AMS); and
- *Average Recurrence Interval (ARI)* - the average time period between occurrences equalling or exceeding a given value. Usually the ARI is derived from a Peak over Threshold series (PoTS) where every value over a chosen threshold is extracted from the period of record.

Details of AMS and PoTs and the background to these alternative techniques for extracting flood series from recorded data are presented in Book 3, Chapter 2. Included in this discussion are the assumptions necessary for conversion of one probability terminology to the other using the Langbein formula (Langbein, 1949).

Using the Langbein formula, in probability terms, there is little practical difference for events rarer than 10% AEP. Historically, however, there has been a reluctance to convert from the approach used for derivation of the design flood estimate. Furthermore, terminology was attached to particular design flood estimation techniques; for example, when AMS were used to derive design flood estimates, the resultant probability was expressed as an AEP while when a PoTS was used for the same purpose, the resultant probability was expressed as an ARI.

In many situations, this distinction between an ARI and an AEP was imprecise as the design flood prediction methodology adopted did not explicitly note the use of either an AMS or a PoTS in the methodology. As a result, use of ARI and AEP was considered to be interchangeable. This interchangeable use often resulted in confusion.

The National Committee on Water Engineering believes that within Australian Rainfall and Runoff a terminology should be used which, while being technically correct, is consistent with other uses. Furthermore, the terminology adopted should be easily understood both by the profession and by other stakeholders within the community.

2.2.4. Practicality and Acceptability

The National Committee on Water Engineering is aware that while the terminology adopted must be technically correct it must also be relatively simple and suitable for use in practice. Terminology that meets this criterion will be accepted by the profession and by other stakeholders.

The interaction of the profession with the community and the increased public participation in decision making means that terminology needs to be clear not only to the profession but also to the community and other stakeholders, other professions involved in flood management, and to the managers of flood-prone land. This need has resulted in a move away from the terminology adopted in the 1987 Edition of Australian Rainfall and Runoff towards a clear and unambiguous terminology supported by the National Committee on Water Engineering of Engineers Australia and the National Flood Risk Advisory Group (NFRAG, a reference group under the Australian and New Zealand Emergency Management Committee). All parties believe that terminology involving annual percentage probability best conveys the likelihood of flooding and is less open to misinterpretation by the public.

2.2.5. Adopted Terminology

To achieve the desired clarity of meaning, technical correctness, practicality and acceptability, the National Committee on Water Engineering has decided to adopt the terms shown in Figure 1.2.1 and the suggested frequency indicators.

Frequency Descriptor	EY	AEP (%)	AEP (1 in x)	ARI
Very Frequent	12			
	6	99.75	1.002	0.17
	4	98.17	1.02	0.25
	3	95.02	1.05	0.33
	2	86.47	1.16	0.5
	1	63.21	1.58	1
Frequent	0.69	50	2	1.44
	0.5	39.35	2.54	2
	0.22	20	5	4.48
	0.2	18.13	5.52	5
	0.11	10	10	9.49
Rare	0.05	5	20	19.5
	0.02	2	50	49.5
	0.01	1	100	99.5
Very Rare	0.005	0.5	200	199.5
	0.002	0.2	500	499.5
	0.001	0.1	1000	999.5
	0.0005	0.05	2000	1999.5
Extreme	0.0002	0.02	5000	4999.5
			↓	
			PMP/ PMP Flood	

Figure 1.2.1. Australian Rainfall and Runoff Preferred Terminology

Navy outline indicates preferred terminology. Shading indicates acceptable terminology which is depends on the typical use. For example in floodplain management 0.5% AEP might be used while in dam design this event would be described as a 1 in 200 AEP.

As shown in the third column of Figure 1.2.1, the term Annual Exceedance Probability (AEP) expresses the probability of an event being equalled or exceeded in any year in percentage terms, for example, the 1% AEP design flood discharge. There will be situations where the use of percentage probability is not practicable; extreme flood probabilities associated with dam spillways are one example of a situation where percentage probability is not appropriate. In these cases, it is recommended that the probability be expressed as 1 in X AEP where 100/X would be the equivalent percentage probability.

For events more frequent than 50% AEP, expressing frequency in terms of annual exceedance probability is not meaningful and misleading , as probability is constrained to a maximum value of 1.0 or 100%. Furthermore, where strong seasonality is experienced, a recurrence interval approach would also be misleading. An example of strong seasonality is where the rainfall occurs predominately during the Summer or Winter period and as a consequence flood flows are more likely to occur during that period. Accordingly, when strong seasonality exists, calculating a design flood flow with a 3 month recurrence interval is of limited value as the expectation of the time period between occurrences will not be consistent throughout the year. For example, a flow with the magnitude of a 3 month recurrence interval would be expected to occur or be exceeded 4 times a year; however, in situations where there is strong seasonality in the rainfall, all of the occurrences are likely to occur in the dominant season.

Consequently, events more frequent than 50% AEP should be expressed as X Exceedances per Year (EY). For example, 2 EY is equivalent to a design event with a 6 month recurrence interval when there is no seasonality in flood occurrence.

Different users of Australian Rainfall and Runoff, in general, will use different segments of the relationship between flood magnitude and exceedance probability. To reduce confusion, that may arise from switching between different terminologies, it is recommended that consistent terminology in accordance with one of the columns of Figure 1.2.1 be used within an industry segment.

These expressions of estimated frequencies relate directly to the particular time period for which data have been analysed and frequencies determined with no consideration given to the long term effects of climatic change. Nonetheless, the adopted terminology is considered to be equally applicable to both stationary and non-stationary climatic environments, as there is no requirement for the annual exceedance probabilities to be constant over time. Consequently, where flood characteristics are changing as result of long term climatic change, the AEP of a flood characteristic for a future time period may be different or, conversely, a flood characteristic magnitude corresponding to a given AEP may change.

2.3. Difference Between Design Events and Actual Events

Much confusion has resulted from lack of recognition of the fundamental differences between these two types events and associated of flood estimation problems. Although the same mathematical procedures may be involved in both cases, the implications and assumptions involved, and the validity of application, are quite different. The emphasis in this document is largely on design floods.

A design flood is a probabilistic or statistical estimate, being generally based on some form of probability analysis of flood or rainfall data. An Annual Exceedance Probability is attributed to the estimate. This applies not only to normal routine design, but also to probable maximum estimates, where no specific probability can be assigned but the intention is to obtain a design value with an extremely low probability of exceedance. In the flood estimation methods based on design rainfalls, the probability relationship between design rainfall events and design flood events is not a direct one. Occurrence of a rainfall eventwhen the catchment is wet might result in a very large flood, while occurrence of the same rainfall event when the catchment was dry might result in relatively little, or even no runoff. For the design situation, the combinations of different factors combining to produce a flood event are not known and must be assumed, often implicitly in the design values that are adopted.

The approach to estimating an actual (or historic) flood from a particular rainfall event is quite different in concept and is of a deterministic nature. All causes and effects are directly related to the specific event under consideration. The actual antecedent conditions prevailing at the time of occurrence of the rain are directly reflected in the resulting flood and must be allowed for in its estimation. No real information on the probability of the on flood probability can be gained from consideration of a single actual flood event.

Although the differences in these two types of events are often not recognised, they have three important practical consequences. The first is that a particular procedure might be might be appropriate for analysing actual flood events but quite unsuitable for probabilistic design flood events.

The second concerns the manner in which values of parameters are derived from recorded data, and the manner in which designers regard these values and apply them. If actual floods are to be estimated, values for use in the calculations should be derived from calibration on individual observed events. If design floods are to be estimated, the values should be derived from statistical analyses of data from many observed floods.

The third practical consequence concerns the manner in which parameters are viewed by designers and analysts. For example, design initial losses for bursts can be very different from event initial losses derived from actual events, yet practitioners still often compare them without understanding the differences.

2.4. Probability Concepts

2.4.1. Probability Relationship Between Design Rainfall and Design Flood Characteristics

In the flood frequency based design flood estimation approaches covered in Book 3, the probabilities of a specific event magnitude being equalled or exceeded are estimated directly for the flood characteristic of interest (e.g. peak flow or flood volume). However, for the catchment simulation and hydrograph estimation procedures covered in Book 4, Book 5 and Book 7, the exceedance probability associated with design rainfall, as the primary probabilistic input to the design flood estimation procedure, needs to preserved in its transformation to a design flood. This concept is often referred to as AEP neutrality.

However, each of the processes represented in a model that converts rainfall to runoff and forms a flood hydrograph at the point of interest introduces some joint probability, resulting in the fundamental problem that the true probability of the derived flood characteristic may be obscure, and its magnitude may be biased with respect to the true flood magnitude with the same probability as the design rainfall, especially at the low probabilities of interest in design.

Since publication of ARR 1987 (Pilgrim, 1987) there has been a steady shift towards methods that better account for the stochastic nature of how floods of different magnitude and exceedance probabilities are generated. Procedures of different complexity to deal with this fundamental issue are discussed in Book 1, Chapter 3.

2.4.2. Choosing a Quantile Estimator

The 1 in Y AEP quantile corresponds to the flood magnitude with annual probability of exceedance equal to 1/Y. Because the parameters of the flood frequency distribution have to be estimated from limited data, the true quantile is not known. Different quantile estimates are available depending on the application. These are described in Book 3, Chapter 2.

In cases where the interest is principally on the accurate estimation of the AEP that corresponds to a specified flood magnitude (e.g. the flood level at which a particular flood protection structure is expected to fail), an expected AEP (or expected probability) quantile should be used. The use of such a quantile ensures that, on average, its AEP equals the true value. In cases where the mean-squared-error in the flood magnitude is to be minimized for a given AEP, expected parameter quantiles should be used.

The difference between these quantile estimates is typically not of significance when there is little or no extrapolation of the observed range of data, and especially if the skew is small. However, if extrapolation is required and high skews are involved, the difference can be appreciable. The methods in Book 3, Chapter 2 describes how to estimates these quantiles.

2.4.3. Avoiding Inconsistencies in Procedures and Resolution

The important step often overlooked by practitioners is mistakenly using an input or parameter that was derived a particular way and at a particular resolution in a manner that is different to how it was derived. This is particularly difficult to avoid with digital data sets compiled from different sources and resolutions. Historically problems have arisen when a method was derived from one scale map and used at a different scale.

2.5. Risk-Based Design

Floods can cause significant impacts where they interact with the community and the supporting natural and built environment. However, flooding also has the potential to be the most manageable natural disaster as the likelihood and consequences of the full range of flood events can be understood, enabling risks to be assessed and where necessary managed. There is strong move from managing floods by a by simple standards approach, where a certain frequency of flooding is deemed acceptable, to risk-based approaches, where the consequence and probability of design capacity being exceeded are assessed explicitly. Risk and design flood estimation concepts are discussed in detail in Book 1, Chapter 5.

2.5.1. Route Serviceability

A particular aspect of risk based approaches is where total system risk is of main interest. With a railway or major road, flooding of any one of many stream crossings will cause closure of the route. The item of real interest is the probability of this closure, and not of failure at any particular site. This probability of closure will be much greater than that at an individual site. Closure of the route at any site may cause major disruption and economic losses. Upgrade works can be targeted at reducing the probability of closure.

This problem is receiving increasing attention from transport managers and is discussed in detail in Book 1, Chapter 5.

2.6. The Importance of Data

Data is fundamental to flood estimation. Data is needed to understand the processes involved in the formation of floods and to ensure that models are accurate and reflect the real world issues being analysed. Flood estimation primarily uses data that describes the rainfall, streamflow and water levels. The procedures and guidelines presented in ARR could not have been developed without historical data, and often the reliability of the methods presented depends on the extent of data that has been used in development.

For the first time, ARR has been based completely on Australian data to better reflect Australia's variable landscape, including a national database of extreme flood hazards. A major task of the current ARR update was assembling a national databases of rainfall and streamflow data for developing inputs and methodologies. ARR 1987 (Pilgrim, 1987) used 600 pluviographs rainfall gauge (measures the amount of rainfall which fell) with greater than 6 years data and 7500 daily rainfall gauges with over 30 years record. ARR 2016 uses almost 30 years of extra rainfall and streamflow data, including data from over 2200 pluviographs and over 8000 daily rainfall gauges. Over 900 streamflow gauges were analysed. Over 100, 000 storm events were analysed. This data provides a valuable resource for the development of future methodologies.

Major improvements have been made to design flood estimation methods but national databases will allow the use and parameterisation of more complex methods. Major advances will continue that will allow us to leverage the limited data we can afford to collect on the continent nation. Many projects have opened the eyes of researchers and practitioners on what could be done with more time, money and the still limited data available. The data sets developed as part of this update should be enhanced and applied to for future improvements.

Book 1, Chapter 4 provides a summary of the types of data used for flood estimation.

2.7. Climate Change

ARR 1987 (Pilgrim, 1987) while acknowledging climate change did not address climate change or non-stationarity or provide guidance on the inclusion of climate change impacts in flood estimation. One key aim of this edition was the incorporation of the best available information of climate change impacts on flooding.

This edition of ARR funded research projects which investigated the following aspects:

- How climate change will affect flooding and the factors influencing flooding;
- How to incorporate climate change into the investigation methodologies used by the engineering profession to estimate design floods;
- Updating of the methodology in Australian Rainfall and Runoff so that the outcomes from climate change research (e.g. regional dynamic downscaling) can be incorporated easily into the investigation methodology as the science and results become available.

The impacts of climate change on design flood estimation are discussed in detail in Book 1, Chapter 5, Section 10. More detail can be found in the ARR Climate Change Research Plan and ARR Project 1: Climate Change Synthesis report (Bates and Westra, 2013; Bates et al., 2015).

2.7.1. Climate Change Impacts on Flooding

Global warming has been observed over several decades, and has been linked to changes in the large-scale hydrological cycle including increasing atmospheric water vapour content; changing precipitation patterns, intensity and extremes; changes in soil moisture and runoff; and increasing melting of snow and ice (Bates et al., 2008).There is increasing evidence that human-induced climate change is changing precipitation extremes, and that extreme flooding globally has increased over the 20th century (Trenberth, 2011). There is confidence that these changes in the hydrological cycle will lead to increased variability in precipitation and increased frequency of flood events over many areas (IPCC, 2007; Bates et al., 2008). Changes in climate will result in changes in the frequency, intensity, spatial extent, duration, and timing of extreme weather and climate events, and may lead to unprecedented extreme weather and climate events (IPCC, 2012).

The major areas where climate change will impact flooding are:

- Design rainfall intensity-frequency-duration;
- Storm type, frequency, and depth;
- Rainfall spatial and temporal patterns;
- Antecedent conditions;
- Changes in sea level; and
- The joint probability of storm surge and flood producing rainfall.

2.7.1.1. Climate Change Impacts on Rainfall

Changes in extremes events, such as floods, can be linked to changes in the mean, variance, or shape of probability distributions, or all of these (IPCC, 2012). For example, climate change projections have shown that a relatively small shift in the distribution of precipitation may result in a large change in the frequency and magnitude of extreme precipitation events (Nicholls and Alexander, 2007). Studies have shown that a change in the shape of the distribution of precipitation is likely to have a greater effect on the frequency of extremes than a shift in the mean precipitation (White et al, 2010; Groisman et al., 1999), and that climate change is most likely to increase climate variability, particularly affecting the extremes (Jones et al., 2012; Fowler and Ekstrom, 2009).

A warming climate leads to an increase in the water holding capacity of the air, which causes an increase in the atmospheric water vapour that supplies storms, resulting in more intense precipitation. This effect is observed, even in areas where total precipitation is decreasing (Trenberth, 2011). Indeed, some of the largest impacts of climate change are likely to result from a shift in the frequency and strength of climatic extremes, including precipitation (White et al, 2010). It is likely that the frequency of heavy precipitation will increase by the end of the 21st century, particularly in the high latitudes and tropical regions and there is likely to be an increase in heavy rainfalls associated with tropical cyclones (IPCC, 2012).

There have been many studies globally that have found increases in the intensity or frequency of extreme precipitation events (Bates et al., 2008; Westra et al., 2013). It is likely that since the 1970s the frequency of heavy precipitation events has increased over most areas (Bates et al., 2008). From 1950 to 2005, extreme daily rainfall intensity and frequency has increased in north-western and central Australia and over the western tablelands of New

South Wales, but decreased in the south-east and south-west and along the central east coast (CSIRO and Australian Bureau of Meteorology, 2007). Projections analysed by CSIRO and Australian Bureau of Meteorology (2007) showed that an increase in daily precipitation intensity is likely under climate change. The study found that the highest 1% of daily rainfalls tends to increase in the north of Australia and decrease in the south, with widespread increases in summer and autumn, but not in the south in winter and spring when there is a strong decrease in mean precipitation (CSIRO and Australian Bureau of Meteorology, 2007).

The increases in precipitation are more evident in sub-daily rainfalls and major changes in the intensity and temporal patterns of sub-daily rainfalls can be expected by the end of the 21st century (Westra et al., 2013). In a study of downscaled outputs from climate models, Abbs and Rafter (2008) found that by 2070 the models projected an increase of an average of 40% in intensity for 24 and 72 hour events around the Queensland-New South Wales border, and an increase of more than 70% in the two hour rainfall events in the high terrain inland from the Gold Coast.

2.7.1.2. Antecedent Conditions

Changes in the patterns of precipitation and in evaporation will lead to changes in antecedent conditions prior to flood events, affecting soil moisture and thus loss rates in the catchment (Bates et al., 2008). Potential evaporation is projected to increase almost everywhere on a global scale due to an increase in the water-holding capacity of the atmosphere with higher temperatures combined with little projected change in relative humidity (Bates et al., 2008).

Projections of potential evapotranspiration over Australia show increases by 2030 and 2070. The largest projected increases are in the north and east, where the change by 2030 ranges from little change to a 6% increase, with the best estimate being a 2% increase. By 2070, the A1FI scenario gives increases of 2% to 10% in the south and west with a best estimate of around 6%, and a range of 6% to 16% in the north and east with a best estimate around 10% (CSIRO and Australian Bureau of Meteorology, 2007).

Projected decreases in rainfall over much of Australia combined with increases in evaporation may result in disproportionate decreases in runoff due to a disconnection between surface and groundwater, as was experienced in parts of Australia during the Millennium drought (CSIRO, 2012).

2.7.1.3. Sea Level

The relatively small rise in sea level that is seen in observed data over the past century has already caused a significant change in the frequency of extreme sea-level events, and associated flooding (Hunter, 2007). Studies of observed sea level data worldwide have shown that sea level rise is the predominant cause of increases in the frequency of extreme sea level events (IPCC, 2007; Hunter, 2007). There is high confidence that there has been an increase in the frequency of high coastal sea level events of a given magnitude, and that extreme flooding events due to sea level rise will increase significantly, dependent on location (Church et al., 2012). The likely range of global-mean sea level rise between the 1980 – 1999 and 2090 – 2099 periods is given by (IPCC, 2007) as 0.18 – 0.59 m. There is high confidence that the global rate of sea level rise has increased between the mid-19th and the mid-20th centuries. The average rate was 1.7 ± 0.5 mm/yr for the 20th century and 3.1 ± 0.7 mm/yr for 1993–2003 (Bates et al., 2008). The observed rate of sea level rise in the Australian region from 1993 - 2011 has high spatial variability, with a maximum in the north and north-west coasts of Australia of 9mm/yr, and a rate of 2 to 4 mm/yr on the south-eastern and eastern Australian coastline (Church et al., 2012).

2.8. Dealing with Uncertainty

2.8.1. Introduction

This section provides an overview of the uncertainties in the design flood estimation. The specific aims are to:

- Identify the types of uncertainty in design flood estimation;
- Motivate practitioners on the value of undertaking uncertainty analysis; and
- Raise awareness of the various sources of uncertainty in common techniques for design flood estimates.

2.8.2. Types of Uncertainty in Design Flood Estimation

It is typical in current practise for design flood estimation to ignore the uncertainty in the estimates of the design flood. This is despite the considerable uncertainties that are introduced when undertaking a Flood Frequency Analysis using short data records and extrapolating the fitted flood frequency distribution to estimate the 1% or 0.5% Annual Exceedance Probability flood. Similarly, when using a catchment modelling approach to obtain estimates of the design flood, the typical situation is that the catchment modelling system is calibrated to data from a few selected flood events, and the calibrated model is then extrapolated using design rainfall estimates (which itself is an extrapolation of observed rainfall data, Book 2, Chapter 3) to provide estimates of the 1% or 0.5% Annual Exceedance Probability flood. Both these type of approaches introduce significant uncertainties in estimates of the design flood.

The causes of these uncertainties are that practitioners are required to: (1) Use mathematical algorithms to represent the complexity of catchment processes that transform rare rainfall into rare flood events. (2) Calibrate and validate these algorithms using measurements of the catchment process that are highly uncertain. It is widely acknowledged that there is significant spatial variation in catchments and temporal and spatial variation in the antecedent catchment wetness and rainfall events that drive significant flood events. Practitioners use hydrologic models, which are simplified mathematical conceptualisations to represent these complex spatially and temporally distributed hydrological processes. These hydrologic models are calibrated to measurements of data on variables such as rainfall, evaporation and flow. It is widely acknowledged that these data can have significant measurement errors (refer to Book 1, Chapter 4). Rainfall is spatially heterogeneous, however, typically there are only a small number rainfall gauges in a given catchment. Streamflow is based on river height (stage) measurements and a rating curve, which can be difficult to reliability estimate for large flood events. Typically these uncertainties are ignored in the design flood estimation process.

Uncertainty analysis provides the tools with which to handle this uncertainty and incorporate it into the design flood estimates. To enable the use of uncertainty analysis tools, it is first important to distinguish two broad types of uncertainty:

- *Aleatory (or inherent) Uncertainty* - refers to uncertainty that arises through natural randomness or natural variability that we observe in nature; and

- *Epistemic (or knowledge-based) Uncertainty* - refers to uncertainty that is associated with the state of knowledge of a physical system (our estimation of reality), our ability to measure it and the inaccuracies in our predictions of the physical system.

These definitions are consistent with the broad definitions provided by Ang and Tang (2007) in wider context of general engineering and the specific context of flood risk by Pappenberger and Beven (2006). The major differences between the two types of uncertainty is that epistemic uncertainty can be reduced, through advances in process understanding or improvement in measurement techniques, while aleatory uncertainty cannot be reduced, and therefore needs to be characterised. Both types of uncertainty can be characterised using tools of uncertainty analysis. Ang and Tang (2007) provide a wealth of examples of the two types of uncertainty in a general engineering context.

In the context of design flood estimation, a simple example to understand the differences between these two types of uncertainty is to consider an example of a flood frequency distribution, as shown in Figure 1.2.2, with probability limits on the design flood estimates over the range of Annual Exceedance Proabilities.

An illustration of aleatory uncertainty is the natural variability in annual maximum floods which is due to the climate variability in extreme rainfall and antecedent soil moisture condition from year to year. This aleatory uncertainty influences the shape of the flood frequency distribution, and influences the values of 1% Annual Exceedance Probability design flood estimates. The aleatory uncertainty is why practitioners undertake a risk-based design approach to estimate the likelihood of flooding. At different catchments, the flood frequency distribution changes due to the natural variability in the climate and catchment processes, hence this is also of type aleatory uncertainty.

An illustration of epistemic uncertainty is the uncertainty in the estimate of the design flood for a given Annual Exceedance Probability, e.g. Figure 1.2.2, the design flood for a 1% Annual Exceedance Probability has an expected flow of 100 m^3/s and the 95% probability limits are 65 and 155 m^3/s. This uncertainty in the design flood estimate for a given Annual Exceedance Probability is primarily of type epistemic (or knowledge based) uncertainty. There is an opportunity to reduce this uncertainty, if there were longer flow records which would reduce the uncertainty in the parameters of the flood frequency distribution fitted to the annual maximum floods. Similarly, for catchment modelling, or if there was a better understanding on the catchment processes obtained through better data to calibrate and verify the catchment modelling system, this would reduce the uncertainty in the flood estimates of the catchment model.

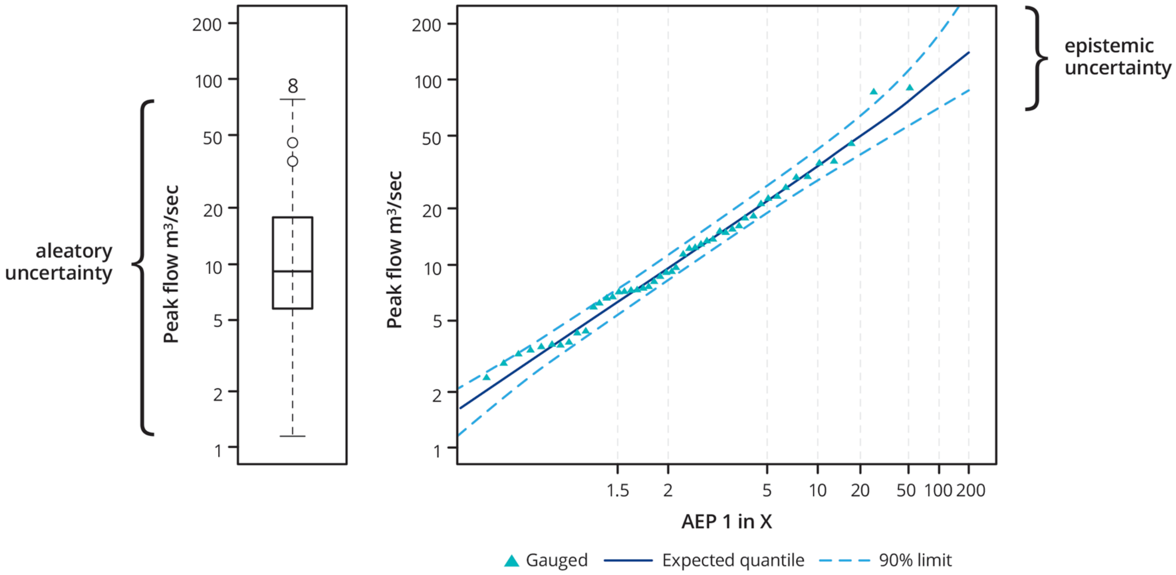


Figure 1.2.2. Different Types of Uncertainty, Aleatory and Epistemic, in the Context of Design Flood Estimation

Despite the simplicity of the two illustrations of aleatory and epistemic uncertainty, given in the flood frequency distribution in Figure 1.2.2, there are occasions where the distinction between the two different types of uncertainty is not always clear. For example, the illustration of Figure 1.2.2 implies that as level of information increases and the epistemic uncertainty is reduced then "true" flood frequency distribution for a given catchment will emerge. There is practical limit on the level of information (data and/or process understanding) available on a given catchment hence the concept of a single "true" flood frequency distribution for a given catchment is likely to unobtainable. Hence the epistemic uncertainty given in Figure 1.2.2, will have a component of aleatory uncertainty.

The concepts of aleatory and epistemic uncertainty are similar to concepts of flood likelihood and uncertainty from risk-based decision-making (Book 1, Chapter 5).

2.8.3. Motivation for Incorporating Uncertainty Into Design Flood Estimates

There are a range of approaches for dealing with uncertainty, the simplest of which is to ignore it, to qualitative descriptions (highly uncertain) or relative rankings, (option 1 is more uncertainty than option 2) ie. to rigourous quantitative approaches which use uncertainty analysis techniques to characterise the individual sources of uncertainty, and use advanced techniques to estimate their impacts on the uncertainty in the design flood estimations (refer to Book 4, Chapter 3 for an overview of the various approaches). The greater the rigour in uncertainty analysis approach the more effort and resources is required. The reward for this greater effort is more informed decision making.

An example of the potential benefits of incorporating uncertainty for more informed decision making is provided in Figure 1.2.3. Consider two different designs; Design A and Design B. The practitioner needs to choose the design that reduces the flood magnitude for given catchment location. Design A has a higher value for the most likely estimate of the design flood, but has a lower uncertainty than Design B. The differences in the uncertainty estimates could arise because Design B is a more complicated design option than design A and requires the use of more complex catchment modelling approach (e.g. fully distributed model (Book 5)) and there was a lack of spatial data in the catchment to calibrate the distributed model and hence parameter estimates had to be based on regional information. In contrast Design A was based on catchment modelling approach that was well-calibrated using high quality data that was readily available in the catchment. If the uncertainty is ignored then Design B would be the preferred choice of the practitioner, because the most likely estimate of the flood magnitude is lower than Design A. If the uncertainty in the flood magnitude incorporated than a practitioner who is risk-averse may prefer to choose Design A, because it the probability of a large magnitude flood with major/catastrophic consequence is lower than Design B. This example illustrates how the uncertainty in the design flood estimates, when combined with risk attitude (risk-averse, risk-neutral, or risk-seeking) of the practitioner provides a more information on which to base the design choice.

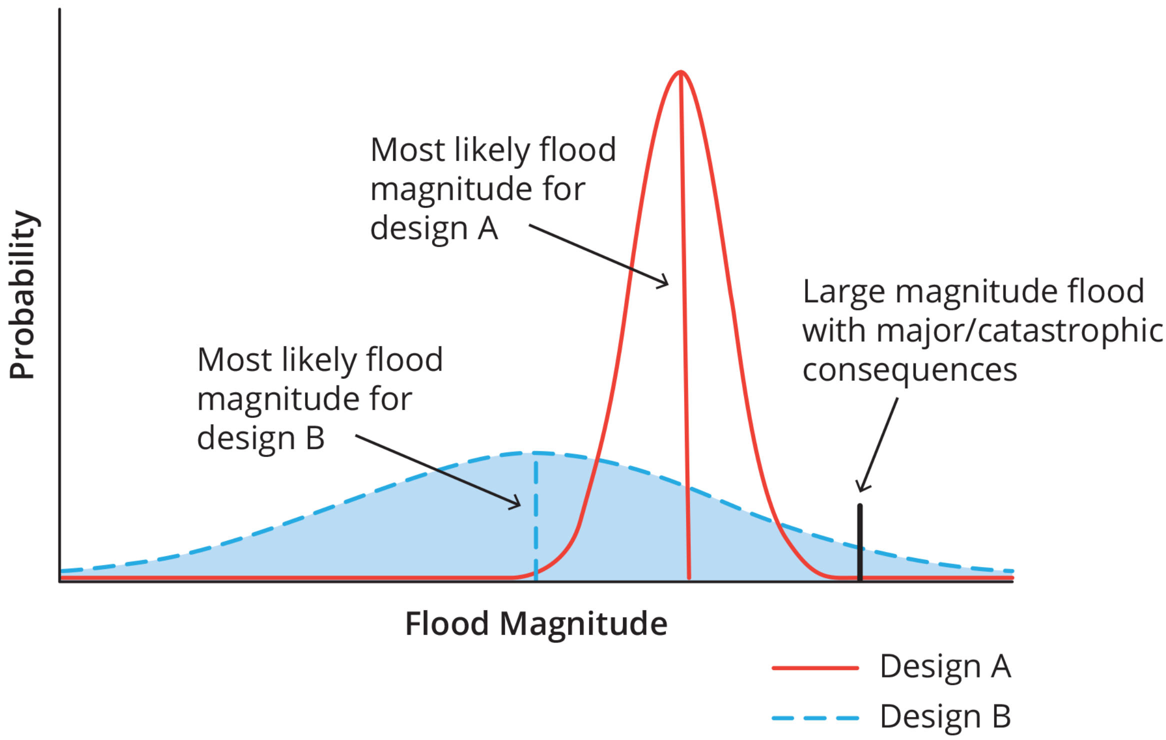


Figure 1.2.3. Impact of Uncertainty on a Design Flood Estimate for Two Design Cases

From a practical and scientific perspective Pappenberger and Beven (2006) provide an overview of the common reasons for not undertaking uncertainty analysis for hydrologic and hydraulic models and argue that these arguments are not tenable. A summary of the reasons provided by Pappenberger and Beven (2006) and their counter arguments are summarized as follows:

1. *Uncertainty Analysis is Not Necessary Given Physically Realistic Models*

   Pappenberger and Beven (2006) states there are a group of practitioners who believe that their models are (or at least will be in the future) physically correct and thus parameter calibration or uncertainty analysis should not be necessary (or only minimal) if predictions are based on a true understanding of the physics of the system simulated. This position is difficult to justify considering published discussions of the modelling process in respect of the sources and impacts of uncertainties (Beven, 1989; Beven, 2006; Oreskes et al., 1994). It is argued that this group of practitioners have too much faith in the model representation of physical laws or empirical equations. An alternative is a group of practitioners who inherently accept uncertainties in the modelling process, at least as a result of errors and natural variability in time and space.

2. *Uncertainty Analysis is Not Useful in Understanding Hydrological and Hydraulic Processes*

   To be able to learn about how water flows through the landscape and the best model to represent this water flow requires the use of a hypothesis testing framework . In real applications, this hypothesis testing framework would evaluate different competing hypothesis (ie. models) against the observations, and should explicitly consider the potential sources of uncertainty in applications to real systems to enable the results to be stated in a probabilistic rather than a deterministic manner. This would enable evaluation of whether the differences in model performance, can be reliability identified given the uncertainty in the predictions and observations.

3. *Uncertainty (Probability) Distributions Cannot be Understood by Policy Makers and the Public*

   Pappenberger and Beven (2006) cite several scientific studies that suggest practitioners actually want to get a feeling for the range of uncertainty and the risk of possible outcomes. Furthermore, policy-makers derive decisions on a regular basis under severe uncertainties. If uncertainty is not communicated and there is a misunderstanding of the certainty of modeling results this can lead to a loss of credibility and trust in the model and the modelling process.

   However, it is acknowledge that there are a wide range of different perceptions of “risk” and “uncertainty” and that effort is required on the part of both pracitioners and policy-makers to work together to achieve a common understanding of uncertainty.

4. *Uncertainty Analysis Cannot be Incorporated into the Decision-Making Process*

   There are two supporting arguments to this reason (1) Decisions are binary; (2) Uncertainty bounds are too wide to be useful in decision making. Pappenberger and Beven (2006) conclude there is no question that, for many environmental systems, a rigorous estimate of uncertainty leads to wide ranges of predictions. There are certainly cases in which the predictive uncertainty for outcomes of different scenarios is significantly larger than the differences between the expected values of those scenarios. This leads to the perception that decisions are difficult to make. To counter these arguments, Pappenberger and Beven (2006) present numerous examples from the literature on decision support systems and decision analysis which provide a range of methods for decision making under uncertainty based on assessments of the risk and costs of possible outcomes. Examples of decisions under uncertainty for Flood Frequency Analysis are illustrated by Wood and Rodriuez-Iturbe (1975) and more recently by Botto et al. (2014). The key outcome from Botto et al. (2014) was that incorporating uncertainty in estimating the design floods (by minimising the total expected costs) leads to substantial higher estimates of the design flood compared to standard approaches when uncertainty is ignored. This suggests incorporating uncertainty leads to reduce expected costs and highlights the benefits of incorporating uncertainty.

5. *Uncertainty Analysis is Too Subjective*

   Pappenberger and Beven (2006) identify that in the application of uncertainty analysis methods, certain decisions must be made, some of which include an element of subjectivity, including the choice of probability distributions for data errors, prior distributions for parameter uncertainty or predictive errors. In principle, many of these assumptions can be checked as part of the analysis but it is common to find that not all assumptions can be fully justified or some assumptions cannot be checked, and hence this leads to the conclusion that predictions with uncertainty are too subjective. Pappenberger and Beven (2006) conclude that any analysis which does not considering uncertainties in the modeling can be objective. This view is based on a misplaced faith in deterministic modeling in the light of the inevitable uncertainty in the modeling process (refer to also argument 1 above). Even a fully deterministic model run requires necessarily subjective assumptions about model inputs and boundary conditions and performance evaluation. The important issue is that the nature of the assumptions should be made explicit so that they can be assessed and discussed. Uncertainty analysis provides a set of tools to make these assumptions transparent and subject them to explicit scrutiny.

6. *Uncertainty Analysis is Too Difficult to Perform*

   Pappenberger and Beven (2006) note this is a common attitude amongst practitioners and is consequence of the need to spend more time and money on assessing the different potential sources of uncertainty in any particular application, coupled with a lack of clear guidance about which methods might be useful in different circumstances. Pappenberger and Beven (2006) note that in general, uncertainty analysis is not too difficult to perform and provide list of relevant software that is available. Since, Pappenberger and Beven (2006) review, the research publications on uncertainty analysis in hydrologic modelling has increased substantially, with many new tools/techniques and reviews available (for example the recent review by Uusitalo et al. (2015)). These tools will be reviewed to provide guidance for practitioners on which is applicable for different situations in the context of design flood estimation. The continued increases in computational power have reduced the computational costs of uncertainty analysis, which reduces the difficulty in undertaking uncertainty analysis.

In summary, Pappenberger and Beven (2006) conclude that in the past many modelling and decision making processes have ignored uncertainty analysis and it could be argued that under many circumstances it simply would not have mattered to the eventual outcome. However, they note that the arguments for uncertainty analysis are compelling because:

1. It makes the practitioner think about the processes involved and the decisions made based on model results;

2. It makes predictions of different experts more comparable and leads to a transparent science;

3. It allows a more fundamental retrospective analysis and allows new or revised decisions to be based on the full understanding of the problem and not only a partial snapshot; and

4. Decision makers and the public have the right to know all limitations in order to make up their own minds and lobby for their individual causes.

2.8.4. Sources of Uncertainty in Context of Design Flood Estimation

Book 1, Chapter 2, Section 8 outlined the practical advantages of undertaking uncertainty analysis. The first step of undertaking uncertainty analysis is to identify the various sources of uncertainty in the modelling processes. To raise awareness of the various sources of uncertainty in the context of design flood estimation, this section will outline the various sources of uncertainty and identify how these sources of uncertainty manifest themselves in the two common techniques used for design flood estimation; the Flood Frequency Analysis and catchment modelling approaches to design flood estimation. The primary drivers of each of the sources of uncertainty will then also be discussed.

The various sources of uncertainty that are relevant to design flood estimation are outlined as follows:

- *Predictive Uncertainty*

Predictive uncertainty represents the total uncertainty in the predictions of interest, typically the estimates of the design flood. It is comprised of the various sources of uncertainty that are outlined below, including data uncertainty, parametric uncertainty, structural uncertainty, regionalisation uncertainty (if relevant) and deep uncertainty (if relevant). This total predictive uncertainty is what used as input to the decision making uncertainty framework, to provide reliable predictions. The magnitude of the total predictive uncertainty and the relative contribution of the various sources of uncertainty is of obvious interest. The magnitude provides an indication of the total uncertainty of the predictions, while the relative contribution highlights which sources of uncertainty are the key contributors and which can be reduced.

- *Data Uncertainty*

Data uncertainty is a key source of predictive uncertainty. The more uncertain the data used to inform the methods used to estimate the peak flows, the more uncertainty in the predictions of the peak flows. The definition of “data” is a challenging one in the context of design flood estimation since in each step of the modelling process, the data used an input maybe based on the output of a prior modelling process, rather than actual measurements. Data uncertainty is dependent on the quality and number of measurements undertaken to inform that data.

- *Parametric Uncertainty*

Design flood estimates relay on using mathematical models to predict design floods. These models are estimated using time series of uncertain data with finite length. These limitations induce uncertainty in the estimates of these parameters, called parametric uncertainty. This parametric uncertainty would occur even if the mathematical model were exact. The magnitude of this parametric uncertainty, decreases as the length of the time series of data increases and increases when the uncertainty of the data increases. When time series are short and/or uncertainty in the data are high then parametric uncertainty can contribute significantly to total predictive uncertainty.

- *Structural Uncertainty*

Structural uncertainty refers to the uncertainty in the mathematical model used to provide the predictions of the peak flows. It is a consequence of the simplifying assumptions made in approximating the actual environmental system with a mathematical hypothesis (Renard et al., 2010). The structural error of a hydrologic model depends the model formulation.

- *Regionalisation Uncertainty*

Regionalisation uncertainty refers to the uncertainty induced when there is a geographical migration of hydrological information from data rich location to a data poor location. This is an extension of the concepts of regionalisation of hydrologic model parameters, as outlined by Buytaert and Beven (2009). In the context of design flood estimation itt refers to any information that is transferred from one site to another, and could include the parameters of the flood frequency distribution, the parameters of the runoff-routing model, the loss model or the design rainfall used in the catchment modelling approach. It is a function of the predictive uncertainty of the original application of the model at the data rich location (which is a dependent on the structural, parametric and data uncertainty at that data rich site) and the regionalisation model used to transfer information from one site to another. Given there are large number of sources of uncertainty in regionalisation uncertainty, it can induce significant predictive uncertainty, when there is very limited at-site data.

- *Deep Uncertainty*

Deep uncertainty refers to the sources of uncertainty that impact on the robustness of design but are difficult to assign apriori probabilities measures to. It acknowledges that practitioners and decision makers may not be able to enumerate all sources of uncertainty in a system nor their associated probabilities (Herman et al., 2014). It is related to the emerging field of robust decision making, where it is assumed that future states of the world are deeply uncertainty and instead of assigning probabilities, it seeks to identify robust strategies which perform well across the range of plausible future states. In the context of design flood estimation, examples of deep uncertainty could include the effects of climate change, because the different scenarios used for future greenhouse gas emissions cannot be assigned probabilities, another example might be future land use changes within a catchment, because it depends on variety of political, social and economic factors, which can be difficult to reliably assign probabilities. This source of uncertainty requires a different approach to the other sources, where scenario analysis is used to test the system and identify thresholds where significant failures occur. This approach has seen recent application in analysing water resources systems for long-term drought planning, however the application in flood design is limited. Given this is still a burgeoning area with significant research required, the approaches to treat this source of uncertainty will not be further considered in the scope of this uncertainty in Australian Rainfall and Runoff.

2.8.5. Raising Awareness of the Sources of Uncertainty in Techniques Used for Design Flood Estimation

In this section, it will be illustrated how to identify the sources of uncertainty for the two common techniques used for design flood estimation; Flood Frequency Analysis and catchment modelling. The identification of the sources of uncertainty involves the following steps:

1. Identify the information required for each step of the methods; and

2. Identify the potential sources of uncertainty in the information required for each of the steps.

Uncertainty is related to the level of information (ie. available of at-site data, its length and quality). For the purposes of this illustration, two different scenarios of available information will be considered (a) Using at-site data (b) No at-site data available, using regional information only. In practise, the level of information will be commonly be somewhere in between these two scenario, nonetheless these two scenarios provide convenient “use” case, to illustrate the identification of the sources of uncertainty.

The relative contribution of each of these sources of uncertainty to the total predictive uncertainty is catchment specific, and depend on a range of factors (outlined below). Hence, to evaluate and determine the dominant source of uncertainty in a particular catchment requires a rigourous uncertainty analysis. Hence the following description will focus on describing the various source of uncertainty for each of the steps in both Flood Frequency Analysis and catchment modelling and identify the factors that will impact on the magnitude of that particular source. In any particular combination of information available means that one source could dominant the other. Hence in the following descriptions, each uncertainty source will not be described as low or high, rather the description will identify what increases or decreases the magnitude of the sources uncertainty.

2.8.5.1. Flood Frequency Analysis

1. *Estimate Flood Frequency Distribution Parameters*

   a. Using At-site Data

   Data Uncertainty

   When using at-site streamflow data to estimate the Flood Frequency Distribution, the data uncertainty in this streamflow data is a source of uncertainty. The factors that effect the magnitude of this source of uncertainty are primarily the quality of the rating curve used to estimate the streamflow, the number of gaugings (and their quality), the degree of extrapolation of the rating, the stability of the rating curve, among others (Le Coz et al., 2013).

   Parametric Uncertainty

   As parameters of the Flood Frequency Distribution are estimated based on limited time series of data, this induces uncertainty in the parameters. This parametric uncertainty is determined by the length of data (uncertainty increases as the length decreases) and the quality of the data (parametric uncertainty increases as data uncertainty increases).

   Structural Uncertainty

   The source of structural uncertainty is the assumed form of the food frequency distribution probability model, ie. log-Normal, Log Pearson III etc. When calibrating to at-site data, this source of uncertainty can be checked by comparing against the observed data, to determine if the quality of the fit to observed data.

   b. Using Regional Information without At-site Data

   Data, Parametric, and Structural Uncertainty

   When there is no at-site data, then regional information is used to inform the parameters and the choice of the probability model used for the flood frequency distribution. For this case, there data uncertainty is not a source of uncertainty, however the parametric uncertainty is higher than case (a), because no at-site data is available, and the structural uncertainty is also high than case (a) because no at-site data is available to evaluated if the chosen probability model for the flood frequency distribution is appropriate.

   Regionalisation Uncertainty

   When using regional information there is also regionalisation uncertainty because the parameters of the flood frequency has been transferred from another catchment. All the sources of uncertainty that contribute to the regionalisation uncertainty as described previously will be relevant to this source of uncertainty.

2. *Predicting Design Floods using Flood Frequency Analysis*

   In this Step 2 of predicting design floods using Flood Frequency Analysis, the data, parametric and structural uncertainty sources identified in Step 1 will be present. A additional contributor to the structural uncertainty when predicting design floods with Annual Exceedance Probability beyond the range of the streamdata (e.g. 1 in 100 Annual Exceedance Probability based on 30 years of streamflow data) is the assumption that the chosen probability model will provide a reliable estimate of design floods under extrapolation to the 1 in 100 or 1 in 200 Annual Exceedance Probability flood. This additional source of structural uncertainty will be present, irrespective of case (a) or case (b) levels of information. A longer time series of at-site streamflow data, and hence a lower degree of extrapolation will decrease, but not eliminate, the magnitude of this source of uncertainty.

2.8.5.2. Catchment Modelling Approach to Estimating Design Floods

The catchment modelling approach to design flood estimation relies on estimates of the design rainfall, which is converted into effective rainfall using a loss model and then used as input into runoff-routing model (calibrated to a limited number of flood events) to simulated flood events and therefore provide estimates of the design flood. The steps of this approach are at (1) Estimate runoff-routing model and loss model parameters (2) Estimating design rainfall and the temporal and spatial patterns (3) Predicting design floods using catchment modelling systems. These steps are outlined:

1. *Estimate Runoff-Routing Model and Loss Model Parameters*

   The parameters for the runoff-routing model and the loss model are usually calibrated jointly using flooding events in a given catchment. There are distinct components of the catchment modelling processes, however as their sources of uncertainty are similar, they will be discussed together.

   a. Using At-site Data

      Data Uncertainty

      Runoff-routing models (e.g. RORB) and loss models (e.g. required in the catchment modelling approach are typically calibrated to at-site flood event data. In this calibration step, the data uncertainty is the uncertainty in the streamflow data (discussed previously) and the additional uncertainty in the rainfall data, which as discussed previously, increases as the rainfall gauge density within the catchment decreases.

      Parametric Uncertainty

      The runoff-routing model loss model have parameters estimated through calibration to a limited number of flood events. This source of parametric uncertainty will decreases as the number of events decreases, and the consistency of the parameter estimates between events also increases. If the parameter estimates vary significantly between events, this will increase the parametric uncertainty.

      Structural Uncertainty

      As the runoff-routing model and the loss models represents a mathematical simplification of the actual catchment processes, will be a source of structural uncertainty. As the fit to the data used for calibration increases this source of uncertainty will decrease, but will not be eliminated. If the complexity of the runoff-routing model increases, e.g. move from lumped to a spatially distributed model, may potentially decreased the structural uncertainty, however, with a spatially distributed model the challenge becomes estimating the parameters over a spatial grid. Hence, if there is a lack of spatial streamflow and rainfall data to calibrate the model, than there is a potentially a shift from structural uncertainty to parametric uncertainty, which may results in no reduction the total predictive uncertainty.

b. Regional Information Only

Data, Parametric, and Structural Uncertainty

Similar to Flood Frequency Analysis, when there is no at-site data, the regional information is used to inform the parameter estimates, and choice of runoff-routing and loss model. For this case, there is data uncertainty is not a source of uncertainty, however the parametric uncertainty is higher than case (a), because no at-site data is available, and the structural uncertainty is also high than case (a) because no at-site data is available to evaluated if the runoff-routing model or loss model is appropriate.

Regionalisation Uncertainty

When using regional information there is also regionalisation uncertainty because the parameters of the runoff-routing model and loss model have been transferred from another catchment. All the sources of uncertainty that contribute to the regionalisation uncertainty as described previously will be relevant to this source of uncertainty, but they will apply both to the loss model and the runoff-routing model. In comparison to regionalisation of flood frequency distribution which is relatively well advanced , the regionalisation of runoff-routing models and loss models is still relatively unreliable and hence the regionalisation uncertainty of runoff-routing and loss models is likely to far larger than regionalisation of flood frequency distributions.

2. Estimating Design Rainfall and the Temporal and Spatial Patterns

In the majority of cases practitioners will use the design rainfall estimates provided by the Bureau of Meteorology, rather than undertake an Intensity Frequency Duration analysis of the observed rainfall data within a catchment, hence only the case when regional information is available will be considered in this description. There are many similarities to sources of uncertainty in the Flood Frequency Analysis, except the goal is to estimate extreme rainfall events rather than flow events.

Data, Parametric, Structural and Regionalisation Uncertainty

The source of data uncertainty is rainfall gauge density and the length of rainfall data across Australia, is highly variable in different parts of Australia and with far lower gauge density and shorter records for sub-daily rainfall data then daily. This can induce significant data uncertainty in the design rainfall estimates. Similar to Flood Frequency Analysis, a probability model is used to estimate the extreme rainfall events (e.g. 1 in 100 AEP) based on the limited rainfall data available. This probability model has parametric uncertainty, which increases as the length and quality of the rainfall data decreases. There is structural uncertainty in the choice of the probability model for extreme rainfall, and this is increased when the probability model is used to extrapolate to from shorter rainfall time series to extreme events. This is particular problematic for sub-daily rainfall, because records are typically shorter than daily rainfall data. There is regionalisation uncertainty because the design rainfall estimates are regionalised to areas with limited gauged data.

This design rainfall for an event is then disaggregated into a time series using temporal patterns, they have their own sources of data, parametric, structural and regionalisation uncertainty, because they are estimated based on rainfall data from outside the catchment of interest. If spatial patterns are used to distribute design rainfall spatially across a catchment, then they will similar sources of uncertainty.

Considering the high spatial and temporal variability of rainfall process these uncertainties in design rainfall are unlikely to be small.

3. Predicting Design Floods using Catchment Models

   When a catchment modelling approaches is used to predict design floods, the data, parametric, structural and regionalistion uncertainty identified in Steps 1 and 2 will be present. There are two sources of addition uncertainty, parameter uncertainty and structural uncertainty. These sources of uncertainty are because the runoff-routing and loss models in Step 1 are calibrated on runoff events are then extrapolated to larger design flow events, e.g. 1 in 100 AEP. The source of uncertainty is whether the parameters and model structural based on calibrations to (inevitable) smaller flood events can be applied to the larger design flow events.

2.8.5.3. Total Predictive Uncertainty

Table 1.2.1 provides a summary of the various sources of uncertainty for the two different techniques (Flood Frequency Analysis versus catchment modelling) for design flood estimation. It can be seen that due to the larger number of components in the catchment modelling, there are a greater number of sources of uncertainty in this process, compared with Flood Frequency Analysis. Typically when there are a larger number of sources of uncertainty the total predictive uncertainty is higher. Based on this analysis it can be concluded that catchment modelling is likely to have a higher total predictive uncertainty compared with Flood Frequency Analysis. However, the relative magnitude of the total predictive uncertainty for the two different techniques would vary on a catchment basis.

Table 1.2.1. Sources of Uncertainty in Design Flood Estimation

Steps	**Information Available**	**Sources of Uncertainty**			
		Data	**Parametric**	**Regionalisation**	**Structural**
Flood Frequency Analysis (FFA)					
1. Estimate Flood Frequency Distribution Parameters	a. At-site data	yes - streamflow	yes	No	yes
	b. Regional information only	No	yes – higher than case(a)	yes	yes – higher than case(a)
2. Predict Design Floods using Flood Frequency Analysis	Based on step 1	n/a - identified in step 1	n/a - identified in step 1	n/a - identified in step 1	yes - in addition to step 1

Steps	Information Available	Sources of Uncertainty			
		Data	Parametric	Regionalisation	Structural
Catchment Modelling					
1. Estimate Runoff-Routing Model and Loss Model Parameters	a. At-site data	yes – rainfall and streamflow	yes	No	yes
	b. Regional information only	No	yes – higher than case(a)	yes	yes – higher than case(a)
2. Estimate Design Rainfall and the Temporal/ Spatial Patterns	Based on Bureau of Meteorology IFD	yes – rainfall	yes	yes	yes
3. Predict Design Floods using Catchment Modelling Systems	Based on steps 1-2	n/a – identified in steps 1-2	yes – in addition to steps 1-2	n/a – identified in steps 1-2	yes – in addition to steps 1-2

2.8.6. Summary

This overview of the uncertainty in design flood frequency estimation has identified the two different types of uncertainty in the context of design flood estimation, aleatory uncertainty (due to natural variability) and epistemic uncertainty (due to knowledge uncertainty). It then outlined the motivation for undertaking uncertainty analysis, which is to provide more informed and transparent information on the uncertainty in the design flood estimates to enable practitioners and design makers to make better judgements on the appropriate design. The major sources of uncertainty in the context of design flood estimation were then outlined, and include data (uncertainty in measurements), parametric uncertainty of the models used, structural uncertainty in the models mathematical representation of the physical process, regionalisation uncertainty when information is moved from data rich to data poor catchments, and the total predictive uncertainty, which is composed of the elements of the individual sources of uncertainty. To raise awareness of the sources of uncertainty in the different techniques used for design flood estimation were identified. The conclusion, was that comparing Flood Frequency Analysis and catchment modelling, due to the larger number of components, the catchment modelling technique has a larger number of sources of uncertainty than Flood Frequency Analysis, and hence this will likely lead to a higher predictive uncertainty. However, the magnitude of the total predictive uncertainty is catchment specific, depending the availability of data and knowledge of the processes that driver design flood events.

2.9. References

Abbs, D. and Rafter, T. (2008), The Effect of Climate Change on Extreme: Rainfall Events in the Westernport Region, CSIRO.

Ang, A. and Tang, W. (2007), Probability Concepts in Engineering, 2nd edition ed. Wiley, United States of America.

Bates, B., and Westra, S. (2013), Climate Change Research Plan - Summary. Report for Institution of Engineers Australia, Australian Rainfall and Runoff Guideline: Project 1.

Bates, B., Evans, J., Green, J., Griesser, A., Jakob, D., Lau, R., Lehmann, E., Leonard, M., Phatak, A., Rafter, T., Seed, A., Westra, S. and Zheng, F. (2015), Development of Intensity-Frequency-Duration Information across Australia - Climate Change Research Plan Project. Report for Institution of Engineers Australia, Australian Rainfall and Runoff Guideline: Project 1. 61p.

Bates, B. C., Z. W. Kundzewicz, S. Wu, and J. P. Palutikof (2008), Climate Change and Water. Technical Paper of the Intergovernmental Panel on Climate Change, Rep., 210 pp, IPCC Secretariat, Geneva.

Beven, K. (1989), Changing ideas in hydrology - The case of physically-based models, Jounral of Hydrology, 105(1-2), 157-172.

Beven,A manifesto for the equifinality thesis, Journal of Hydrolology, 320: 18-36.

Botto, A., Ganora, D., Laio, F. and Claps, P. (2014), Uncertainty compliant design flood estimation, Water Resour Res, 50(5), 4242-4253. 10.1002/2013WR014981

Buytaert, W. and Beven, K. (2009), Regionalization as a learning process, Water Resour Res, 45(11).

CSIRO (2012), Climate and water availability in south-eastern Australia: A synthesis of findings from Phase 2 of the South Eastern Australian Climate Initiative (SEACI), CSIRO, Australia, September 2012, 41 pp.

CSIRO and Australian Bureau of Meteorology (2007), Climate Change in Australia, CSIRO and Bureau of Meteorology Technical Report, p: 140. www.climatechangeinaustralia.gov.au

Church, J.A., White, N.J., Hunter, J.R. and McInnes, K.L. (2012), Sea level. In A Marine Climate Change Impacts and Adaptation Report Card for Australia 2012 (Eds. E.S. Poloczanska, A.J. Hobday and A.J. Richardson). Available at: http://www.oceanclimatechange.org.au ISBN: 978-0-643-10928-5.

Fowler, H.J. and Ekstrom, M. (2009), Multi-model ensemble estimates of climate change impacts on UK seasonal precipitation extremes, International Journal of Climatology, 29(3), 385-416.

Groisman, P.Y, Karl T.R., Easterling D.R., Knight R.W., Jamason P.F., Hennessy K.J., Suppiah R., Page C.M., Wibig J., Fortuniak K., Razuvaev V.N., Douglas A., Forland E. and Zhai, P.M. (1999), Changes in the probability of heavy precipitation: Important indicators of climatic change,Climatic Change, 42: 243-283.

Herman, J. D., Zeff, H.B., Reed, P. M. and Characklis, G. W. (2014), Beyond optimality: Multistakeholder robustness tradeoffs for regional water portfolio planning under deep uncertainty, Water Resour Res, 50(10), 7692-7713, 10.1002/2014WR015338

Hunter, J. (2007), Estimating sea-level extremes in a world of uncertain sea-level rise, 5th Flood Management Conference, Warrnambool, Australia, [Accessed 12 October. 2007].

IPCC (Intergovernmental Panel on Climate Change) (2007), Climate Change 2007, Synthesis Report. Contribution of Working Groups I, II and III to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change [Core Writing Team, Pachauri, R.K and Reisinger, A. (eds.)]. IPCC, Geneva, Switzerland, 104 pp.

IPCC (Intergovernmental Panel on Climate Change) (2012), Workshop Report of the Intergovernmental Panel on Climate Change Workshop on Socio-Economic Scenarios [Edenhofer, O., R. Pichs-Madruga, Y. Sokona, V. Barros, C.B. Field, T. Zwickel, S. Schloemer, K. Ebi, M. Mastrandrea, K. Mach, C. von Stechow (eds.)]. IPCC Working Group III Technical Support Unit, Potsdam Institute for Climate Impact Research, Potsdam Germany, p: 51.

Jones, M.R., (2012), Characterising and modelling time-varying rainfall extremes and their climatic drivers. PhD Thesis, Newcastle University.

Langbein, W.B. (1949), Annual floods and the partial-duration flood series, Transactions, American Geophysical Union, 30(6), 879-881.

Le Coz, J., Renard, B., Bonnifait, L., Branger, F. and Le Boursicaud, R. (2013). Uncertainty Analysis of Stage-Discharge Relations using the BaRatin Bayesian Framework. 35th IAHR World Congress, 08/09/2013 - 13/09/2013, Chengdu, China.

Nicholls, N. and Alexander, L. (2007), Has the climate become more variable or extreme? Progress 1992?2006, Progress in Physical Geography, 31: 77-87.

Oreskes, N., Shrader-Frechette, K. and Belitz, K. (1994), Verification, Validation, and Confirmation of Numerical Models in the Earth Sciences, Science New Series, 263: 641-646.

Pappenberger, F. and Beven, K. J. (2006), Ignorance is bliss: Or seven reasons not to use uncertainty analysis, Water Resour Res, 42(5), 8, W05302 10.1029/2005wr004820

Pilgrim, DH (ed) (1987) Australian Rainfall and Runoff - A Guide to Flood Estimation, Institution of Engineers, Australia, Barton, ACT, 1987.

Renard, B., Kavetski, D., Kuczera, G., Thyer, M. and Franks, S.W. (2010), Understanding predictive uncertainty in hydrologic modeling: The challenge of identifying input and structural errors, Water Resour. Res., 46(5), W05521, 10.1029/2009wr008328.

Trenberth KE (2011), Changes in precipitation with climate change. Clim Res 47: 123-138.

Uusitalo, L., Lehikoinen, A., Helle, I. and Myrberg, K. (2015), An overview of methods to evaluate uncertainty of deterministic models in decision support, Environmental Modelling & Software, 63: 24-31, http://dx.doi.org/10.1016/j.envsoft.2014.09.017.

Westra, S., L.V. Alexander, F.W. Zwiers (2013), Global increasing trends in annual maximum daily precipitation. Journal of Climate, in press, doi:10.1175/JCLI-D-12-00502.1.

White, C.J., Grose, M.R, Corney, S.P., Bennett, J.C., Holz, G.K., Sanabria, L.A., McInnes, K.L., Cechet, R.P., Gaynor, S.M. and Bindoff, N.L. (2010), Climate Futures for Tasmania: extreme events technical report, Antarctic Climate and Ecosystems Cooperative Research Centre, Hobart, Tasmania.

Wood, E.F. and Rodriguez-Iturbe, I. (1975), Bayesian inference and decision making for extreme hydrologic events, Water Resour Res, 11(4), 533-542, 10.1029/WR011i004p00533.

Chapter 3. Approaches to Flood Estimation

Rory Nathan, James Ball

Chapter Status	Final
Date last updated	14/5/2019

3.1. Introduction

Design flood estimation is a focus for many engineering hydrologists. In many situations, advice is required on flood magnitudes for the design of culverts and bridges for roads and railways, the design of urban drainage systems, the design of flood mitigation levees and other flood mitigation structures, design of dam spillways, and many other situations. The flood characteristic of most importance depends on the nature of the problem under consideration, but it is often necessary to estimate peak flow, peak level, flood volume, and flood rise. The analysis might be focused on a single location (such as a bridge waterway or levee protecting a township) or it may be necessary to consider the performance of the whole catchment as a system, as required in urban drainage design.

Design objectives are most commonly specified using risk-based criteria, and thus the focus of this guidance is on the use of methods that provide estimates of flood characteristics for a specified probability of exceedance (referred to as flood quantiles, see Book 1, Chapter 2, Section 2).

The general nature of the estimation problem is illustrated in Figure 1.3.1. This figure shows the annual maxima floods (blue circular symbols) from 75 years of available gauged records. These flood maxima have been ranked from largest to smallest and are plotted against an estimate of their sample exceedance probability (as described in Book 3). Such information can be used directly to identify the underlying probability model of flood behaviour at the site at which the data was collected. The flood peaks are usually considered to be independent random variables, and it is often assumed that each flood is a random realisation of a single probability model. The gauged flood peaks shown in Figure 1.3.1 do appear to be from a homogeneous sample (ie. a single probability model), but in many practical problems the relationship between rainfall and flood may change over time, and it may be necessary to either censor the data or identify appropriate exogenous factors to condition the fit of the adopted probability model.

The best estimate of the relationship between flood magnitude and Annual Exceedance Probability (AEP) (Book 1, Chapter 2, Section 2) obtained by fitting a probability model is shown by the solid red curve in Figure 1.3.1. The gauged data represent a finite sample of a given size, and thus any estimate of flood risk using a fitted probability model is subject to uncertainty, as illustrated by the increasingly divergent dashed red curves in Figure 1.3.1 (referred to as confidence limits). The computation of such confidence limits usually only reflects the limits of the available sample, or perhaps the increasing uncertainty involved in the extrapolation of the relationship between recorded stage and estimated flood peak. The computed confidence limits are also conditioned on the assumed underlying probability model. However, it needs to be recognised that these factors only represent the uncertainties most easily characterised; other factors, such as the influence of a non-stationary climate, changing land-use during the period of record, and the changing nature of flood response with event magnitude, confound attempts to identify the most appropriate probability model. Accordingly, the true uncertainty around such estimates will be larger than that based solely on consideration of the size of the available sample. Of course, data are rarely available at the location of design interest, and additional uncertainty is involved in the scaling and/or transposition of flood risk estimates to the required site.

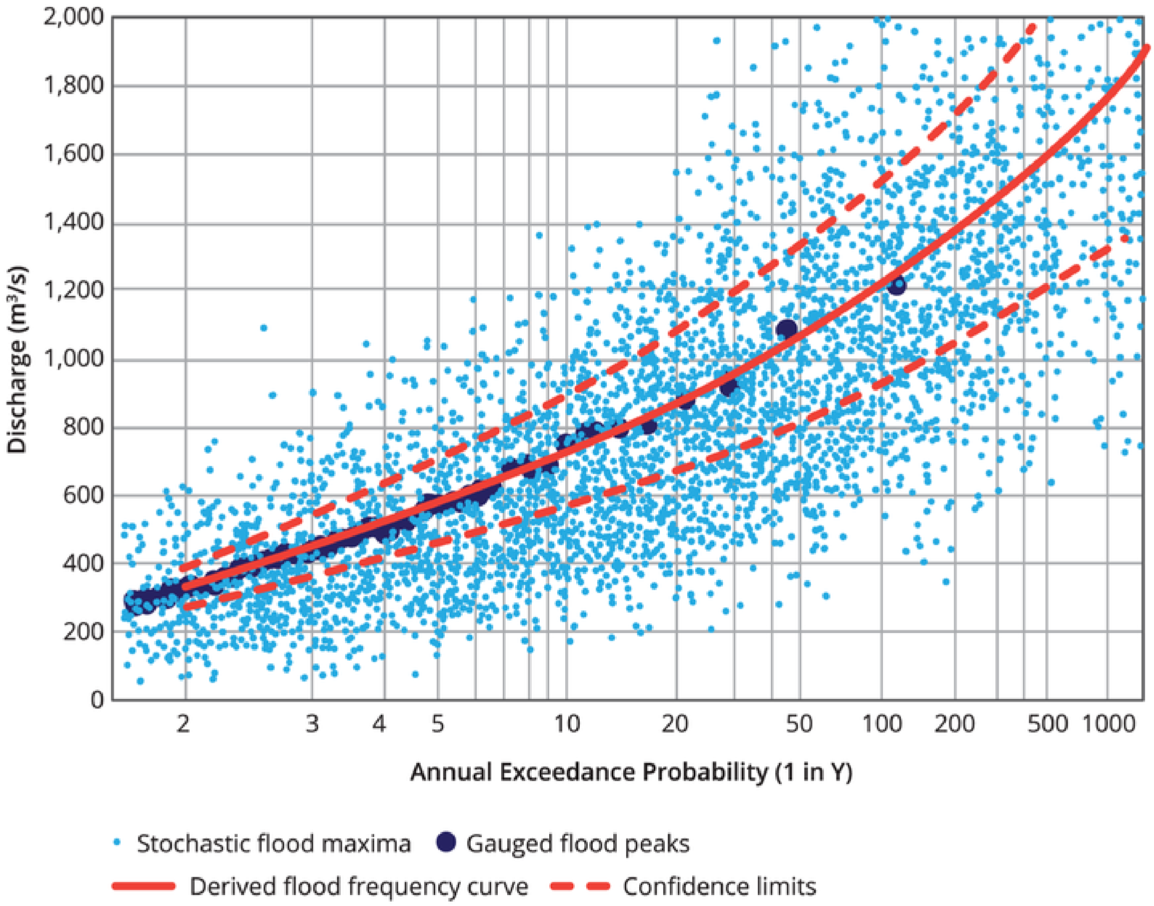


Figure 1.3.1. Illustration of Stochastic Influence of Hydrologic Factors on Flood Peaks and the Uncertainty in Flood Risk Estimates Associated with Observed Flood Data

One of the great advantages of fitting a probability flood model to observed data is that the approach avoids the problem of considering the complex joint probabilities involved in flood generation processes. Floods are the result of the interaction between many random variables associated with natural and anthropogenic factors; natural factors include interactions between the characteristics of the rainfall event, antecedent conditions, and other stochastic factors such as tide levels and debris flows; anthropogenic factors might include the influence of dam and weir operations, urbanisation, retarding basins, flood mitigation works, and land-management practices.

Figure 1.3.1 also illustrates the influence of natural variability on flood generation processes, and is based on the stochastic simulation of flood processes using 10 000 years of rainfall data under the assumption of a stationary climate. The stochastic flood maxima were obtained by varying key factors that influence the production of flood runoff, namely rainfall depth, initial and continuing losses, and the spatial and temporal patterns of catchment rainfalls. The flood peaks in Figure 1.3.1 are plotted against the AEP of the causative rainfall, and the scatter of the stochastic maxima illustrates the natural variability inherent in the production of flood runoff. While these maxima have been derived from mathematical modelling of event rainfall bursts, an indication of this variability can be seen in the relationship between observed rainfalls and runoff in gauged catchments (though of course with real-world data we do not have 10 000 years of observations).

The scatter of stochastic flood maxima resulting from different combination of flood producing factors illustrates the inherent difficulty in removing bias from “simple design event” methods. Such methods use a flood model to transform probabilistic bursts of rainfall (the design rainfalls as presented in Book 2) to corresponding estimates of floods. For example it is seen from Figure 1.3.1 that the flood peaks resulting from 1% AEP rainfalls range in magnitude between around 500 m^3/s and 2000 m^3/s; it is also seen that the rainfall that might generate a flood with a 1000 m^3/s peak might vary between a 20% and 0.1% AEP. Traditional practice has been to adopt fixed values of losses and rainfall patterns for use with design rainfalls to derive a single flood that is assumed to have the same AEP as its causative rainfall (probability neutrality). If chosen carefully it is possible to select a set of values that yields an unbiased estimate of the design flood for a particular catchment, but without taking steps to explicitly cater for the joint probabilities involved, there is a considerable margin for error (Kuczera et al., 2006; Weinmann et al., 2002).

Accordingly, a key difference between this and earlier versions of ARR is the focus on how best to achieve “probability neutrality” between rainfall inputs and flood outputs when using rainfall-based techniques. A number of more computationally intensive procedures are introduced (such as ensemble event, Monte Carlo event, and continuous simulation approaches) to help ensure that the method used to transform rainfalls into design floods is undertaken in a fashion that minimises bias in the resulting exceedance probabilities. An overview of these concepts is provided in Book 1, Chapter 3, Section 3, and more detailed description of the procedures is provided in Book 4.

The methods discussed here are divided into two broad classes of procedures based on:

i. the direct analysis of observed flood and related data (Book 1, Chapter 3, Section 2); and

ii. the use of simulation models to transform rainfall into flood maxima (Book 1, Chapter 3, Section 3).

All methods involve the use of some kind of statistical model (or transfer function) to extrapolate information in space or time. Each method also has its strengths and limitations and they vary in their suitability to different types of data and design contexts, and this is discussed in Book 1, Chapter 3, Section 4.

3.2. Flood Data Based Procedures

3.2.1. Overview

An overview of the procedures commonly used to analyse flood data directly is provided in Table 1.3.1. Flood frequency techniques (Book 1, Chapter 3, Section 2) are used to estimate the probability of flood exceedances directly from observed flood maxima, and are often used to extrapolate to probabilities beyond that inferred by the length of available record. Flood Frequency Analyses are most commonly applied using only the data at the site of interest using Peaks-over-Threshold and Annual Maxima Series (“at-site analyses”), but the resulting estimates of flood risk can be significantly improved by the consideration of flood behaviour at multiple sites that are judged to have similar flood frequency distributions (“at-site/regional analyses”). This concept of pooling information from multiple sites is often referred to as “trading space for time” for, with appropriate care, the information on flood exceedances across a region can improve the fit of the probability model at a single site with a short period of record.

One drawback of frequency analyses is that it can only provide quantile estimates at sites where data is available. Accordingly, a range of procedures have been developed to estimate flood risk at sites with little or no data (Book 1, Chapter 3, Section 2). These procedures generally involve the use of regression models to estimate the parameters of probability models (or the flood quantiles) using physical and meteorological characteristics, although simpler scaling functions can sometimes be used for local analyses.

Table 1.3.1. Summary of Common Procedures used to Directly Analyse Flood Data

	Frequency Analysis of Frequent Floods	Frequency Analysis of Rare Floods	At-Site/Regional Flood Frequency Analysis	Regional Flood Frequency Estimation
Inputs	Peak-over-Threshold series	Annual Maxima Series at single site of interest	Gauged flood maxima at multiple sites with similar flood behaviour	Catchment characteristics and flood quantiles (or parameters) derived from frequency analyses
Analysis	Selected probability model is fitted to flood maxima (e.g. exponential distribution fitted by L-moments)	Selected probability model is fitted to flood maxima (e.g. Log Pearson III/GEV distributions fitted by L-moments)	Information from multiple catchments is used to improve fit of probability model (e.g. regional L-moments or Bayesian inference)	Regression on model parameters or flood quantiles (e.g. RFFE method), or local scaling functions based on catchment characteristics
Outputs	Flood quantiles for AEPs > 10% at a gauged site	Flood quantiles for AEPs < 10% at a gauged site	Improved flood quantiles at multiple sites of interest	Flood quantiles at ungauged sites
ARR Guidance	Book 3, Chapter 2, Section 4 and Book 3, Chapter 2, Section 7	Book 3, Chapter 2, Section 4 and Book 3, Chapter 2, Section 6	Book 3, Chapter 2, Section 6 (Bayesian Calibration)	Book 3, Chapter 3

3.2.2. Flood Frequency Techniques

Flood Frequency Analysis involves the fitting of a probability model to recorded maxima to relate the magnitude of extreme events to their frequency of occurrence. The method can be applied directly to flood peaks (as described in Book 3) or rainfall (as used in Book 2), or indeed to any set of flood characteristics for which it is desired to determine the relationship between event magnitude and exceedance probability. The technique is generally not applicable to flood level maxima as the manner in which flood levels increase with flood magnitude is heavily dependent on channel geometry and thus is not suited to statistical extrapolation.

Flood Frequency Analyses can be broadly divided into three types of applications (Table 1.3.1), namely:

- At-site - the parameters of the probability distributions are fitted to annual maxima series to derive estimates of flood risk rarer than 10% AEP (or to peaks above a given threshold for more common floods) solely using information at the site of interest.

- At-site/regional - the information used to fit the model parameters is obtained from the site of interest as well as from other sites considered to exhibit similar flood behaviour.

- Regional - the information used to fit the model parameters is obtained from a group of sites considered to exhibit similar flood behaviour, where, as described in the following section, regression-based procedures may be used to estimate the model parameters (or probability quantiles) at the ungauged sites of interest.

Flood frequency methods are particularly attractive as they avoid the need to consider the complex processes and joint probabilities involved in the transformation of rainfall into flood. However, the utility of these methods is heavily dependent on both the length of available record and its representativeness to the catchment and climatic conditions of interest, as they are based on the assumption of stationary data series. Details on what distributions should be used, and how to select the sample of maxima and fit the distribution, are provided in Book 3.

There is advantage in undertaking frequency analyses at multiple sites in a local region of interest as this provides information on how local flood behaviour changes with catchment area, and other factors such as rainfall intensity can also be considered for more detailed analyses. Simple quantile regression models (ie. the development of a regression relationship between, say, catchment area and 10% AEP flood peak) are readily derived and are well suited to transposing flood risk estimates to locations upstream or downstream of a gauging site. Such simple scaling functions can also be applied to estimates derived using rainfall-based procedures.

3.2.3. Regional Flood Methods

Regional flood methods generally involve the application of a regression technique in which flood characteristics are related to catchment and relevant meteorological characteristics; the regression equation can be fitted to the flood quantiles directly ("quantile regression technique"), or else they can be fitted to the parameters of a probability model ("parameter regression technique").

Book 3 provides details of the application of the latter approach to data sets for different Australian regions in which the three parameters of the probability model are estimated from catchment characteristics using a Bayesian regression approach (Rahman et al., 2014). The developed procedure provides a quick means to estimate the magnitude of peak flows between the 50% to 1% AEPs, with the additional attraction that uncertainty bounds are provided. The regression equations presented in Book 3 were developed using parameters obtained from at-site/regional flood frequency analyses, and thus represent a rigorous example of Regional Flood Frequency Estimation based on parameter regression.

In some situations it might be useful to obtain an additional independent estimate based on local data, and if so then prediction equations can be developed by regressing catchment characteristics against flood quantiles obtained from at-site/regional flood frequency analyses. The most common example of this is to develop a relationship between flood quantiles and catchment area for nested sites located in the same catchment (typically this is undertaken using log-transformed data). The utility of such an approach when compared to the procedure presented in Book 3 depends on the relevance of the data to the problem at hand, and on the extent to which the assumptions of the fitted model have been satisfied.

3.3. Rainfall-Based Procedures

3.3.1. General

Rainfall-based models are commonly used to extrapolate flood behaviour at a particular location using information from a short period of observed data; this can be done using either event-based or continuous simulation approaches, as described in Book 1, Chapter 3, Section 3 and Book 1, Chapter 3, Section 3 below. The parameters of such models can also be transposed to a different location (or modified to represent different catchment conditions) and used to estimate flood characteristics for which no gauging information is available.

Table 1.3.2 summarises the different characteristics of the event-based and continuous simulation approaches. The three broad approaches to event-based simulation all use the same hydrologic model to convert design rainfall inputs into hydrograph outputs, the main difference is in the level of sophistication used to minimise bias in the probability neutrality of the transformation. Continuous simulation approaches utilise model structures which generally differ markedly from those used in event-based models.

Event-based approaches are based on the transformation of rainfall depths of given duration and AEP (“design rainfalls”) into flood hydrographs by routing rainfall excess (obtained by applying a loss model to rainfall depths) through the catchment storage. Such models can include the allowance of additional pre- and post-burst rainfalls to represent complete storm events, and can separately consider baseflow contribution from prior rainfall events to represent total hydrographs. The defining feature of such models is that they are focused on the simulation of an individual flood event and that antecedent conditions need to be specified in some explicit fashion. Simple Design Event methods are applied in a deterministic fashion, where key inputs are fixed at values that minimise the bias in the transformation of rainfall into runoff. Alternatively, stochastic techniques can be used to explicitly resolve the joint probabilities of key hydrologic interactions; ensemble techniques provide simple (and approximate) means of minimising the bias associated with a single hydrologic variable, whereas Monte Carlo techniques represent a more rigorous solution that can be expanded to consider interactions from a range of natural and anthropogenic factors. It should be noted that the guidance provided in ARR only focuses on the use of stochastic techniques to cater for (random) variability of key inputs, and its use to characterise epistemic uncertainty is assumed to be the domain of specialist statistical hydrologists.

Continuous simulation approaches remove the need to specify antecedent conditions as these are implicitly considered in the successive updating of state variables via the simulation of continuous rainfall (and other) input time series. The continuous simulation of key state variables also has the potential to simplify the consideration of the complex joint probabilities involved in flood generation processes. The conceptual basis of continuous simulation is the simulation of data that would have been recorded at a location if a gauge were present at that location. Hence estimation of design flood characteristics from data generated through application of a continuous simulation modelling system requires the undertaking of subsequent statistical analysis, as outlined in Book 1, Chapter 3, Section 2. The advantages of continuous simulation may be offset by the need to consider additional complexity which are avoided by event-based approaches, though the relative merits of each approach is dependent upon the available data and the nature of the design problem being considered.

Table 1.3.2. Summary of Recommended Rainfall-Based Procedures

<table>
<tr><th></th><th>Simple Design Event</th><th>Ensemble Event</th><th>Monte Carlo Event</th><th>Continuous Simulation</th></tr>
<tr><td>Hydrologic Inputs</td><td colspan="3">Design rainfalls (ie. rainfall depth for given burst duration and Annual Exceedance Probability)</td><td>Observed (or synthetic) time series of rainfall and evaporation.</td></tr>
<tr><td>Hydrologic variability</td><td>Fixed patterns of rainfall and other inputs</td><td>Ensemble of N temporal patterns</td><td>Ensemble (or distribution) of temporal patterns, losses, and other factors.</td><td>As represented in the time series of inputs – if not in time series then not represented</td></tr>
<tr><td>Model</td><td colspan="3">Event-based model based on routing rainfall excess through catchment storage (see Book 5 for details of technique)</td><td>Model of catchment processes influencing runoff generation</td></tr>
<tr><td>Framework</td><td>Single simulation for each combination of rainfall depth and AEP</td><td>N simulations for each combination of rainfall depth and AEP (N >10)</td><td>Stochastic sampling of input distributions using continuous or stratified domain (potentially thousands of simulations)</td><td>Continuous simulation at time step for N years</td></tr>
<tr><td>Flood AEP</td><td colspan="2">Assumed same as input rainfall</td><td rowspan="2">Statistical analysis of joint probabilities (e.g. frequency analysis of maxima or Total Probability Theorem)</td><td rowspan="2">Computed from frequency analysis of N annual maxima</td></tr>
<tr><td>Flood magnitude</td><td>Single estimate derived from each set of inputs</td><td>Simple average (or median) of N simulations</td></tr>
<tr><td>ARR guidance</td><td>Book 4</td><td>Book 4</td><td>Book 4</td><td>Book 4</td></tr>
</table>

3.3.2. Event-Based Simulation

The simple design event method represents common industry practice in Australia and overseas, and traditionally includes the use of the Rational Method, Unit Hydrograph, SCS, Gradex and runoff-routing procedures (Haan and Schulze, 1987; Cordery and Pilgrim, 2000; McKerchar and Macky, 2001; Smithers, 2012). With this approach, a rainfall event with pre-selected AEP and duration is transformed into a flood hydrograph by a simple hydrologic model (or transfer function). The approach is termed "deterministic" in the sense that the single resulting flood output is uniquely derived from a set of inputs that are explicitly selected. The transformation often involves the application of two modelling steps, namely:

i. a *runoff production model* - to convert the storm rainfall input at any point in the catchment into rainfall excess (or runoff) at that location, and;

ii. a *hydrograph formation model* - to simulate the conversion of rainfall excess into a flood hydrograph at the point of interest.

The AEP of the derived flood is assumed to be the same as the input rainfall. This assumption is made on the basis that the hydrologic factors that control runoff production are set to be probability neutral. In practice this means that factors related to the temporal and spatial distribution of rainfall, antecedent conditions and losses, are set to “typical” values (from the central tendency of their distributions) that are associated with the input rainfall. Factors related to formation of the hydrograph are generally assumed to be invariant with rainfall. Design events for different rainfall durations are simulated, and the one producing the highest peak flow (corresponding to the critical rainfall duration) is adopted as producing the design flood for the selected AEP (flood quantile).

The ensemble event method represents a modest increase in computational requirements. Rather than adopting typical fixed values of inputs in the hope of achieving probability neutrality, modelled inputs are selected from an ensemble of inputs and the simulation results are based on the central tendency of the outputs (ie. the average or the median, as judged appropriate for the degree of non-linearity involved). If the members of the ensemble do not occur with equal likelihood (as would usually be the case with temporal patterns) then it will be necessary to weight the results by the relative likelihood of the selected inputs occurring. A representative hydrograph from the ensemble can be scaled to match the derived peak for design purposes. This approach represents a simple means of accounting for the hydrologic variability of a single dominant factor (ie. temporal patterns), and testing has demonstrated (Sih et al., 2008; Ling et al., 2015; WMAwater, 2015) that this approach provides results for many practical purposes that are similar to that obtained from more rigorous methods.

The basis of the Monte Carlo event method is a recognition that flood maxima can result from a variety of combinations of flood producing factors, rather than from a single combination as is assumed with the design event approach. For example, the same peak flood could result from a large, front-loaded storm on a dry basin, or a moderate, more uniformly distributed storm on a saturated basin. Such approaches attempt to mimic the joint variability of the hydrologic factors of most importance, thereby providing a more realistic representation of the flood generation processes. The method is easily adapted to focus on only those aspects that are most relevant to the problem. To this end, it is possible to adopt single fixed values for factors that have only a small influence on runoff production, and full distributions (or data ensembles) for other more important inputs, such as losses, and temporal patterns, or any influential factor (such as initial reservoir level) that may impact on the outcome. The approach involves undertaking numerous simulations where the stochastic factors are sampled in accordance with the variation observed in nature and any dependencies between the different factors. In the most general Monte Carlo simulation approach for design flood estimation, rainfall events of different durations are sampled stochastically from their distribution (Weinmann et al., 2002). Alternatively, the simulations can be undertaken for specific storm durations (applying the critical rainfall duration concept) and the exceedance probability of the desired flood characteristic may be computed using the Total Probability Theorem (Nathan et al., 2002). The latter approach is simpler and more aligned to available design information, and is more easily implemented by those familiar with the traditional design event approach.

The simple design event approach gives a single set of design hydrographs that can be used for subsequent modelling steps, such as input to a hydraulic model to determine flood levels for a given exceedance probability. With the Ensemble and Monte Carlo event methods an ensemble of hydrographs is produced and it is often not practical to consider all these hydrographs in subsequent simulation steps. With both the ensemble and Monte Carlo approaches a representative hydrograph can be simply scaled to match the probability neutral estimate of the peak flood; the representative hydrograph needs to capture the typical volume and timing characteristics for the selected duration and severity of the event, though some of the advantages of ensemble and Monte Carlo event methods are lost if an ensemble of events is not used through all the key modelling steps.

3.3.3. Continuous Simulation

With continuous simulation approaches, a conceptual model of the catchment is used to convert input time series of rainfall and evaporation into an output time series of streamflow; the flood events of interest are then extracted from the simulated streamflow record and analysed by conventional frequency analysis. The models used to transform the input rainfall into streamflow tend to be rather more complex than those commonly used in the design event or stochastic approaches. The main reason for this complexity is the ability of the models to account for changes in state variables (e.g. soil moisture and other catchment stores) during the simulation period. While these models have been used for the past 40 years for the prediction of continuous flow sequences, their dominant purpose has been for estimation of flow sequences for either yield analysis or for environmental considerations (Chiew, 2010). However, their use has been extended to the estimation of design floods (Cameron et al., 2000; Boughton and Droop, 2003; Blazkova and Beven, 2004; Blazkova and Beven, 2009).

"Hybrid" approaches have the potential to capitalise on the advantage of both event-based and continuous simulation approaches. Typically, hybrid approaches use statistical information on rainfall events in combination with continuous simulation and event-based models. With these approaches, long term recorded (or stochastic) climate sequences can be used in combination with a continuous simulation model to generate a time series of catchment soil moisture and streamflows. This information is used to specify antecedent conditions for an event-based model, which is then used in combination with statistical information on rainfall events to generate extreme flood hydrographs. For example, SEFM (MGS Engineering Consultants, 2009) and SCHADEX (Paquet et al., 2013) are examples of the hybrid approach. In both these models a continuous hydrological simulation model is used to generate the possible hydrological states of the catchment, and floods are simulated on an event basis. While there are a number of conceptual advantages to these methods, significant development would be required for their implementation for routine design purposes.

3.4. Selection of Approach

3.4.1. Overview

The methods described above have their differing strengths and weaknesses, and this means that each method is suited to a particular range of data availability and design contexts. While the broad differences in the applicability of the different methods are discussed below, it should be recognised that there is considerable overlap in their ranges of applicability and it is strongly advisable to apply more than one method to any given design situation. The comparison of different methods yields insights about errors or assumptions that might otherwise be missed, and the process of reconciling the different assessments provides valuable information that aids adoption of a final "best estimate".

In developing guidance on the selection of an approach it is first worth briefly summarising the strengths and weakness of the different methods. This is done separately for flood data based procedures and rainfall-based procedures, and this is then followed by general guidance for selection of an approach.

3.4.2. Advantages and Limitations of Flood Data Based Procedures

The prime advantage of Flood Frequency Analyses is that they provide a direct estimate of flood exceedance probabilities based on gauged data. Peak flood records represent the integrated response of a catchment to storm events and thus are not subject to the potential for bias that can affect rainfall-based procedures. Furthermore, Flood Frequency Analyses are quick to apply compared to rainfall-based procedures and have the ability to provide estimates of uncertainty, most easily those associated with the size of sample and gauging errors. These represent very considerable advantages, and thus it is not surprising that flood frequency analysis is an important tool for the practicing flood hydrologist.

However, there are some practical disadvantages with the technique. The available peak flood records may not be representative of the conditions relevant to the problem of interest: changing land-use, urbanisation, upstream regulation, and non-stationary climate are all factors that may confound efforts to characterise flood risk. The length of available record may also limit the utility of the flood estimates for the rarer quantiles of interest. Also, peak flow records are obtained from the conversion of stage data and there may be considerable uncertainty about the reliability of the rating curve when extrapolated to the largest recorded events. There is also uncertainty associated with the choice of probability model which is not reflected in the width of derived confidence limits: the true probability distribution is unknown and it may be that different models may fit the observed data equally well, yet diverge markedly when used to estimate flood quantiles beyond the period of record.

Perhaps the most obvious limitation of Flood Frequency Analysis is that it relies upon the availability of recorded flood data. This is a particular limitation in urban drainage design as there are so few gauged records of any utility in developed catchments. But the availability of representative records is also often a limitation in rural catchments, either because of changed upstream conditions or because the site of interest may be remote from the closest gauging station.

For this reason, considerable effort has been expended on the development of a regional flood model that can be used to estimate flood quantiles in ungauged catchments (Book 3, Chapter 3). The prime advantage of this technique is that it provides estimates of flood risk (with uncertainty) using readily available information at ungauged sites; the estimates can also be combined with at-site analyses to help improve the accuracy of the estimated flood exceedance probabilities. The prime disadvantage of the technique is that the estimates are only applicable to the range of catchment characteristics used in development of the model, and this largely excludes urbanised catchments and those influenced by upstream impoundments (or other source of major modification).

The main advantages and limitations of flood data based procedures are summarised in Table 1.3.3. In addition to the points made above, specific mention is made of the applicability of Peak-over-Threshold analysis to events more frequent than 10% AEP, and the use of Annual Maxima Series for the estimation of rarer events. Also included in this table is reference to the use of large scale empirical techniques. While these techniques have the advantage of providing an indication of the upper limiting bounds on the magnitude of floods using national and global data sets (Nathan et al., 1994; Herschy, 2003), it is difficult to assign exceedance probabilities to such events and thus such procedures are better seen as a complement, and not an alternative, to traditional regional flood frequency techniques (Castellarin, 2007).

Table 1.3.3. Summary of Advantages and Limitations of Common Procedures used to Directly Analyse Flood Data

Method	Advantages	Limitations	Comments on Applicability
Peak-over-Threshold analysis	• Exceedance threshold can be selected to suit frequency range of most interest	• Sensitive to adopted independence criteria • Fewer generic software packages available to aid analysis	• Particularly suited to exceedance probabilities more frequent than 10% AEP • Requires development of transposition/ scaling functions for application to ungauged sites
At-site Flood Frequency Analysis based on Annual Maxima Series	• Well established procedures that are strongly supported by literature • Software readily available that includes assessment of uncertainty • Estimates obtained for modest investment of effort	• Rare estimates sensitive to length of available record, a small number of rare events, and assumptions of stationarity • Extrapolation best undertaken with knowledge of changing channel geometry and rating curve errors	• Requires development of transposition/ scaling functions for application to ungauged sites
At-site/ regional frequency analysis based on Annual Maxima Series	• Well established procedures that are strongly supported by literature • Provides more robust estimates of rare events, especially for sites with limited length of record	• Dependent on degree of homogeneity of gauged sites used in the analysis • Requires more specialist expertise than at-site analysis	• Functions for transposition to ungauged sites readily derived from regional information used to undertake the analysis
Regional flood model	• Based on rigorous statistical procedure that takes advantage of large processed data sets • Estimates include uncertainty and are derived with small investment of effort	• Largely restricted to catchments smaller than 1000 km² • Flood response needs to be within range of characteristics used in development of the method • larger degree of uncertainty (wider confidence limits)	• Ease of application allows this to be used as independent estimate for all other methods

Method	Advantages	Limitations	Comments on Applicability
		than flood estimates from at-site analysis • Representativeness of the gauges used	
Large scale empirical	• Estimates readily obtained once relevant data sets have been sourced • Generally a useful indicator of the upper bound of flood behaviour	• Enveloped characteristics may not be relevant to site of interest • Not suited to inferring probabilities of exceedance	• Useful as a sanity check on results obtained from other procedures • Regional nature of information allows for application to ungauged sites

3.4.3. Advantages and Limitations of Rainfall-Based Procedures

A key advantage of rainfall-based approaches is that they provide the means to derive flood hydrographs. The derivation of a full hydrograph rather than a single attribute (such as flood peak) allows the design loading condition to be assessed in terms of both peak and volume, which is of prime importance when considering the mitigating influence of flood storage.

Of arguably greater importance is the ability of rainfall-based approaches to take advantage of the extensive availability of rainfall data. This is a very important advantage as rainfall characteristics vary across space in a more predictable and generally more uniform fashion than floods. This feature, along with the greater length and density of rainfall gauging, allows the derivation of probabilistic estimates of rainfalls that are much rarer and more easily transposed than flood characteristics.

However, these significant advantages are offset by the need to transform rainfalls into floods using some kind of design event transfer function or simulation model. Common examples of the former include the Rational Method and Curve Number method of the US Soil Conservation Service; while such methods provide an attractive means of simplifying the complexity involved in generation of flood peaks, their use in this edition has been replaced by the more defensible implementation of the Regional Flood Model (Book 3, Chapter 3). The focus of this guidance is thus on the use of event-based and continuous simulation approaches. While these models provide a conceptually more attractive means to derive flood hydrographs arising from storm rainfall events, they present the very real potential for introducing probability (AEP) bias in the transformation. That is, the methods are well suited to the simulation of flood hydrographs, but great care is required when assigning exceedance probabilities to the resulting flood characteristic.

The advantages and limitations of some common approaches to rainfall-based procedures are summarised in Table 1.3.4. The first row of this table summarises the attributes of continuous simulation approaches, and the remaining rows refer to event-based approaches.

The continuous simulation approach has the major advantage that it implicitly allows for the correlations between the flood producing factors over different time scales. This can be a great advantage in some systems (such as a cascade of storages or complex urban environments) where the volume of flood runoff is the key determinant of flood risk. However, its major drawback for flood estimation is that considerable modelling effort is required to reproduce the flood characteristics of interest; the structure of continuous simulation models is geared towards reproduction of the complete streamflow regime, and not on the reproduction of annual maxima. This has implications for model structure, as well as for how the model is parameterised and calibrated to suit the different flood conditions of interest. With continuous simulation, the vast majority of the information used to inform model parameterisation is not relevant to flood events other than to ensure that the right antecedent conditions prevail before onset of the storm. Under extreme conditions, many state variables inherent to the model structure might be bounded, and the process descriptions relevant to such states may be poorly formulated and yield outcomes that are not consistent with physical reasoning; while this is the case for flood event models, the more complex structure generally used with continuous simulation models may confound attempts to detect the occurrence of such behaviour. In addition, if the length of historic (sub-daily) rainfalls is not long enough to allow estimation of the exceedance probabilities of interest, it will be necessary to use stochastic rainfall generation techniques (or some down-scaling technique) to produce synthetic sequences of sufficient length. Lastly, given the interdependence between model parameters and the difficulty of parameter identification, it can be difficult to transpose such models to ungauged catchments.

The deterministic application of “design-event” models based on linear and non-linear routing has a long history of application in Australia. However, considerable care needs to be taken when selecting “typical” values of the key inputs to avoid the introduction of probability bias in the transformation of design rainfalls into floods. Ensemble event approaches have the potential to mitigate this bias, but these are only likely to be defensible for those problems influenced by a single dominant factor in addition to rainfall. Monte Carlo techniques can be used to derive expected probability quantiles of selected flood characteristics arising from the joint interaction of many factors, but the defensibility of these estimates rests upon the representativeness of the inputs and the correct treatment of correlations which may be present.

Table 1.3.4. Summary of Advantages and Limitations of Common Rainfall-Based Procedures

Method	Advantages	Limitations	Comments on Applicability
Continuous Simulation	• Well suited to assessing flood risk in complex systems that are sensitive to flood volume • Most applicable to range of very frequent to frequent events	• Difficult to parameterise model to correctly reproduce the frequency of flood exceedance in manner that adequately captures shape of observed hydrographs	• Useful for hindcasting streamflows for sites with short periods of record • Model parameters not easily transposed to ungauged locations
Simple Event	• Long tradition of use thus familiar to most practitioners	• Difficult to demonstrate that probability - neutrality is achieved	• Little justification to use this simplistic method with currently available computing resources, but suited to derivation of preliminary estimates

Method	Advantages	Limitations	Comments on Applicability
Ensemble Event	• Simple means of minimising probability bias for modest level of effort • Well suited to accommodating single source of hydrologic variability in simple catchments	• Not suited to considering multiple sources of hydrologic variability or other joint probability influences • Difficult to determine if probability bias remains in the estimates	• Provides easy transition for practitioners familiar with design event method • The required sets of ensemble temporal patterns are now available
Monte Carlo event	• Rigorous means of deriving expected probability estimates for range of factors considered • Readily extended to consider multiple sources of variability and additional joint probability factors (both anthropogenic and natural)	• Requires specialist skills to develop bespoke solutions and thus dependent on availability of software • For more complex applications care needs to be taken to ensure correlations between dependent factors are appropriately considered	• Non-dimensional loss distributions and temporal pattern ensembles are now available • The expected probability estimates account for hydrologic variability not parameter uncertainty as the necessary information on governing distributions is generally not available.

3.4.4. Relative Applicability of Different Approaches

The broad nature of applicability of the different methods is illustrated in Figure 1.3.2. Figure 1.3.2 is not intended to be prescriptive, but rather it is intended to illustrate the relative ability of the different methods to provide unbiased estimates of flood characteristics in the given AEP range. Figure 1.3.2 is best interpreted with reference to Table 1.3.3 which summarises the strengths and limitations of each method and provides some brief comments on their application.

Flood Frequency Analyses are most relevant to the estimation of peak flows for Very Frequent to Rare floods. Flood Frequency Analysis methods can also be applied to other flood characteristics (e.g. flood volume over given duration) but this involves additional assumptions.

Peak-Over-Threshold analysis (Book 3, Chapter 2, Section 7) is most relevant to the estimation of flood exceedances that occur several times a year, up to floods more frequent than around 10% AEP. For rarer events the use of an Annual Maximum Series is preferred (Book 3, Chapter 2, Section 6), and with good quality information at-site frequency analyses are suited to the estimation of Rare floods of 2% and 1% AEP. The use of regional flood data provides valuable information that can be used to help parameterise the shape of the flood distribution, and thus where feasible it is desirable to use at-site/regional flood frequency methods (Book 3, Chapter 2, Section 6). The use of regional information can support the estimation of flood risks beyond 1% AEP and can greatly increase the confidence of estimates obtained using information at a single site.

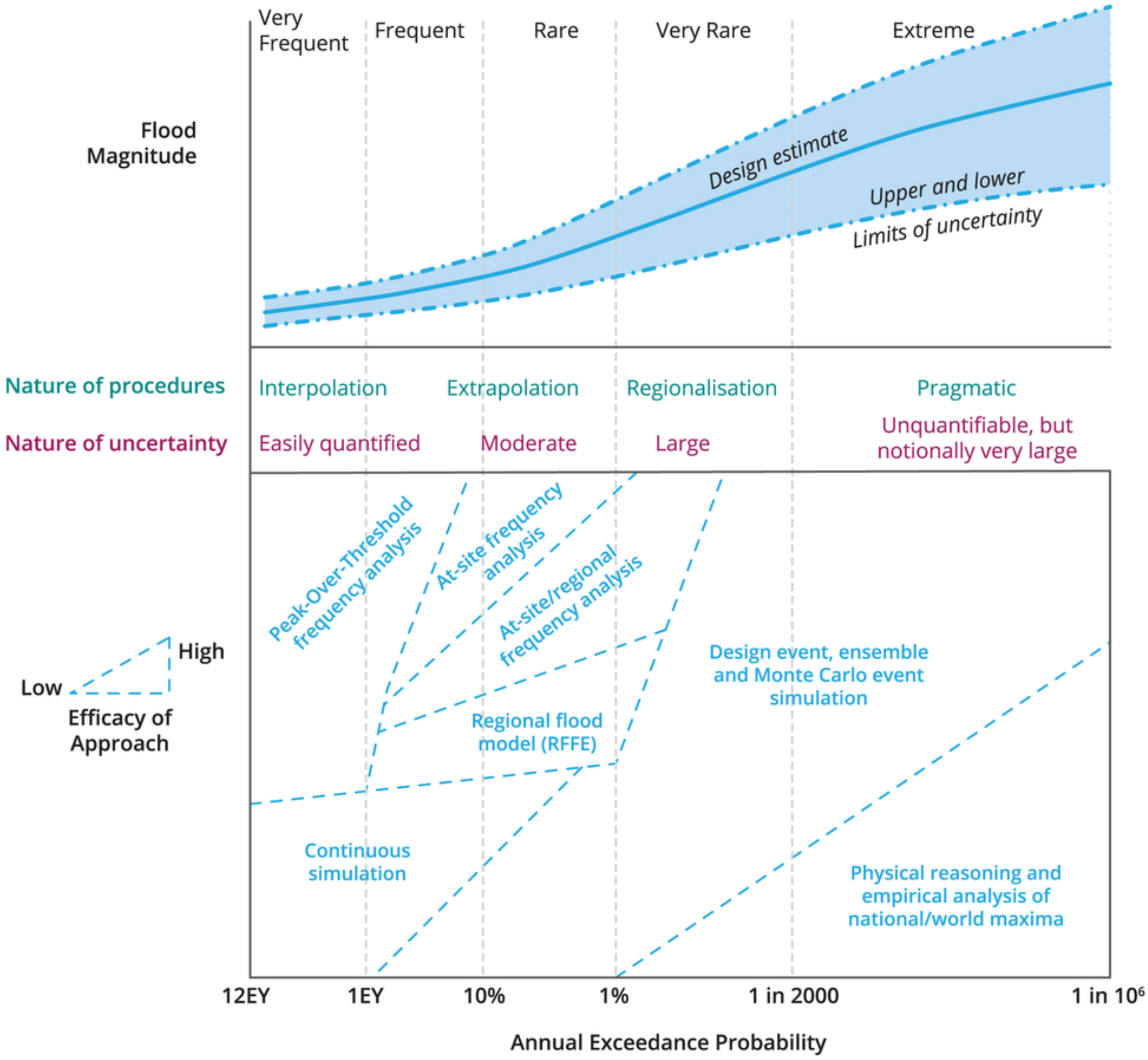


Figure 1.3.2. Illustration of Relative Efficacy of Different Approaches for the Estimation of Design Floods

The RFFE model (Book 3, Chapter 2, Section 6) (Rahman et al., 2014) provides estimates of peak flows for Frequent to Rare floods for sites where there is no streamflow data. While its primary purpose is for the estimation of flood quantiles, the resulting estimates can also be used to develop scaling functions to support the transposition of results obtained from rainfall-based procedures to ungauged sites. This is the same concept as the simple quantile regression approach discussed above, but as it is based on a more rigorous statistical procedure it is more suited to transposition of results where factors other than merely area are important. The RFFE method is quick to apply and provides a formal assessment of uncertainty, and thus is well suited to provide independent estimates for comparison with other approaches.

Figure 1.3.2 also illustrates the areas of design application most suited to rainfall-based procedures. These are applicable over a wider range of AEPs than techniques based directly on the analysis of flow data as it is easier to extrapolate rainfall behaviour across space and time than it is for flow data. But while these methods can capitalise on our ability to extrapolate rainfall data to rarer AEPs and infill spatial gaps in observations more readily than flows, their use introduces the need to model the transformation of rainfalls into floods.

Continuous simulation procedures are well suited to the analysis of complex systems which are dependent on the sequencing of flood volumes as the method implicitly accounts for the joint probabilities involved. Application of these methods require more specialist skill than event-based procedures; for example, it is important that the probabilistic behaviour of the input rainfall series relevant to the catchment (either historic or synthetic) is consistent with design rainfall information provided in Book 2, and that the model structure yields flood hydrographs that are consistent with available evidence. Transposition of model parameters to ungauged sites presents significant technical difficulties which would require specialist expertise to resolve. Given these challenges it is presently recommended that the main benefit of continuous simulation approaches is for the extension of flow records at gauged sites with short periods of record, where system performance is critically dependent on the sequencing of flow volumes; if flow data are not available, then it may be appropriate to consider their application to small scale urban environments where runoff processes can be inferred from an analysis of effective impervious areas. Its position in Figure 1.3.2 indicates the degree of accuracy of results that can be expected from this method relative to at-site frequency analysis.

By comparison with continuous simulation models, event-based models are far more parsimonious and more easily transposed to ungauged catchments; it is easier to fit the fewer model parameters involved to observed floods, and their structure has been tailored specifically to represent flood behaviour. However, while such models are easily calibrated and their parameterisation is generally commensurate with the nature of available data, their use generally involves the simulation of floods beyond the observed record. As such, it is necessary to make assumptions about the changing nature of non-linearity of flood response with flood magnitude and trust that the model structure and adopted process descriptions are applicable over the range of floods being simulated. These assumptions introduce major uncertainties into the flood estimates, and this uncertainty increases markedly with the degree of extrapolation involved. This issue is discussed in greater detail in Book 8.

The event-based methods considered in these guidelines generally involve a similar suite of storage-routing methods (Book 5). There are some conceptual differences in the way that these models are formulated, but in general these differences are minor compared to the constraints imposed by the available data. Australian practice has generally not favoured the use of unitgraph-based methods combined with node-link routing models (Feldman, 2000); in principle such models are equally defensible as storage-routing methods, and the strongest reason to prefer the latter is the desire for consistency when used to estimate Extreme floods that are well beyond the observed record, and also for the local experience with regionalisation of model parameters.

Perhaps the greatest choice to be made with event-based models is the adopted simulation environment (as discussed in Book 4). For systems that are sensitive to differences in temporal patterns there is little justification to use simple event methods: the additional computational burden imposed by ensemble event models is modest, and the resulting estimates are much more likely to satisfy the assumption of probability neutrality. However, this additional effort may not be warranted in those urban systems which are dominated by hydraulic controls, and in such cases the most appropriate modelling approach is likely to be a hydraulic modelling system with flow inputs provided in a deterministic manner. Monte Carlo event schemes provide a rigorous solution to the joint probabilities involved, and the solution scheme ensures expected probability quantiles that are probability neutral, at least for the given set of ensemble inputs and distributions used to characterise hydrologic variability in the key selected inputs. For those catchments or systems where flood outputs are strongly dependent on the joint likelihood of multiple factors, it is necessary to adopt a Monte Carlo event approach.

The greatest uncertainties in terms of both flood magnitude and exceedance probabilities are associated with the estimation of Extreme floods beyond 1 in 2000 AEP. There is very little data to support probabilistic estimates of floods in this range, and it is prudent to compare such estimates with empirical analysis of maxima based from national (Nathan et al., 1994) or even global (Herschy, 2003) data sets.

It should be noted that the procedures based directly on the analysis of flood data can readily provide an assessment of uncertainty. Additional uncertainty is introduced when transposing flood information to locations away from the gauging site used in the analysis, and the Regional Flood Frequency Estimation Method (RFFE) is the only method where this is provided in a form easily accessed by practitioners. The Monte Carlo event approach provides an appropriate framework to consider uncertainty in a formal fashion, though this will only provide indicative uncertainties: the greater the degree of extrapolation the greater the influence of uncertainty due to model structure and this is a factor that is not easily characterised. The uncertainty bounds shown in the top panel of Figure 1.3.2 are clearly notional and merely reflect the fact that uncertainty of the estimates increase markedly with event magnitude. It must be accepted that when the above procedures are applied to locations not included in their calibration that the associated uncertainties will be perhaps up to an order of magnitude greater.

Lastly, it needs to be recognised that the ranges of applicability of the different methods illustrated in Figure 1.3.2 are somewhat notional, and that there is considerable overlap in their ranges of applicability. It is thus strongly advisable to apply more than one method to any given design situation, where adoption of a final “best estimate” is ideally achieved by weighting estimates obtained from different methods by their uncertainty. Estimates of uncertainty for flood frequency analyses and regional flood estimates are provided in Book 3, and methods for use with rainfall-based techniques are provided in Book 4, with examples showing how uncertainty propagates through to the design outcome being provided in Book 7. In practice, the information required to assign relative uncertainties to different methods is either limited or difficult to obtain, and careful judgment will be required to derive a single best estimate with associated confidence intervals.

3.5. References

Blazkova, S. and Beven, K. (2004), Flood frequency estimation by continuous simulation of subcatchment rainfalls and discharges with the aim of improving dam safety assessment in a large basin in the Czech Republic. Journal of Hydrology, 292(1-4), 153-172. doi:10.1016/j.jhydrol.2003.12.025

Blazkova, S. and Beven, K. (2009), A limits of acceptability approach to model evaluation and uncertainty estimation in flood frequency estimation by continuous simulation: Skalka catchment, Czech Republic, Water Resour. Res., 45, W00B16, doi: 10.1029/2007WR006726.

Boughton, W. and Droop, O. (2003), Continuous simulation for design flood estimation - a review, Environmental Modelling and Software, 18: 309-318.

Cameron, D., Beven, K. and Naden, P. (2000), Flood frequency estimation by continuous simulation under climate change (with uncertainty), Hydrology and Earth System Sciences, 4(3), 393-405.

Castellarin, A. (2007), Probabilistic envelope curves for design flood estimation at ungauged sites. Water Resources Research, 43(4), pp.1-12. doi:10.1029/2005WR004384

Chiew, F.H.S. (2010), Lumped Conceptual Rainfall-Runoff Models and Simple Water Balance Methods: Overview and Applications in Ungauged and Data Limited Regions. Geography Compass, 4(3), 206-225. doi:10.1111/j.1749-8198.2009.00318.x.

Cordery, I. and Pilgrim, D.H. (2000), The State of the Art of Flood Prediction. In: Parker DJ (ed.) Floods. Volume II. Routledge, London. pp: 185-197.

Feldman, A.D. (2000), Hydrologic Modeling System HEC-HMS Technical Reference Manual. CDP-74B, U.S. Army Corps of Engineers.

Haan, C. and Schulze, R. (1987), Return Period Flow Prediction with Uncertain Parameters. American Society of Agricultural Engineers, 30(3), 665-669. doi:10.1109/IWCFTA.2011.52

Herschy, R. (2003), World catalogue of maximum observed floods. IAHS Publ No. 284, 285 pp.

Kuczera, G., Lambert, M.F., Heneker, T. Jennings, S., Frost, A. and Coombes, P. (2006) Joint probability and design storms at the crossroads, Australian Journal of Water Resources, 10(2), 5-21.

Ling, F., Pokhrel, P., Cohen, W., Peterson, J., Blundy, S. and Robinson, K. (2015), Australian Rainfall and Runoff Project 12 - Selection of Approach and Project 8 - Use of Continuous Simulation for Design Flow Determination, Stage 3 Report.

MGS Engineering Consultants (2009), General Stochastic Event Flood Model (SEFM), Technical Support Model. Manual prepared for the United States Department of Interior, Bureau of Reclamation Flood Hydrology Group.

McKerchar, A. I. and Macky, G. H. (2001), Comparison of a regional method for estimating design floods with two rainfall-based methods. Journal of Hydrology New Zealand, 40(2), 129-138.

Nathan, R.J., Weinmann, P.E. and Hill, P.I. (2002), Use of a Monte-Carlo framework to characterise hydrologic risk. Proc., ANCOLD conference on dams, Adelaide.

Nathan, R.J., Weinmann P.E. and Gato S. (1994), A quick method for estimation of the probable maximum flood in southeast Australia. International Hydrology and Water Resources Symposium: Water Down Under, November, Adelaide, I.E. Aust. Natl. Conf. Publ. No.94, pp: 229-234.

Paquet, E., Garavaglia, F., Gailhard, J. and Garçon, R. (2013), The SCHADEX method: a semi-continuous rainfall-runoff simulation for extreme flood estimation, J. Hydrol., 495: 23-37.

Rahman, A., Haddad, K., Haque, M., Kuczera, G. and Weinmann, E. (2014), Project 5 Regional Flood Methods: Stage 3, report prepared for Australian Rainfall and Runoff Revision Process.

Sih, K., Hill, P. and Nathan, R. J. (2008), Evaluation of simple approaches to incorporating variability in design temporal patterns. Proc Water Down Under Hydrology and Water Resources Symposium, pp: 1049-1059.

Smithers, J.C. (2012), Methods for design flood estimation in South Africa, Water SA, 38(4), 633-646.

WMAwater (2015), Australian Rainfall and Runoff Revision Project 3: Temporal Patterns of Rainfall, Part 3 - Preliminary Testing of Temporal Pattern Ensembles, Stage 3 Report, October 2015.

Weinmann, P.E., Rahman A., Hoang, T.M.T., Laurenson, .E.M., Nathan, R.J. (2002), Monte Carlo simulation of flood frequency curves from rainfall - the way ahead, Australian Journal of Water Resources, IEAust, 6(1), 71-80.

Chapter 4. Data

James Ball, William Weeks, Grantley Smith, Fiona Ling, Monique Retallick, Janice Green

Chapter Status	Final
Date last updated	14/5/2019
Minor edits	27/08/2024

4.1. Introduction

Data, in a range of types, is essential for all water resources investigations, especially the topics involving design flood estimation covered by Australian Rainfall and Runoff. This data is needed to understand the processes and to ensure that models are accurate and reflect the real world issues being analysed.

While standard hydrologic data includes rainfall, water levels and streamflow, a range of other data is also useful or even essential for flood investigations. This chapter provides some background on the types of data needed, and specific issues related to each of these.

It also needs to be pointed out that most the procedures and guidelines presented in Australian Rainfall and Runoff could not have been developed without historical data, and often the reliability of the methods presented depends on the extent of data that has been used in its development.

4.2. Background

Because of variability in water resources data (especially in Australia), long historical records are important to ensure that this variability is well sampled. Long records help to ensure that extremes of both wet and dry periods have been sampled.

However, having long term records means that trends in the data may be important. Trends may be natural or human-induced and may be difficult to detect because of variability and the infrequent occurrence of rare events. Trends can result from human-induced climate change, land use changes, or poorly understood long term climate cycles (e.g. related to the Inter-decadal Pacific Oscillation and other large scale phenomena). Careful analysis is needed to ensure that the long historical records are considered in the context of long and short term natural variability and trends.

There are many organisations that collect and maintain data that is useful for flood estimation. Some of these organisations are major authorities that can be clearly identified and have well organised data in accessible formats. However, there is also a considerable amount of data that is harder to find and often valuable information can be found in unexpected locations. This chapter provides information on the types of data that may be useful, sources where this data can be found and the accuracy that can be expected. A useful discussion on the value of hydrologic data, specifically streamflow data, is included in the paper by Cordery et al. (2006).

Routine data collection programmes are important, but it is often valuable to expend some effort in finding and verifying other data for particular projects. It is also important to note that data useful for these projects may be anecdotal rather than formal and often valuable information can be gathered by simply holding discussions with local residents or other stakeholders. Many projects have a consultation programme which can uncover useful information.

As well, specific formal data collection programmes are often needed for particularly large projects, where the scale of the project justifies expenditure on data collection. For example, this type of programme is often carried out as part of environmental impact studies for major projects during the approval process.

4.3. Risks From Inadequate Data

The following comments are taken from the paper by Cordery et al. (2006).

Australia is the driest inhabited continent and has a more variable climate than other continents. As a result water resources in Australia are often scarce and are therefore critical to the nation's prosperity. At the other end of the scale, large floods often cause devastating damage to property and endanger lives. While present generations are benefiting from the data collection activities of our predecessors, it is our responsibility to ensure that future generations are not disadvantaged by the changes we are implementing now. Data collection is about reducing the risks and increasing the benefits the current and future generations receive from the expenditure of the limited funds available for water management.

Water resources data are used for:

- Flood warning (e.g. the Nyngan floods);
- Groundwater and dry-land salinity assessment and management (e.g. throughout the Murray Darling Basin and much of WA, the Great Artesian Basin and inland sub-artesian aquifers);
- Drinking water quality (e.g. coliform counts as health indicators);
- Design of bridges, dams, stormwater and sewer systems;
- River water quality (e.g. blue-green algae outbreak in the Darling River, habitat protection);
- Water supply for urban and rural communities (e.g. water restrictions and new dams);
- Irrigation for agriculture (e.g. the cap on extractions from the Murray Darling Basin);
- Assessing climate change and its effects on future availability of freshwater;
- Extreme flood estimation (e.g. Warragamba Dam spillway upgrade);
- Water trading – agreed volumes and timing must be reconcilable, and be measured accurately, compliance with licence entitlements; and
- Development of water plans and policies.

Considering the specific concerns for Australian Rainfall and Runoff, inadequate data or the lack of data leads to uncertainty in the results of the analysis and will tend to require additional freeboard allowance for example to compensate for the uncertainty. While there are available procedures that are regional specific and can be implemented on ungauged catchments, there will be more uncertainty in these applications and therefore an increase risk in the flood estimation application. Practitioners need to utilise as much local information, even if this is anecdotal and limited, as possible to reduce this risk.

4.4. Stationarity

Detection of changes in river discharge and magnitude of flood peaks, whether it is abrupt or gradual change is of considerable importance, being fundamental for planning of future water resources and flood protection (Kundzewicz, 2004). Generally flood analysis and planning design rules, including data collection programmes, are based on the assumption of stationary hydrological data sets. If the stationarity assumption is proved to be invalid through global climate change then the existing procedures for designing water-related infrastructures will need revision. This has been recognized in the US with increased emphasis on maintaining stations with long data records (National Research Council, 2004). Long data sets and ongoing analysis are essential to promote accurate design of systems to perform adequately for their design probability and not be over designed resulting in higher costs or under-designed resulting in large damage bills, loss of life and perhaps ultimate failure of structures with resultant community destruction.

A range of human activities including man-made structures such as dams, reservoirs and levees can change the natural flow regime. Land cover and land-use changes including deforestation and urbanization controls many facets of the rainfall–runoff process increasing the peak flows and increasing the amount of runoff. Water conveyance in rivers is altered by river regulation measures (such as channel straightening and shortening, construction of embankments, construction of weirs and locks) or the rehabilitation of rivers with increased stabilisation using trees and logs to provide a better environment for native species. Abstractions from river systems can cause them to run dry and further change the natural channel system and henceforth impinge on the magnitude of larger floods stage height by the considerable amount of debris in the rivers.

Hydrologic data series have generally been considered to be stationary series i.e. there are no long-term shifts in the time series statistical parameters. However, it is recognised that with the “greenhouse effect” analyses might need to take into account the non-stationary effects when performing hydrologic designs. There is therefore a demonstrated need to continue data collection to avoid potential large errors in hydraulic structures design and water resources management due to inadequate streamflow data (Wain et al., 1992).

All flood investigations need to consider the potential for non-stationarity in any data applied to the project, and make appropriate adjustments as required.

4.5. Hydroinformatics

4.5.1. Introduction to Hydroinformatics

An important component for prediction of design flood characteristics is the consideration of the data available for the purpose of predicting both, the magnitude and probability of a flood characteristic. Since the publication of the previous edition of Australian Rainfall and Runoff (Pilgrim, 1987), the increasing computational power available has seen changes in availability and perceptions of data. These changing perceptions resulted in development of hydroinformatics as a conceptual framework for various techniques and approaches to deal with information about water in an electronic format.

Though Abbott (1991) first proposed the term "Hydroinformatics" as a generic term describing the utilisation of information and data about water, the most encompassing and concise definition was presented by Meynett and van Zuylen (1994) who stated:

> ... hydroinformatics deals with the electronic knowledge encapsulation of various sources of information related to the hydro sciences.

With this definition, it is clear that the term 'hydroinformatics' covers a wide range of subject areas that are beyond the classical hydrological and hydraulic sciences involved in design flood prediction and management. This definition also includes data and information from the political, social, economic and legal spheres relevant to design flood events. As suggested, though the scope of hydroinformatics is extensive, however, only those aspects relevant to prediction of design flood characteristics will be discussed in this chapter.

4.5.2. Components of a Hydroinformatics System

Since the specifications of each hydroinformatics system are different and their components will also differ for each application, it is impossible to define all the components that together will constitute a system, however, the concept is that a hydroinformatics system deals with data processing in hydro-environmental sciences. Therefore, any software that assists in this regard can be considered to form part of a hydroinformatics system.

Generally, for a system concerned with the prediction of design flood quantiles, the following components are expected :

- Databases for the storage, retrieval and display of spatial, temporal and statistical data;
- Models for prediction of design flood quantiles using the information contained in the relevant databases;
- Models for generation of data about catchment response to storm bursts or complete events; and
- Decision support systems for enhanced modelling and analysis.

This guide does not discuss all the aspects of a hydroinformatics system, rather the purpose of this guide is to introduce the concept of a hydroinformatic system for design flood estimation in sufficient detail for design flood analysts. Further information on development in and application of hydroinformatics systems can be found in Vathananukij and Malaikrisanachalee (2008), Malleron et al. (2011), Hersh (2012), Popescu et al. (2012), and Moya Quiroga et al. (2013).

4.6. Data Categories and Issues

4.6.1. General

There are two broad groups in which hydrologic data can be categorised, as follows:

- Routine
- Project specific

Routine data collection includes the standard and widely available data, such as streamflow or rainfall data collected on a routine basis by major government agencies such as the Bureau of Meteorology. This data is collected to provide a long term understanding of Australia’s climatic conditions for a representative selection of sites throughout the country. These stations are the basis of many flood estimation procedures and for projects, though the data is appropriate for many other applications.

Project specific data is collected especially for a particular project, and may include observations for major floods in the project area and other specific information to assist in a particular project.

4.6.2. Data Source Organisations

Data can be obtained from a wide range of organisations and individuals, and effort expended in sourcing and checking all available flood data and information is worthwhile. Data can be sources from a number of immediately obvious organisations, but can also be found from others that may not be so obvious.

Major water authorities, such as the Bureau of Meteorology and state water agencies are immediately obvious sources, and these organisations are expected to hold most of the data from formal data collection programmes. These agencies generally have well designed websites, where data can be reviewed and downloaded, and almost all data required for flood investigations can usually be downloaded from these agency websites for no charge.

Other sources include:

- Local authorities. Councils are responsible for planning process and flooding is an important constraint to their planning. Councils therefore usually hold historical data on flood levels as well as other observations.

- Transport agencies. The major state government road and rail agencies, as well as privatised road and rail organisations usually hold extensive data on flooding as it affects their infrastructure.

- Other government agencies. Activities of other agencies such as those responsible for the primary industries, environment or mining are impacted by flooding and they will often hold flood or other meteorological data relevant to flooding.

- Commercial organisations. Mining or agribusiness companies require flood data as it affects their activities and may hold relevant data.

- Farmers and graziers. The weather is critical to agricultural industries and many farmers operate a raingauge and can at least provide data for major storm events, but they may also hold flood records as they affect their irrigation performance for example.

- Individuals. Many individuals have an interest in flooding especially if it affects them so flood levels and other observations can often be obtained from individual property owners.

- Others. There may be specific stakeholders who could supply flood data for a particular flood investigation, and these can vary depending on the actual project and location.

In general therefore, it is important to search widely for flood data during projects. Even anecdotal information will usually be of value in setting model parameters and improving local understanding of flood conditions.

4.6.3. Data on Historical Events

This information is particularly important for most projects, and can often provide a significant improvement in the quality of the analysis. While data on historical floods may be difficult to obtain at times, efforts expended in finding and analysing this data is extremely valuable.

There are three types of data referred to here. These are:

Significant events.

If a major event occurs, it is important for the Bureau of Meteorology, Council or other appropriate agency to collect as much relevant information as possible soon after the event and publish this, even if only in an internal report. Because major events occur rarely and unexpectedly, it is often difficult to mobilise the resources in time and appropriately. As well it may not always be obvious that this data will be useful, so there may not be an immediate interest in the data collection. The Bureau of Meteorology produce reports following major events and these reports usually contain information that is very helpful in particular projects.

Historical events.

Where especially significant events have occurred in past, there are often historical records of the event. These records may be in reports by relevant government agencies, but often there may be useful information in newspaper reports, historical societies or museums or information can be gathered from old long term residents. Particularly significant events such as the Clermont storm of 1916 and the Brisbane floods of 1893 have good published data, but other events may be more difficult to locate.

"Routine" flood data.

As well as the major events noted above, data on more routine (though still large) events can be sourced from discussions with residents and other stakeholders. This data is usually descriptive, but often actual flood levels can be surveyed based on the data held by residents and flood marks on buildings and elsewhere. This data is especially useful if there has been a major flood in reasonably recent times, and local residents can recall details. Photos or videos can be obtained as part of these programmes.

The accuracy of this type of data may be extremely variable and careful review and checking is essential. The requirements for checking are difficult to specify, but the checks should involve review of the consistency between individual data points and a general check of "reasonableness".

Usually this type of data is of variable quality, but with careful collection and checking, is almost always very valuable in implementation of projects.

As well as "numerical data", other less formal data can be collected for historical events. These can include photos or videos taken during the flood or eye witness descriptions and accounts. While this type of information may not be directly applicable for model calibration, it is invaluable in many applications to ensure that the model is representing the general flow conditions and distribution.

This type of data is often sourced from local residents during consultation programmes, when the flood specialist specifically searches for it.

4.7. Discussion on Hydrologic Data Issues

4.7.1. Data Types

4.7.1.1. General

Data is defined as the value of qualitative or quantitative variables. While this definition is simple, the interaction of data with knowledge and data warrants discussion, particularly because the concept of the terms data, information and knowledge frequently overlap each other.

The main difference between these terms is in the level of abstraction being considered. Data is the lowest level of abstraction, while information is the next level, and knowledge is the highest level among all three however, data on its own carries no meaning. For data to become information, it must be interpreted and should take on a meaning. For example, the height of Mt. Everest is considered generally as 'data', but a book on the geological characteristics of Mt. Everest may be considered as 'information', while a report containing practical information on the best way to reach the peak of Mt. Everest may be considered as 'knowledge'. This distinction between the terms is consistent with Beynon-Davies (2002) who uses the concept of a sign to distinguish between data and information; data are symbols while information occurs when symbols are used to refer to something.

In the following discussion of types of data, the lowest level of abstraction is used, namely data has a value but no meaning.

4.7.1.2. Deterministic

Deterministic data can be defined as data that has a unique value in spatial and temporal dimensions. There are many examples of deterministic data used in prediction of design flood quantiles; these examples include Digital Elevation Model (DEM) of the catchment, the surface roughness parameter, and the continuing loss rate parameter.

4.7.1.2.1. Probabilistic

In contrast to deterministic data, probabilistic data does not have a unique value, rather it has a range of values described by a statistical relationship. Each time a data value is sought from probabilistic data, the data value will be different. An example of a probabilistic parameter would be the continuing loss rate for use in a Monte Carlo simulation; in this example, the continuing loss rate will be sampled from distribution of potential continuing loss rates, with each sample likely to differ from previous samples.

4.7.1.3. Spatial

Data can be unique to a point or cover a spatial extent. Additionally, the data may vary in the spatial dimensions but be invariant with time. There are many examples of spatial data types in design flood prediction, including:

- Soil types;
- Spatial patterns of rainfall; and
- Flood surface elevations.

4.7.1.4. Temporal

Temporal data, in contrast to spatial data, consists of data that varies in the time dimension but, usually, has a fixed location . There are many examples of temporal data types in design flood prediction, including:

- Temporal patterns of rainfall; and
- Historical flood hydrographs.

4.7.1.5. Meta-data

For optimal utility in use of data, it is imperative that potential users have the possibility of tracing the data passage stored in an electronic database to its initial source. Questions such as “How were the values obtained?”, “What is the reliability of the values?”, “What editing of the data has occurred?” are vital in assisting the user to interpret the data in a manner appropriate for resolution of the issue under consideration. The development of hydroinformatic systems offers the possibility of facilitating access to meta-data and enhancing the utility of data. The inclusion of meta-data, therefore, is an essential and necessary aspect for suitable use of data in a hydroinformatic system.

A further example of the utility of meta-data for interpretation of data is obtained from consideration of the data necessary for floodplain management. It is possible that a review of the meta-data contained within the hydroinformatic system used for flood management of the catchment may result in the finding that all data within the hydroinformatic system was derived from application of catchment modelling systems and that complementary monitored data was not available. The interpretation of the stored data, therefore, will be different from what would have been the case if the meta-data were not available and it had been assumed that the data were from catchment monitoring.

It is worth noting that prior to the widespread availability of computerised data (i.e. hydroinformatic systems), the analyst preparing the recommendation for freeboard probably would have been aware of the data sources (i.e. the meta-data) and, therefore, would have incorporated this knowledge into the interpretation and ultimate recommendation. Consequently, the inclusion of meta-data in the hydroinformatic system does not generate new knowledge in itself but merely incorporates knowledge currently available only in a non-electronic form.

4.7.2. The Data Cycle

Fundamental to the concept of hydroinformatics is the management of data and its passage from the time it is generated to the time when it is used or presented to stakeholders and other interested parties. During this time, data can be considered to pass through a number of conceptual components. The passage of data through these components is not uniform and linear, rather will pass through the cycle in a random and nonuniform manner. The passage of the data and the components through which it passes, can be considered as the data cycle (Ball and Cordery, 2000).

Conceptually, the data cycle is analogous to the hydrologic cycle where the passage of data through the data cycle can be considered analogous to the passage of water through the hydrologic cycle. Also, similar to the hydrologic cycle, the data cycle can be considered in a systems format with the data flowing between different components.

If the data necessary for prediction of design flood characteristics is considered in this manner, then it is apparent that the components of the data cycle are relevant to the design flood problem. Hence, the concepts associated with hydroinformatics and the data cycle are relevant to the prediction of design flood quantiles.

One of the conceptual views of components forming the data cycle is shown in Figure 1.4.1. The conceptual components shown in the figure are Generation, Editing, Storage, Analysis and Presentation. Also, as indicated , there is a circularity in the flow of information, which arises from analysis of data resulting in generation of new data that needs to be edited and stored in a manner similar to previous data. Considered in this manner, an individual component within a hydroinformatic system is both, a supplier and the data user.

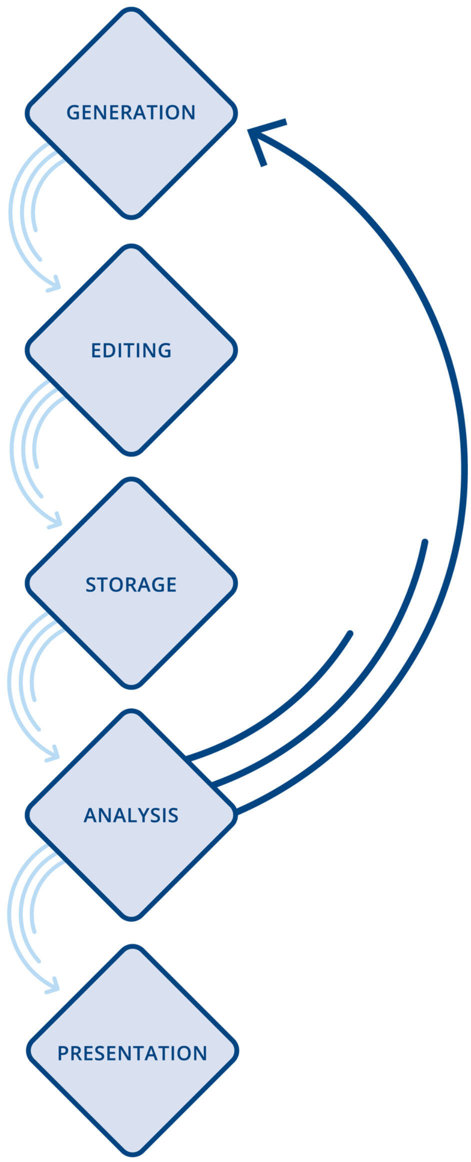


Figure 1.4.1. The Data Cycle

Within each of these conceptual components of the data cycle, the pertinent aspects are:

- *Generation* - In this component the data is sourced; this can be restated as this is the component where the data is created or collected. Furthermore, it is the component where data is recognised relevant for prediction of the design flood characteristics.

  For example, in a catchment monitoring program aimed at collecting discharge data, the necessary steps, as discussed by Chow et al. (1988), would be:

  - sensing the phenomenon;
  - recording the value of the phenomenon; and
  - transmission of these values to a storage repository. This storage depository may be centralised or distributed according to the needs of the stakeholders involved.

Further details on data creation through monitoring programs are presented Book 1, Chapter 4, Section 12.

In addition to the data generation through technical approaches, there are other methods of data generation, where data can be collected through social surveys, census surveys, and historical reviews for example. This data, similar to the technical data, will require management using the concepts of hydroinformatics and the data cycle.

- *Editing* - An important aspect of data is knowledge of its accuracy (sometimes referred to as its uncertainty) and original source. Prior to insertion into a database, it is necessary to define these parameters; in other words, the relevant meta-data has to be attached to the data. The term meta-data is used here to describe the background material about the data which is referred to sometimes as the data about the data. Meta-data is discussed further in Book 1, Chapter 4, Section 7.

  For monitored data, items of primary relevance include issues regrading how the data was observed, the reliability of monitoring (for example, the sensitivity of the sensing equipment, the robustness of the rating table for monitored discharges, the detection limit of contaminants for water quality constituents), and editing changes to the data and the philosophy behind these changes. These meta-data are of great importance for catchment monitoring since at least some of the monitored data will be inaccurate; in other words, some of the monitored data will contain undiscovered errors.

  In a similar manner, data generated by catchment modelling systems needs to be defined by the software (and version) used, the input data inclusive of adopted parameter values or distribution of parameter values. It is should be noted that input data covers both the parameters necessary for operation of the software and the data necessary for implementation of the modelling system. Therefore, one can conveniently state that editing of data generated from catchment modelling systems, i.e. the attachment of meta-data, should be sufficient to define how the data was generated and enable replication.

  The step involving data editing is a vital part of the data cycle and should prioritised above data insertion into relevant database and its subsequent usage by others not involved in its generation. The inclusion of the meta-data and its availability to future users is becoming increasingly important as the availability of digital data increases and data users become more remote from the generation of the data.

- *Storage* - The storage of the data is performed in this component of the data cycle. In general, the storage of data will comprise the insertion of the data into a digital database. It is important to note that the manner of data storage should ensure that its retrieval is both practical and feasible. If retrieval is not easy, there is no addition to the data available for design flood estimation.

  There are many different forms of data stored in a database which in turn influences the data storage design. Commonly, spatial databases are referred to as Geographic Information Systems (GIS) while databases used to store temporal data can be referred to as a Time-Series Managers (TSM). These computerised storage facilities have superseded, in general, the previous techniques based on data storage through maps and charts. There are a large number of alternative GISs and TSMs that can be used for data storage. It is not the purpose of Australian Rainfall and Runoff to recommend a particular GIS or TSM but rather to note their use for storing data relevant to design flood estimation.

  Since 2008, the Bureau of Meteorology (BoM) has been responsible for delivering water data throughout Australia. As part of this role, the Bureau of Meteorology has been collecting water data from more than 200 organisations across the nation and is using this data to report on water availability, condition and use in a nationally consistent way. To facilitate this role, the Bureau of Meteorology is building the Australian Water Resources Information System (AWRIS) as a secure repository for water data and as a means to deliver high quality water data to all Australians.

The aim of AWRIS is to allow the Bureau of Meteorology to process and publish water data in new and powerful ways. The Bureau of Meteorology will be able to merge historical water data records with current observations to suit a variety of user needs. By spatially enabling this data we will be able to query and report the data in many different ways. Data stored in AWRIS will be delivered to the web, to mobile devices and various hydrologic forecasting systems to be operated by the Bureau of Meteorology.

- *Analysis* - It is common that analysis of data will be required. The analysis techniques form this component of the data cycle. The steps involved in this component can be summarised as data retrieval, and data usage. It is worth noting that the data obtained from the analysis could be considered as part of the data generation component. Hence there is some similarity between the generation and analysis components.

  There are many alternative analysis techniques. For design flood estimation purposes, the most commonly used analysis techniques would be statistical modelling (for example, Flood Frequency Analysis) and catchment response modelling using a catchment modelling system. Both of these techniques result in the generation of additional data that, in turn, requires editing and storage.

- *Presentation* - The final conceptual component is the presentation component. Within this step, the stored or analysed data is presented in a manner that is understandable to relevant stakeholders. The technical level of the presented data would not be constant for all presentations but, rather, would vary with the technical expertise of the audience. The important point about the presentation of data is that it is presented in a manner that is clear and precise for the audience.

4.8. Hydrologic Data

This section has an outline of the types of data that is needed for hydrology and hydraulic analysis required for flood estimation and the issues associated with each type.

The data types that are needed are as follows, with discussion or each in the following sections.

- Rainfall;

- Other precipitation types;

- Water levels;

- Streamflow;

- Catchment data, including topography, survey, digital terrain, land use and planning data; and

- Other hydrologic data, including tidal information, meteorological, sediment movement and deposition and water quality.

This data needs to be collected, reviewed for completeness and accuracy and then archived and disseminated to practitioners as required. Discussion on specific details is included below.

4.9. Rainfall Data

4.9.1. Overview

Rainfall is a primary data input for almost all water resources projects, and rainfall data forms the basic input to the development of the rainfall chapters in "Australian Rainfall and Runoff" as well as a key input to other components.

The Bureau of Meteorology is the primary agency responsible for collection of rainfall data in Australia, but there are many other agencies which have significant records of rainfall data. The other agencies include local authorities and water agencies, but some organisations have particular local data programmes that may be useful in specific projects. As well rainfall data can be collected from various sources for major historical floods as discussed further below.

In many major flood events, it is often valuable to look for unofficial rain gauges where data has been collected by members of the public. In rural areas, most property owners have rain gauges and they are also not unusual in towns and cities.

Data endorsed by the Bureau can be regarded as accurate, but some checks for consistency and the reasonableness of the data should also be carried out. In particular, tests for missing (accumulated) data need to be considered but it is also possible that gauge overflows mean that the larger events are not well measured. Data from other agencies may be also of a high standard, but these agencies sometimes have poorer quality data. More careful checks are needed on this data. Data collected at unofficial rain gauges operated by members of the public may be sometimes of very poor quality, with poor exposure for example, and records from these sources must be checked very carefully. However the value of this data means that it is often worth further analysis to ensure that useful data is not discarded. Data from unofficial gauges is especially important for major events where it can be used to supplement information from official gauges.

Most publicly available rainfall data should be available through the Bureau of Meteorology's AWRIS database.

The types of rainfall data that may be useful include:

- Daily rainfall records; and
- Pluviometer records.

Normally data endorsed by the Bureau of Meteorology can be relied upon. However, users should check the data for consistency and logic, before application. In particular, tests for missing or accumulated data need to be considered, along with assessing the potential for gauge overflows.

4.9.2. Rainfall Observations

The standard instrument for manual measurement of rainfall is the 203 mm rain gauge (see Figure 1.4.2). In essence, this instrument is a circular funnel, with a diameter of 203 mm and the top located 0.3 m above the ground surface,that collects the rain into a graduated and calibrated cylinder. Any excess precipitation is captured in the outer metal cylinder. Most manually read gauges are used for daily observations.

Daily rainfall is nominally measured each day at 9:00 am local time. At most rainfall sites, observations are taken by volunteers who send in a monthly record of daily precipitation at the end of each month. A subset of observers at strategic locations, as well as automatic weather stations, send observations electronically to the Bureau of Meteorology each day. Very few stations have a complete unbroken record of climate information. Missed observations may be due to observer illness or equipment failure. If, for some reason, an observation is unable to be made, the next observation is recorded as an accumulation, since the rainfall has been accumulating in the rain gauge since the last reading.

Figure 1.4.2. Standard Rain Gauge (Source: Bureau of Meteorology)

An alternative to the manual measurement is to use a continuous recording rain gauge resulting in either an analogue chart record or a digital record. While some chart recorders remain in operation, the more common form of continuous rain gauges is the Tipping Bucket Rain Gauge (see Figure 1.4.3). Like the manual rain gauge, the aperture of the funnel for a TBRG is 203 mm.

Figure 1.4.3. Tipping Bucket Rain Gauge (Source: Bureau of Meteorology)

Advantages of the TBRG are claimed to include unattended, automatic operation, and the ability to record the rate at which the rain is falling. Operation of a TBRG is based on the generation of an electronic pulse when the water volume collected in the bucket results in bucket tipping. While the usual volume of water collected is equivalent to a depth of 0.2 mm, some early TBRGs required a depth of 0.5 mm before the bucket would tip. When analysing data from TBRGs, users should check the bucket size to ensure the validity of the analysis; this information should be available from the meta-data attached to the recorded data.

Traditionally rainfall is measured to the nearest 0.2 mm (prior to 1974 records were in Imperial units and measurements were to the nearest 1 point, approximately 0.25 mm). However, in recent years some observations have been reported to 0.1 mm. Hence, users check the meta-data attached to the data records to note the measurement accuracy rather than the inferred accuracy from the database records.

The Bureau of Meteorology undertake a number of quality control processes to detect errors in the rainfall data forwarded from the many volunteer and professional readers. This data checking includes:

- Values that extend beyond what is considered realistic;
- Inconsistent observations (for example, high rainfall combined with clear skies); and
- Discontinuous or abrupt changes in values over a short period of time.

While the Bureau of Meteorology undertakes these checks it is recommended that individual users ensure that their rainfall data is suitable for purpose. This may entail undertaking additional quality control processes.

4.9.3. Review of Rainfall Data

While rainfall data is frequently regarded as reliable and accurate, there are some issues with the accuracy and consistency of rainfall data and these need to be considered while applying data to practical applications. Issues often encountered are.

- Accumulated records. Rainfall data, especially from daily read gauges may have missing days of record. In some cases, these missing days are simply not recorded while on other occasions, the total for a number of days is accumulated. This occurs since thee rainfall is collected in the raingauge and several days record are recorded on a single day at the end of the accumulated period. These records need to be reviewed in conjunction with records from neighbouring gauges and adjustments made as necessary. Accumulated records may give an excessively high daily record for the day where the records are accumulated.
- Missing data. In some cases, for both daily read and continuous gauges, there may be missing periods of record. In this case, the record should be reviewed carefully in conjunction with records from neighbouring catchments and appropriate adjustments made.
- Gauge quality. Rain gauges operated by the Bureau of Meteorology are expected to meet the Bureau of Meteorolgy’s standards, however other gauges, especially privately operated gauges which may be used to supplement rainfall records for major events, may not meet the Bureau of Meteorolgy’s stringent standards. Where privately operated gauges appear inconsistent with nearby stations, the siting of the gauge needs consideration and it may be necessary to remove the gauge from the analysis.

4.9.4. Rainfall Databases

4.9.4.1. Introduction

The Bureau of Meteorology has developed a number of databases for storage of rainfall data and its meta-data. These databases include:

- Australian Data Archive for Meteorology (ADAM);
- Site Meta-data (SitesDB); and
- Australian Water Resources Information System (AWRIS).

4.9.4.2. Australian Data Archive for Meteorology

The Australian Data Archive for Meteorology (ADAM) stores meteorological observations from observing systems over mainland Australia and from neighbouring islands, the Antarctic, ships and ocean buoys that are operated by the Bureau of Meteorology. It also stores a limited number of observations from other local and international sources to support research and improve Bureau of Meteorology services.

The most common observation type stored in ADAM is daily rainfall. Dating back to the mid-1800s, these total more than 200 million records from a network of over 16 000 locations. Other types of weather data that are stored in ADAM include air temperature, humidity, wind velocity, sunshine, cloud cover, soil temperatures, upper atmospheric wind and temperature, and observed weather phenomena (for example, thunder, frost and dust).

To support this large database, the ADAM system contains supporting database tables and software tools required to enter, retrieve and quality control data efficiently. A set of detailed rules and procedures ensure consistent treatment of information.

4.9.4.3. Site Meta-data

Meta-data about the Bureau of Meteorology’s rainfall stations is stored in 'SitesDB'. It contains meta-data for each of the Bureau of Meteorology operated rainfall stations and includes, as a minimum, the following information:

- Rainfall station name and number;
- Rainfall station location in latitude and longitude;
- Rainfall station elevation; and
- Details of the current instrumentation.

However, for many stations the following meta-data are available also:

- Maps showing location of rainfall station;
- Schematic of rainfall station layout;
- Photos of rainfall station;
- Photos for each of the four main compass points showing siting, clearance and proximity to trees, buildings and other factors likely to influence measurement of rainfall;
- History of instrumentation installed at site; and
- Record of dates of site visits, maintenance undertaken, problems identified and resolution adopted.

4.9.4.4. Australian Water Resources Information System

The Bureau of Meteorology is building the AWRIS as a secure repository for water data and as a means to deliver high quality water information to all Australians. Under the Water Regulations 2008, the Bureau of Meteorology receives information about river discharges and groundwater levels, water volumes in storage, water quality in rivers and aquifers, water use and restrictions, water entitlements and water trades. The intention is that AWRIS will store and manage this data in a central database.

To achieve this aim, AWRIS is a powerful hydroinformatic system capable of receiving, standardising, organising and interpreting water data from across the nation. The Australian Hydrological Geospatial Fabric (also known as the Geofabric refer to Book 1, Chapter 4, Section 13) is a vital component of AWRIS. It is a specialised Geographic Information System that enables a spatial context to the data stored within AWRIS. With this spatial context, the utility of water data of the data will be enhanced through ease of access. Additionally, the Geofabric encodes the spatial connections and relationships between most of Australia's hydrological features including rivers, dams, lakes, aquifers, diversions, supply channels, drains and monitoring points. Distribution of the data stored in AWRIS is through the Bureau of Meteorology website and Water Online.

4.9.4.5. Gridded Rainfall Data

In addition to the recorded rainfall data from rain gauges located in and near the catchment, the Bureau of Meteorology publish a grid of daily rainfall data for the whole of Australia covering a period from 1900 to date. This is a component of the Australian Water Availability Project (AWAP) (Jones et al., 2007; Raupach et al., 2009). This grid is at a resolution of approximately 5 km (0.05 degrees) and includes daily rainfall value for each grid cell for each day from 1900. The quality of the grid points varies depending on the period of interest since there is a better coverage of gauges in more recent times. The quality also varies depending on the location, with less accurate records for locations with high rainfall gradients and for less populated regions where there is a more sparse gauge density.

4.9.5. Application of Rainfall Data for Flood Estimation

Rainfall data is a critical input to the development of ARR and is also essential in many flood applications, with two principal applications.

Firstly extensive statistical analysis of rainfall data has been carried out to prepare the Intensity Frequency Duration (IFD) input applied to many assessments. Details of the rainfall data analysed for the IFD development is discussed in Book 2, Chapter 3.

Secondly rainfall data is applied for analysis of historical events for flood analysis, and in this application, recorded rainfall data is required for these historical events.

4.10. Other Precipitation Types

Other sources of precipitation include snow, hail or dew. These are usually a relatively minor component of the water balance in Australia, but there are some locations and occasions where this data may be of interest or value for particular projects.

In parts of Australia, snow may contribute a significant amount of precipitation affecting water resources, but this is not common.

The Bureau of Meteorology and specialist agencies in mountainous and southern regions collect this data, and use it for specific studies and supply it on request.

4.11. Water Levels

4.11.1. Overview

Water level data is a principal data type, and as well as being used in its own right, is also used to calculate streamflow data.

A major source of water level data is at formal stream flow stations operated by the major water authorities. This data is usually converted into streamflow data as discussed further below, but there are some locations where only water level data is collected or published. Water authorities publish data and it is usually freely available. The published data usually has an indication of the accuracy and completeness, but some checking is needed.

In addition, there are many stations, especially those operated by the Bureau of Meteorology and local authorities, mainly for flood forecasting and warning, where stream flows are not calculated and water levels only are available. The larger agencies usually make this data available, sometimes on line.

Water level data may be in the form of continuous records monitored by an automatic recorder or as manually read records. Most of the earlier records were manually read at staff gauges, but many of these are now replaced by automatic water level recorders. Manually read records were usually recorded once a day with supplementary more frequent readings during flood events. Because of the rapid response of many streams, the manually read records may not provide the peak levels and may even totally miss short duration flood events. Manually read records are usually better quality for large slowly responding streams and this data can be used with confidence, but smaller catchments may be significantly in error. More often than not, manually read records show smaller flood peaks and lower discharges than automatic recorders. However practically all records from before the 1960s are manually read, so this data forms the only available information for long term stations. Therefore data from manually read stations is usually the only available record and has to be used, but careful consideration is needed to make sure the records are interpreted correctly.

In addition to the formal water level records, informal records can also be obtained usually following a major flood event. These records are usually obtained by a Council or other stakeholder who sends surveyors on-site soon after an event to survey flood marks to indicate the maximum water levels reached. This data provides an indication of the variation of water levels across the floodplain and an indication of the flow patterns. The quality of this data may sometimes be questionable, and the records need to be carefully checked. These checks can include checking for consistency and reasonableness as well as a review of the reliability of the agency or person who has collected the data. When this type of data is collected, it is important that the records include careful descriptions of the circumstances of the collection and an indication of the expected accuracy.

Common concerns with this data is the level of observed debris marks, whether the water levels have been collected at the peak level of the flood and the source of the water level, either local drainage or backwater for example. Therefore while very useful data can be obtained, it must be carefully reviewed otherwise the data may lead to incorrect conclusions in the resulting analysis.

However water levels are often only used as the source of streamflow or discharge data, as discussed below, and while water levels are useful in many applications, streamflow data is usually of far greater value for many water resources studies.

4.11.2. Historical Flood Level Data

4.11.2.1. Continuous Water Level Recorders

Continuous water level recorders measure water levels at nominated intervals and, where a rating curve (stage-discharge relationship) exists, these can be converted to discharge. Flow is derived from stage using a stage-discharge relationship and it is critical that maximum gauged flow is known so that the extent of extrapolation underlying the ‘recorded’ flow is clear (refer to Book 1, Chapter 4, Section 12). These records are very important as, if intact, they will show the complete hydrograph (i.e. the rise, peak and fall of the flood). Historical records for continuous water level recorders are typically stored by government agencies and one needs to be careful with the datum of these records as may not be to AHD (Australian Height Datum) and some conversion may be required. In case of large events, these recorders can fail and the data needs to be inspected for 'flat' areas, which may indicate failure of the gauge or they may rise steeply in case of a landslip occurs. Typically, each stage record has an accuracy code assigned and these should be noted before use.

4.11.2.2. Maximum Height Gauges

Maximum height gauges simply record the peak flood level reached during a particular event. Again, data is often held by a government agency and one needs to be careful while converting the gauge datum to AHD. Failure of these gauges is difficult to detect as they are simply recording the peak level, and if the gauge fails before the peak of an event, it may still provide a 'peak level' value, which will refer the flood level reached prior to the peak at the time of gauge failure. Time of peak is sometimes also available, however, it is recommended that this is checked to ensure that the peak was recorded at approximately the correct time.

4.11.2.3. Peak Level Records

If the flood event has been of a significant nature, it is likely that stakeholders have been able to collect some actual flood levels at a variety of locations. This is typically done by mobilising agency staff to place markers (paint, stakes, nails, surveyor’s tape) either indicating maximum flood extent (e.g. spray paint on a road, stake in ground) or peak flood levels (e.g. nail in a tree). Ideally, each marker should have the time and the staff member’s name recorded at the marker site. Following the event, surveyors can measure *x,y* location data and z flood level data at each of these markers. It is also useful to photograph the site and to record ground level. Surveyors should also include the time at which the markers were placed, by whom and type (e.g. nail in tree) in their meta-data. Residents often also record peak flood levels, particularly if the flood has inundated any buildings on their property. Post event flood levels can be collected by residents by a questionnaire and reliable marks surveyed. An assessment as to the reliability of these levels can only be made after viewing the marks themselves and noting the care with which the recording has been made. Have different event dates been recorded by the resident or is the resident relying on memory to determine one event from another? Has the location of the marks changed in any way since the record was made? For example, if the marks are made near the front door, has the house been raised at any time since? Detailed discussion with the resident can often unearth important details otherwise unknown.

4.11.2.4. Debris Marks

Debris marks are a typical means of measuring the maximum flood level and are best measured as soon as possible after the event, when the debris or scum line is still fresh. This ensures that the mark is attributable to the event of interest and has not been subsequently degraded.

Debris marks can be inaccurate for a number of reasons. They can be influenced by dynamic hydraulic effects such as waves, eddies, pressure surges, bores or transient effects, which may not be accounted for in a hydraulic model. For example, if the debris mark is located within a region of fast flowing floodwater it is possible that the floodwater has pushed the debris up against an obstacle, lodging it at a higher level than the surrounding flood level. More common though is the fact that debris often lodges at a level lower than the peak flood level. The reason for this is that for debris to be deposited it needs to have somewhere to lodge and this elevation is not always at the peak flood level. For example, the classic place for debris lodgement is a barbwire fence with horizontal strands of wire. If the flood level almost reached the top strand of barbwire, debris will not lodge in the top strand but rather on the second from top strand, which may be about 0.3 m lower than the peak flood level. It is recommended that the surveyor be asked to record as much information as possible about the mark itself (e.g. debris on barbwire fence spacing 0.35 m) so the modeller is able to consider reasons for discrepancies in the calibration process, if they arise.

4.11.2.5. Anecdotal Information

Anecdotal information is usually qualitative in nature but can be very valuable in determining flow behaviour and subsequently verifying that the flood analysis represents these observations in the hydraulic modelling undertaken. Photograph and video evidence can also be beneficial in this regard and can often assist long-term residents remember details of historical floods long past. The flood modeller will need to be mindful of the fact that memories can sometimes fade or be skewed by other events that have occurred particularly when several floods occur close together. In addition, information providers may not be able to provide unbiased information due to a vested interest (e.g. pride or financial gain etc) in the level to which an historic event reached. Again, detailed discussions with residents and stakeholders can provide the modeller with a general feel for the reliability of all anecdotal evidence. Inconsistent facts have to be identified and discarded and discrepancies have to be studied and explained.

4.11.3. Application of Water Level Data in analysis

The principal application of observed water level data in flood projects is in the calibration of hydraulic models and to ensure that the models represent reality.

4.12. Streamflow Data

4.12.1. Introduction to Streamflow Records

Streamflow data is one of the most important data requirements for individual projects and for development of regional procedures. As noted above, streamflow data is calculated from records of water levels, usually collected by major water authorities. The water levels are used to calculate streamflow data by the application of a stage-discharge relationship (rating curve) developed for the station. Continuous records of streamflow can be calculated from the continuous records of water levels. The stage-discharge relationship is often uncertain and application is one of the major sources of uncertainly in the data.

As with water level data, the major water authorities have well established systems for storage and dissemination of their streamflow data. This data is usually available from these agencies, often on line, and almost always free of charge. The data dissemination systems are well organised and data can be supplied accurately and quickly.

Different agencies around Australia maintain appropriate databases of their records. These systems include considerable detail on the type, accuracy and reliability of the gauged data including rating curve accuracy, periods of missing record, number of gaugings and variation in rating curves. In many cases, the system includes photos and maps of the station. This is valuable information and is valuable to ensure that the data is used most effectively. The water agencies are state based and there are differences between the states. Their on-line documentation should be consulted to ensure that the quality and limitations of the record are understood.

There are few other agencies that collect streamflow data, because of the difficulties of calculating the flows from water levels. Where there are other sources of this data, it is usually limited as a part of the research project for a limited duration and locality, and the data is sometimes difficult to obtain. This data is also usually only for a short period of record, but sometimes, it may include an important event. However it is also possible for practitioners to calculate discharge records from water level data using individually developed rating curves that can be prepared from hydraulic models or other theoretical methods.

There are many checks needed when analysing streamflow data. The principal check is on the accuracy and completeness of the stage-discharge relationship. This can be checked by assessment of the number of discharge measurements that have been taken and the maximum discharge (as compared to the maximum recorded water level). As well the variability in the stage-discharge curve indicates that the relationship has changed over time and therefore may be less reliable for particular events. The stage-discharge relationship may be poor for the lower flows because of regular changes in low flow controls. As well it may also be poor at higher flows because of the lack of discharge measurements at higher flows. There are difficulties in extrapolation of the relationships, where there is a change in conditions, for example where the river overtops the banks.

Different gauges in the same catchment can be compared to test for consistency and the water balance and there is a range of other checks that can be carried out. Having more than one gauge in a catchment though is not particularly common.

Poor quality streamflow data may mean poor quality model calibration, so a high standard for checks of data is important. However it is noted that it is very difficult to check the accuracy of the discharge records for a station, and poor quality data may be accepted.

Streamflow records are the basic data source used in developing reliable surface water resources because the records provide data on the availability of streamflow and its variability in time and space. The records, therefore, are used in planning and design of surface water related projects, and are also used in management or operation of projects after construction of the projects is complete.

In addition, streamflow records are used for calibrating catchment modelling systems that, for example, are used for predicting flood behaviour and for predicting hazards arising from design flood events. Records of flood events obtained at gauging stations also serve as one of the basic data sources for design flood estimation, necessary for designing bridges, culverts, dams and flood control reservoirs, floodplain delineation and flood warning systems; and in development of methods applicable to locations where data is not available.

It is essential, to have valid records for a full range of streamflows. The streamflow records referred to above primarily are continuous records of discharge at stream-gauging stations; a gauging station being a site instrumented and operated so that a continuous record of stage and discharge can be obtained. A network of continuous recording gauging stations, however, often is augmented by auxiliary networks of partial record stations to fill a particular need for streamflow data at a relatively low cost. For example, an auxiliary network of sites, instrumented and operated to provide only instantaneous peak level data, is often established.

4.12.2. General Stream Gauging Procedures

4.12.2.1. Introduction

Gauging stations are installed where the need for streamflow records at a site has been recognised. This gauging station will comprise of instruments for measuring the river stage and a location selected to take advantage of the best locally available conditions and discharge measurement and for developing a stable stage-discharge relationship, sometimes referred to as a rating curve. While there are instruments that simultaneously monitor river stage and discharge, the more common instrumentation requires the use of a stage-discharge relationship to convert the monitored river stage into an equivalent river discharge rate. Artificial controls such as low weirs or flumes are constructed at some stations to stabilise the stage-discharge relationships in the low discharge range. These control structures are calibrated by stage and discharge measurements in the field.

Selection of the gauging station site and the development of the stage-discharge relationship are important components in the management of a gauging station and hence the discussion herein will focus on these aspects of management of a gauging station. While there are many other aspects important in management of a gauging station, these two aspects have the most significant impact on prediction of design flood characteristics.

It is rare to find an ideal site for a gauging station and it is more common that the limitations of a site must be considered with respect to the desired data from the site. There are numerous guides on gauging station site selection with the aim of these guides on ensuring the data is reliable and criteria suggested by ISO (2013) are often adopted. When applying data recorded at a stream gauging station, the practitioner must review details of the station carefully and make an assessment of the quality and limitations of the record.

4.12.2.2. Data Collected at a Gauging Station

There are many different approaches to collection of data at a stream gauging station and flood investigations are not necessarily the principal objective of any particular gauging station. As stated previously, the purpose of a gauging station is to collect data about the time history of discharge at that point in the catchment drainage network. In general, the data collected consists of the gauge heights, sometimes referred to as stages. These gauge heights are used as the independent variable in a stage-discharge relationship to estimate the discharge at that point in time. Reliability of the discharge record is dependent on the accuracy and precision of the gauge-height record as well as the accuracy and precision of the stage-discharge relationship.

Gauge-height records may be obtained by systematic observation of a non-recording gauge, or with automatic water level sensors and recorders. Furthermore, various types of transmitting systems are used to relay gauge-height information from remote gauging stations to storage databases.

New technology, especially in the field of electronics and computer based management of field data, has led to a number of innovations in sensing, recording, and transmitting gauge height data. In the past most gauging stations used floats in stilling wells as the primary method of sensing gauge height and these are still in common use today. However, the current trend is toward the use of submersible or non-submersible pressure transducers which do not require a stilling well. Additionally, electronic data recorders and various transmission systems are being used more extensively.

Details of the instruments and associated structures for a stream gauging station are outlined by ISO (2013), Rantz (1982), and World Meteorological Organisation (2010) and hence are not repeated herein. Nonetheless, users of stream discharge data for design flood estimation should ensure that they are conversant with the instruments and structures employed for the collection of streamflow data.

4.12.2.3. Stage-Discharge Relationships

The conversion of a record of gauge-height to a record of discharge is through use of a stage-discharge relationship. The physical element or combination of elements in the stream channel or floodplain that maintains the relation is known as a control. One major classification of controls differentiates between section controls and channel controls. Another classification differentiates between natural and artificial controls. Artificial controls are structures built for the specific purpose of controlling the stage-discharge relation, such as a weir, flume, or small dam. A third classification differentiates between complete, partial and compound controls.

The two attributes of a satisfactory control are stability and sensitivity. If the control is stable the stage-discharge relationship will be stable. If the control is subject to change, the stage-discharge relationship will be subject to change and frequent discharge measurements will be required for the continual re-calibration of the stage-discharge relationship. This increases the operating cost of the gauging station and also increases the uncertainty of streamflow records extracted from the database. Additional data about controls can be found in Herschy (1995).

The traditional way in which a stage discharge relationship is derived for a particular gauging station is the measurement of discharge at convenient times. Traditionally, this measurement is undertaken with a current meter measuring the discharge velocity at enough points over the river cross-section so that the discharge rate can be obtained for that particular stage. By taking such measurements for a number of different stages and corresponding discharges over a period of time, a number of points can be plotted on a stage-discharge diagram, and a curve drawn through those points, giving what is hoped to be a unique relationship between stage and discharge, the stage discharge relationship, as shown in Figure 1.4.4. This rating curve is used in a manner whereby the routinely measured stages are converted to discharges by assuming that the corresponding discharge can be obtained from the curve.

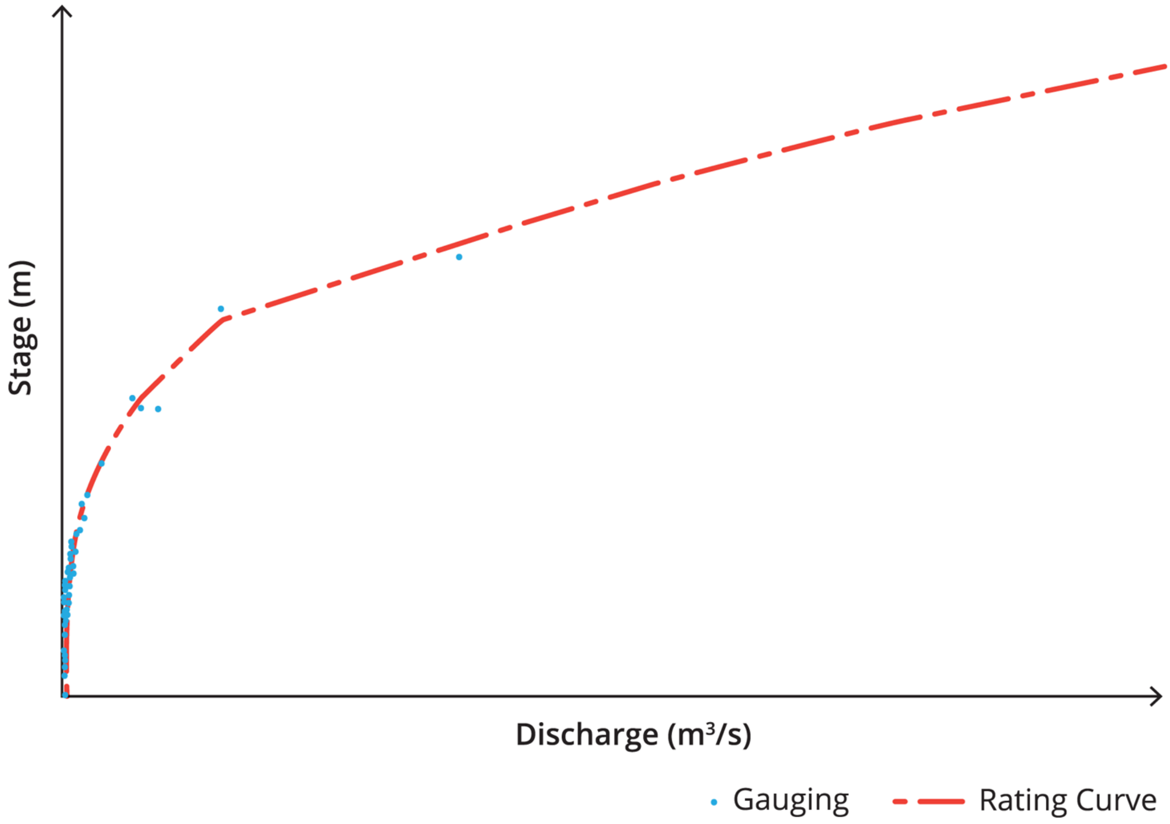


Figure 1.4.4. Typical Rating Curve

There are a number of factors which might cause the rating curve not to give the actual discharge, some of which will vary with time. Fenton (2001) quotes (Boyer, 1964) as describing a list of factors affecting the rating curve, or what he called a *shifting control*. These include:

- The channel and hydraulic control changing as a result of modification due to dredging, bridge construction, or vegetation growth;
- Sediment transport - where the bed is in motion, which can have an effect over a single flood event, because the effective bed roughness can change during the event. As a flood increases, any bed forms present will tend to become larger and increase the effective roughness, so that friction is greater after the flood peak than before, so that the corresponding discharge for a given stage height will be less after the peak. This will also contribute to a flood event showing a looped curve on a stage discharge diagram as shown on Figure 1.4.5. Both Simons and Richardson (1962) and Fenton and Keller (2001) have examined this phenomenon and presented approaches for dealing with this issue;
- Backwater effects - changes in the conditions downstream such as the construction of a dam or flooding in the next waterway downstream;
- Unsteadiness - in general the discharge will change rapidly during a flood, and the slope of the water surface will be different from that for a constant stage, depending on whether the discharge is increasing or decreasing. The effect of this is for the trajectory of a flood event to appear as a loop on a stage-discharge diagram as shown in Figure 1.4.5;
- Variable channel storage - where the stream overflows onto floodplains during high discharges, giving rise to different slopes and to unsteadiness effects; and

- Vegetation - changing the roughness and hence changing the stage-discharge relationship.

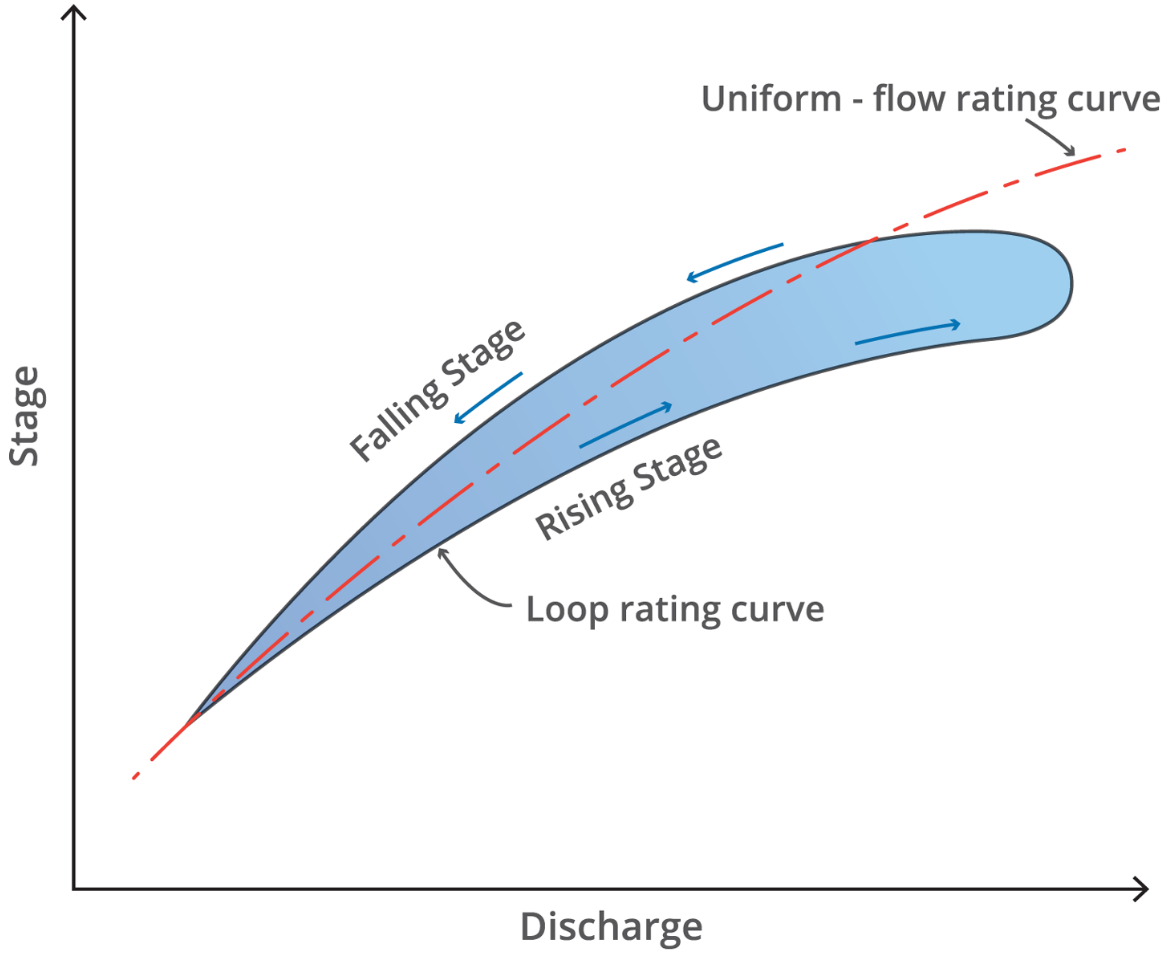


Figure 1.4.5. Loop in Rating Curve

In addition to these generic problems associated with the use of rating curves, there are several problems associated with the use of rating curves for prediction of a design flood characteristic. These include:

- The assumption of a unique relationship between stage and discharge, in general, is not justified;

- Discharge is rarely measured during a flood, and the quality of data at the high discharge end of the curve typically is quite poor because there are usually few velocity measurements at high flow. As a result estimation of the peak discharge of a flood event usually involves extrapolation of the stage-discharge relationship beyond the recorded data points;

- The relationship is usually a line of best fit through the data points defining the stage-discharge relationship. The approach recommended for estimation of this line of best fit in many guidance documents (for example, World Meteorological Organisation (2010)) is a visual fit. This approach provides minimal data on the uncertainty of the relationship and the reliability of any extrapolation of the relationship. This limits the estimation of the propagation of the uncertainty in the flood characteristic prediction approach; and

- It has to describe a range of variation from no discharge through small but typical discharges to very large extreme flood events.

As highlighted in the previous discussion, the unsteadiness of the discharge during a flood event (i.e. the variation of discharge with time) and its influence on a discharge estimate is ignored in the traditional application of a rating curve. In a flood event the slope of the water surface for a given stage will be different from that for the same stage during steady flow conditions; this difference will depend on whether the discharge is increasing or decreasing. As the flood increases, the surface slope in the river is greater than the slope for steady flow at the same stage, and hence, according to conventional hydraulic theory more water is flowing down the river than the rating curve would suggest. The effect of this is shown in Figure 1.4.5. When the water level is falling, the slope and, hence, the discharge inferred is less. The effects might be important - the peak discharge could be significantly underestimated during highly dynamic floods, and also since the maximum discharge and maximum stage do not coincide, the arrival time of the peak discharge could be in error and may influence flood warning predictions. Finally, the use of a discharge hydrograph derived inaccurately by using a single-valued rating relationship may distort estimates for resistance coefficients during calibration of an unsteady flow model.

4.12.2.4. Extrapolation of Stage-Discharge Relationships

The stage-discharge relationship can be considered to consist of two zones. These zones are:

- An interpolation zone where the relationship is within the range of the stage measurements used to develop the relationship; and
- An extrapolation zone where the relationship is not defined by gaugings taken to develop the relationship.

A diagrammatic illustration of these two zones is shown in Figure 1.4.6.

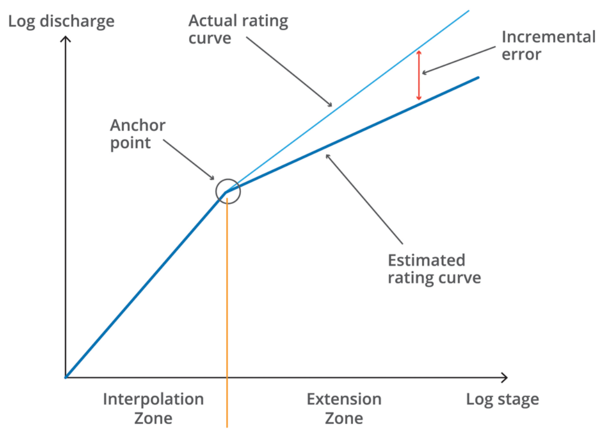


Figure 1.4.6. Stage-Discharge Relationship Zones

While it is preferable that all stage measurements are within the interpolation zone, the nature of the data needed for design flood estimation, and for flood prediction in general, the reliability of data from measurements within the extrapolation zone will require consideration of the extrapolation methodology. The need for extrapolation is shown in Figure 1.4.7 where the discharges for the Annual Maxima Series extracted for the Stream Gauge are plotted as a function of the rating ratio (the rating ratio is the ratio of the recorded discharge to the highest gauging used to develop the stage-discharge relationship). All points in the Annual Maxima Series where the rating ratio is greater than 1 require use of the extrapolation zone of the stage-discharge relationship.

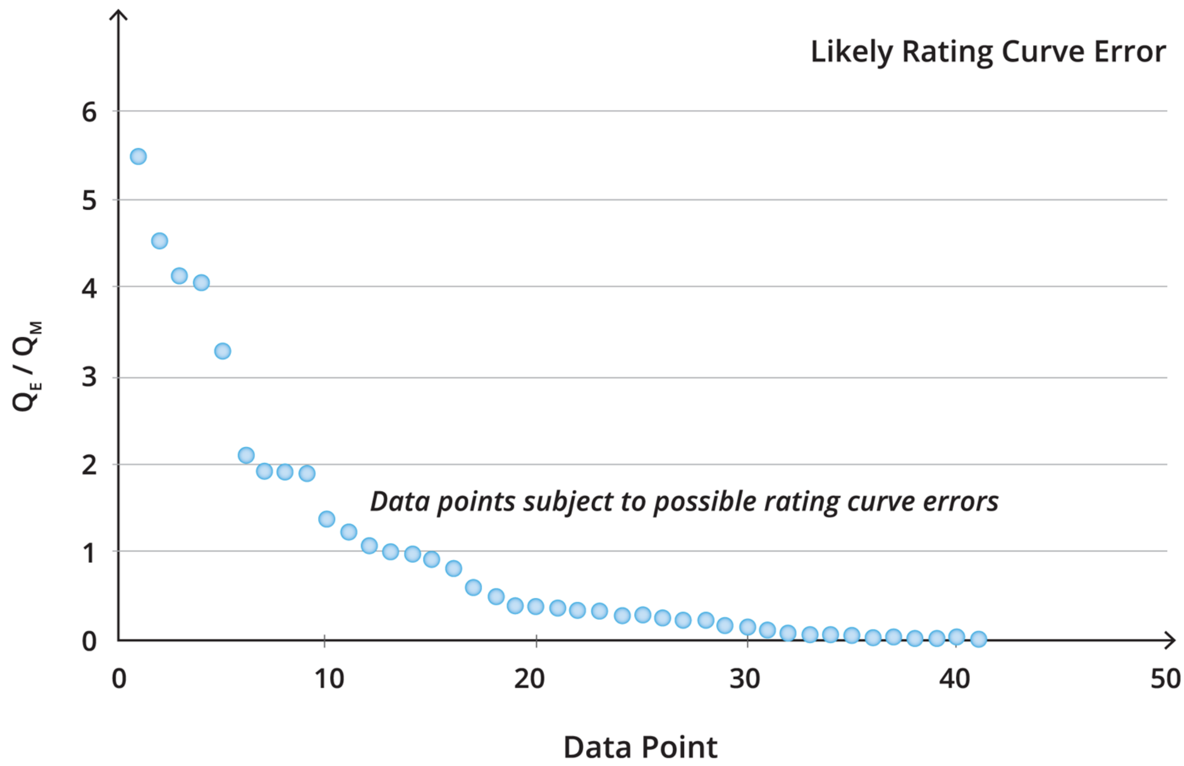


Figure 1.4.7. Annual Maximum Series

As shown in Figure 1.4.7, a number of the values in the Annual Maxima Series are in the extrapolation zone. The accuracy of the values in the extrapolation zone has the potential to influence fitting of the statistical model to the Annual Maxima Series, thereby influencing the predicted design flood quantiles. Fitting a statistical model to data points where the higher values are subject to estimate errors is discussed in Book 3, Chapter 2.

There are a number of alternative techniques for development of the extrapolation zone of the stage-discharge relationship, with a logarithmic extrapolation being often recommended. This approach however may not be applicable because in many cases, the extrapolation may extend from a confined channel into a floodplain.

An alternative approach is the use of a hydraulic model to develop the extrapolation zone of the stage-discharge relationship. Similar to the application of a logarithmic technique, the suitability of this approach needs to be confirmed prior to its application. Of particular concern is the modelling of the energy losses associated with flow in the channel and adjacent floodplains where it is necessary to assume that the parameter values obtained during calibration are suitable for the larger discharges being simulated in the extrapolation zone of the stage-discharge relationship.

The important point in this discussion, however, is a recognition that the values of the data extracted from a discharge record for fitting of a statistical model will contain values where the conversion of the recorded level to an equivalent discharge occurred through extrapolation of the stage-discharge relationship. Consideration of this in the fitting of a statistical model to the Annual Maxima Series is discussed later in Book 3, Chapter 2.

4.12.2.5. Uncertainty of Discharge Measurements

The accuracy, or uncertainty, of a discharge measurement is very important for purposes of assessing the quality and reliability of that measurement. The concept of error and error analysis is a long-standing practice in the field of hydraulics and hydrology. The concept of uncertainty, however, is relatively new. Nonetheless, methods for evaluating, defining and expressing the uncertainty of streamflow measurements have developed.

Uncertainty and accuracy are terms that are sometimes used interchangeably even though they have two very distinct meanings. Accuracy (or error) refers to the agreement, or disagreement, between the measurement of stream discharge and the true or correct value of the discharge at the time of measurement. Since we can never know the true value of the discharge, we can never know the exact amount of error in the discharge measurement.

The uncertainty of a discharge measurement, on the other hand, acknowledges that no measurement is perfect. It is defined, therefore, as a parameter associated with the result of a measurement that characterises the dispersion of values that could reasonably be attributed to the measurement. It is typically expressed as a range of values in which the measurement value is estimated to lie, within a given statistical confidence. It does not attempt to define or rely on a unique true value. To summarise, common usage of the word 'accuracy' for quantitatively describing the characteristics of a discharge measurement is incompatible with its correct meaning. The proper term for expressing the statistical confidence of possible values for a discharge measurement is uncertainty.

The sources of uncertainty in discharge measurements can be categorised as:

- Measurement - these are the uncertainties associated with taking the measurements. The primary component in this category is instrument accuracy. The accuracy of both the velocity meter and the level recorder need to be considered.
- Methodology - these are the uncertainties associated with the analysis of the recorded measurements to enable the development of a point on the stage-discharge relationship. In this category, features such as the assumption of linear variation in the cross-section between the bathymetric points, the assumption of a logarithmic vertical velocity profile, wind effects, changing stage during measurement, etc need to be considered.

World Meteorological Organisation (2010) guidance on the likely standard errors encountered in undertaking a gauging point are:

- For rod suspension, the standard error ranges from 2% for an even, firm, smooth and stable streambed, to 10% for a mobile, shifting sand, or dunes streambed;
- For cable suspension, the standard error ranges from 2% to 15% for an unstable streambed, high velocity, and vertical angles; and
- For acoustic depth measurements, the standard error ranges from 2% for a stable streambed to 10% for a mobile, shifting sand, and dunes streambed.

4.13. Catchment Data

4.13.1. General

Catchment data is an essential component for estimation of design flood characteristics and there are various types of catchment data required. Furthermore, data is available from different sources and with a range of accuracies. Generally, it is advisable that practitioners seek the most suitable data in each instance and assess the required accuracy of that data in respect of the desired accuracy of the outputs.

4.13.2. Types of Catchment Data

There are many alternative types and forms of catchment data relevant to estimation of design flood characteristics, one of the major forms being those associated with assessment of flood behaviour. To predict this flood behaviour, the following types of data may be required:

- Topographic and infrastructure data including structures within the floodplain including culverts, bridges, and pipe networks;
- Land use information;
- Vegetation data; and
- Soil data.

4.13.3. Topographic and Infrastructure Data

4.13.3.1. General

Topographic data is an important component of any design flood investigation. Proper scoping of topographic and infrastructure data collection can have a significant impact on the cost effective delivery of flood investigations. The scope of the required topographic and infrastructure data is driven by the nature of flood behaviour for a given area. The desired elements of topographic and infrastructure data include:

- Catchment extent;
- Catchment slope;
- Drainage topology (i.e. the drainage flow paths and network of channels);
- Channel cross-sections;
- Waterway structures (weirs, levees, regulators, dams, culverts and bridges etc);
- Overland flowpath definition; and
- Infrastructure (bridges, culverts, pits, pipes etc).

There are a number of alternative approaches to obtaining the necessary topographic data including:

- Field survey;
- Airborne techniques; and
- Available spatial mapping.

4.13.3.2. Field Survey

Discussed in this section are details of the scoping of the field survey component for an investigation for design flood estimation. The focus is on those features where field survey is needed to capture the desired information. These features include:

- Channel and overland flow path cross-sections (and potentially long-sections);
- Waterway Structures - weirs and regulators;
- Infrastructure - bridges, pipes, pits and culverts;
- Road and rail embankments;
- Levees; and
- Property data (floor level, type of building, size, location).

While it is possible to obtain existing field survey data from other sources, it is important to assess its suitability for the intended purpose. In other words, it is necessary to obtain both the data and the meta-data, which includes items related to date of capture, accuracy, etc.

4.13.3.2.1. Cross-Sections

A survey of cross-sections is required only when the design flood estimation requires application of a catchment modelling system to generate data that is not available from a catchment monitoring program. Hence, the scope of a cross-section survey depends on the type of the catchment modelling being used. Where catchment modelling is focussed on hydrologic simulation (using, for example, a conceptual rainfall-runoff model), the cross-section data required is minimal. However, where the catchment modelling requires hydraulic simulation (using, for example, a one dimensional network model), the cross-section data required will be more extensive.

The important characteristics of the cross-section data include the lateral extent and longitudinal spacing of cross-sections. The lateral extent of the cross-section must be sufficient to include key in-bank elements and extend to above the highest flood level, which often extends onto the floodplain and outside the stream channel. When surveying in-bank cross-sections, good field notes and/or photos describing the nature of the channel are vital for proper interpretation post collection. There are numerous references such as Stewardson and Howes (2002) that describe flood study cross-section survey requirements in detail. The overriding principle being that the cross-section data is adequate to estimate the shape and slope of the channel so that suitable estimates of the flow conveyance capacity of the channel can be calculated.

The influence of in-bank features on flood discharge behaviour tends to reduce as the magnitude of the flood discharges increases. For example, bank-full capacity of a river channel may represent 100% of a 0.2 or 0.5 EY discharge but less than 10% of a 1% AEP discharge.

4.13.3.2.2. Structures and Drainage Infrastructure

Structures in the waterway and on the floodplain may have a significant influence on flood behaviour. Structures requiring survey include:

- Levees;
- Road and rail embankments;
- Hydraulic structures such as weirs, bridges and culverts;
- Fences; and
- Drainage infrastructure such as pipes and pits.

Often these structures constrict and obstruct flood discharges thereby influencing the design flood estimation. The effects on flood behaviour may be intentional (such as a weir) or unintentional (such as blockage at a culvert). Where the effect on flood discharge is unintentional, it is worth noting that the effect may be stochastic and hence the likelihood of the effect needs to be considered by a suitable joint probability technique. Irrespective of whether the influence is intentional or unintentional, these structures will have an influence on the estimation of the design flood characteristic.

Hydraulic structures generally act to control the discharge behaviour in accordance with a particular management strategy. As most management strategies are concerned with frequent discharges, there are control structures designed to operate under flood conditions as part of an operational flood management strategy. Hence, the impact of these structures will vary with the management strategy and flood magnitude.

The purpose of a levee is to divert discharges as part of an operational flood management strategy. As part of this strategy, levees are designed only to protect floodplains for a specified portion of the relationship between the likelihood and magnitude of a flood event; in other words, there is a defined probability that a levee can be overtopped (with or without failure of the levee bank) by a flood rarer than the levee was designed to protect against. Hence, in design flood estimation, the geometric properties of a levee are important to enable suitable estimation of the flood magnitudes (and hence probabilities) of design events likely to result in hydrologic (and/or structural) failure of the levee.

Similar to field survey of cross-sections, field practicalities such as vegetation, access and water depth and flowrate may influence the location and details surveyed for a given structure.

4.13.3.2.3. Field Survey Techniques

The techniques used for collection of field survey data (typically by surveyors) are discussed in this section. Details of three techniques are presented; namely Traditional Ground Survey, Real Time Kinematic (RTK) Global Positioning Systems (GPS)/ Differential GPS, and Photogrammetry. Typical accuracies for each of these techniques are provided in Table 1.4.1. The techniques to measure topography and other survey features generally fall into two main categories:

- Direct measurement - where the survey technique involves a ground based instrument measuring features by physical contact and relating the measurement directly related to know ground control such as a State Survey Mark (SSM); or
- Remote sensing - where features are measured without physical contact with the object, and generally refers to measurement by an airborne or satellite mounted instrument.

Table 1.4.1. Typical Accuracies of Field Survey

Survey Technique	Nominal Accuracy (+/- m)	
	Vertical	Horizontal
Traditional Ground Survey	0.01	0.01
RTK GPS	0.05	0.05
Photogrammetry	0.1-0.3	0.2-0.5
ALS (LiDAR)	0.15-0.4	0.2-0.5

4.13.3.2.3.1. Traditional Ground Survey

Traditional ground survey (that is, survey collected by traditional or total station ground survey techniques) is the most accurate survey technique with vertical and horizontal accuracies as shown in Table 1.4.1. However, this technique is manual and labour intensive and is therefore best suited to small and/or difficult areas for other techniques, to supplement data obtained from other techniques, and to validate data obtained from other techniques.

In the context of a design flood estimation, traditional ground survey methods are often used for:

- Checking remote sensed data sets; and/or
- Supplementing remote sensed data in:
  - Areas that are impenetrable from the air (for example, satellites and aeroplanes have difficulty in sensing the ground in areas like the bank of channels and/or heavily vegetated areas);
  - Areas that are critical for the data being sought from the catchment modelling system (for example, critical hydraulic controls such as levees and weirs) where topographic accuracy is important.

4.13.3.3. Real Time Kinematic Global Positioning Systems and Differential Global Positioning Systems

Real Time Kinematic Global Positioning Systems (RTK GPS) involves coordinated use and comparison of two Global Positioning Systems (GPS) using the same satellite signals. The first GPS, sometimes referred to as the 'base station', is positioned over a known location (typically a permanent survey mark) and maintains a continuous record of the location relative to numerous satellite positions. The second GPS, referred to as a 'roving GPS', can be hand-held or mounted to a car, boat or an all-terrain vehicle; this unit collects data defining the location of the roving GPS position from the same satellites. The accuracy of the roving GPS location is enhanced by comparison of the satellite signals for the two stations. Shown in Figure 1.4.8 is the concept underpinning field survey using RTK GPS techniques.

RTK GPS has the advantage that it can collect a reasonable amount of data at higher accuracy than remote sensed data and much faster than by traditional means. The disadvantage is that if the vehicle in which the RTK GPS is mounted is unable to access an area, the system may have to be dismantled to gain access by some other means or measurement of data is not possible in the inaccessible area. RTK GPS methods also rely on the instrument having line of sight access to an array of satellites. This can be a limitation to the technique in areas underneath a tree canopy.

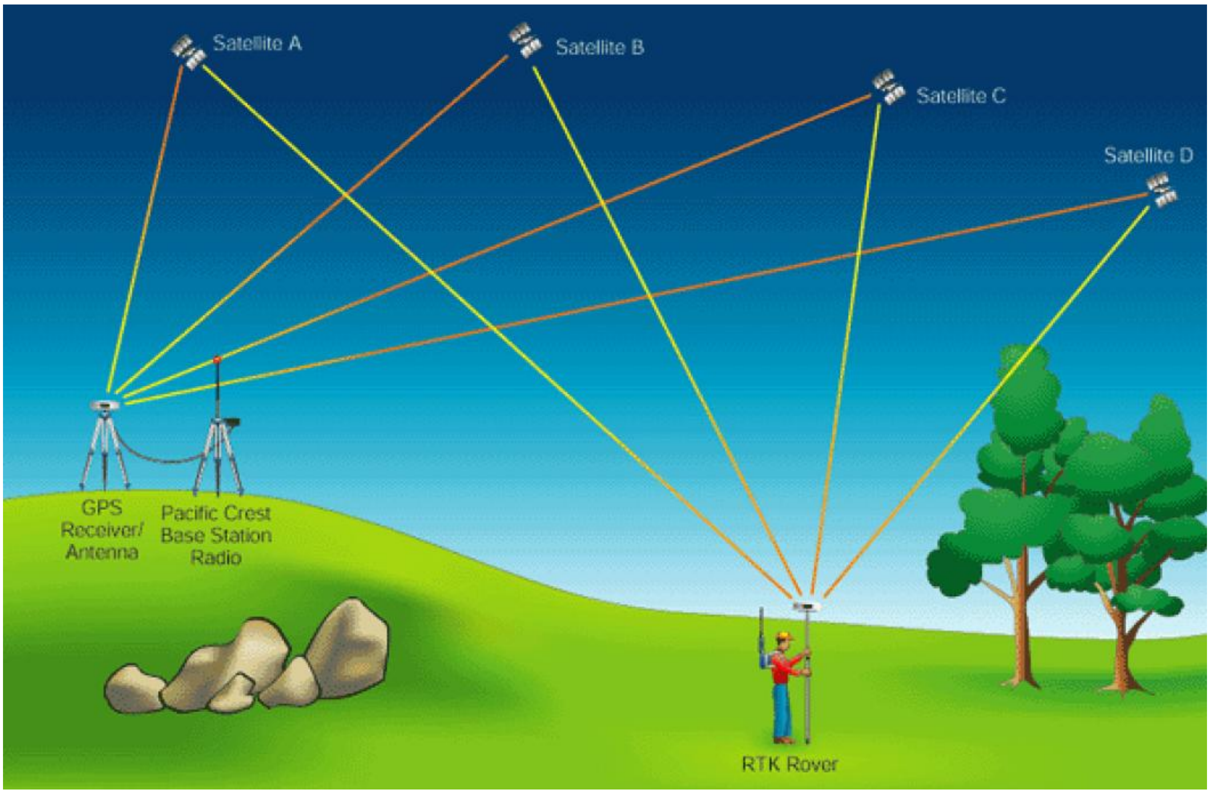


Figure 1.4.8. Concept of RTK GPS Technique of Field Survey

4.13.3.4. Airborne Techniques

Defining flood behaviour within an area containing overland flowpaths requires extensive topographic data. Aerial techniques are well suited to capture this topographic data across a broad area. Commonly, this topographic data is represented as a Digital Elevation Model (DEM). It should be noted that there is no universal definition of the terms Digital Elevation Model (DEM), Digital Terrain Model (DTM) and Digital Surface Model (DSM) in scientific literature. In most cases, the term DSM represents the earth's surface and includes all objects on it. In contrast to a DSM, the DTM represents the bare ground surface without any objects like plants and buildings. Both DTM and DSM may be referred to as DEM.

Usually, a DEM is represented as a raster (a grid of squares) or a vector-based Triangular Irregular Network (TIN). According to Toppe (1987), the TIN DEM data set is referred to as a primary (measured) DEM, whereas the Raster DEM is referred to as a secondary (computed) DEM; this definition, however, predates the widespread availability of Airborne Laser Scanning (ALS) (also known as Light Detection and Ranging - LiDAR) data for definition of the catchment surface topography.

Before embarking on any aerial data capture, it is worth liaising with other agencies that may hold topographic data to ensure that the existing spatial extent of data does not cover the desired area.

At present, the two principal techniques used in aerial survey for obtaining topographic data are:

- Photogrammetry; and
- Airborne Laser Scanning.

A general description of photogrammetry is that it is the science of making measurements from photographs; in the context being used herein, it is the science of making measurements about the topographic surface of catchments. Generally, topographic data obtained through photogrammetry consists of spot elevations plus linear breaklines and the topographic surface is then determined from the TIN model through spot elevations. The breaklines are lines in the TIN that represents distinct interruptions in a surface slope, such as a ridge, road, or stream. No triangle in a TIN may cross a breakline (in other words, breaklines are enforced as triangle edges). Elevation levels along a breakline can be constant or variable.

ALS or LiDAR is a remote sensing technology that measures distance by illuminating a target (in this case, the ground surface) with a laser and analysing the reflected light. Raw ALS data consists of a dense cloud of spot elevations classified into ground and non-ground strikes. This raw ALS data usually has the non-ground strikes removed prior to provision. Analysis of the raw ALS data usually will result in a raster DEM.

It should be noted that neither photogrammetry nor ALS can penetrate water surfaces. Only the water surface level at the time of capture can be measured. If the bathymetry under the water surface is relevant in the context of the numerical modelling, bathymetric data must be collected and incorporated separately. Similarly, neither of the two methods can penetrate dense vegetation (such as trees, sugar cane and mangroves) to produce ground elevations. Hence, ground survey may be necessary to fill gaps in the topographic data under heavy vegetation.

It is important in a DEM to ensure key linear features such as levees, embankments and other infrastructure are adequately represented. These features can be incorporated into the topographic description using field data as breaklines.

The ANZLIC Committee on Surveying and Mapping have developed guidelines on the acquisition of LiDAR (ANZLIC, 2008) in terms of accuracy, data formats and meta-data. They have also developed the National Elevation Data Framework.

Further discussion of these two aerial survey techniques is provided in the following sections.

4.13.3.4.1. Photogrammetry

Photogrammetry is a measurement technique where the three dimensional (x,y,z) coordinates of an object are determined by measurements made from a stereo image consisting of two (or more) photographs; usually, these photographs are taken from different passes of an aerial photography flight. In this technique, the common points are identified on each image. A line of sight (or ray) can be built from the camera location to the point on the object. It is the intersection of its rays (triangulation) that determines the relative 3D position of the point as the known control points can be used to give these relative positions absolute values. More sophisticated algorithms can exploit other information on the scene known as priori.

The accuracy of the photogrammetric data is a function of flying height, scale of the photography and the number and density of control points. Typically, the accuracy requested when scoping photogrammetric data collection for flood study purposes ranges from +/- 0.1 m to +/- 0.3 m. Note that the accuracy of the developed design flood profile cannot have an accuracy better than the catchment data used to estimate the design flood profile.

As the technique is based upon the comparison of photographic images, shading and obscuring of the ground surface by vegetation can reduce coverage in specific areas.

However, as photogrammetric analysis can utilise manual inspection of the stereo pair of photographs, the photogrammetrist is sometimes able to pick the odd ground surface visible through tree or crop cover. In this way, photogrammetry is sometimes able to provide some reliable points in vegetated areas.

Shown in Figure 1.4.9 is a region over which photogrammetric data coverage and ALS data coverage will be demonstrated. Shown in Figure 1.4.10 is a typical sample of the raw data obtained from a photogrammetric technique while shown in Figure 1.4.11 is the raw data in finer detail for the area indicated in Figure 1.4.10. These figures demonstrate the following specifically in relation to photogrammetry:

- The measured points are well spaced, but not always on a grid. Some manual manipulation has occurred in locating these points when necessary.
- Breaklines (seen as intervals along which measured elevations form the vertices of the interval) are evident along tops and bottom of banks. These are most likely to have been manually derived by viewing the stereo pair of photographs.

Figure 1.4.9. Aerial Photograph Example (Region A)

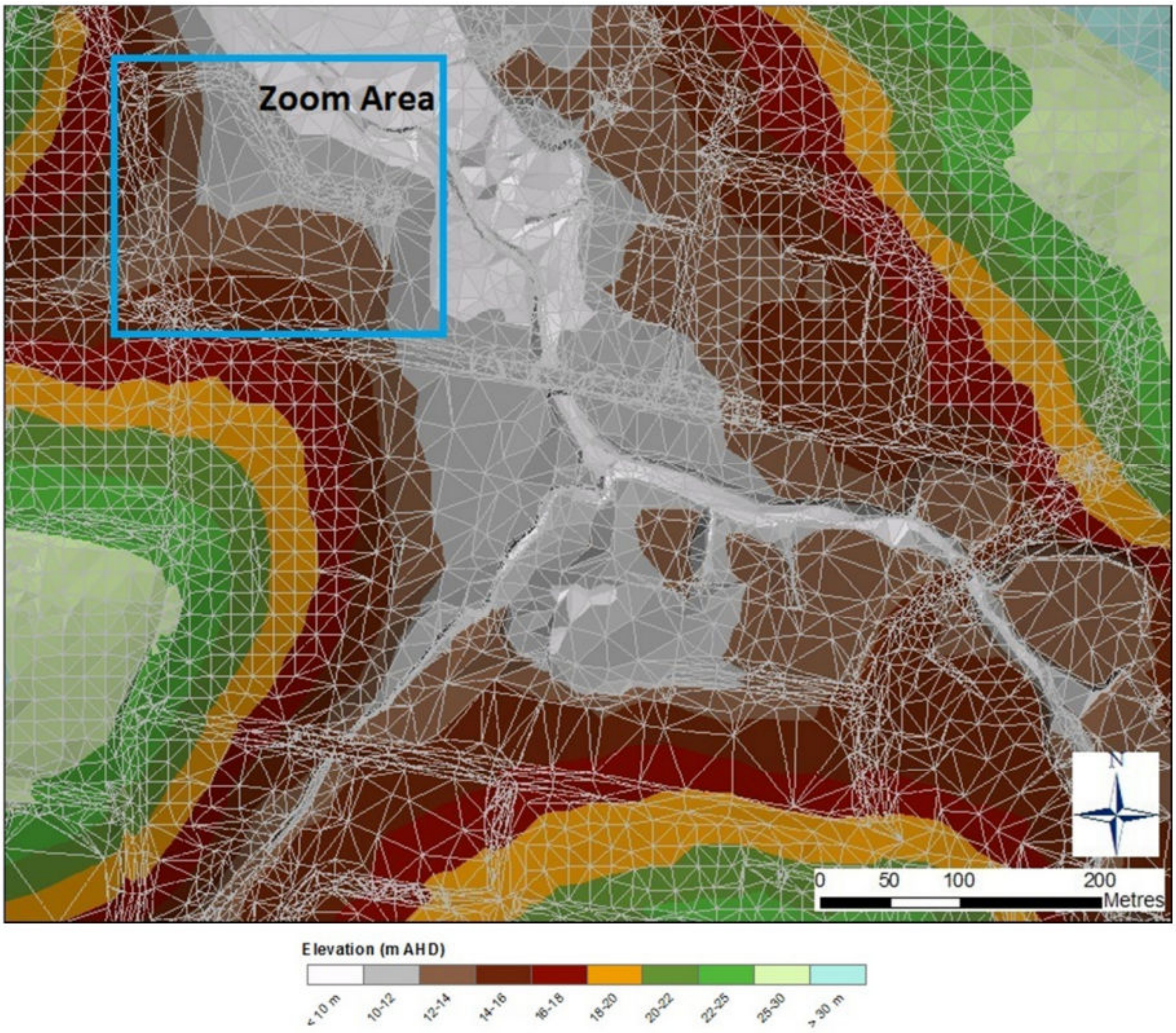


Figure 1.4.10. Sample of Processed Photogrammetry data set (Region A)

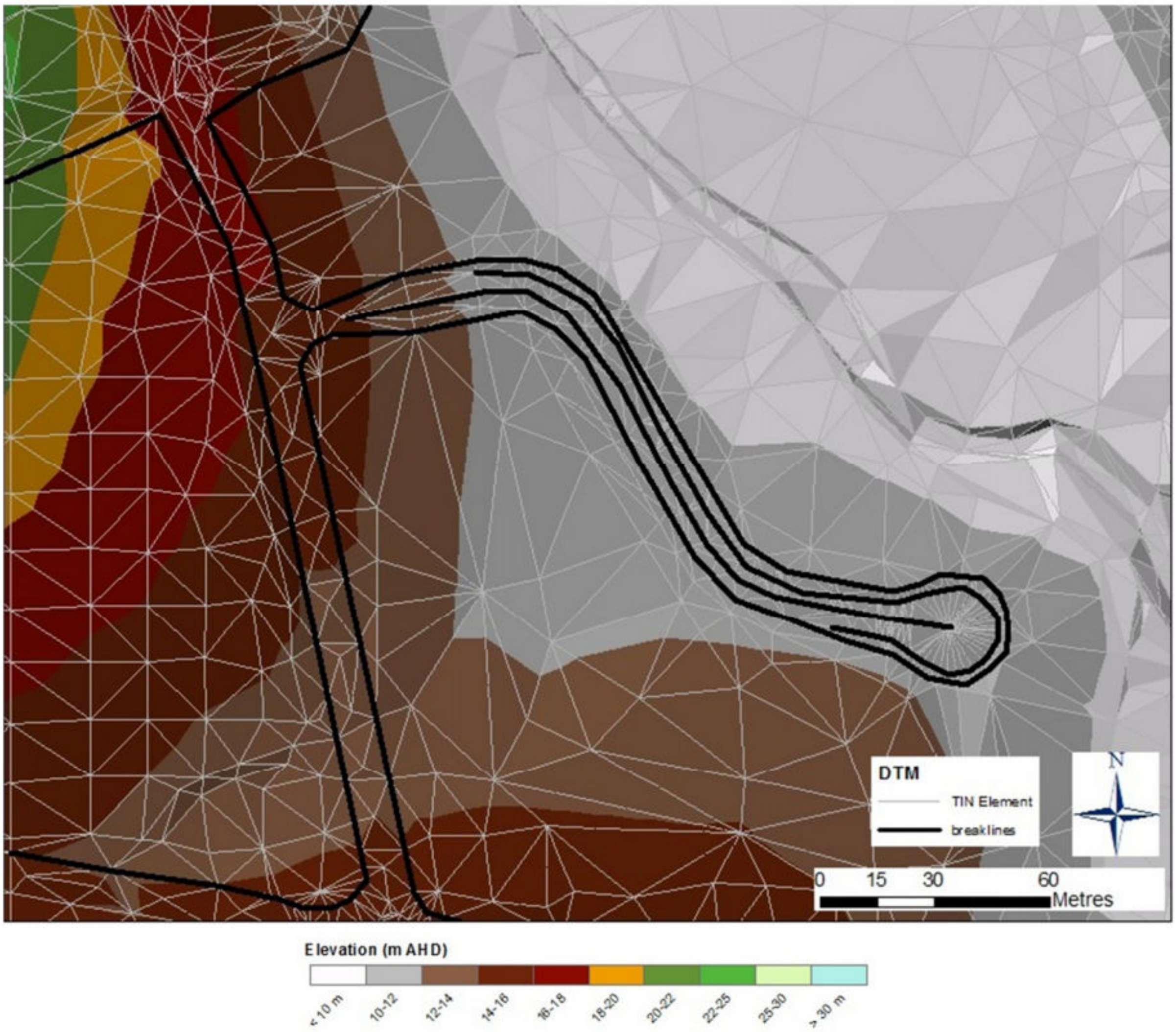


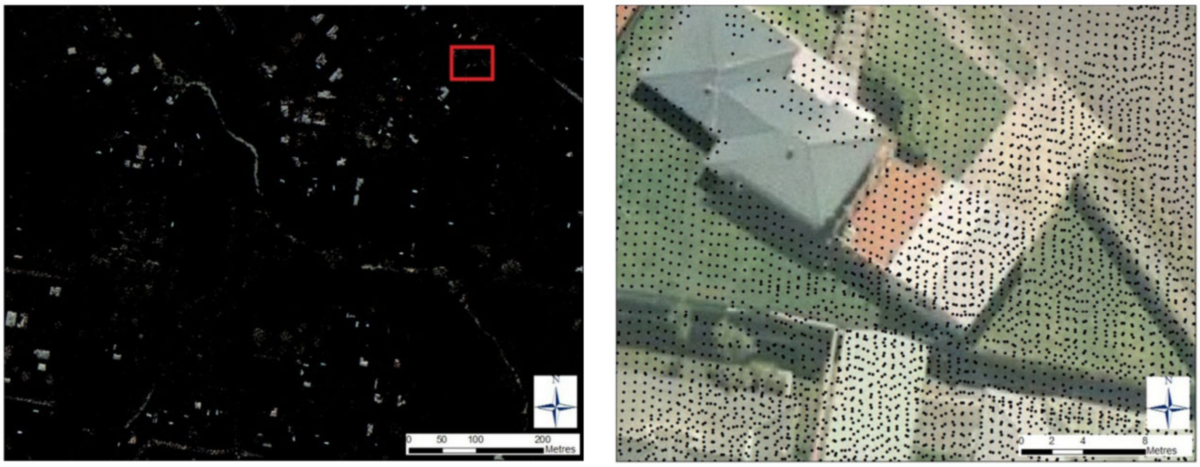
Figure 1.4.11. Sample of Processed Photogrammetry data set (Region A detail)

Photogrammetry is also often used to develop contours of the land surface directly as polylines with an attributed elevation; these contours are then used to create the desired elevation data set.

4.13.3.4.2. Airborne Laser Scanning

ALS or LiDAR consists of a high frequency laser emitter and scanner, coupled with a GPS and an Inertial Measurement Unit (IMU), all mounted in fixed winged aircraft. Rapid pulses of light are fired toward the earth by the laser instrument. These light pulses rebound from a target and are sensed. The scanner records the time differential between the emission of the laser pulses and the reception of the return signal. The time taken is used to determine the distance between the emitter and the target. The position and orientation of the scanner is determined using differential kinematic GPS and the IMU to account for aircraft pitch.

ALS produces a dense cloud of points (See Figure 1.4.12). These points can be classified as ground or non-ground points. While ALS requires little ground control in acquisition, ground control is important for quality control of the ALS measurements. For example, while it may be easy to scan inaccessible or sensitive areas without ground survey, the accuracy and reliability of the collected data may be low; in other words, ground control is important for ensuring the accuracy and reliability of the ALS measurements.



(a) (b) Zoom area indicated by red square in (a)

Figure 1.4.12. Sample of Raw ALS data set (Region A)

The vertical and horizontal accuracy of ground surface level measurement by ALS is a function of the laser specification, flying height, ground control and the surface coverage. Hard road surfaces, for example, normally are able to be measured accurately, but other surface types (for example, swamps or heavily vegetated areas) are not easy to measure and hence the measurements must be treated with caution. It is useful to note that many quoted accuracy values for ALS data are in reference to the data accuracy for clear hard ground. For clear, hard ground (that is ground with no surface coverage), the nominal accuracy for technology commonly applied in Australia is:

- Horizontal accuracy:
  - 1/3000 x altitude at which the aeroplane is flown; for a flying height of 1000 m, the horizontal accuracy is about +/-0.33 m; and
- Elevation accuracy:
  - < +/-0.15 m @ 1100 m flying height
  - < +/-0.25 m @ 2000 m flying height
  - < +/-0.4 m @ 3000 m flying height

The width of the land terrain sampling per pass, commonly referred to as the swath varies with the flying height. While typical values are given in Table 1.4.2, users are advised to obtain the meta-data regarding their ALS to ensure suitability for purpose.

Table 1.4.2. Typical Swath Values

Typical Swath (m)	Altitude (m)
800	1100
1456	2000
2184	3000

A disadvantage of the ALS data capture method (compared to, for example, low-level photogrammetry) is the absence of breaklines in the data to define distinct, continuous topographic features and significant changes in grade. While the horizontal density of points usually is quite high (average point spacing of 1 to 2 metres depending on flying height and sampling frequency), features such as narrow banks/levees or channels will only be resolved if the data are sampled on a very small grid (less than 1 to 2 m grid). This can result in large and unwieldy terrain files.

There are a number of approaches that can be taken in relation to the treatment of breaklines in ALS data sets:

1. Sample the entire survey area at a fine resolution - say on a regularly spaced 1 m Digital Elevation Model (DEM) grid and manually identify important salient topographic features and hand enter these features into the models;

2. Use local knowledge, GIS, aerial photos, satellite imagery or historic plans to identify locations of important features and hand-digitise over the fine resolution DEM in a manner similar to the first approach (approach 1) and then drape values from the DEM to develop 3D breakline strings;

3. Use observations as in approach 2 to determine locations of key features and then use field survey to develop 3D breakline strings; and

4. Use auto-processing/filtering algorithms to extract breaklines from the raw ALS data.

Experienced users favour a combination of technique approaches (2), (3) and (4). While approach (4) nominally provides the widest coverage and extracts the most information from the ALS data, the processes cannot be considered reliable as no method has been developed for testing the validity of the breaklines produced.

Although requiring a greater manual input, it is considered that approaches (2) and (3) are better approaches for the provision of reliable estimates of the surface level at critical locations with in the floodplain. This arises from the manual checking that occurs during the progress of approaches (2) and (3). Furthermore, long-sections from the ALS can be checked for consistency and a sub-sample tested against field measurements as a validation process.

Capturing ALS data results in surface level estimates from various ground coverages including bare earth, vegetation, and buildings. Thorough processing of this raw data is required to ensure a true representation of the ground surface is obtained.

Raw ALS data files contain all returns can be very large leading to difficulties with data storage and the manipulation of this data. As a result, a common approach is to use 'thinned' ALS data; this is data that has been processed to remove data points providing limited additional definition of the terrain surface. Shown in Figure 1.4.13 is an example of the processed ALS data set to produce the thinned ALS data set. Illustrated in this figure are the following aspects:

- The measured spot heights are provided as a grid; in other words, each spot height is representative of an area defined by the grid dimensions. This grid has been created by "thinning" the raw ALS point cloud data set. While the dimensions of the grid are defined by the user, it is useful to note that the smaller the grid, the larger and perhaps more unwieldy the data set being used but the surface topography will have an apparent higher definition. Conversely, the larger the grid, the smaller the data set but the surface topography will have an apparent lower definition. It is also worth noting that definition of the surface topography only needs to be adequate to provide the necessary information and that any additional definition will not provide either additional flood data or accuracy of the predicted flood data.

- There are no breaklines in the model of the surface topography. Breaklines can be created according to the approaches discussed previously but typically are not determined solely from the ALS data.

- Areas where there are no measurements and hence no points available to build the model of the surface topography. In these areas, the ground surface was not visible due to, for example, heavy vegetation or surface water. These areas can be removed during the processing as "non-ground" points. When using a processed ALS data set, there will be situations where a non-ground elevation point (for example, a point that has hit the canopy of trees) has not been removed during processing. As a result, the spot elevation remains in the data set and would be treated as a surface elevation. In some situations. Errors of this type are obvious and can be removed manually. In other situations, they may not be obvious and hence will form part of the DEM (Digital Elevation Model) used in the analysis. Users need to be cognisant of errors of this type in ALS data and the consequent significant impacts on the predictions obtained from the catchment modelling system.

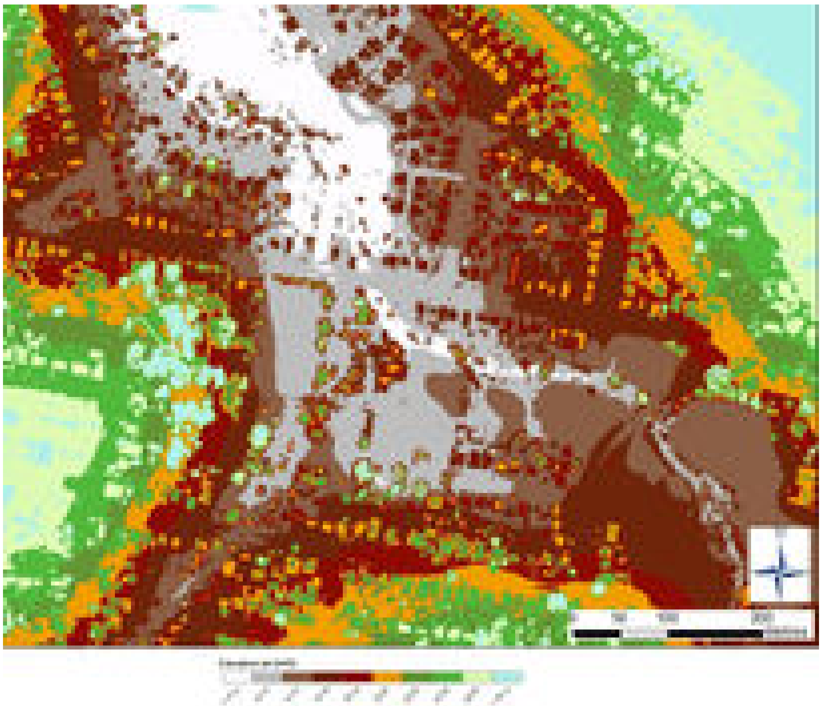


Figure 1.4.13. Sample of Processed ALS data set (Region A)

4.13.3.4.3. Aerial Survey Quality Checks

As discussed in Book 1, Chapter 4, Section 13, survey data is not exact and will have a tolerance or a stated level of accuracy. Data obtained from aerial survey is the same. With data obtained from aerial survey, accuracy is dependent upon a number of factors, including flying height and the number of control points. In general, the required accuracy is advised prior to commencement of data collection and the surveyor designs the data collection program needed to achieve the desired accuracy.

An important point to note is that the accuracy of the collected data is checked and confirmed. The resultant stated accuracy typically does not apply to the DTM derived from the survey data but rather to the individual elevation points sampled during the data generation program. This is an important distinction and users of a DTM derived from aerial survey data needs to be aware of this feature.

There are a number of methods that may be used to check the accuracy of the aerial survey.

1. POINT v DTM SURFACE: Independent field surveys of selected quality check points can be compared to the DTM. However, this surface has been interpolated from the aerial elevation data and, unless the accuracy stated by the aerial surveyor referred to the DEM, the correlation in accuracy values is not guaranteed. But, this method can provide a preliminary indication of accuracy, particularly if the independent field survey points with known accuracy are already available from another source;

2. POINT v POINT: Independent field surveys of selected quality check points can be compared to the individual aerial data elevation points. The selected check points must exactly match the coordinates of the aerial data points to ensure that a valid comparison is being made. To do this, the aerial survey must first be received in order to select the points at which comparison will be made, which may slow the data collection phase. Alternatively, early liaison with the aerial surveyor may allow the location of a number of points to be known before data provision, which may save time; and

3. STRING v DTM SURFACE: A more appropriate approach may be to field survey a number of breakline strings along key linear features. These strings may be some 50 m - 200 m in length with points at regular spacing (say every 5 -15 m). A comparison can then be made of the profiles along the feature determined from both the aerial and field survey. In addition, if the strings are across conveyance paths (i.e. they are cross-sections), the modeller can check that conveyance cross-sectional areas are adequately represented. These comparisons enable a qualitative assessment of aerial survey accuracy for a given region within the study area. A number of surveyed strings may be required across the study area to gain an overall appreciation of accuracy.

4.13.3.5. National Digital Elevation Model

National grids of terrain information are available from Geoscience Australia. These data sets use information obtained from a variety of sources inclusive of satellites. As a result, these data sets tend to be coarse with grid sizes varying from 1 second (1 seconds in longitude and latitude is approximately 30 m) to 9 seconds (9 seconds in longitude and latitude is approximately 250 metres). As development of these databases occur, it is likely that these specifications will change. Therefore, as with other sources of data, users should ensure that they use the most appropriate data for their problem.

4.13.3.6. Bathymetric (Underwater) Techniques

Many methods for generating surface data are not applicable for collecting bathymetric data (ground data below the water surface) in permanent or semi-permanent water bodies. Where a water body has not been surveyed adequately, a specific survey will be required to supplement the ground surface data.

If the water body is shallow or small, then a traditional surface survey technique may be suitable. For deeper, larger water bodies, a specialised bathymetric survey may be required. Instruments such as echo sounders, side scan sonar systems and acoustic doppler profilers may be used for this purpose.

In most cases, the bathymetric survey will need to be merged with ground surface data.

While bathymetric surveys have been conducted for most major rivers, if the data is not recent and the riverbed is subject to change then the data should be checked for suitability prior to its use.

4.13.3.7. Aerial Photographs

Aerial photographs are an important source of qualitative data and can be collected during an aerial survey. Geo-referenced (or ortho-rectified) aerial photos can be supplied as part of a photogrammetric survey. These geo-referenced aerial photographs are aerial photos where spatial coordinates have been added to locate the photograph. It is important to recognise that the raw aerial photograph is spatially distorted, being a planar image of a curved surface of variable height. In ortho-rectifying the image, the image is scaled, rotated and stretched so that various reference locations move to their correct coordinate locations.

A consequence of this is the location of features on an aerial photograph will have a degree of uncertainty. For example, if a rectified aerial photograph is used to locate a flood mark, the attributed location will be subject to a tolerance. In an area of high flood gradient this can result in differences between observed and simulated flood levels that do not accurately represent the true differences.

Aerial photographs, while not providing quantitative data directly, can provide additional information about flowpaths, flow obstacles and floodplain vegetation that is not always immediately evident or accessible on a site inspection. Additionally, aerial photos can be a useful guide when defining parameters for floodplain characteristics (for example, roughness coefficients) and can be used to develop a spatial map of the floodplain parameter.

Another example of the use of aerial photography is its use in urban areas to define building outlines or fence lines where these are to be included within the hydraulic model and can be a reasonable source of information for assessing the total imperviousness of a catchment (see Book 5, Chapter 4).

Finally, when historical aerial photography is available, it is useful in assessing catchment development or sourcing information on the floodplain development when historical events occurred.

4.13.3.8. Historical Topography and Infrastructure

All data collection methods covered thus far have been concerned with present day catchment conditions. However, when catchment modelling systems are used for design flood estimation, calibration to historical events is required and the catchment and floodplain conditions at the time of the historical event need to be considered particularly as these conditions may not be the same as present day conditions. In addition, if a number of events are to be used during the calibration process, changes to catchment conditions may occur between events.

Conditions at each of the relevant historical points in time must be established and used in the model development; this is discussed in more detail in Book 7. Changes to conditions that may affect flood behaviour include dam construction, initial dam storage levels, dredging or siltation of river channels and particularly of river mouths, construction of levees and other associated flood mitigation works, road construction including the raising or duplication of roads, the realignment of road embankments, the construction of new culverts and/or bridges, upgraded drainage networks both in rural and urban environments, developments on the floodplain, the construction of new weirs or the removal of old weirs, different crop types or stage in the growing season, and others that have not been mentioned here. Depending upon the length of time since the occurrence of the calibration event, record of these changes may be only available anecdotally.

The availability of data for historical events needs to be considered when the event is used as part of the design flood estimation. For example, there may be anecdotal or even good formal measurement evidence of a record flood that occurred 100 years ago but details of this flood event may not be adequate for its use as a calibration event for validation of a catchment modelling system. On the other hand, the data may be adequate for it to be included as a high discharge censored event in a flood frequency analysis (Book 3, Chapter 2).

4.13.3.9. Land Use Information

Land use data is important for several aspects of projects, and can be obtained from land use maps, field observations or consultation with local authorities, land managers or property owners. Local authorities are often a valuable source of land use data.

Land use data is used in hydrology models to determine suitable parameters to calculate runoff and is also used in hydraulic models to assist in the determination of Manning's n values.

Land use data is normally supplied as a map or a GIS layer. When obtained from local authorities, the data is usually supplied on request for no charge.

Information on land use can be used in the hydrologic model to determine percentage impervious or in hydraulic models to inform hydraulic roughness. Land use information may be sourced from:

- Local or State Government Authorities in spatial layers of existing development zonings;
- Local or State Government Authorities in spatial layers of future development zonings; and
- Inferred from Aerial photographs (current and historical).

4.13.3.10. Vegetation Data

Information on vegetation type can be used in the hydrologic model to determine runoff characteristics or in hydraulic models to inform hydraulic roughness values (Manning's n). This data may be sourced from:

- Vegetation maps;
- Field inspections; and
- Inferred from Aerial photographs.

Care needs to be taken with vegetation maps as, in general, the maps are based on limited sampling and inferring the results of this survey to be the representative of a larger area. Additionally, the individual species within an area designated as one vegetation type may vary.

4.13.3.11. Bureau of Meteorology Geofabric

The Bureau of Meteorology Australian Hydrological Geospatial Fabric (Geofabric[1]) consists of a number of GIS layers which include hydrological features such as rivers, water bodies, aquifers, and catchments. The current geofabric includes (Bureau of Meteorology, 2015):

- Geofabric Surface Cartography - Cartographic representation of surface hydrological features;

[1]http://www.bom.gov.au/water/geofabric/index.shtml

- Geofabric Surface Network - Network representation of hydrological features;
- Geofabric Surface Catchments - Catchment boundaries derived from the 9 second Digital Elevation Model;
- Geofabric Groundwater Cartography - Cartographic representation of groundwater hydrology features;
- Geofabric Hydrology Reporting Catchments- Contracted nodes, contracted catchments and node-link network; and
- Geofabric Hydrology Reporting Regions - Reporting regions based on aggregations of contracted catchments.

It is worth noting that the catchment area is a function of the scale at it was estimated and therefore is likely to have inaccuracies at a fine scale. The current data is based on a 9 Second DEM and GA GEODATA TOPO 250K Series 1 (GEODATA 1) and GEODATA TOPO 250 K Series 3 (GEODATA 3).

Subsequent versions of the Geofabric will have upgrades to data and include (Bureau of Meteorology, 2015):

- hydrometric monitoring features;
- more detailed surface water hydrology; and
- Digital Elevation Model (DEM) derived streams and catchment boundaries based on a 1 second resolution DEM.

4.13.3.12. Soil Data

Some hydrologic models require information on the catchment soil properties (for example, information on the A and B horizon depths and their water holding capacity, or the soil type) to estimate losses from the rainfall.

Soil property data is available spatially for the whole country from the Atlas of Australian Soils (McKenzie et al., 2000); this information is available in a GIS format from the Australian Soil Resource Information System website. These maps are broad scale (typically 1:250000 - 1:500000) and were completed between 1960 and 1968. State based maps are available, also. Care should be taken when using soil maps as variations in soil over short distances occur frequently and cannot be resolved by the reconnaissance style mapping used in their development (McKenzie et al., 2000).

While it is possible to estimate land subject to inundation by floods through consideration of the soils and geomorphology, this does not provide any guidance on the likelihood of the flood hazard and therefore can be misleading. Furthermore, there is a need to ensure that the soil and geomorphoric data is obtained at a fine scale to ensure spatial variations over short distances are adequately recognised when using soil information to assess potential flood hazard.

4.13.3.13. Property Data

In order to assess the magnitude of the flood hazard to people and property, property data (including building type, condition and floor level) typically are required.

Property databases form the basis of most flood damage assessments. These databases typically require a description of the property attributes and features on a property by property basis. Typical information required for each residential property includes:

- Street address;
- Representative ground level;
- Habitable Floor levels;
- Building construction type (e.g. brick veneer, timber, slab on ground, on piers etc);
- Building age;
- Single/double storey; and
- House size.

Commercial and industrial properties require similar information, but also require information on the type of business undertaken at the site as this can have a significant bearing on the value of flood damages from business to business.

Ideally, this data are collected via field survey. However, it can be a costly process depending upon the number of properties for which data are required. Alternatively, there may sometimes be records available from the local authority, other government agency or the census. For broad assessments, property data may be estimated. A panel of people with relevant skills should review the method of estimation for soundness. As an example, property data may be estimated from aerial photography or from a general understanding of local conditions.

A number of national data sets are also available from Geoscience Australia such as:

- Australian Flood Risk Information Portal (Geoscience Australia) will be a central depository for information on flood studies conducted throughout the country and associated spatial data;
- Water Observations from Space historical surface water observations derived from satellite imagery for all of Australia for the period of 1998 to 2012; and
- State borders, city locations, topographic maps.

4.14. Other Data Considerations

4.14.1. Storage of Data and Meta-data

Most large data sets on a project are produced by combining multiple sources of information. Most large data sets are too big to be checked and must be machine quality controlled. It is important that meta-data associated with quality checking is recorded to assist future users of the data set.

4.14.2. Co-ordinate Systems and Datums

Most large national data sets are in latitude and longitude. Smaller data sets are in Map Grid of Australia (MGA 94) which is Universal Transverse Mercator projection and the Geocentric Datum of Australia 1994. Some older data sets may use ISG - Integrated Survey Grid or

AMG - Australian Map Grid. Care should be taken when translating from one projection to another; of particular concern is the use of the correct local conversion as these conversions are not the same across the country.

4.15. Other Hydrological Data

4.15.1. Tidal Data

In many coastal areas and areas adjacent to coasts, ocean and tidal data can be an important component of the design flood estimation process. Tidal data may be collected by manual observations or by automatic recorders and needs to include astronomical tides as well as storm surge and long-term trends in sea levels. In some circumstances, wave data may also be relevant.

Historical tidal data for particular events can be useful model calibration while long-term records can be used for statistical analysis for design flood estimation purposes (refer to Book 6, Chapter 5). With increased sea levels induced by global warming, it is likely that long-term records of tides and sea levels will exhibit non-stationary behaviour; refer to Book 1, Chapter 6[2] .

Tidal data is collected regularly by relevant government agencies, being concerned for the coastal environment or engineering. This data is published in handbooks or websites. In addition, there are research or other short-term projects carried out in coastal areas, which may include data on tides. However, these projects are generally localised and of short duration.

4.15.2. Meteorological Data

As well as rainfall and other precipitation, other meteorological data is used in water resources studies. This data is used to assess soil moisture and evapotranspiration for example. This data includes pan evaporation, temperature, humidity, wind speed and other parameters.

The Bureau of Meteorology is the principal agency that collects this data, however there are records held by water agencies and agricultural departments as well as small localised records held by different organisations. Regional maps of key meteorological data, especially pan evaporation and evapotranspiration are published by the Bureau of Meteorology, and this regional information is often adequate for many requirements.

Meteorological data is usually available free of charge from the Bureau of Meteorology and major agencies.The records from other organisations may be difficult to locate and then there may be contractual difficulties in obtaining and using the data.

4.15.3. Sediment Movement and Deposition

Sediment movement, including scour and deposition, is one of the most important water quality impacts of drainage systems, both natural and man-made and can cause environmental problems in downstream receiving waters as well as damage and disruption to drainage systems.

Data collection on sediment movement is particularly difficult and there is only limited available data. Most routine data collection programmes are carried out by water and

[2] This section was written before the latest climate change guidance in Book 1, Chapter 6 (2024). A minor change to the text has been made to reflect the change in guidance.

environmental agencies, but these are usually somewhat limited. Tthere are some small specialised programmes carried out for specific projects, often as part of an environmental impact study. However these programmes are generally limited in scope and also limited in the duration for the data collection programme.

This data is normally available only directly from the agency or organisation that collects the data, and may be difficult to find that it exists and then it may be difficult to obtain.

Once data is located, it is then often difficult to access and use because of differences in the methods adopted for collection, analysis and processing. There are also differences in the treatment of bed and suspended loads and measurements of turbidity, all of which are used at times to measure sediment movement.

Sediment deposition may be monitored by owners of affected assets, but the data is difficult to apply in investigations.

Therefore application of sediment movement and deposition data is difficult and needs considerable skill to interpret and apply. Where this is an important aspect of a project, efforts should be exerted to find and use the data.

4.15.4. Water Quality

As well as sediment, there are many other water quality parameters that are relevant to water resources and drainage programmes. The water quality parameters that can be monitored cover a wide range from the relatively routine such as nutrients and salinity to quite specialised contaminants.

As with sediment, data collection on this topic is particularly difficult and there is only limited available data. Most data collection programmes are carried out by water and environmental agencies, and some of this data (especially the routine parameters, such as salinity) is available in formal data archiving systems. In addition to these programmes, there are some small specialised programmes carried out for specific projects, often as part of an environmental impact study.

Some types of water quality data, especially salinity and nutrients are available as historical records that can be used to calibrate models and assess changes in conditions with time, but much of the data is short term and variable. Considerable skill and expertise is needed to apply this data to project requirements.

4.16. Climate Change Data

In this section, data available to consider the impacts of climate change on estimation of design rainfalls and floods is described. As further research and development of climate change data occurs, the discussion and guidance presented here will change. Practitioners, therefore, should ensure that they are aware of these changes and the impact on available data.

4.16.1. Types of Climate Change Data

Quantifying the effects of climate change on the factors that affect flood estimation is a difficult task, and any estimates of impacts of future climate on the inputs to flood assessments will include large uncertainties. The fact that the occurrence of flood events, and their associated causal factors, is rare limits the data available to assess changes in their frequency or intensity (IPCC, 2012).

Observed data is often used to investigate whether there are any trends apparent in historical flood data. The data used to project the impacts of future climate on flooding is generally sourced from climate modelling. “Any useful technique for the assessment of future risk should combine our knowledge of the present, our best estimate of how the world will change, and the uncertainty in both” (Hunter, 2007). To study the impact of climate change, a plausible and consistent description of a possible future climate is required. The construction of such climate change scenarios relies mainly on results from model projections, although some information from past climates can be used (IPCC, 2001). Refer to Book 1, Chapter 6 for more information on climate change and flood estimates.

4.16.1.1. Observed Data

Stationarity is one of the fundamental assumptions in traditional design flood estimation. Climate change challenges this assumption as observed historical rainfall and flow data may not be a good indicator of future conditions. This has implications for the use of historical data in assessment of flood risk including in estimation of design rainfalls, flood frequency analysis, sea-level and storm surge, and estimates of design flood model parameters that account for losses.

Detecting changes in the frequency or intensity of precipitation or flood events in recorded data presents a number of difficulties (Jones et al., 2012; Milly et al., 2008). The ability to assess climate-driven observed changes in the magnitude and frequency of floods at regional scales is limited by the lack of observed records and their coverage in space and time, and by changes in catchments due to land-use and development (IPCC, 2012). Attributing trends in discharge data to climate change is particularly difficult, as changes in catchment conditions, or river operations can contribute to trends in the data (Bates et al., 2008). Long-term records are needed to be able to detect trends in data, and the availability of consistent, quality controlled data is a major limitation in any study to detect trends in large to extreme rainfall or flood data (Bates et al., 2008). There is evidence that climate change will result in a larger increase in extreme sub-daily rainfalls than at a longer duration (Westra, 2011). The availability of long records of sub-daily rainfall data is limited, with an average length of Australian sub-daily rainfall stations of approximately 19 years compared to an average length of 65 years for daily rainfall stations (Johnson et al., 2012).

One of the most fundamental issues in detecting trends in precipitation or discharge data due to climate change is separating the influences of climate variability from long-term climate change in a relatively short record. Due to the normal range of climate variability, there is limited information available to establish the probability of a flood event currently, even without consideration of climate change (White et al., 2010). Local and regional changes in precipitation due to climate change are greatly affected by patterns of atmospheric circulation. Patterns of change in precipitation associated with ENSO can dominate the global patterns of variations, particularly in the tropics and over much of the mid-latitudes (Trenberth, 2011). There is little observed data available to investigate relationships between hemispheric scale modes of the atmosphere (such as ENSO) and climate change.

The observed data available to directly estimate the impacts of climate change on antecedent conditions includes seasonal rainfalls, evapotranspiration, and soil moisture. Detecting changes in these parameters can be undertaken with a trend analysis, however long records with appropriate spatial coverage are required for this task. As for detecting changes in extreme precipitation or discharge events, the difficulty is in separating the impacts of climate change from natural variability or the influence of changes in catchment conditions, in the relatively short records available. The coverage of directly measured evapotranspiration data is relatively sparse across Australia and is very limited globally (Bates et al., 2008). Evapotranspiration data available from global analysis data is sensitive to the type of analysis and the uncertainty in the data makes it unsuitable for trend analysis (Bates et al., 2008). Direct measurements of soil moisture are available for only a few regions and are often very short in duration (Bates et al., 2008).

Changes in storm surge events can be investigated using data from tide gauge records. Tide gauge data can be used to evaluate the Annual Exceedance Probabilities of extreme sea levels, however a reliable analysis of the risk of extreme events or trends in the data is limited by the short duration of records collected at many gauges. Church et al. (2006) found only two gauges of sufficient length for use in this type of analysis in Australia, and only nineteen records with lengths of 40 years or greater (and some of these were intermittent). The limited number of tide gauges means that there is no data available for large stretches of coastline, which inhibits the assessment of this hazard even under present climate conditions, let alone future conditions due to climate change (McInnes et al, 2007).

4.16.1.2. Climate Modelling

- **Global scale modelling**

Whilst observed historical data can be used to investigate trends, Global Climate Models (GCMs) are most often used to generate data to investigate the impacts of climate change into the future on a global or continental scale.

Climate models are mathematical representations of the climate system, expressed as computer codes and run on computers (IPCC, 2007). The models would be too complex to run on any existing computer if all variables in the climate system were explicitly included in the models, so simplifications are made so that the system has reduced complexity and computing requirements (IPCC, 2001). Outputs from GCMs cover many variables that impact the hydrologic cycle including precipitation, evaporation, soil moisture and sea level. GCMs have been developed by a range of international agencies. The Intergovernmental Panel on Climate Change's (IPCC) Fourth Assessment Report (2007) used climate modelling output from 23 different GCMs as the basis of their global assessment of climate change (IPCC, 2007). GCMs differ in their representations of climatology and thus using an ensemble including a range of GCMs can enhance the representation of specific weather patterns (Abbs and Rafter, 2009; Grose et al., 2010).

In order to address the uncertainty in future greenhouse gas emissions, a range of plausible futures are often run with the GCMs. IPCC (2007) developed a range of emissions scenarios (SRES emissions scenarios) that are commonly used with GCMs. The SRES emissions scenarios are divided into six families based on different likely emissions considering future technological and societal changes (Corney et al., 2010). The current GCM results for Australia can be accessed through the Climate Change in Australia website (http://www.climatechangeinaustralia.gov.au/en/) and the Climate Futures web tool (http://www.climatechangeinaustralia.gov.au/en/climate-projections/climate-futures-tool/introduction-climate-futures/).

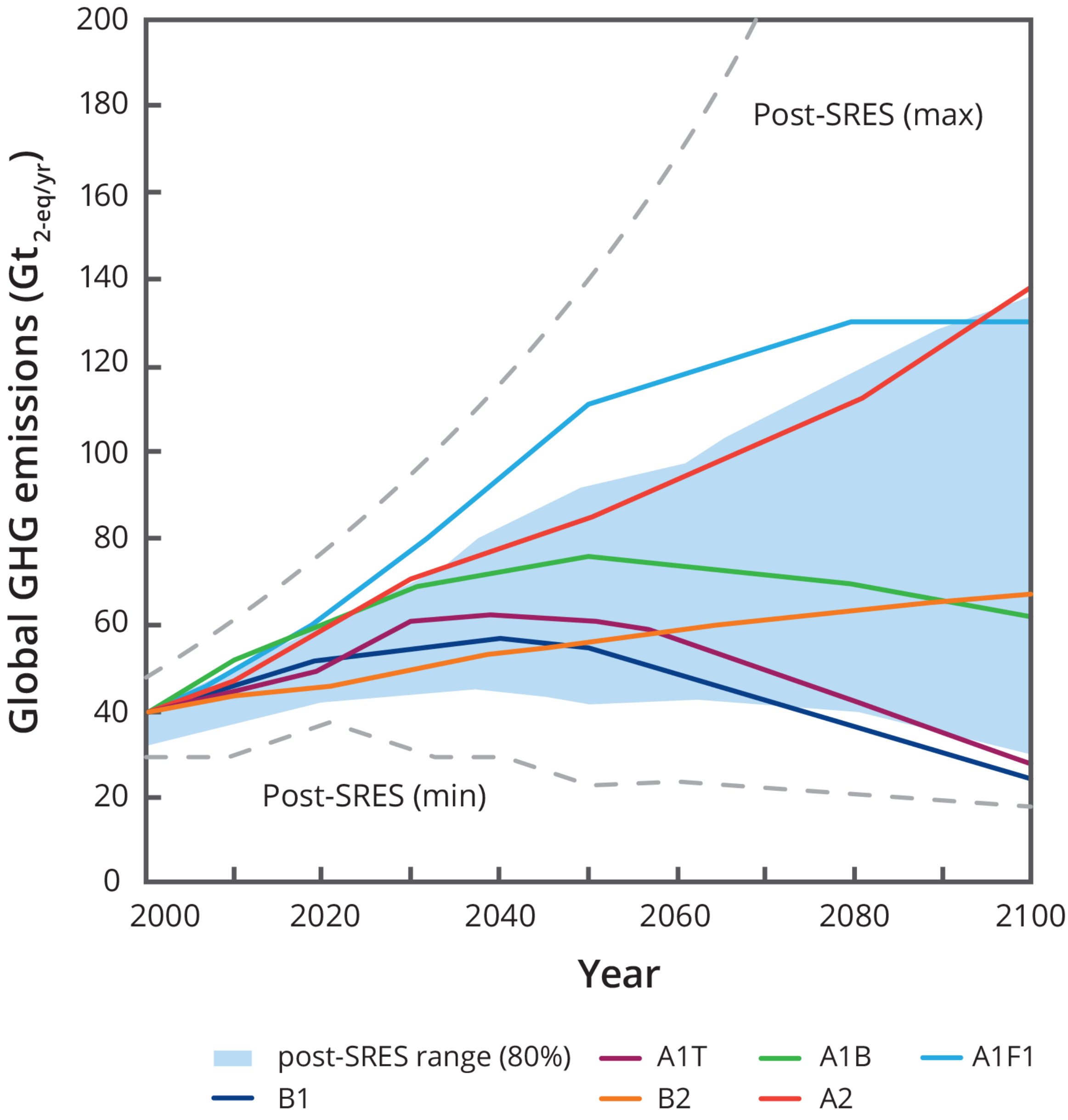


Figure 1.4.14. Global Greenhouse Gas Emissions Scenarios for the 21st Century (from IPCC, 2007)

- **Confidence in GCM data**

The fact that climate model fundamentals are based on established physical laws, such as conservation of mass, energy and momentum, along with a wealth of observations gives some confidence in the ability of models to represent the global climate. The models have shown a good ability to simulate important aspects of the current climate, and reproduce features of past climates and climate changes (IPCC, 2007).

Areas of uncertainty in the GCM projections include uncertainty in future levels of greenhouse gas emissions, the response of the climate to the emissions, and changes in regional climate (CSIRO, 2012). The cascade of uncertainties in projections is shown in Figure 1.4.15. The uncertainty in the levels of emissions results from a lack of knowledge of the future social, economic and technological development of the world, and the associated greenhouse-gas emissions. The model uncertainty is due to deficiencies in the knowledge of the science of climate change, in setting initial conditions for the models, and in the representation of the global climate by the models (Hunter, 2007). There are also deficiencies in the simulation of tropical precipitation, the El Niño- Southern Oscillation and the Madden-Julian Oscillation. Most of these errors are due to the fact that many important small-scale processes cannot be represented explicitly in GCMs, and so must be included in approximate form as they interact with larger-scale features. This is partly due to limitations in computing power, but also results from limitations in scientific understanding or in the availability of detailed observations of some physical processes (IPCC, 2007).

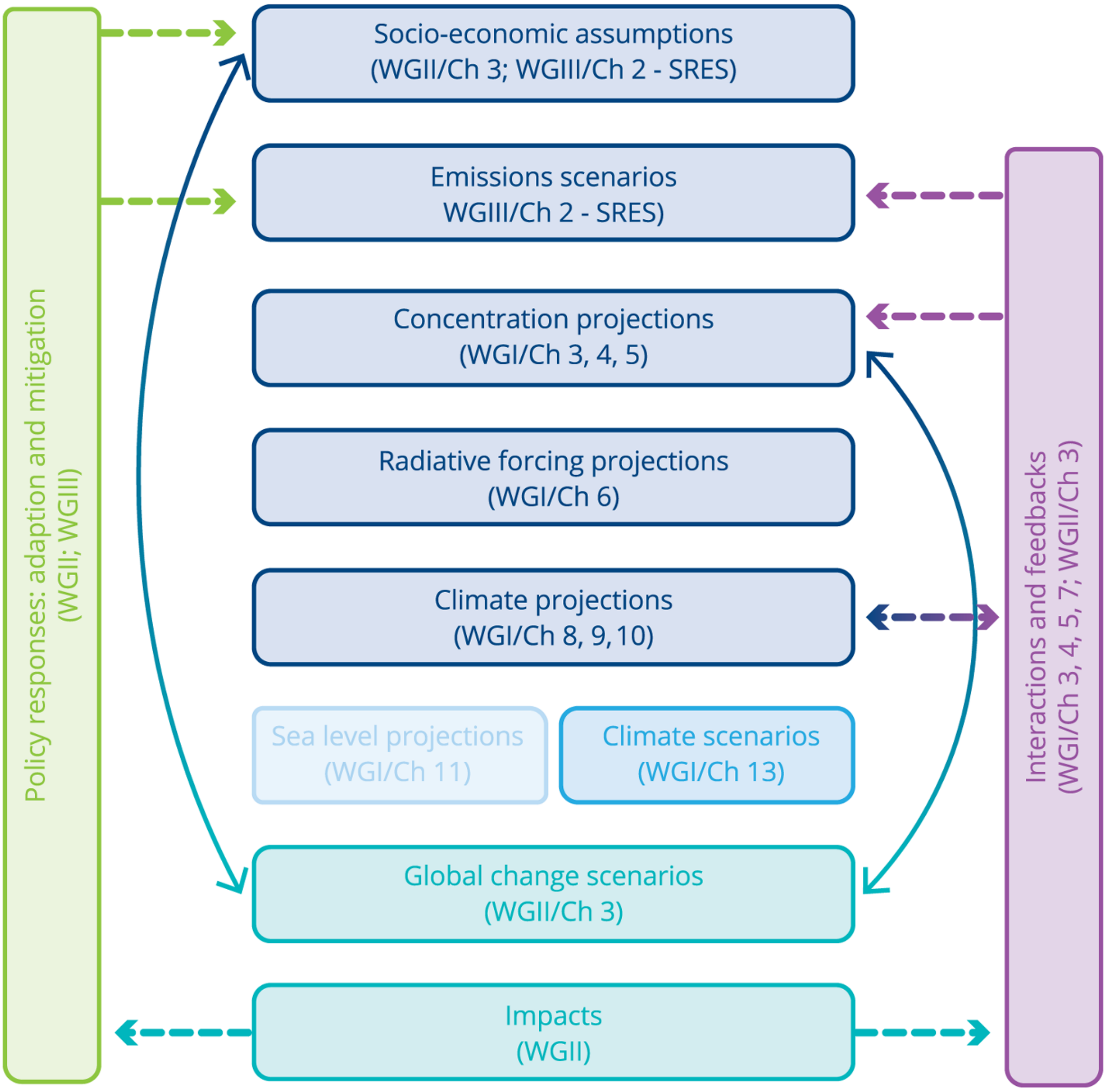


Figure 1.4.15. From (IPCC, 2001)

- **Regional Scale Modelling**

The outputs from GCMs give information at a global scale with a limited resolution, so cannot provide a detailed picture of climate variables at the regional scale required to investigate the factors influencing flood events (Corney et al., 2010). Some form of downscaling is required to investigate the impacts of future climate on specific variables at a local scale, in particular for precipitation and climate extremes. To address the decrease in confidence in the changes projected by global models at smaller scales, other techniques, including the use of regional climate models and downscaling methods, have been specifically developed for the study of local-scale climate change (IPCC, 2007). Three methods are commonly used to scale outputs from GCMs: temperature scaling, statistical downscaling, and dynamical downscaling (Westra, 2011). Combinations of these methods are also used.

- **Temperature Scaling**

Temperature scaling has been used give downscaled estimates of precipitation from GCMs. Extreme precipitation is directly related to the water holding capacity of the atmosphere. A warming climate leads to an increase in the water holding capacity of the air, which causes an increase in the atmospheric water vapour that supplies storms, resulting in more intense precipitation (Trenberth, 2011). The Casius-Clayperon relationship gives an increase of water holding capacity of approximately 7% per degree Celsius of warming (Trenberth, 2011). This relationship has been found to hold for some sub-daily rainfalls, however daily extreme rainfalls have been found increase at a lower rate (Lenderink and van Meijgaard, 2008). The relationship also appears to hold only to a threshold temperature (Hardwick-Jones et al., 2010). The simple scaling of rainfall with temperature does not reflect all the processes that produce rainfall events. The true scaling relationship is more complex and is affected by the extremity and duration of the rainfall event, the atmospheric temperature, and access to atmospheric moisture (Westra et al., 2013). This results in differing local impacts of climate change and, in particular, different impacts are seen dependent on the duration of the rainfall event.

- **Statistical Downscaling**

Statistical downscaling uses relationships between large-scale climate variables and local scale weather to develop estimates at a local scale. In the simulation of extreme rainfalls, a common approach is to use extreme value distributions to describe precipitation extremes (Abbs and Rafter, 2009). Another approach is to use a model to simulate precipitation and to then analyse the extremes (Mehrotra and Sharma, 2010). The advantage of statistical downscaling is that it is not computationally intense, and can be undertaken relatively quickly over large areas. The limitations of statistical downscaling include the assumption that the current observed relationships between large scale climate variables and local climate will persist in a changed climate regime. Another limitation is that the observational data set being used for the downscaling should cover the range of projected future climate responses (Grose et al., 2010).

- **Dynamical Downscaling**

Dynamical downscaling takes the outputs from a host GCM as inputs to either a limited area model or stretched grid global climate model. The result is a fine scale dynamical model over the area of interest, often called a Regional Climate Model (RCM). Because a RCM focuses on a small area, it can provide more detail over that area than is possible with a GCM alone (Grose et al., 2010). Dynamical downscaling allows representation of local scale features, such as orographic effects, land-sea contrast and other land surface characteristics, and smaller scale physical processes that influence extreme precipitation (Marauan et al., 2010). By modelling the atmosphere and local environment at a much finer scale than is possible using a GCM, it is expected that the specific processes that drive regional weather and climate will be better represented. Bias corrected outputs from dynamically downscaled models have been shown to be able to be used directly in projections of changes in extreme precipitation (White et al., 2010). A number of studies have used RCMs to investigate changes to daily precipitation extremes, however a lack of available sub-daily RCM data has limited studies on shorter duration events (Hanel and Buishand, 2010). The advantages of dynamical downscaling are in the ability to represent changes in rainfall spatial and temporal patterns, as well as impacts of local scale features. The disadvantage of dynamical downscaling is in computational time. There are assumptions inherent in the structure of each RCM and ideally a range of RCMs would be used in conjunction with a range of GCMs to give a more comprehensive description of local climate. The ability to undertake such studies is inhibited by the computational intensity of the task, and thus studies are generally limited to use of one or two RCMs with a range of GCMs.

4.17. References

ANZLIC (2008), ICSM Guidelines for Digital Elevation Data Version 1, August 12.

Abbott, M.B. (1991), Hydroinformatics - Information Technology and the Aquatic Environment, Avebury Technical, Aldershot, UK.

Abbs, D. and Rafter, T. (2009), Impact of Climate Variability and Climate Change on Rainfall Extremes in Western Sydney and Surrounding Areas: Component 4 - dynamical downscaling, CSIRO.

Ball, J.E. and Cordery, I. (2000), Information and hydrology, Proc. Hydro 2000 - Hydrology and Water Resources Symposium, Perth, WA, Australia, pp: 997-1001.

Bates, B. C., Z. W. Kundzewicz, S. Wu, and J. P. Palutikof (2008), Climate Change and Water. Technical Paper of the Intergovernmental Panel on Climate Change, Rep., 210 pp, IPCC Secretariat, Geneva.

Beynon-Davies, P. (2002), Information Systems: An introduction to informatics in organisations, Basingstoke, UK: Palgrave Macmillan. ISBN 0-333-96390-3.

Boyer, M.C. (1964) Streamflow measurement, Handbook of Applied Hydrology, Ed. V. T. Chow, Chapter 15, McGraw-Hill.

Bureau of Meteorology (2015), Australian Hydrological Geospatial Fabric (Geofabric). [ONLINE] Available at: http://www.bom.gov.au/water/geofabric/index.shtml. [Accessed 18 August 15].

CSIRO (2012), Climate and water availability in south-eastern Australia: A synthesis of findings from Phase 2 of the South Eastern Australian Climate Initiative (SEACI), CSIRO, Australia, September 2012, 41 pp.

Chow, V., Maidment, D. and Mays, L. (1988), Applied hydrology. New York: McGraw-Hill.

Church, J.A., Hunter, J.R., McInnes, K.L. and White, N.J. (2006), Sea-level rise around the Australian coastline and the changing frequency of extreme sea-level events. Aust. Met. Mag., 55(4), 253-260.

Cordery, I., Weeks, B., Loy, A., Daniell, T., Knee, R., Minchin, S. and Wilson, D. (2006), 'Water Resources Data Collection and Water Accounting', Australian Journal of Water Resources, 11(2), 257-266.

Corney, S.P., Katzfey, J.J., McGregor, J.L., Grose, M.R., Bennett, J.C., White, C.J., Holz, G.K., Gaynor, S.M. and Bindoff, N.L. (2010), Climate Futures for Tasmania: climate modelling technical report, Antarctic Climate & Ecosystems Cooperative Research Centre, Hobart, Tasmania.

Fenton, J.D. (2001), Rating curves: Part 1-Correction for surface slope. OR Fenton, J.D. (2001) Rating curves: Part 2-Representation and approximation.

Fenton, J.D. and Keller, R.J. (2001), the Calculation of Streamflow from Measurements of Stage, Technical Report Report 01/6, Cooperative Research Centre for Catchment Hydrology, Monash University, Clayton, Vic, Australia.

Grose M.R., Barnes-Keoghan I., Corney S.P., White C.J., Holz G.K., Bennett J.B., Gaynor S.M. and Bindoff N.L. (2010), Climate Futures for Tasmania: general climate impacts technical report, Antarctic Climate & EcosystemsCooperative Research Centre, Hobart, Tasmania.

Hanel, M. and T.A. Buishand (2010), On the value of hourly precipitation extremes in regional climate model simulations, Journal of Hydrology, 393: 265-273.

Hardwick-Jones, R, S.Westra and A. Sharma (2010), Observed relationships between extreme sub-daily precipitation, surface temperature and relative humidity, Geophysical Research Letters, 37(L22805).

Herschy, R.W. (1995), Streamflow Measurement, E & FN Spon, Chapman & Hall, London, UK. Huber and Dickinson, (1988)

Hersh, E. S. (2012), The long tail of hydroinformatics: Implementing biological and oceanographic information in hydrologic information systems.Hunter, J., (2007): Estimating sea-level extremes in a world of uncertain sea-level rise [http://staff.acecrc.org.au/~johunter/vic_flood_10102007_v12.pdf], 5th Flood Management Conference, Warrnambool, Australia, 9-12 October 2007.

Hunter, J. (2007), Estimating sea-level extremes in a world of uncertain sea-level rise, 5th Flood Management Conference, Warrnambool, Australia, [Accessed 12 October. 2007].

IPCC (Intergovernmental Panel on Climate Change) (2012), Workshop Report of the Intergovernmental Panel on Climate Change Workshop on Socio-Economic Scenarios [Edenhofer, O., R. Pichs-Madruga, Y. Sokona, V. Barros, C.B. Field, T. Zwickel, S. Schloemer, K. Ebi, M. Mastrandrea, K. Mach, C. von Stechow (eds.)]. IPCC Working Group III Technical Support Unit, Potsdam Institute for Climate Impact Research, Potsdam Germany, p: 51.

IPCC (Intergovernmental Panel on Climate Change) (2001), Climate Change 2001, The Scientific Basis. Contribution of Working Group I to the Third Assessment Report of the Intergovernmental Panel on Climate Change [Houghton, J.T.,Y. Ding, D.J. Griggs, M. Noguer, P.J. van der Linden, X. Dai, K. Maskell, and C.A. Johnson (eds.)]. Cambridge University Press, Cambridge, United Kingdom and New York, NY, USA, 881 p.

IPCC (Intergovernmental Panel on Climate Change) (2007), Climate Change 2007, Synthesis Report. Contribution of Working Groups I, II and III to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change [Core Writing Team, Pachauri, R.K and Reisinger, A. (eds.)]. IPCC, Geneva, Switzerland, 104 p.

ISO (International Standards Organisation) (2013), Guide: 18365:2013 Hydrometry. Selection, establishment and operation of a gauging station, International Standards Organisation.

Johnson, F., K.Haddad, A.Rahman, and J.Green (2012), Application of Bayesian GLSR to estimate sub daily rainfall parameters for the IFD Revision Project. Hydrology and Water Resources Symposium 2012, Sydney. Engineers Australia.

Jones, D.A., Wang, W. and Fawcett, R. (2007), Climate Data for the Australian Water Availability Project: Final Milestone Report. National Climate Centre, Bureau of Meteorology, Melbourne.

Jones, M.R., (2012), Characterising and modelling time-varying rainfall extremes and their climatic drivers. PhD Thesis, Newcastle University.

Kundzewicz Zbigniew, W. (2004), Editorial Hydrological Sciences-Journal-des Sciences Hydrologiques, 49(1), 3-6.

Lenderink, G. and E. van Meijgaard (2008), Increase in hourly precipitation extremes beyond expectations from temperature changes, Nature Geoscience, 1, pp: 511-514

Malleron, N., Zaoui, F., Goutal, N., and Morel, T. (2011), On the use of a high-performance framework for efficient model coupling in hydroinformatics. Environmental Modelling & Software, 26(12), 1747-1758.

Marauan, D.F., Wetterhall, A.M., Ireson, R.E., Chandler, E.J., Kendon, M., Widmann, S., Brienen, H.W., Rust, T., Sauter, M., Theme, V.K.C., Venema, K.P., Chun, C.M., Goodess, R.G., Jones, C., Onof, M., Vrac, I. and Thiele-Eich (2010), Precipitation downscaling under climate change: recent developments to bridge the gap between dynamical models and the end user, Reviews of Geophysics, 48.

McInnes, K.L., Hubbert, J.D, Macadam, I. and O'Grady, J.G. (2007), Assessing the Impact of Climate Change on Storm Surges in Southern Australia, In Oxley, L. and Kulasiri, D. (eds) MODSIM 2007 International Congress on Modelling and Simulation. Modelling and Simulation Society of Australia and New Zealand, ISBN: 978-0-9758400-4-7.

McKenzie, N.J., Jacquie, D.W, Ashton, L.J., and Cresswell, H.P. (2000), Estimation of soil properties using the atlas of Australian soils, CSRIO Land and Water, Canberra ACT.

Mehrotra, R., and Sharma, A. (2010), Development and Application of a Multisite Rainfall Stochastic Downscaling Framework for Climate Change Impact Assessment, Water Resources Research, 46.

Meynett, A.E. and van Zuylen, H.J. (1994), On the concept of hydroinformatics, Proc. Hydroinformatics '94, Delft, The Netherlands, AA Balkema, pp: 19-24.

Milly, P.C.D., Betancourt, J., Falkenmark, M., Hirsch, R.M., bigniew, W.Z., Kundzewicz, Z.W., Lettenmaier, D.P. and Stouffer, R.J. (2008), Stationarity is Dead: Whither Water Management? Science, 319(5863), 573-574.

Moya Quiroga, V., Mano, A., Asaoka, Y., Kure, S., Udo, K., and Mendoza, J. (2013), Snow glacier melt estimation in tropical Andean glaciers using artificial neural networks. Hydrology and Earth System Sciences, 17(4), 1265-1280.

National Research Council (2004), Assessing the National Streamflow Information Program, National Academy of Sciences, US.

Pilgrim, DH (ed) (1987) Australian Rainfall and Runoff - A Guide to Flood Estimation, Institution of Engineers, Australia, Barton, ACT, 1987.

Popescu, I., Jonoski, A., and Bhattacharya, B. (2012), Experiences from online and classroom education in hydroinformatics, Hydrol. Earth Syst. Sci., 16: 3935-3944, doi: 10.5194/hess-16-3935-2012.

Rantz, S.E. (1982), Measurement and computation of stream flow. Volume 1: Measurement of stage and discharge; Volume 2: Computation of discharge. US Geological Survey water-supply paper, 2175, 631.

Raupach, M.R., Briggs, P.R., Haverd, V., King, E.A., Paget, M. and Trudinger, C.M. (2009), Australian Water Availability Project: CSIRO Marine and Atmospheric Research Component: Final Report for Phase 3, CAWCR Technical Report, No.013, July.

Simons, D.B. and Richardson, E.V. (1962), The effect of bed roughness on depth-discharge relations in alluvial channels, U.S. Geological Survey Water-Supply Paper 1498-E.

Stewardson, M.J. and Howes, E.A. (2002), The number of channel cross-sections required for representing longitudinal hydraulic variability of stream reaches. Proceedings of the 27th Hydrology and Water Resources Symposium. Engineers Australia, pp: 21-25.

Toppe, R. (1987), Terrain models - A tool for natural hazard Mapping, Avalanche Formation, Movement and Effects, IAHS Publication Number 162: 629-638.

Trenberth KE (2011), Changes in precipitation with climate change. Clim Res 47: 123-138.

Vathananukij, H and Malaikrisanachalee, S. (2008), Hydroinformatic system (implementation in Thailand), Water SA, 34(6), 725-730.

Wain, A.T., Atkins, A.S. and McMahon, T.A. (1992), The Value of Benefits of Hydrologic Information, AWRAC Research Project P87/24, Centre for Applied Hydrology, University of Melbourne, p: 59

Westra, S. (2011), Implications of Climate Change on Flood Estimation. Discussion Paper for the Australian Rainfall and Runoff Climate Change Workshop No.2, 5th November 2010, UNSW.

Westra, S., L.V. Alexander, F.W. Zwiers (2013), Global increasing trends in annual maximum daily precipitation. Journal of Climate, in press, doi:10.1175/JCLI-D-12-00502.1.

White, C.J., Grose, M.R., Corney, S.P., Bennett, J.C., Holz, G.K., Sanabria, L.A., McInnes, K.L., Cechet, R.P., Gaynor, S.M. and Bindoff, N.L. (2010), Climate Futures for Tasmania: extreme events technical report, Antarctic Climate and Ecosystems Cooperative Research Centre, Hobart, Tasmania.

World Meteorological Organization (2010), Manual on Stream Gauging - Volume I - Fieldwork, WMO-No. 1044, Geneva, Switzerland, ISBN 978-92-63-11044-2.

Chapter 5. Risk Based Design

Duncan McLuckie, Rhys Thomson, Leo Drynan, Angela Toniato

Chapter Status	Final
Date last updated	14/5/2019
Minor edits	27/08/2024

5.1. Introduction

Floods can cause significant impacts where they interact with the community and the supporting natural and built environment. However, flooding also has the potential to be the most manageable natural disaster as the likelihood and consequences of the full range of flood events can be understood enabling risks to be assessed and where necessary managed.

Design flood estimation plays a key role in understanding flood behaviour and how this may change with changes within the floodplain and catchment and in climate and how these changes can influence both decisions that influence the growth and management of flood risk.

Design flood estimation provide essential information on a range of key factors that need to be considered in understanding and managing the consequences of flooding. These include: flood frequency; flow rates, velocities and volumes; flood levels and extents; duration of inundation. ARR provides essential analytical tools to assist in estimating design floods and in understanding these factors. Estimates of design floods are an essential element in understanding flood behaviour and making informed decisions on:

- Managing flood risk through a risk based decision making process (AEMI, 2013). Such approaches generally provide an understanding of flood behaviour across the full range of flood events, up to and including extreme events, such as the Probable Maximum Flood (PMF). They can inform decision making in flood risk management, a broad range of land use planning activities, emergency management for floods and dam failure, and in estimating flood insurance premiums;
- Managing flood risk through the use of design standards related to the probability or frequency of occurrence, rather than the broader assessment and management of risk;
- Setting infrastructure performance criteria based upon a design standard, generally a probability or frequency of an event, rather than the broader assessment and management of risk;
- Managing flood risk in short term projects through a risk based decision making process considering the life of a project; and
- Understanding and managing the impacts changes within the catchment and floodplain may have on flood behaviour and risk.

This chapter provides advice and examples on using the analytical tools outlined in ARR to inform decision making for flood risk management, road and rail design, mining and agriculture and design of dams. It does not provide details on the design standards, risk assessment frameworks, the assessment of impacts on the community or built or natural environment, nor to estimate residual risks. Further information on risk management approaches, processes and frameworks can be found in (AEMI, 2013), ANCOLD (2003) ISO (2009a).

The remainder of this chapter is structured as follows:

- Book 1, Chapter 5, Section 2 provides a background on flood risk;
- Book 1, Chapter 5, Section 3 discusses risk analysis;
- Book 1, Chapter 5, Section 4 discusses managing risk;
- Book 1, Chapter 5, Section 5 discusses managing flood risk to communities;
- Book 1, Chapter 5, Section 6 discusses managing flood risk to mining, agriculture and infrastructure projects;
- Book 1, Chapter 5, Section 7 discusses the management of flood risk in relation to dams.;
- Book 1, Chapter 5, Section 8 discusses the management of flood risk using basins;
- Book 1, Chapter 5, Section 9 discusses the consideration of effective service life of infrastructure;
- Book 1, Chapter 5, Section 10 discusses how flood risk changes over time due to a range of factors;
- Book 1, Chapter 5, Section 11 provides further reading material;
- Book 1, Chapter 5, Section 12 provides some examples of calculations; and
- Book 1, Chapter 5, Section 13 provides references.

5.2. Flood Risk

Flood risk results from the interaction of the community, through human occupation or use of the floodplain, with hazardous flood behaviour. It is the risk of flooding to people, their social or community setting, and the built and natural environment (AEMI, 2013).

Flood risk is not simply the probability of an event occurring. The *International Standard on Risk Management,* (ISO, 2009a) defines risk as the effect of uncertainty on objectives. In addition, *ISO Guide 73:2009 Risk Management Vocabulary* (ISO, 2009b) notes that uncertainty is the state, even partial, of deficiency of information related to, understanding or knowledge of an event, its consequence, or likelihood. An effect is a positive or negative deviation from the expected outcome. Objectives can have different aspects (financial, health and safety, environmental) and apply at different levels (local, state, site).

AEMI (2013) and ANCOLD (2003) express risk in terms of combinations of the likelihood of events (generally measured in terms of Annual Exceedance Probability (AEP)) and the severity of the consequences of the event (see Figure 1.5.1). Risk is higher the more frequently an area is exposed to the same consequence or when the same frequency of event has higher consequences.

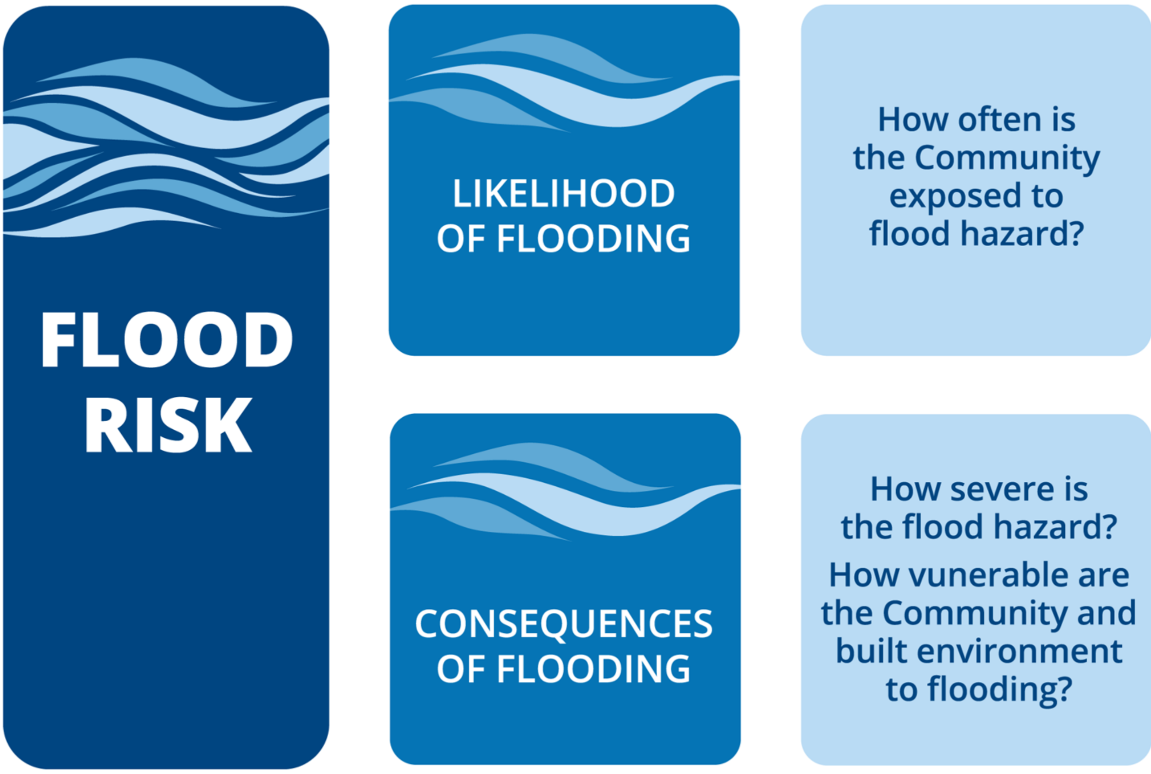


Figure 1.5.1. Components of Flood Risk (After McLuckie (2012)

AEMI (2013) discusses the consequences of flooding on the community. The consequences depend upon the vulnerability of the community and the built environment to flooding. Vulnerability varies with the element (people, property and infrastructure) at risk and within the different cohorts within the elements outlined below. *AEMI (2015)* advises that this may be measured in terms of the impacts upon:

- *People* – in terms of fatalities and injuries;
- *The economy and assets* - in terms of reduced economic activity and asset losses;
- *The social setting* – in terms of consequences to the community as a whole (rather than individuals) that can lead to the breakdown of community organisations and structures. This can include the temporary or permanent loss of community facilities or culturally important objects or events;
- *Public administration* – in terms of changes to the ability of the governing body for the community to be able to deliver its core functions;
- *The environment* – in terms of destruction and degradation of critical environmental assets (and their processes and structures) and/or species extinction and habitat range reduction; and

The consequence to different elements from the same exposure to flooding can be different. For example, a flood may have major consequences for community assets (such as a water or sewerage treatment plant) but have only minor or moderate consequences in terms of potential fatalities and injuries in the population.

The likelihood of exposure to flooding and therefore flood risk varies significantly between and within floodplains and flood events of different magnitudes. Figure 1.5.2 shows areas exposed to flooding from events of different AEPs.

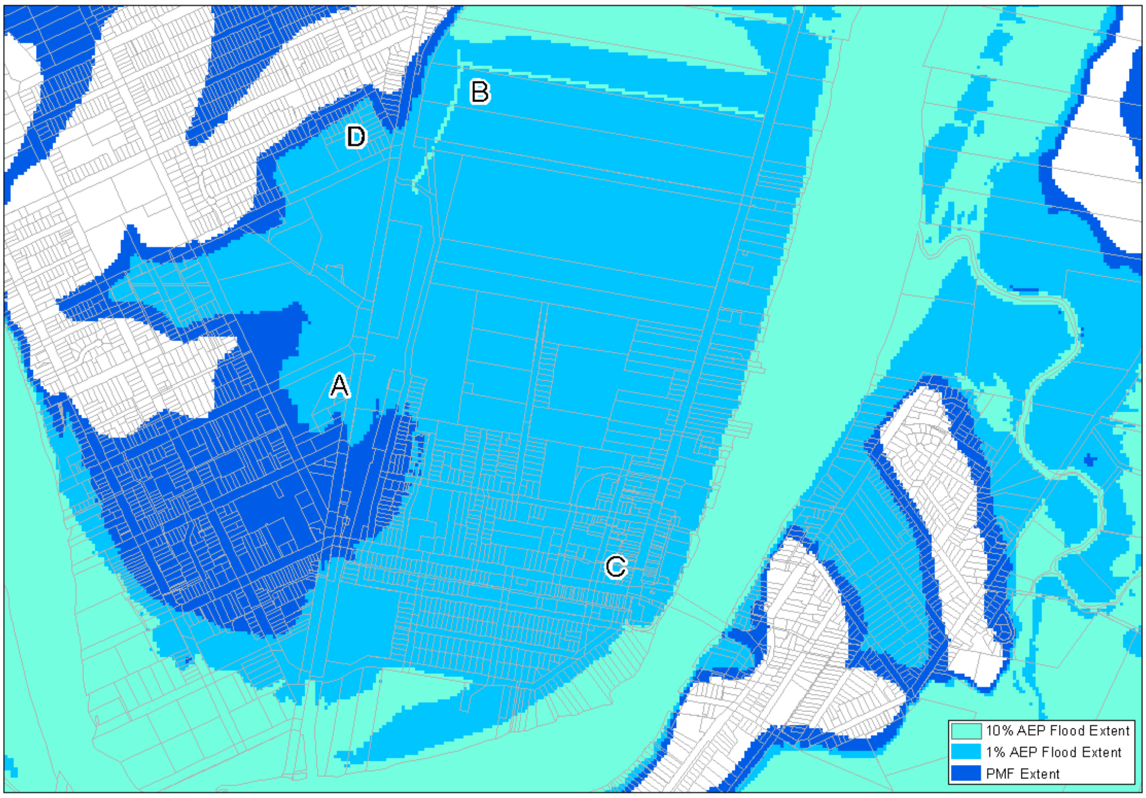


Figure 1.5.2. Map Showing Different AEP Flood Extents Including an Extreme Event

5.3. Risk Analysis

Risk analysis is a systematic approach to understanding the nature of and deducing the level of risk. It involves developing an understanding of the nature of, driver for, hazard and the associated consequences to rank the relative severity of risk. It is one of the steps in the risk management process (ISO, 2009a) and for example is generally undertaken as part of a floodplain management study in the flood risk management framework *(AEMI, 2013).*

Risk analysis involves understanding the varying likelihood of events (that result in a consequence), and the severity of their consequences. It should also involve an assessment of confidence, which considers factors such as the divergence of opinion, level of expertise, uncertainty, quality, quantity and relevance of data. These factors are combined to assign a relative risk rating for an event through development of a risk matrix or other tools. Risk analysis may be quantitative or qualitative. In both cases the probability of events affecting communities may be able to be estimated through design flood estimates.

Quantitative analysis is often used where both the probability and the consequences can be measured. For example, consequences may be estimated in terms of tangible flood damage to the community for events of different AEPs. Tangible damages are those damages that are more readily able to be estimated in economic terms and lend themselves to quantitative assessment, including:

- Direct damages to structures and their contents due to water contact; and
- Indirect damages of clean-up of debris and removal of damaged material, loss of wages, sales, production and costs of alternative accommodation, and opportunity costs due to loss of services.

Qualitative analyses are generally undertaken where consequences are difficult to quantify. For example, these can include social and environmental impacts and the costs of fatalities and injuries which are intangible damages that cannot readily be put in economic terms. Table 1.5.1 provides an example of a qualitative risk matrix.

Table 1.5.1. Example Qualitative Risk Matrix

Consequence Level					
Likelihood Level	Insignificant	Minor	Moderate	Major	Catastrophic
Almost Certain	Medium	Medium	High	Extreme	Extreme
Likely	Low	Medium	High	Extreme	Extreme
Unlikely	Low	Low	Medium	High	Extreme
Rare	Very Low	Low	Medium	High	High
Very Rare	Very Low	Very Low	Low	Medium	High
Extremely Rare	Very Low	Very Low	Low	Medium	High

Risk analysis can be used to inform decisions on both the acceptability of residual risk and the effective and efficient use of scare resources to better understand and manage risk.

5.4. Managing Flood Risk

Managing flood risk generally involves a combination of:

- Managing changes within the floodplain that may alter flood behaviour;
- Altering the likelihood (how frequently exposure to flooding occurs); and
- Managing the consequences of flooding (reducing vulnerability to flooding when exposed) to reduce the risks.

Managing risk needs to consider the different elements at risk which may require different management techniques and standards. It also needs to consider the risks to the existing community and built and natural environment and the additional risk created by introducing new development and infrastructure into the floodplain.

5.4.1. Managing Changes to Flood Behaviour

Changes within the floodplain that can significantly affect flood behaviour include:

- Development (including filling) within the floodplain (particularly in flow conveyance and flood storage areas);
- Development within the catchment even though outside the floodplain; and

- Construction or upgrade of above ground infrastructure across waterways and the floodplain.

These activities may result in significant changes to flood behaviour including changes to flow paths, peak flow and velocities, flood levels and extents, distribution of flood waters, and the timing and duration of flooding. These changes can lead to adverse impacts on the existing community and the built and natural environment and the built and natural environment through changes to flood behaviour and the ability of the community to effectively respond to flood emergencies.

New developments and infrastructure projects generally have constraints placed on them through government approvals processes relating to negating or minimising adverse impacts of the project on existing development and the environment.

Broader processes such as the floodplain specific management process AEMI (2013) often examine changes through scenario testing. These may be managed though floodplain, catchment or community based techniques as discussed in Book 1, Chapter 5, Section 5.

However decisions on infrastructure projects, particularly those developed by entities that do not manage floodplain or catchment development, are often made without being able to influence development directions or mitigation efforts. Therefore decisions of infrastructure projects, may need to consider these potential changes as part of non-stationarity considerations as part of project investigations. This is discussed in Book 1, Chapter 5, Section 10 of this chapter.

5.4.2. Managing Flood Risk by Limiting Likelihood and Consequences

Approaches to managing the likelihood or consequences of flood risk on or to a community or asset generally fall into the following types:

- Use of design standards that relate to a particular flood event;
- Providing a certain level of service; and
- Use of risk based management approaches.

Risk based management approaches are generally more complex than the use of design flood and level of service standards.

5.4.2.1. Design Flood Standards

The establishment of design flood standards for infrastructure design and as a basis for minimum protection levels for the community have often been based upon decisions to reduce the frequency of exposure to risk. This involves a balance between protection of an asset or affected communities and stakeholders from an event against the cost of the infrastructure to provide protection.

Design flood standards are generally aimed at limiting the frequency of exposure to flood risk. For example, the use of a minimum floor levels for a building relative to a design flood level aims to reduce the exposure to flooding by excluding flooding from above the floor level of the building in the design flood event. This approach is based upon accepting that consequences that result from the building flooding in events larger than the design flood event.

Design flood standards are typically adopted across an entire floodplain, or government service area. Generally, there is only limited ability to incorporate location specific issues into the design flood standards.

When used in isolation, this approach makes the assumption that the residual risks remaining after development is constructed to standards are acceptable. It also assumes that:

- the location is suitable for the development;
- the development will not impact upon flood behaviour and therefore have an adverse impact elsewhere in the community; and
- the impacts of flooding on the building and its occupants, including the associated residual risks, are acceptable or can be managed by other means.

Where used in isolation this can limit the effectiveness of this approach and may lead to decisions that leave individuals, communities and the built environment exposed to risks that may be considered unacceptable to the community.

Design flood standards are also generally based upon existing floodplain, catchment and climatic conditions, what can be called stationary conditions. However, these conditions can change over time. Consideration in estimating design floods is discussed in Book 1, Chapter 5, Section 5 to Book 1, Chapter 5, Section 5. Further discussion on the factors that can lead to non-stationarity and current literature on non-stationary risk assessment is provided in Book 1, Chapter 5, Section 10.

5.4.2.1.1. Design Flood Standard Terminology

There are two key ways in which design flood standards are typically expressed:

- Annual Exceedance Probability (AEP): the likelihood of occurrence of a flood of a given size or larger in any one year; usually expressed as a percentage (e.g. a flood protection levee may adopt of Flood Design Standard that offers protection up to the a 1% AEP event); and
- Service Life Exceedance Probability (SLEP): The likelihood of exceedance during a project's adopted service life, rather than as an annual likelihood. It is recommended that this should be the Effective Service Life (Book 1, Chapter 5, Section 9), rather than the Design Service Life.

AUSTROADS, the national association of road transport and traffic authorities in Australia use both the SLEP approach and the AEP method depending on the context.

The AEP method is used to define the levels of service of roads:

“Freeways and arterial roads – should generally be designed to pass the 50 or 100 year ARI flood without interruption to traffic. However for arterial roads in remote areas, a reduced standard is commonly adopted where traffic densities are low, Austroads (1994).”

The SLEP method is used in the design of bridges:

“All bridges are to be designed so that they do not fail catastrophically during a flood that has a 5% chance of being exceeded during the Design Service Life of the structure. Assuming a 100 year Design Service Life, this equates to a flood with an ARI of 2000 years’ Austroads (1994).”

It is considered that while design standards are commonly expressed as an annual exceedance probability, it may be the case that stakeholders actually interpret and apply this more as a SLEP or an Exceedance Frequency over the adopted service life. For example, when stakeholders refer to a 1% AEP flood standard (floor level) for residential developments, conceptually they may interpret this to mean that the floor level for the property will only be exceeded once in every 100 years.

The SLEP approach may be more readily understandable for short term or temporary structures (refer to Book 1, Chapter 5, Section 6). Similarly, where infrastructure may be particular susceptible to damage from overtopping (e.g. a bridge superstructure that is not designed for inundation or a secondary spillway), then it may be important to understand the likelihood of being exceeded during the structure's effective service life (discussed in Book 1, Chapter 5, Section 9).

5.4.2.2. Level of Service Standards

Level of service standards are generally aimed at maintaining the serviceability during an event of a particular magnitude. For example, having a road trafficable in a design flood event of a certain AEP or having a waterway structure under the road pass the peak of a design flood event without overtopping the road. Considered in isolation, this approach assumes that the:

- Road will not impact upon flood behaviour and have adverse impact on the community;
- Impacts of flooding on the road are acceptable or can be managed by other means. For example, the road is expected to overtop and the design allows for damage minimisation or replacement in such events;
- Level of service provided is acceptable. For example, loss of access along the road is expected during a flood event and this is considered in emergency management planning; and
- Residual risk remaining to the community is acceptable.

Similarly to design flood standards, the assumptions of this approach can limit its effectiveness, particularly where it is used in isolation.

5.4.2.3. Risk Based Decision Making Processes

Risk based decision making processes (such as those outlined in AEMI (2013)) are used to develop management strategies that consider the risks associated with a full range of potential flood events and the associated consequences to the community and its supporting built and natural environment. It can be used to:

- manage the residual risks associated with design flood standards, or
- for management of risks for non-standard and critical infrastructure where a broad design flood standard may not be appropriate.

5.4.2.3.1. Non-Standard or Critical Infrastructure

Some examples of this type of infrastructure include:

- Dam safety risks (ANCOLD, 2003), and design of spillways and outlets;

- Risk management decisions for projects with a relatively short time frame, such as construction projects as well as temporary infrastructure (such as a coffer dam during construction);
- Structures that are particularly susceptible to overtopping or inundation. For example, a bridge superstructure that cannot withstand active flow or impacts from debris, or a flood levee that is not designed for overtopping and will likely result in failure.
- Critical infrastructure that may result in significant consequences should they fail or be inundated. This may include economically important infrastructure (for example transportation routes such as those between ports and major economic hubs, trunk communication networks (internet, phone) or key elements of the electricity network) or emergency response infrastructure (e.g. hospitals, evacuation centres etc).

In undertaking a risk based design process to develop management strategies for non-standard and critical infrastructure, it is important to understand the stakeholders involved in the decision making process. In many cases, these stakeholders may have different risk preferences (risk averse versus risk accepting). For example, a mining company may have a different risk preference for inundation of a mine, versus the community preferences for flood impacts downstream of the mine. It will be important to fully understand these different preferences to assist in informing the appropriate mitigation measures that might be required as the same likelihood and consequences could lead to a higher risk categorisation where there is a risk averse rather than a risk accepting preference.

5.4.2.3.2. Management of Residual Risk

When examining risks to existing and future development within a community the approach can be used in a manner that is complementary to the use of design flood and levels of service standards by examining the residual risk to the community and examining whether additional risk management measures may be necessary. This approach may involve testing whether the design flood or level of service standard is appropriate in the circumstances or whether additional management measures may be warranted to address residual risks.

For example, in a particular instance the use a design flood event as a standard for development within a community may reduce the frequency of exposure of people and property to flooding. However, additional management measures may be required to address residual risks. For example:

- the degree of flood damage to new buildings built to design standards mean that risks to property remain high. This may result in consideration of the use of a larger design flood event as a standard for development or other damage reduction approaches to broadly reduce flood damages.
- limitations to the ability to effectively warn and evacuate the community during the available warning time may mean that risk to life may remain high. Implementation of options to reduce risks to life, such as:
  - Development or improvement of a flood warning system so the community can be advised of a flood event and have more time to respond to calls for evacuation; or
  - Upgrade of evacuation routes to improve traffic capacity to enable the community to be more effectively evacuate within the available warning time.
- the effect development has on flood behaviour outside the development area may be significant. This may result in the need to implement, additional management measures

such as allowance in design for areas within the floodplain to continue to provide their essential flood conveyance and storage functions.

5.5. Managing Flood Risks to Communities

Flooding has the potential to be the most manageable natural disaster as the location of flood impacts and its effect upon the community and the built and natural environment can be understood for the full range of events.

Best practice in flood risk management in Australia (AEMI, 2013) works towards the vision:

“Floodplains are strategically managed for the sustainable long-term benefit of the community and the environment, and to improve community resilience to floods.”

Best practice promotes the consideration and, where necessary, management of flood impacts to existing and future development to improve community flood resilience using a broad risk management hierarchy of avoidance, minimisation and mitigation to: reduce the health, social and financial costs of occupying the floodplain; increase the sustainable benefits of using the floodplain, and improve or maintain floodplain ecosystems dependent on flood inundation (AEMI, 2013).

Managing flood risk provides an informed basis for the effective and efficient use of scare resources to:

- Better understand flood risk;
- Manage the growth in flood risk to the community due to the introduction of new development into the floodplain; and
- Reduce risks to the existing community where warranted.

This enables investment to be focused on understanding and managing flood risk where the need and benefit is greatest.

Different treatment solutions may be necessary depending upon the element at risk (people, property and infrastructure) and the location. Treatment options may involve a combination of flood mitigation, emergency management, flood warning and community awareness, together with strategic and development scale land-use planning arrangements that consider the flood situation and hazards.

Different options are also used dependent upon whether the aim is to manage risk to existing or future development within the community.

For the existing development it is important to understand the current exposure of the community to the full range of flooding, how the associated risks to different elements within the community are currently being managed and whether changes would be required to reduce risks to a more acceptable level. Where treatment options to reduce risks are being considered the impacts these measures may have on flood behaviour need to be understood and considered in decision making.

For flood risk to future development it is important to understand how the flood behaviour varies across the floodplain so that the constraints that this may place on development can be considered in deciding where to develop (and where not to develop), the types of development that may be suitable in different areas, and the flood related development constraints necessary to reduce risks to acceptable levels (in areas suitable for development).

It is also important to consider how flood behaviour and the associated risk will change over time due to development in the catchment and due to climate change and its impacts on both sea level and the intensity of flood producing rainfall events (discussed in Book 1, Chapter 6). Assessment of these changes on design flood estimates are discussed in Book 1, Chapter 5, Section 5 to Book 1, Chapter 5, Section 5.

More information on understanding and managing flood risk is available in AEMI (2013).

5.5.1. Using Flood Estimation to Inform Flood Risk Management

Design flood estimation can support management of flood risk to the community by improving knowledge of the potential range of flood behaviour and providing tools and information to support decision making. The selection of a modelling approach needs to consider the capability the approach provides relative to the requirements of the project specification. Many management decisions, such as emergency management planning, rely upon an understanding of the full range of floods rather than a specific design event, and need time varying information across a whole event rather than just the peak of the event. Managing flood risk involves a range of different groups with different information needs AEMI (2013).

5.5.1.1. Analysis of Historic Flood Events

Managing flood risk to the community generally requires more knowledge than can be gained from historic flood events. The information available on historic flood events is generally incomplete and is unlikely to represent the full range of potential flood events. In addition, it is likely that there have been changes within the floodplain or catchment since the historic flood event occurred that would alter the behaviour or impacts of a flood of the same magnitude if it were to occur today.

The use of historic flood event information in isolation without an understanding of the potential range and severity of flood events at a location and an understanding of how this may vary within a floodplain can result in poor management decisions – leaving the community unsustainably exposed to risk.

Knowledge and experience of historic flood events provides a starting point for understanding flood risk. Modelling historic flood events can assist to:

- Calibrate and validate models against known data and the community’s experience of flooding;
- Better understand historic flood events by filling in gaps in our knowledge of flood behaviour and its variation along a watercourse and across the floodplain; and
- Understand the probability of floods of the scale of historic flood events being exceeded in future.

The consequences of historic flood events can also provide valuable information for understanding flood risk. An appropriately calibrated and validated model can provide a sound basis for updating of the model to current conditions in light of changes in the catchment and floodplain since historic flood events.

5.5.1.2. Analysis of Design Flood Estimates for Current Conditions

Design flood estimates can be used for a range of purposes including:

- Understanding flood behaviour (flow paths, distribution, velocity, depth, level, timing and length of inundation) and risk and how this varies across the floodplain, over the duration of a flood event, and between flood events of different magnitudes;
- Understanding how flood behaviour, hazards and risks may change due to floodplain, catchment and climate changes;
- Establishing design standards based upon a specific design event;
- Assessing whether desired levels of service are met;
- Making decisions on the need for risk treatment, comparing and assessing treatment options, and deciding on which options to implement; and
- Designing waterway structures, basins, levees and other treatment options.

Design flood estimation for the full range of flood behaviour provides the basis for assessing the frequency and severity of flood exposure of different parts of the floodplain and the consequences of flooding to the community,providing a spatial understanding of:

- Flood extents to understand where floods of different magnitudes will impact;
- The variation in the flood functions of flow conveyance and storage within the floodplain for key events. Areas with these functions are generally areas where change in topography, vegetation or development can significantly alter flood behaviour which may lead to detrimental impacts to the existing community;
- The variation in hazard across the floodplain for key flood events (refer to Book 6, Chapter 7). This can delineate where flood behaviour in events is hazardous to people, vehicles and buildings (AEMI, 2014b); and
- The variation in flood evacuation difficulty from areas within the floodplain (AEMI, 2014c).

Outputs from design flood estimation and flood risk management processes are essential in informing government and industry through input to information systems. This can improve the accessibility of information on flood risk so it can be considered in investment and management decisions by government, industry and the community.

5.5.1.3. Analysis of the Impacts of Changing Infrastructure on Design Flood Estimates

Infrastructure crossing a floodplain can often provide some control on flood behaviour within a floodplain. Therefore the introduction of new infrastructure or modification of existing infrastructure crossing the floodplain can create or modify the flood controls within the floodplain which can influence flood behaviour and risk to the existing community.

The significance of these impacts can be assessed by altering calibrated and validated models of existing conditions to allow for changes and assessing the associated impacts. Management of impacts can lead to modifying the design of the infrastructure in consideration of its impacts upon the community or examination of ways to offset impacts upon the community.

5.5.1.4. Analysis of the Impacts of Changing Development on Design Flood Estimates

Future development within the catchment can have significant impacts on flood risk to the existing community. The impacts of development can be assessed by modifying the calibrated and validated models of existing flood behaviour to allow for changes to flow conveyance, storage and runoff conditions within the catchment.

The natural flood functions of flow conveyance and flood storage occur in flow conveyance and flood storage areas. Filling of flow conveyance areas can impact upon upstream flood levels and can result in redistribution of flows, with the potential for new flow paths being activated, affecting other areas. Filling of flood storage areas can affect both upstream and downstream flood levels.

Any decision to modify the flow conveyance and flood storage characteristics of the floodplain need careful consideration as these ramifications can be significant. The significance of these impacts can be assessed by altering calibrated and validated models of existing conditions to allow for changes and assessing the associated impacts. Management of impacts can lead to modifying the allowable changes in these areas in consideration of its impacts upon the community.

Flows at a downstream point in the catchment can also change significantly with upstream development within the catchment, even where flow conveyance and flood storage areas maintain their essential flood functions. These changes can occur due to increase in impervious areas and flow paths being shortened or having a higher proportion of impervious area. This can reduce losses leading to a higher proportion of rainfall running off and the time of concentration of flows within the catchment being reduced.

Assessment of the potential cumulative impacts of such broad changes is best undertaken in community rather than development scale flood investigations. It can provide the basis for understanding the relative significance of this change and where considered significant, assessing options to offset these impacts on a catchment basis. For example impacts on peak flood flows may, in some cases, be able to be managed using centralised or strategic scale basins or distributed (development site related) treatment measures.

5.5.1.5. Analysis of Climate Change Impacts Upon Design Flood Estimates

Climate change can have impacts on both flood producing rainfall events and on sea level and this can influence flood behaviour, frequency and impacts on the community in waterways. Assessment of the potential scale of impacts of climate change on flood producing rainfall events and its significance are discussed in Book 1, Chapter 6.

Sea level rise will directly influence ocean conditions that can influence flood behaviour in coastal waterways. Any rise in ocean conditions will directly translate to an increase in any relevant ocean boundary condition that is used in flood studies and will influence both the scale and balance of interaction of oceanic inundation with catchment flooding. Book 6, Chapter 5 provides information on the potential for coincidence of oceanic inundation and catchment or river flooding. Other guidance, such as OEH 2015 for NSW, may also be relevant and need consideration in particular jurisdictions.

The significance of these impacts of flood behaviour can be assessed by altering calibrated and validated models of existing conditions to allow for changes in flood producing rainfall events and the coincidence of oceanic inundation. Understanding these impacts can lead to an understanding of where they occur and whether management may be necessary. Management may involve strategies to allow for changes upfront or strategies that allow for adaptation over time.

Climate change may also have influence on the effective service life of infrastructure. This is discussed in Book 1, Chapter 5, Section 9.

5.5.2. Using Design Flood Estimation to Support Management of Future Development

Flood risk to future development is primarily managed by incorporating consideration of flood risk and the associated constraints (Book 1, Chapter 5, Section 5) into strategic planning and the relevant land use planning system.

Land use planning systems often use a flood standard as a basis for many flood related controls and decisions. These systems may also consider changes in climate and the influence development within the catchment will have on flood behaviour. These considerations are often made in addition to existing conditions to provide an understanding of how changes may impact upon flood behaviour and the existing community.

Systems may also require consideration of larger or extreme flood events to examine whether additional development constraints are necessary to deal with residual risks to the new development, particularly risk to life.

As new development on the floodplain can impact upon flood behaviour and the flood risk faced by the existing community, land use planning systems generally require these impacts to be assessed and managed.

Developing on the floodplain places the new development and its occupants at risk from flooding. These issues need to be considered in setting strategic directions for the community and determining development constraints.

Design flood estimation provides essential information for understanding constraints (see Book 1, Chapter 5, Section 5) that need consideration in setting strategic land use planning directions for the community, including:

- Information to inform decisions on where to (and where not to) develop and the limits on what type of development to place in different areas of the floodplain. For example, development within a flow conveyance area may have significant impacts upon flood behaviour or cause significant damage to structures. Development in this area should be restricted to enable the flow conveyance area to perform its natural flood function. A further example, such as an area with evacuation issues that is classified as flooded, isolated and submerged (AEMI, 2014b) would not necessarily be an appropriate area for a development whose occupants may be vulnerable in emergency response and therefore difficult to evacuate e.g. aged care facilities or a hospital;

- The assessment of the cumulative impacts of development within the catchment and floodplain on flood flows and behaviour. For example, the assessment of the cumulative impacts of development of the catchment can enable the examination of catchment scale solutions to offset the impacts of development on flood flows in an efficient manner. Such solutions may include a single series of basins whose interaction is considered (see Book 1, Chapter 5, Section 8); and

- Information to inform the derivation and implementation of development constraints that reduce the residual flood risk to the new development and its occupants to an acceptable level.

Design flood estimation also provides essential information to inform consideration of individual development proposals through the planning system. Studies undertaken for specific developments generally aim to assess whether the development will have significant impacts upon existing flood behaviour and the flood risk of the existing community and the impacts of flooding on the proposed development site and the residual risks to the development and its occupants.

For individual development proposals design flood estimation can advise on the:

- Suitability of the specific location for development;
- Suitability of the proposed development for the location;
- Limits on the scale of development to limit impacts on the existing community; and
- Development conditions necessary to manage residual risk to the new development and its occupants and any impacts of the development upon the existing community.

Site specific studies do not generally provide advice for setting strategic land use direction of the community.

5.5.2.1. How Can More Information Aid in Decision Making?

Having more information on flood behaviour and on flood related constraints can support informed decision making. The example provided below in Figure 1.5.3 and Figure 1.5.7 provides an example of how additional information on flood behaviour and factors that influence the risk to people and property can provide a better understanding of flood constraints so these can be considered in investment and development decisions.

Figure 1.5.3 shows the area affected by the design flood event used to set design standards for developments in the area. It does not provide any breakdown of the floodplain to highlight the varying flood function within this area, the varying degrees of hazard; or the differences in emergency response classification within the floodplain. Nor does it provide information on more frequent or more extreme floods. It therefore provides limited information for effective management.

Using the information from Figure 1.5.3 alone, Locations A, B, C and D appear to be exposed to the same degree of risk. The availability of this limited information would, most likely, result in the same development restrictions being applied to each location.

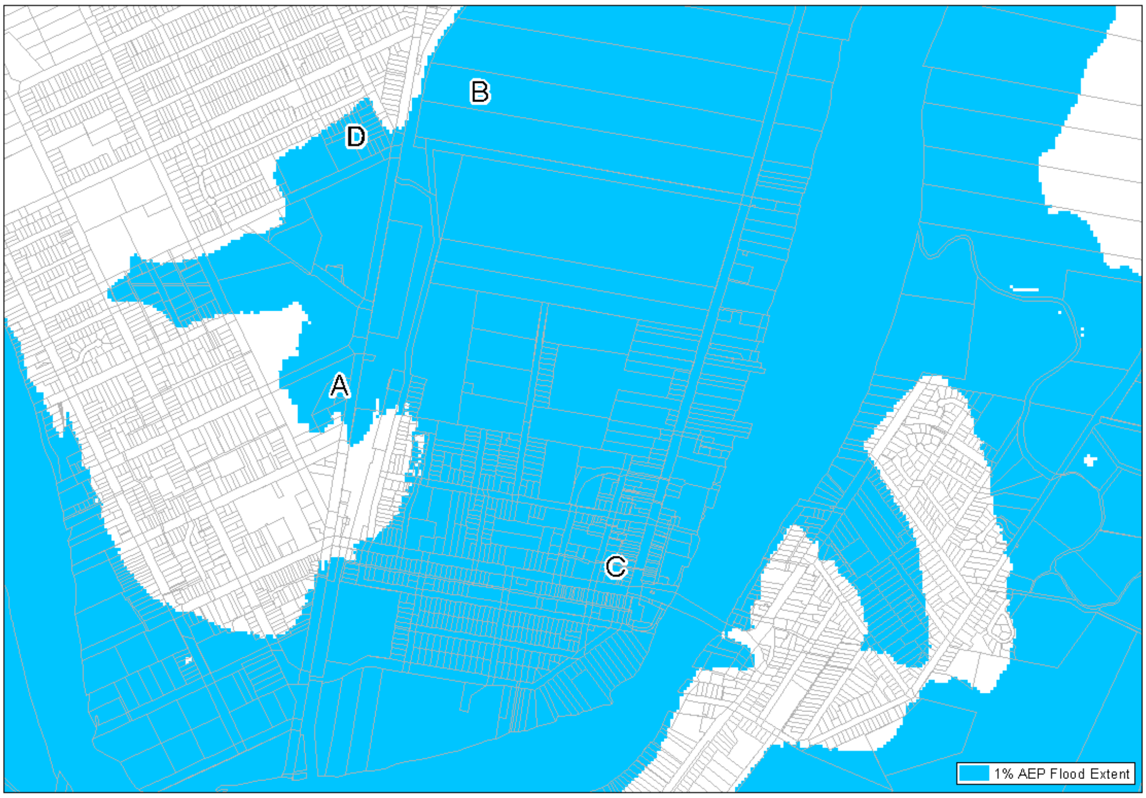


Figure 1.5.3. Map of Flood Extents

However, Figure 1.5.4 to Figure 1.5.6 shows the same floodplain but with information on how flood hazard, flood function and the flood emergency response classification varies across the floodplain. This information shows the flood situations impacting upon Locations A to D to be different and may require different management treatments.

- Location A is easy to evacuate and outside the impacts of flood function and flood conditions are not hazardous to buildings;
- Location B is in a flow conveyance area and development may impact upon flood behaviour and the flood risk of others in the community;
- Location C is isolated and completed inundated by larger floods and has a more difficult flood evacuation situation; and
- Location D is similar to Location A but is an area where floods may be hazardous to houses and people.

If these additional risk factors are considered in setting development constraints flood risk at a location can more effectively be managed at the different locations reducing the residual risks remaining. Mapping the constraints from Figure 1.5.4 to Figure 1.5.6 can provide more clarity on where to apply different development controls as shown in Figure 1.5.7.

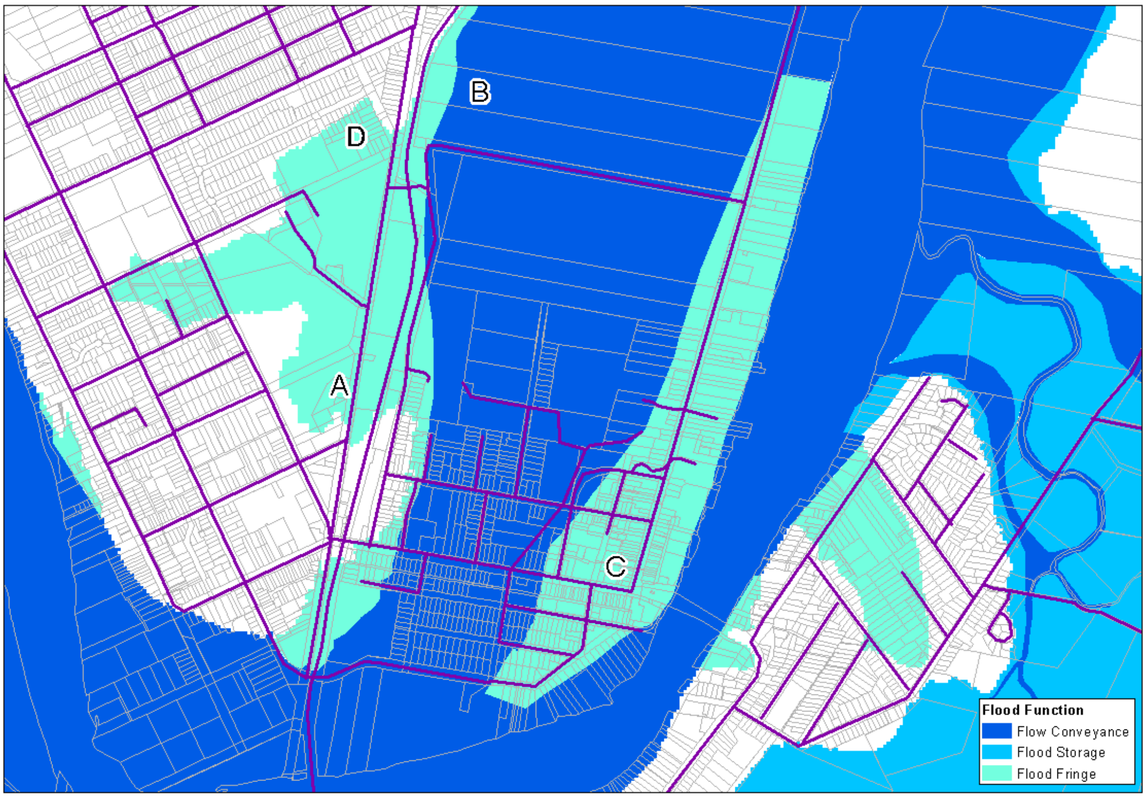


Figure 1.5.4. Map of Flood Extents and Flood Function

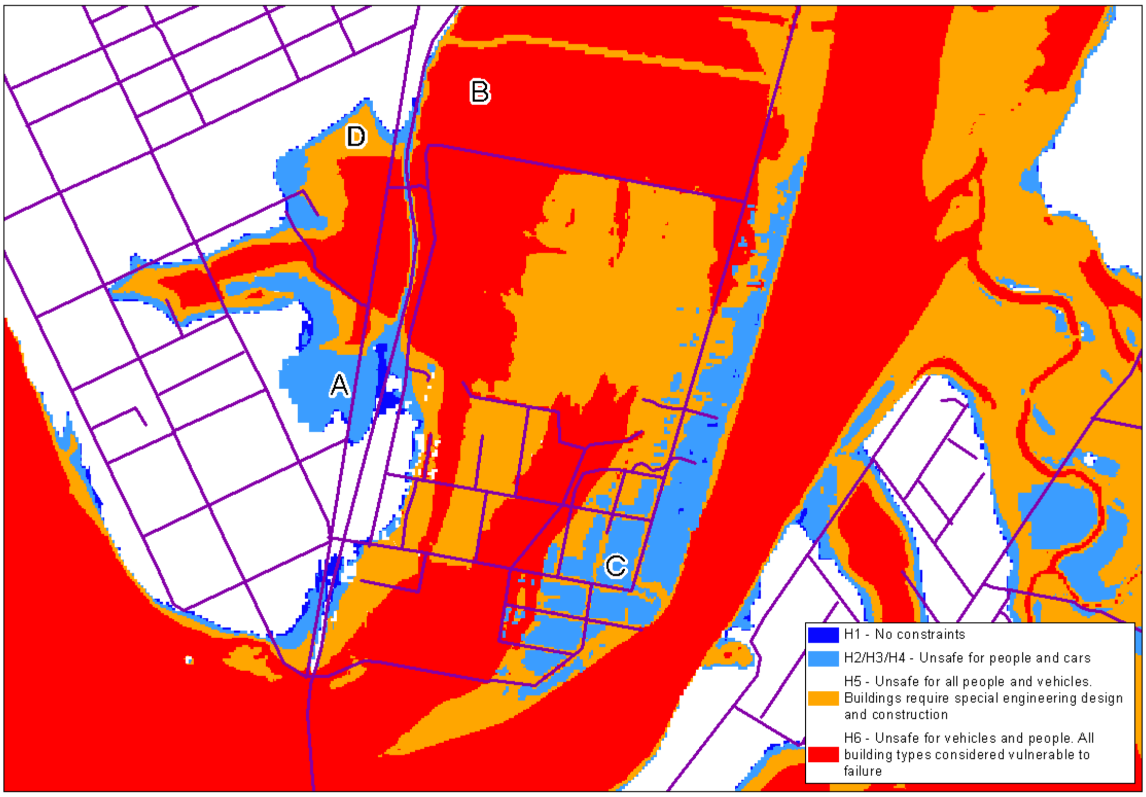


Figure 1.5.5. Map of Flood Extents and Flood Hazard

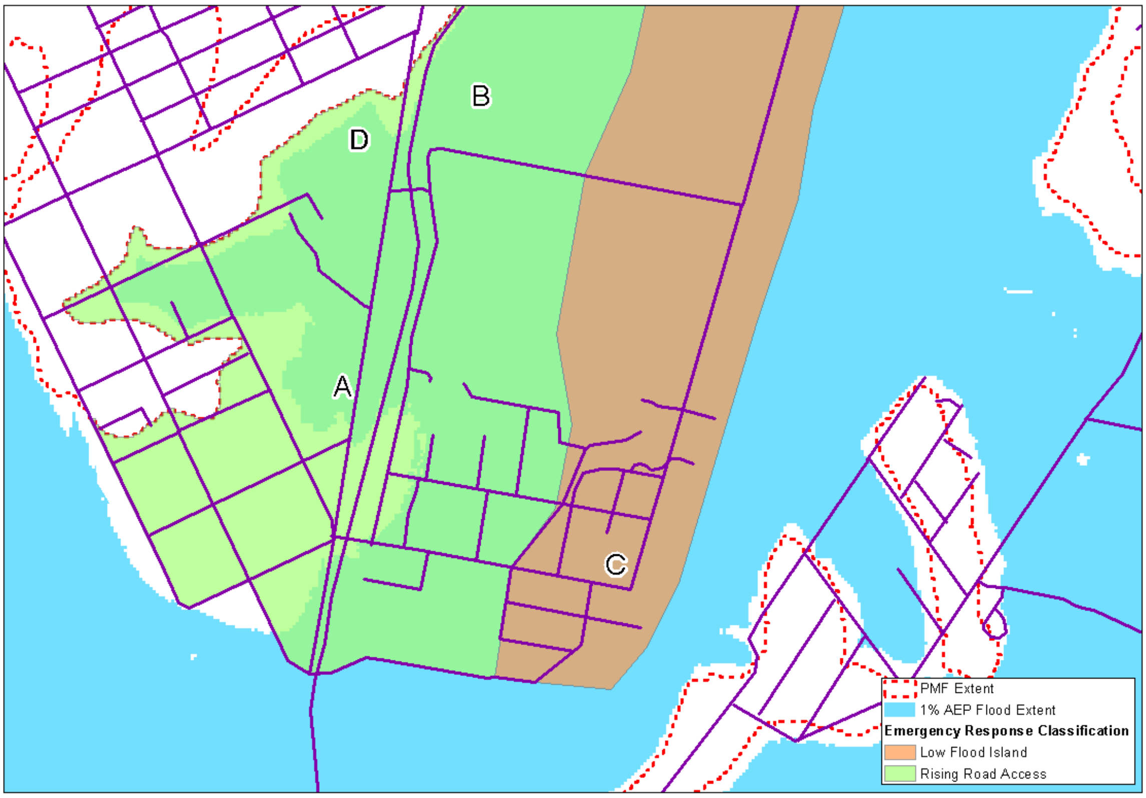


Figure 1.5.6. Map of Flood Extents and Flood Emergency Response Classification

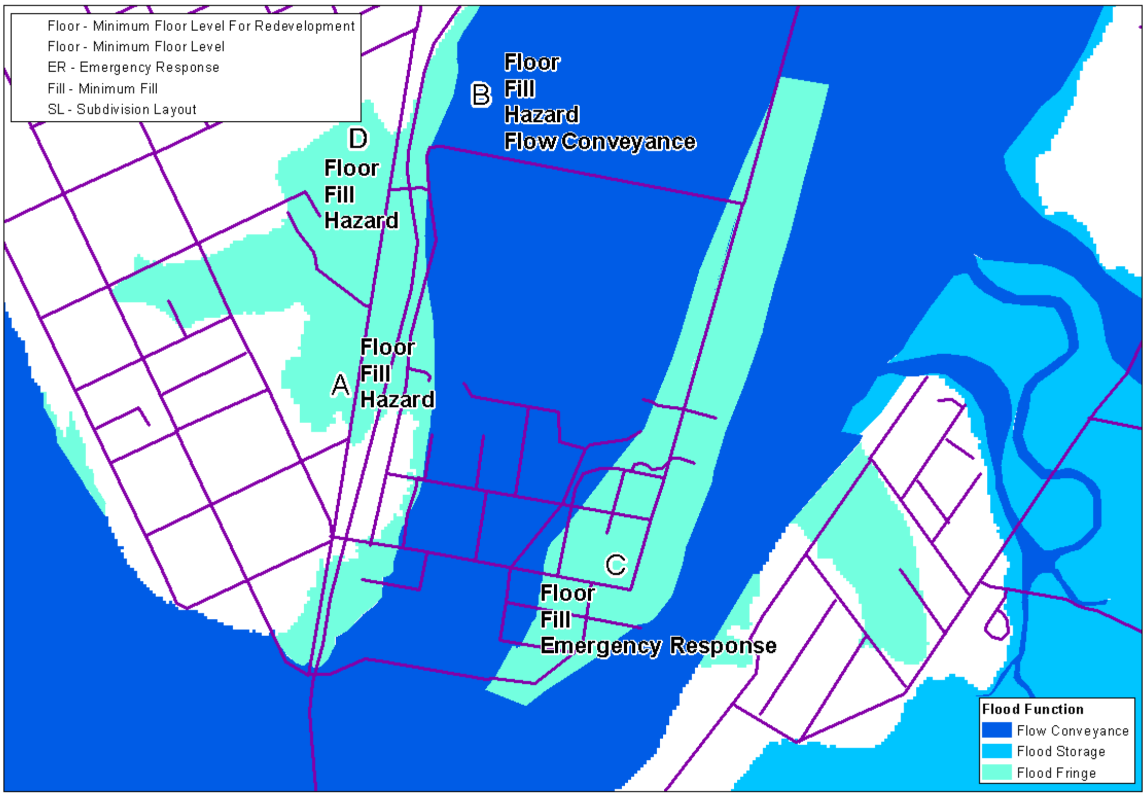


Figure 1.5.7. Map of Variation in Constraints Across the Floodplain

The additional information provided in Figure 1.5.4 to Figure 1.5.7 identifies additional risk factors in different locations without which:

- The need to consider these additional risk factors in decision making may not be evident; and
- Where it has otherwise been recognised that these additional risk factors may need to be considered in decision making it is likely that any associated constraints would be applied broadly across the floodplain. This would require studies for individual developments to determine and address these risk factors.

The additional information provided in Figure 1.5.4 to Figure 1.5.7 has the added benefit of enabling the provision of improved clarity for development conditions by enabling these to be more effectively inform land use planning systems. McLuckie et al. (2016) discusses the extension of this work as part of the development of best practice guidance on flood information to support land use planning being developed by the National Flood Risk Advisory Group and expected to be releases in the second half of 2016.

5.5.3. Understanding and Treating Risk to the Existing Community

In some cases the consequences of flooding and the associated risks may warrant changes to the existing treatment of flood risk in a community to reduce the residual risks to a more acceptable level. In other cases existing treatment of risk may be considered adequate for this purpose.

Design flood estimation can provide the understanding of flood behaviour, and the drivers for this behaviour, across a range of flood events to support management of the flood risk. It can be combined with other information to:

- Assess the consequences of flooding on the community and the natural and built environment. One quantitative measure of consequences is the estimation of flood damages. This section provides an example of the use of flood damage estimation in flood risk management;
- Assess the impacts of floods on community infrastructure such as electricity, water supply, the sewerage system, medical facilities and emergency management infrastructure (evacuation routes and centres), and provide information to consider in recovery planning for the community; and
- Examine the effectiveness of treatment options to reduce this risk where warranted.

5.5.3.1. Estimation of the Current Risk to a Community

Figure 1.5.8 provides an example of a graphical representation of the variation of risk to the different elements (people, community, property) over the full range of flood behaviour for a particular floodplain. This can involve both qualitative and quantitative estimates of consequences for different flood events to determine risk levels using the example risk matrix provided in Table 1.5.1.

In this example, for events more frequent than a 10% AEP event the consequences to people, and the community are insignificant and therefore risks are low. However, the consequences for property are minor and therefore risk is medium. Consequences to people are major for floods rarer than the 10% AEP flood event. Consequences to property rise to major in unlikely, rare, very rare events and extremely rare floods. Impacts upon the community and its supporting infrastructure are moderate for events greater than the 10% AEP and do not reach major levels.

Likelihood of Consequence	AEP Range %	LEVEL OF CONSEQUENCE				
		Insignificant	Minor	Moderate	Major	Catastrophic
Likely	>10	People Community	Property			
Unlikely	1 to 10			Community	People Property	
Rare to very rare	0.01 to 1			Community	People Property	
Extremely Rare	<0.01			Community	People Property	

Legend

Risk Scale	Very Low	Low	Medium	High	Extreme

Figure 1.5.8. Example of Estimated Average Risk to a Community Due to Flooding

Figure 1.5.8 shows the risks to people and property for events between a 10% and 0.01% AEP event. The risk to the community is low except in floods between a 10% and 0.01% AEP event where they are medium.

5.5.3.2. Estimating Flood Damages to an Existing Community

One of the ways to quantitatively assess impacts to the community is to estimate flood damages. This generally involves an aggregation of estimates of flood damages on individual properties considering both direct costs (damages to structures and their content) and indirect costs (such as clean-up and disposal or materials, loss of earnings, sales and production, temporary relocation expenses) and an allowance for or estimate of infrastructure damages. However, it is important to note that this typically does not incorporate all risk factors, such as risk to life, and these additional risk factors may need to be accounted for through a qualitative assessment.

A range of methods are used to derive damages for individual properties. These include:

- *Rapid assessment techniques* - which rely on flood extents to determine the number of properties of different development types (residential, commercial and industrial) affected and apply a fixed damage per property.

- *Techniques based upon the use of stage damage curves* - for different development types and in some cases styles and sizes of buildings (an example for residential buildings derived from DECC (2007) is provided in Figure 1.5.9). This technique provides for variation in damage to structures and yards and their contents with depth above ground level and structure floor level. Approaches, such as this allow for the use of representative buildings within the floodplain. Whereas other approaches may require the style and size of houses to be determined and individual buildings to be considered in more detail.

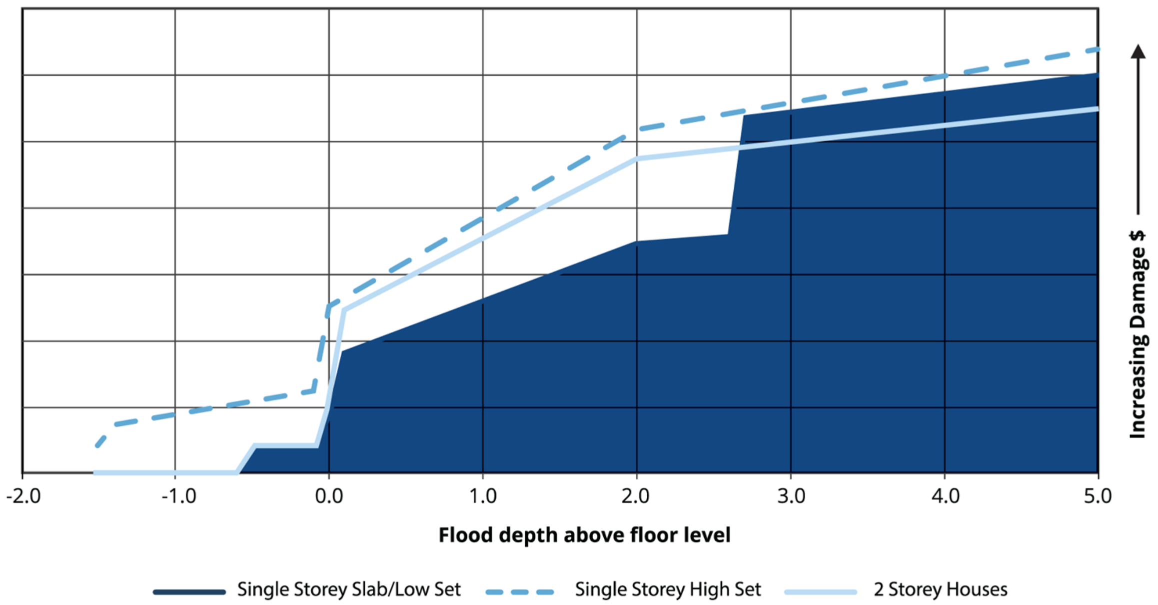


Figure 1.5.9. Indicative Stage Damage Curve for some Residential House Types

Note: There are many different stage damage curve relationships for residential development. Different damage relationships also exist for different types of development (e.g. commercial and industrial). These may be determined based upon the use of historical damage, insured loss information or based upon building component damage at different depths. However, no definitive set of curves exist and work in this area continues to evolve. Commercial and industrial damages can be very complex given the changing nature of the occupation of individual sites. The damage to the structure of the building will not generally change significantly with use but the contents damage can vary significantly. For example, the same light industrial storage area could house aluminium cans for recycling or computer components for assembly and therefore the damages to contents due to flooding would vary greatly.

Assessment based upon stage damage curves requires information on flood extents to determine which properties are affected and flood levels. This information can be used with location, ground level and structure floor level information (determined using survey or approximation methods) to estimate damages at an individual site. These can then be aggregated to estimate damages to a community or area.

Figure 1.5.10 provides an example of an aggregated flood damage curve across the full range of flood events for a community. This provides a quantitative understanding of the impacts of flooding upon the community and the built environment. It can also provide a baseline for considering the benefits of management options or infrastructure projects. Each point on the flood damage curve has a probability of exceedance in any given year. Examining this curve shows that there is no damage in a 20% AEP event with damage in the 0.5% AEP event being approximately $20 Million.

To use this information for flood risk management, particularly when examining the benefits of management measures, this information needs to be translated into an Annual Average Damage (AAD). This is achieved by determining the area under the curve. Book 1, Chapter 5, Section 12 provides an example of calculation of Annual Average Damages based upon the flood damage curve in Figure 1.5.10.

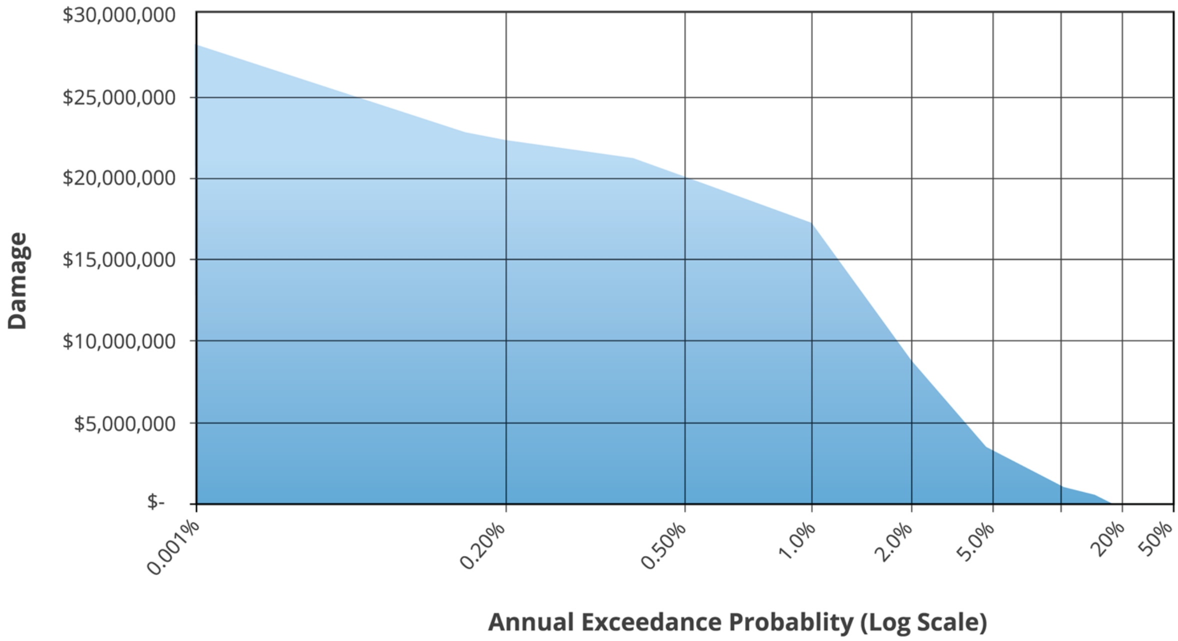


Figure 1.5.10. Example of Flood Damage Curve for a Range of AEP Flood Events

5.5.3.3. Assessing Options to Treat Flood Risk

Where treatment options are proposed that may change flood behaviour the calibrated and validated models that define the existing flood situation need to be altered to incorporate proposed treatment options and design flood estimates developed for the changed conditions.

Comparing this information to the existing flood situation can indicate changes in flood behaviour, and in combination with other information, changes in the consequences of flooding on the community. Flood extents, flood function, flood hazard and emergency response classification and damages may alter for specific areas and different design flood events. Where changes in behaviour are significant there are likely to be areas where flood impacts are reduced and other areas where they may be increased. These changes in consequence can be used to assess the benefits and costs to the community and the limitations of the treatment option.

Section 9.4 and Table 9.3 of AEMI (2013) outline some of the issues that should be considered when selecting and comparing treatment options. The benefits and costs of treatment options may be assessed singularly as well as in combination with complimentary measures as it is rare for a single treatment option used in isolation to effectively manage flood risk to a community. For example, a levee may be built to reduce flood damage in a town in combination with a flood warning system to provide additional warning and upgraded evacuation routes to improve community safety during floods.

One quantitative way of determining the financial efficiency of the project involves understanding the benefits in reduction in flood damages and comparing this to the costs of achieving and maintaining this benefit. A reduction in flood damages can be assessed by determining the reduction in Annual Average Damages and exposure of the community to flooding with treatment options in place. For example the use of minimum floor levels based upon the 1% AEP flood for new development, or a levee designed to exclude a 1% AEP flood from an existing flood affected area will reduce flood damages for events up to but not exceeding the design flood event (in this case 1% AEP event). However, the consequences of floods rarer than the design floods may not change significantly and there may still be substantial impacts upon the community.

Annual Average Damages calculated across the full range of flood events provides a sound basis for understanding the financial benefits and limitations of the project so this can be considered in decision making and enables the calculation of Annual Average Benefits.

Figure 1.5.11 provides an example of the estimation of the financial benefits of a treatment option. It shows the damage curve for the same flood situation as shown in Figure 1.5.10 but both without any treatment and with a treatment option in place.

In this example the treatment option is a levee. The aim of the levee is to reduce flood damages and the frequency of community exposure to flooding and associated risks for events up to the design flood event, in this case the 1% AEP event. Whilst there are some benefits for rarer floods these can be seen to diminish quickly in rarer events. In a 0.2% AEP event the damages with and without the treatment options would be the same.

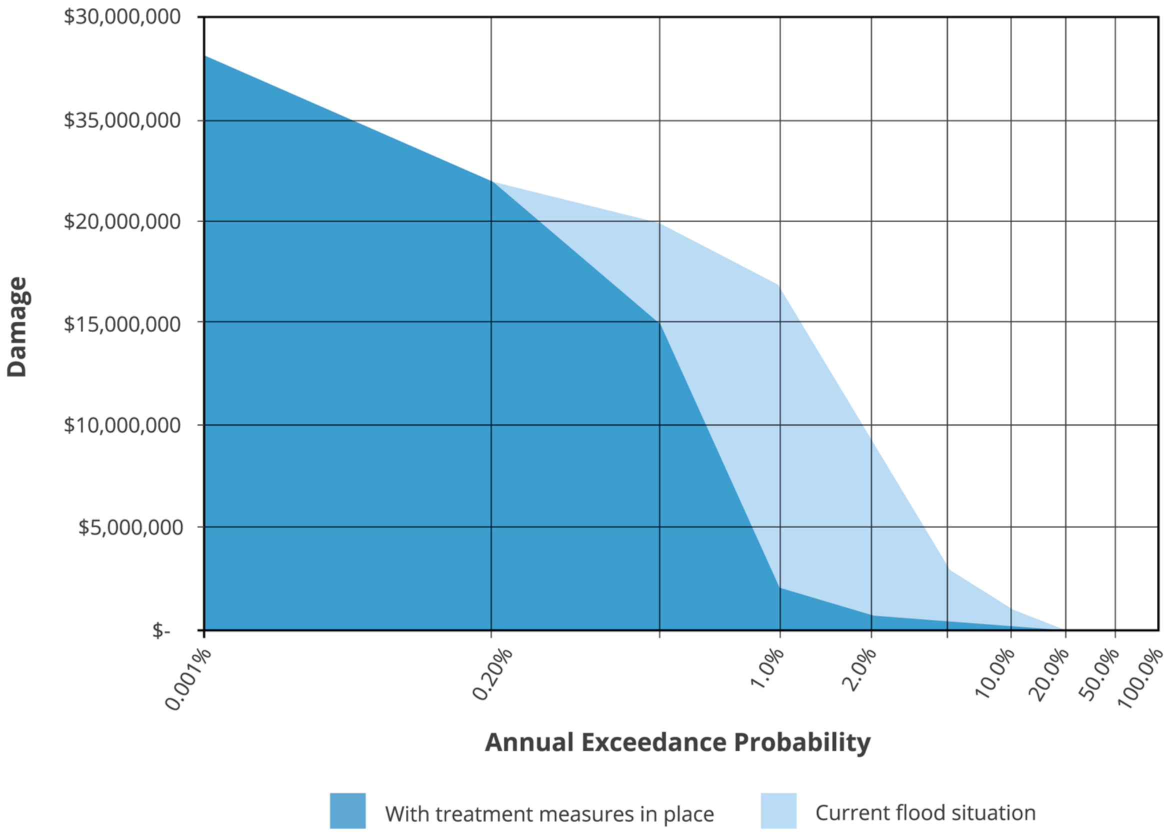


Figure 1.5.11. Example of Flood Damage With and Without Treatment Options for a Range of AEP Flood Events

The reduction in Average Annual Damages (AAD) or the Average Annual Benefit (AAB) can be used to determine the net present value of the benefits. An example is provided in Book 1, Chapter 5, Section 12.

This can then be compared with the Net Present Value (NPV) of life cycle costs of the treatment options to determine the Benefit Cost Ratio (BCR), which provides a measure of the financial efficiency of the project. Book 1, Chapter 5, Section 12 provides an example of estimation of Net Present Value of life cycle costs.

Book 1, Chapter 5, Section 12 provides an example of estimation of the Benefit Cost Ratio. Lifecycle costs and lifecycle benefits for individual years are shown in Figure 1.5.12. Figure 1.5.13 shows the same figures altered under current day dollars assuming a 7% discount rate.

The Benefit Cost Ratio calculated can be used in conjunction with consideration of other benefits, such as reduction in risk to life, reduction in the impacts upon community function and infrastructure and with similar information for other treatment options (including those providing protection for different AEP events) to inform decisions on managing risk.

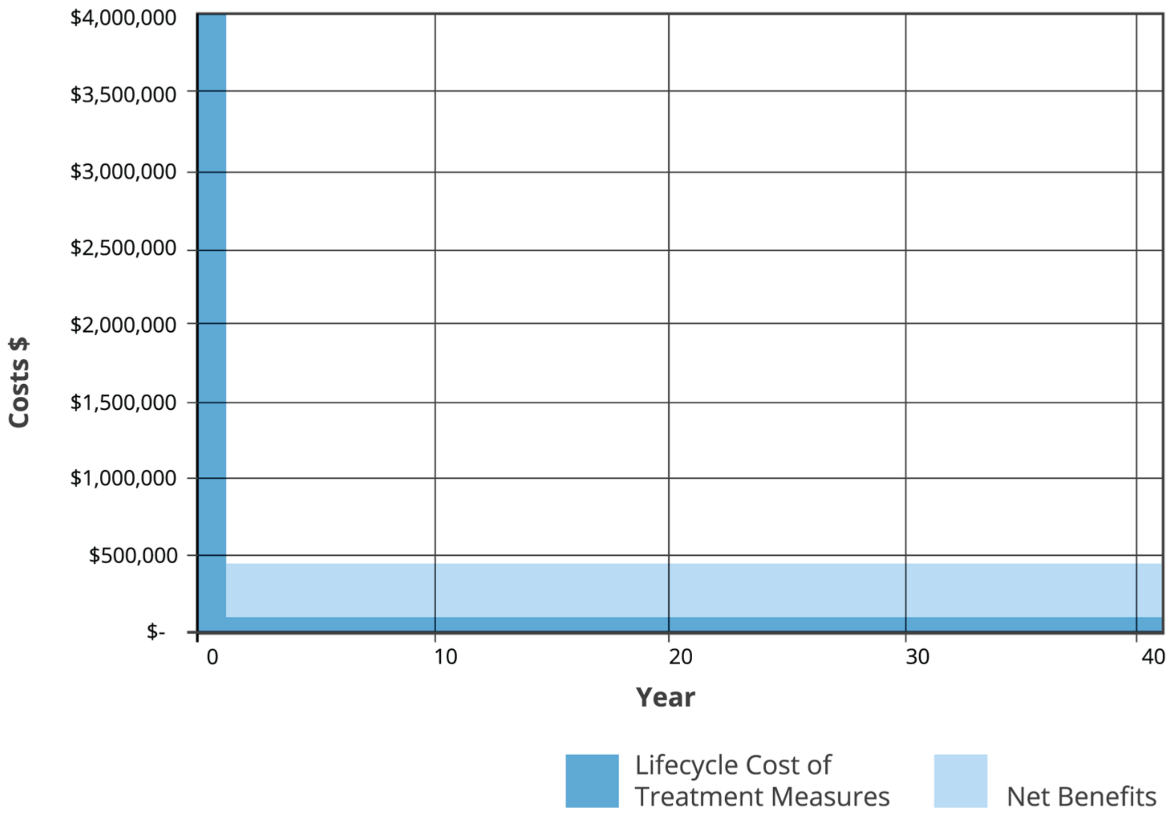


Figure 1.5.12. Example of Annual Lifecycle Benefits and Costs

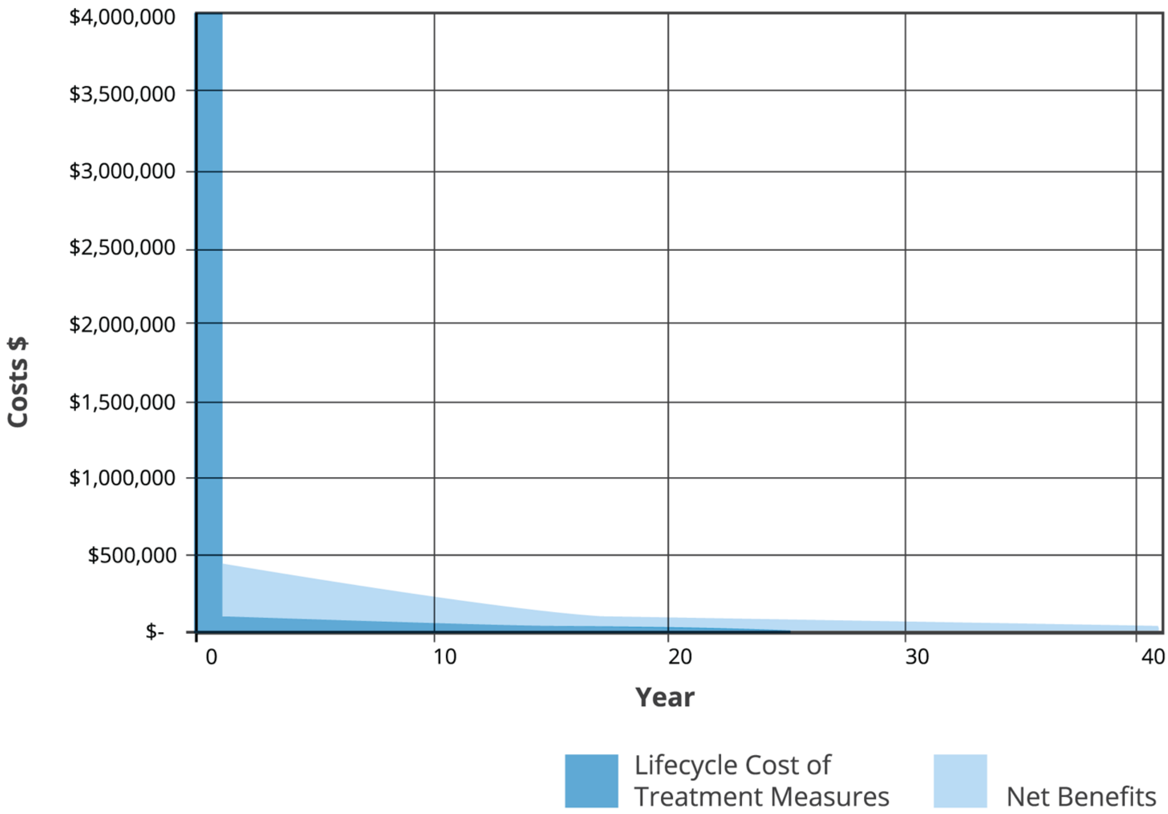


Figure 1.5.13. Example of Lifecycle Benefits and Costs Adjusted to Todays $ Using a 7% Discount Rate

Figure 1.5.14 re-examines the risks identified in Figure 1.5.8 to highlight how these have changed through the implementation of treatment options to provide protection for the 1% AEP event. This example shows a reduction in risk to property from a maximum of high to a maximum of medium in events above a 1% AEP event but low in events less than the design event.

However, risk to people is still high due to the impacts of events greater than the 1% AEP event. This may warrant additional treatment options being considered which, depending upon why this risk remains high, may include flood warning systems, improved emergency management planning and improvements to evacuation routes.

Note: there is no change to the risks in extreme events. The risk to property has been reduced in rare events due to the reduction in damages as a result of risk management measures.

Likelihood of Consequence	AEP Range %	LEVEL OF CONSEQUENCE				
		Insignificant	Minor	Moderate	Major	Catastrophic
Likely	>10	People Community PROPERTY	~~Property~~			
Unlikely	1 to 10		PEOPLE COMMUNITY PROPERTY	Community	~~People~~ ~~Property~~	
Rare to very rare	0.01 to 1			Community PROPERTY	People ~~Property~~	
Extremely Rare	<0.01			Community	People Property	

Legend: Consequences before treatment or where risk is unchanged, CONSEQUENCES AFTER TREATMENT WHERE CHANGED

Risk Scale	*Very Low*	*Low*	*Medium*	*High*	*Extreme*

Figure 1.5.14. Example of Estimating Changing Average Risk to a Community Due to Flooding with Instigation of a Treatment Option

5.6. Managing Flood Risks to Mining, Agricultural and Infrastructure Projects

As well as considering flood risks for existing and future development within communities as discussed above, the interaction of mining, agricultural and infrastructure (particularly linear above ground infrastructure such as road and rail embankments and levees) require management. Where located in the floodplain, these developments are:

- Susceptible to flood risk; and
- May impact upon flood behaviour with detrimental impact to others in the community.

In many cases, a design flood standard may not be available or appropriate, and a risk management approach as described in Book 1, Chapter 5, Section 4 may need to be undertaken. A general overview of some of the issues to be considered are included in Book 1, Chapter 5, Section 6 to Book 1, Chapter 5, Section 6.

Some of these projects or related projects with building of infrastructure can be considered short term projects due to their short term exposure to risk. Assessment of the risks associated with short term projects is discussed in Book 1, Chapter 5, Section 6.

In the same way that short term projects need special consideration, potential changes over the effective service life may need to be considered specifically for longer term infrastructure. Effective service life is discussed in Book 1, Chapter 5, Section 9, while potential implications of changes over the life of the project are discussed in Book 1, Chapter 5, Section 10.

5.6.1. Mines

Mines developed in the floodplain may require levees or similar flood mitigation measures. These levees need to be designed to an appropriate level of flood immunity and may also have an impact on flood levels outside the levee. The risk and potential damage caused by flood inundation both inside and outside the mine needs to be analysed, with a similar process to that used for community development discussed in Book 1, Chapter 5, Section 5.

There are usually key issues of concern for the mine:

- Risks associated with inundation of the mine and its operations. The risk for mining may be from flooding of the mine pit, emplacements, infrastructure, machinery or underground workings, which may disrupt production, damage equipment and result in a risk to life; and
- Risks associated with changes to flood behaviour for communities upstream or downstream. The risk may be associated with changes in flood behaviour, potential significant erosion and sedimentation deposits, polluted water from tailings dams etc.

The key difference between the two elements is that there will generally be two distinct groups of stakeholders in the risk assessment. These different stakeholders may have different risk profiles and this will need to be considered as a part of the assessment. Risks associated with the inundation of the mine will be typically be associated with the mining company, which may also incorporate workers' unions and insurance companies. Risks associated with the community may be associated with government, the local community, community interest and environmental groups.

As a result of these different stakeholders, the risks and associated risk profiles may need to be considered separately. There is also unlikely to be specific design flood standards associated with inundation of the mine, as it may be more driven by acceptable closure periods etc. Therefore, a full risk assessment (Book 1, Chapter 5, Section 4) may be required to derive appropriate management measures.

5.6.2. Agriculture

Flooding in agricultural regions can have concerns for crops, livestock and infrastructure. Crops may be damaged by inundation either: directly by floodwaters; or due to the extended period of inundation (where crops may be susceptible to longer term rather than short term inundation). Livestock may be lost if unable to be relocated to areas outside flood limits.

In order to determine appropriate risk mitigation measures for agriculture, the specific implications for livestock and crops need to be considered, and the risk assessment will need to incorporate these factors. In addition, agricultural infrastructure such as irrigation pipes, fences, buildings and machinery may be damaged and these can have significant value. Book 1, Chapter 5, Section 4 provides guidance on determining appropriate mitigation measures incorporating some of these different factors.

In some areas of high value agriculture, the farm land may be protected by levees (or other infrastructure) and these have similar issues to levees built for other flood mitigation purposes. The appropriate protection level of these infrastructure would be based on a risk assessment considering the above factors, as well as potential impacts to the community upstream and downstream.

The key stakeholder groups for undertaking a risk assessment may include:

- The farming operation(s) who will be directly impacted by the flooding; and
- Community both upstream and downstream, who may be represented by local government, community interest groups, environmental groups etc.

5.6.3. Road and Rail Projects

Road and rail embankments are built across floodplains in many situations. To ensure a suitable level of flood immunity, the infrastructure will be built on embankments which will usually cross watercourses. In this case, the infrastructure must be designed to ensure a suitable level of service (ie. considering the acceptable degree of disruption to transport services for the route), as well as withstand an acceptable risk of damage from scour, submergence, or overtopping of structures. The consideration of disruption to transport depends not only on the frequency of closure, or flood immunity, but also on the duration of closures, both during large floods and as an annual aggregate.

It also needs to consider the intended function of a road during a flood event, particularly where it has an important role in community evacuation or recovery plans. Design of embankments associated with these structures requires analysis of the road or rail level as well as the sizes, locations and types waterway openings. This is to ensure an acceptable level of flood immunity, duration of closure and damage from floods as well as an acceptable impact on upstream flood levels. Discussion of flood assessment and flood risk for road and rail projects can be found in the Austroads Guide to Road Design – Part 5 (Drainage)[1].

Key stakeholders may involve:

- Relevant road authority;
- Community groups, potentially represented by local government, community interest groups etc; and
- Relevant emergency response authorities and groups.

5.6.4. Short Term Projects

Short term or temporary projects are those that will only have a limited effective service life (refer Book 1, Chapter 5, Section 9). Some examples might include:

- Construction projects. For example, a coffer dam protecting an excavated area for a period of 6 months;
- A planned festival or community event in the floodplain, which occurs over a 2 day period; and
- Short term mining operations. For example, a levee to protect a portion of a quarry for a period of 3 months.

With short term projects, it is particularly important to understand the likelihood component of the risk assessment as well as the effective service life (refer Book 1, Chapter 5, Section 9).

Flood design standards are typically developed for long term projects and are based on an assumed effective service life that is generally many years. For example, a residential house might have an effective service life of 50 years. Therefore, when a 1% AEP design flood standard is adopted for the floor level, for example, that is equivalent to an approximate 39% chance that the floor level will be exceeded during the effective service life (using a SLEP terminology, as identified in Book 1, Chapter 5, Section 4).

However, if a 1% AEP flood design level is adopted for a coffer dam for excavation for an effective service life of 6 months (ie. a 6 month construction period), then the chance that it

[1]http://www.austroads.com.au/road-construction/road-design/resources/guide-to-road-design

will be exceeded will be roughly 0.5% during its service life. Therefore, assuming that the consequences remain the same, then the risk profile is significantly more conservative. If a 39% chance of exceedance during its effective service life was assumed to be more appropriate, then that would be equivalent to somewhere between a 50% and a 100% AEP event.

Therefore, a SLEP approach to flood design standards and flood levels can be more readily understandable for short term infrastructure.

However, a full risk assessment will typically be required to understand all the likelihoods and consequences and therefore the risks. For example, the risk to life for a coffer dam may be significant where sufficient warning is not available.

5.7. Managing Flood Risks in Relation to Dams

The guidelines relevant to dam safety is provided by ANCOLD (2003). These guidelines provide an over-arching framework that integrates risk assessment with traditional standards-based engineering practice. They provide guidance on the generic steps involved in undertaking risk assessment for dams, and these are updated periodically with changing understanding and practice.

The dams industry has used risk assessment over the past two decades as a valuable means to establish upgrade priorities and justify the urgency of completing dam safety actions in a transparent and rational manner. However, the national ANCOLD guidelines only support risk assessment as an enhancement to traditional standards-based solutions for important and conclusive decision making. The guidelines are currently being revised, and it is likely that there will be an increased focus on the use of risk-based criteria for final decision making in accordance with changing practice (for example, NSW Dam Safety Committee, 2006).

One of the key differences between dam safety management and floodplain management is the probability domain of interest: the scale and nature of life safety risks posed by dams are generally considerably greater than encountered in floodplain management. The tolerable risks associated with these potential consequences are three to four orders of magnitude rarer than those associated with natural floods. The concept of annualising a high consequence risk based on a 10^{-6} loading condition is mathematically straightforward, but such analyses are not easily combined with more common risks and have little practical utility.

Accordingly, risk assessment for dam safety is focussed on reducing the risks to life (and property) to as low a level as reasonably practicable. It is unusual for dam safety decisions to be governed by the need to balance the costs of upgrading works against damages avoided, and more typically such decisions are dominated by life-safety considerations.

The most relevant guidance on dam safety in this document is provided in Book 8. This provides guidance on the procedures most relevant to the extreme flood risks of interest. It also includes procedures relevant to estimation of the Probable Maximum Flood, which represents the upper limiting magnitude of flood is relevant to standards-based decision making.

5.8. Managing Flood Risks using Basins

Basins can have an important role in reducing downstream flood flows and associated flood risks to the community. They may be built to manage existing community flood problems or to offset the impacts of upstream development on downstream flood risk. A basin may be used in isolation or as part of a series of basins within a catchment to reduce peak design flood flows and risks for the design event(s) at key downstream locations. The design performance requirement is therefore generally either:

- To reduce peak flows for a certain design event(s) to a certain maximum amount. For example, for a basin designed to offset the impacts of upstream new development this may be the pre-development peak downstream flow. For a basin designed to reduce downstream flood impacts this may be to reduce peak basins discharges to a level that reduces flood impacts on the downstream community to an agreed level; and
- To maximise the potential benefit of a basin at the location on downstream flood behaviour to reduce impacts on the community.

An effectively designed basin has to balance restriction of outlet capacity with having available storage capacity near the peak of a flood event. This enables the peak of flood flows to be stored and the stored volume discharged later in the event, as illustrated in the example in Figure 1.5.15.

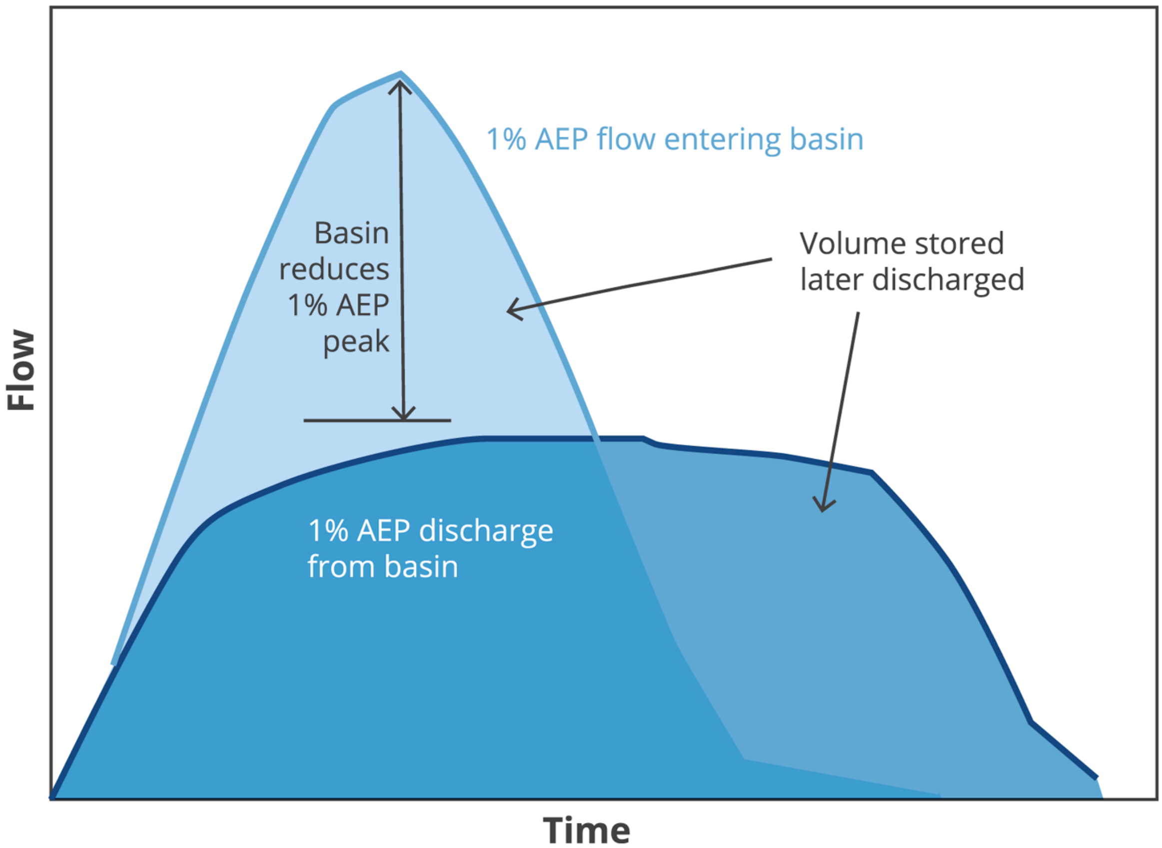


Figure 1.5.15. Example of Impacts of a Basin on Flood Flows in a Design Event

This can significantly alter the critical storm duration with the peak flood flow entering a basin likely to be derived from a shorter duration storm than the peak discharge flow from the basin. Storm pattern can also have a significant impact on basin operation and the storage volume available when the peak of flood flows arrives. For example, if the peak arrives later in the storm the available storage volume may be lower and therefore the basin may have less impact on downstream peak flows.

In addition, critical storm durations are also likely to vary with the AEP of the flood being modelled. It is not unusual for the peak basin discharge in a more frequent flood than the design event to occur in a longer duration storm event as storm volumes are lower and the basin storage will have more impact upon peak discharge. However, for events larger than the design event the opposite is true. There is likely to be less storage volume available at the storm peak so it will have less influence on peak basin discharge. Therefore the critical downstream discharge from rarer events than the design event will likely occur from shorter duration storms. For extreme events this is likely to be closer to the critical storm duration at the location without the basin.

Therefore with a basin in place the peak downstream flood flow is sensitive to both the storm duration and the storm pattern and the routing of flows through the basin. As such basin design can be particularly sensitive to both storm temporal pattern and critical storm duration.

Robust design approaches for basins that test and consider a wide range of storm durations and a range of potential variations of storm pattern for each of these storm durations are recommended for the full range of flood events. Variations in storm pattern should include testing of early, centrally and late weighted storm patterns for the same time duration to test whether the basin can meet the required design criteria with this variation.

Other key points to consider in modelling and designing detention basins include:

- *Considering the impacts of events larger than the design event* -The basin will generally be designed to reduce flood flows in a particular design event. However, in the majority of cases it is unlikely to have significant impacts on peak flows in extreme events. This can mean that there is a significantly larger difference between the extreme and design event flows entering and discharging the basin. Figure 1.5.16 provides an example. This situation is likely to result in a faster rate of rise of downstream flood levels for events larger than the design event (as these events result in high level spillway operation) than would have occurred without the basin. It is essential that this difference is understood and considered in basin and high level outlet design to manage the flood risk downstream of the basin. Residual risk downstream of the basin, including any limitations in emergency response and associated planning, needs to be considered and may require additional management measures including flood warning, community awareness, flood related development controls;

- *Upstream impacts of basins* - The construction of a basin can also have significant upstream impacts on flood behaviour and these are an important consideration in the design of a basin. This should be examined for the full range of flood events to ensure that any impacts on upstream flood risk and the management of this risk (including emergency management planning) are understood;

- *Detentions basins act as dams during flood events* - Therefore, basin design needs to consider dam safety aspects as discussed in Book 1, Chapter 5, Section 7 and Book 8; and

- *The use of multiple basins in a catchment* - Where multiple basins are designed to provide more strategic benefits, ie., away from the downstream boundary of their individual locations they should be designed on a catchment wide basis to ensure their interaction does not result in adverse impacts upon flood behaviour. Use of multiple basins in a catchment without consideration of interaction has the potential to result in adverse impacts on flood behaviour.

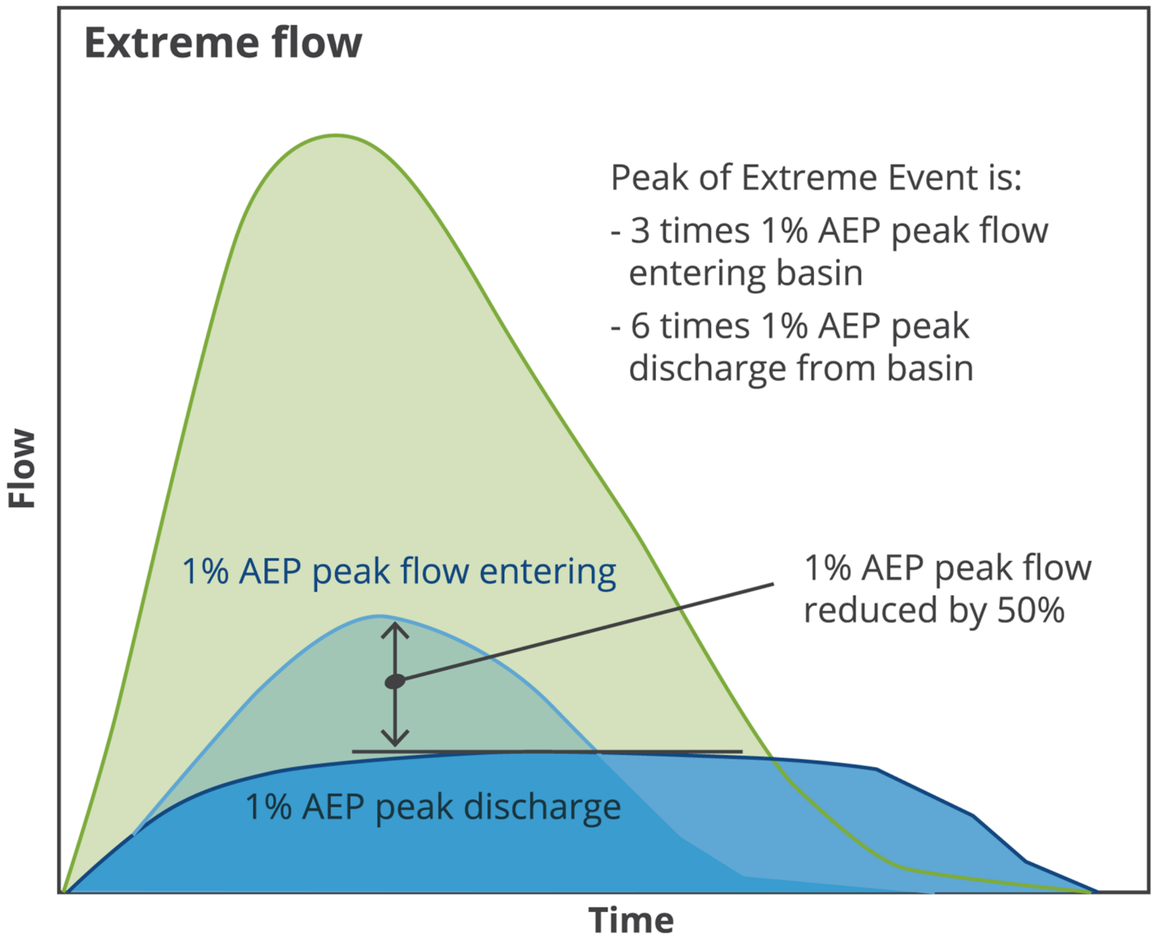


Figure 1.5.16. Example of Difference in Impacts of a Basin on Flood Flows in a Design Event Compared to an Extreme Event

5.9. Effective Service Life of Infrastructure

The longer the operational life of an infrastructure proposal is, the greater the potential for changes to occur in terms of risk, in terms of likelihood and consequences due to either changes in the catchment or floodplain or climatic changes. Typically, the duration of a proposal may be considered in terms of:

- *Economic Service Life* - The total period to the time when the asset, whilst physically able to provide a service, ceases to be the lowest cost option to satisfy the service requirement;
- *Design Service Life (DSL)* - The total period an asset has been designed to remain in use; or
- *Effective Service Life (ESL)* - The total period an asset remains in use, regardless of its Design Service Life.

Currently most guidelines are based around evaluating design service life. However, the difference between the ESL and DSL can be significant and should be recognised in risk assessment of a proposal (Figure 1.5.17).

ESL can be enhanced by factors which increase life such as maintenance, or diminished due to factors that reduce life such as significant weather events. It is considered that a proposal's ESL is of primary importance in risk assessment and should be considered in evaluating risk and setting design flood standards.

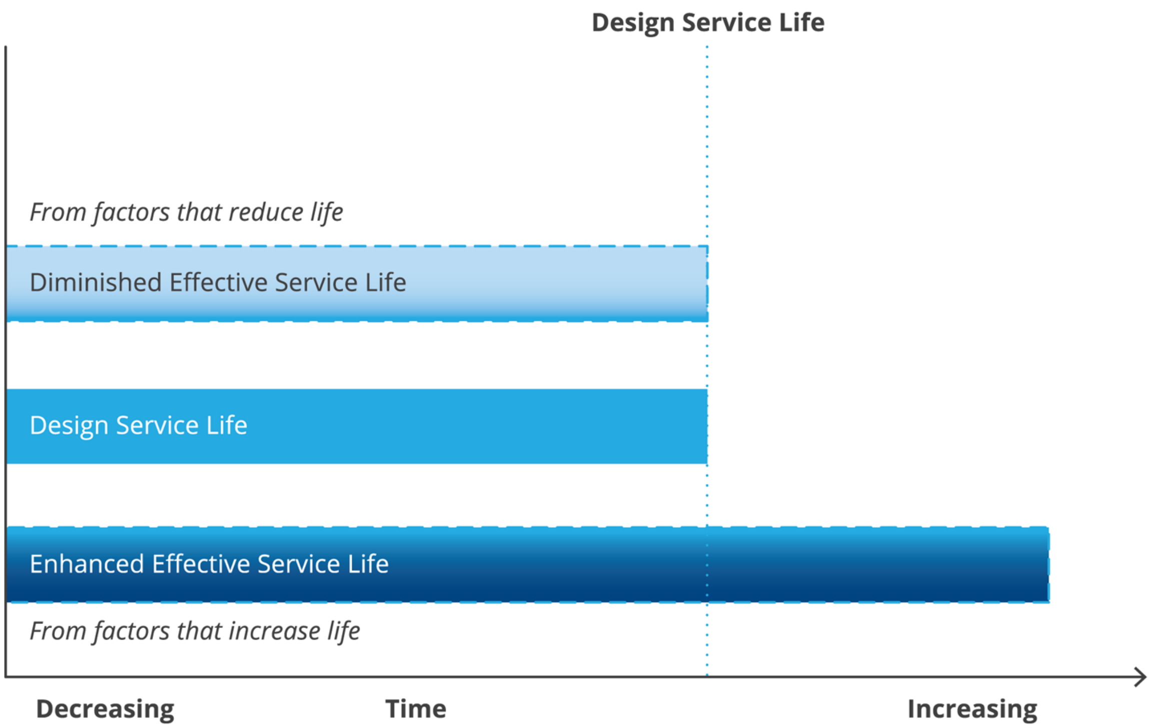


Figure 1.5.17. Design Service Life versus Effective Service Life (derived from United States Environment Protection Agency – 2007)

5.9.1. Estimating Effective Service Life

The effective service life of a particular project may be difficult to estimate. The following conditions are likely to lead to the effective service life extending past the design service life:

- *Magnitude of infrastructure* – very large infrastructure projects are more likely to remain in place longer than smaller projects (e.g. bridges, dams), due to the difficulties in replacing them;
- *High decommissioning or replacement costs* – where decommissioning or replacement costs are high there may be strong economic incentives to continue utilisation of the project (e.g. buried pipes within an urban environment); or
- *Integrated development* – where the project forms part of a broader piece of infrastructure or on-going service there may be economic incentives to continue utilisation of the project, particularly where a change to one component would require a change to others (e.g. road alignment – a road may be reconstructed/ rehabilitated over time, but due to other constraints, will not be able to be changed in terms of elevation or geometry).

Determining the effective service life of infrastructure is complex as it is a product of infrastructure design, materials, environment, maintenance and rehabilitation regime and use. For example, exposed infrastructure (e.g. roads) typically has a lower service life in tropical climates than in sub-tropical climates. Similarly, pipes that lie below a water table typically have shorter service lives than pipes that lie above a water table. The rate of degradation of construction material (e.g. metal, plastic pipes) will also vary with circumstance (e.g. saline vs non-saline conditions). Maintenance and rehabilitation of infrastructure may also seek to extend a project's expected service life (for example, a lining installed in a stormwater pipe).

Table 1.5.2 summarises some of the typical life expectancies (and range in life expectancies) for various infrastructure types. Table 1.5.2 shows that the range within and between infrastructure is high. Within Australia, data for long-lived assets is limited as the majority of the infrastructure has not yet reached its effective service life.

Table 1.5.2. Infrastructure types and potential Effective Service Life[a]

Infrastructure	Effective Service Life expectancy
Water Treatment Plants	20 - 50 years
Concrete Kerb and Gutters	40 - 70 years
Stormwater Pipes	80 - 100 years
Wastewater Systems	50 - 80 years
Residential Buildings	40 - 95 years
Roads	35 - 110 years
Commercial Buildings	15 - 150 years
Open Stormwater Channels	10 - 100 years
Locks and Weirs	40 - 200 years
Dams	50 - 500 years

[a]Data represents a synthesis and interpretation of a number of reports including: IPART (2012), Cardno (2014), USEPA (2014), International Transport Forum (2013), Tonkin (2009).

For projects which have a relatively short design / effective service life (e.g. less than five years), it may be reasonable to assume the risk profiles faced are static for the duration of the project as the likelihood or magnitude of changes to any risks may be negligible in comparison to the overall risk level. In contrast, longer life-span projects will be exposed to a higher level of non-stationary risk which could be considered in design.

5.10. Estimating Change in Risk over Time

Conditions in floodplain are not static. They vary over both in the short and long term and can affect the likelihood or the consequences of flooding. Some of these potential changes (or sources of non-stationarity) are discussed below.

“In many cases, flood studies reflect current conditions at best, and more likely past conditions since the studies often rely on old data …. …..flood risk criteria used to site and design a project should rely on conditions the location is likely to experience during the project's lifetime, not past or current conditions.” (Floodplain Regulations Committee, 2010).

5.10.1. Changes to Likelihood

The likelihood of given magnitude events occurring may change over time due to a large number of variables including:

- *Seasonality* - the seasonality can alter the statistical likelihood of flood events occurring as some events. For instance some catchments tend to have are prone to flooding associated with summer climates;

- *Climatic Variability* - Various weather patterns such as El Niño influence the likelihood of flooding. El Niño events have a life-cycle during which the impacts vary, both in terms of spatial extent and timing;

- *Evolving hydrological / hydraulic estimates* - The likelihood of a given magnitude flood event can vary significantly due to evolving hydrological / hydraulic estimates. For instance expected peak flows derived from flood frequency analysis can be altered by revisions made to the rating table or by extending the record length to include / exclude extreme events;

- *Climate Change* - Fundamental changes in the climate will alter the likelihood of flooding. This is discussed in Book 1, Chapter 5, Section 5 and Book 1, Chapter 6; and

- *Changes within the catchment and floodplain that alter flood flows and flowpaths*. This is discussed in Book 1, Chapter 5, Section 5.

This section discussed the first 3 dot points.

5.10.2. Changes to Consequence

Consequences of given magnitude flood events can change due to a wide range of variables including:

- Land-use change – change to a more vulnerable land use or to the degrees of exposure of development to flooding.

- Economic changes such as inflation.

- Changes to the community exposed to risk through long term or seasonal changes in the total population or its demographics. For example increases in population at holiday destinations and during festivals. A change in community demographics to include a higher proportion of people more vulnerable in emergency response can lead to increased consequences due to a flood.

- level of flood awareness / education in the community. The higher the level of community awareness of flooding the more the community will be to flood risk due to both their understanding of the need to and how to respond to a flood and the knowledge that they may need to take measures, such as having flood insurance to address some of their residual risk to flooding.

5.10.3. Changes to Risk Preference

Further, it is noted that, just as the components of risk (likelihood and consequence) may vary over time, so to an individual's evaluation of the ultimate importance of that risk may alter. This may occur through:

- *Altered risk profiles* – Individuals tolerance for risk varies with their past and recent experience community attitudes to flood risk can vary significantly before and after a flood event; or

- *Risk discounting* – The value of a risk realised at a future time is typically considered of less significance than a risk realised at a current time. The rate at which this is applied may vary over time; typically the discount rate used reflects the economic discount rate in financial systems.

5.10.4. Literature

There is a range of literature that discusses how changes over time non-stationarity can be applied in risk assessments. Much of the work to date has occurred in the academic space (such as those of (Rootzen and Katz, 2013; Åström et al., 2013; Salas and Obeysekera, 2013)). This work indicates that there is potential for non-stationary models to be incorporated into design considerations and that the scale of catchment change may be of sufficient magnitude in some catchments to warrant consideration in design criteria. However, the costs of developing such models and assessments is likely to be prohibitive and unnecessary for the majority of flood- affected infrastructure, and that utilisation of traditional static risk profiles remains the more appropriate form of assessment.

For example, a method for incorporating design flood standards and design life into risk assessments is presented in (Rootzen and Katz, 2013). The paper proposes two methods to quantify risk for engineering design in a changing climate:

- The Design Life Level aims to achieve a desired probability of exceedance (or risk of failure) during the Design Service Life. This method is a SLEP approach (Book 1, Chapter 5, Section 4); or
- The Minimax Design Life Level is closely related, and complementary, but instead focuses on the maximal yearly probability of exceedance during the Design Service Life. This method is an AEP based approach.

The Design Life Level uses a Generalised Extreme Value (GEV) cumulative distribution function (cdf) to present the extremes in year *t*, and with increasing location and scale parameters (the shape parameter is constant) related to *t*, the probability changes. The example likens the increase in location parameter to a possible increase in water level, and scale parameter to an increase in climate variability. Another parameter is also introduced, the Expected Waiting Time (EWT) - the amount of time until a particular level *u* is exceeded.

Under this approach, if what is considered an acceptable level of risk is constant, it may be desirable to design mitigation measures (e.g. Design Flood Standards) such that the likelihood of a given consequence is constant in time. Figure 1.5.18 shows that if risk is increasing through time, then to keep the standard of risk protection constant, it would be necessary to continuously raise a defense. Clearly for many projects it is not possible to continually increase the capacity of flood protection measures, therefore if the mitigation measure is of fixed capacity the standard of risk protection varies with time (Figure 1.5.18).

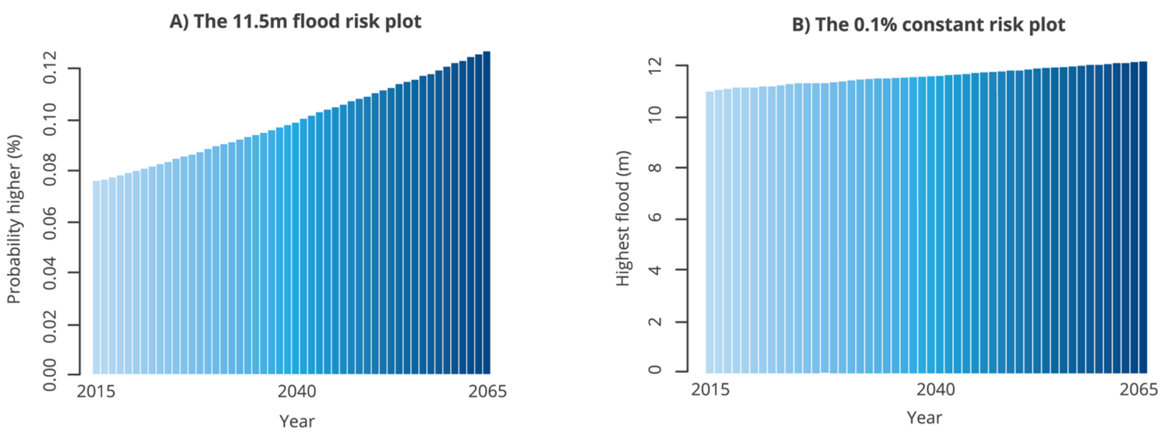


Figure 1.5.18. Flood Risk Plot Versus Constant Risk Plot (derived from Rootzen et al 2012)

Similarly, Salas and Obeysekera (2013) present a framework for addressing non-stationarity in risk assessment. The non-stationarity is considered in terms of increasing frequency of events, decreasing events and random shifting events, with standard return period and risk parameters. In the case of increasing extreme events, the exceedance probability of floods affecting structures also varies through time ie. $p_1, p_2, p_3, \ldots, p_t$. The sequence of p will also be increasing (Figure 1.5.19):

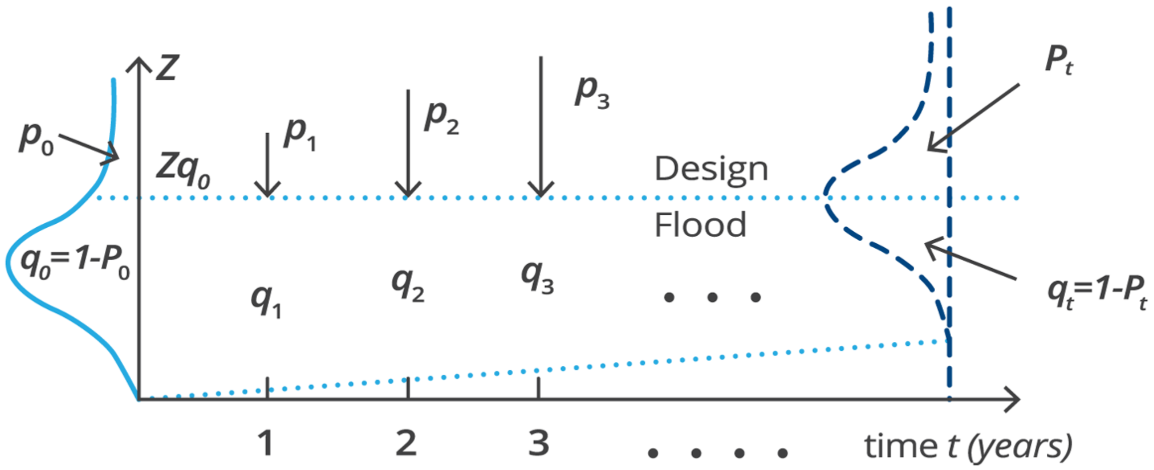


Figure 1.5.19. Schematic of a Design Flood with Exceeding (P_t) and Non-Exceeding (q_t = 1-P_t) Probabilities Varying with Time (Salas & Obeysekera, 2014)

This means that if the probability of the first flood exceeding the Design Flood Standard at time x = 1 is p_1, then the probability at time x =2 is $(1 - p_1)p$. In general, the probability that the first flood exceeding the Design Flood Standard will occur at time x is given by:

$$f(x) = P(X = x) = (1 - p_1)(1 - p_2)(1 - p_3)...(1 - p_{1-x})px \quad (1.5.1)$$

This geometric distribution is developed further for application in a non-stationary framework to allow the waiting time for the first exceedance of the Design Flood Standard to be calculated.

However, given the complexities of non-stationarity, the number of practical studies incorporating non-stationarity within flooding infrastructure is relatively low (e.g. Vogel et al. (2011), Condon et al. (2014), Ng and Vogel (2010) and Woodward et al. (2011)). Furthermore, there is limited application of these techniques in policy framework documents. In contrast, the majority of countries currently adopt coarse climate change adjustment factors to account for non-stationarity. Approaches within Australia are discussed in Book 1, Chapter 6[2].

However, there is broad guidance, such as AEMI (2013) that highlights the importance of considering how the floodplain and catchment will change overtime by encouraging both the understanding of cumulative impacts of new development and also considering the influence of a changing climate. These are generally undertaken separately to identify the sensitivity of changes due to these different changing factors. It is rare that all non-stationarity factors are considered together.

[2] This section was written before the latest climate change guidance in Book 1, Chapter 6 (2024). A minor change to the text has been made to reflect the change in guidance.

5.10.5. Non-Stationary Risk Assessment

Non-stationarity is typically included in community risk based assessments (Book 1, Chapter 5, Section 5) by considering catchment and floodplain changes and climate change in understanding and managing flood risk to a community. However, for other infrastructure project, such as those identified inBook 1, Chapter 5, Section 6 to Book 1, Chapter 5, Section 8, non-stationary factors may need to be considered for the assessment.

5.10.5.1. Is a Non-Stationary Risk Assessment Required

In general, uncertainty in risk likelihood and consequence increases with a project's Effective Service Life. Some discussion on estimating effective service life are provided in Book 1, Chapter 5, Section 9.

Once the effective service life has been determined an assessment of the potentially likelihood of changes to risk (likelihood or consequence factors) should be undertaken. A discussion on climate change, and whether this is important for consideration, is provided in Book 1, Chapter 6.

Where a reasonable risk of change in other sources of non-stationarity is identified, a non-stationary risk assessment may be a preferred risk assessment approach. While this can be difficult to evaluate the following broad guidance is provided: in general, for medium to long term infrastructure (ie. with effective service life of greater than 5 years), it is suggested that:

- If the effective service life is less than 20 years a stationary risk assessment should be undertaken;
- If the effective service life is greater than 20 years but less than 50 years it is recommended that a non-stationary risk assessment be considered, except in areas in which the likelihood of change in local and regional land uses is minimal; and
- If the effective service life is greater than 50 years then it is recommended that a non-stationary risk assessment be undertaken.

The above is a general guidance, and does not take into account project specific issues. For both short-term and long-term infrastructure it is recommended that an initial review be undertaken that evaluates whether or not changes in likelihood and/or consequence (as listed above) are likely to occur over the Effective Service Life of the project and considering whether such changes would impair the project's ability to perform its intended function.

5.10.5.2. Non-Stationary Risk Assessment

Where non-stationary risk assessment is being considered, a process similar to non-stationary risk assessment can be undertaken. However, the non-stationary nature of the risks present will influence the design horizon over which the assessment is undertaken.

Rather than adopt the more complex models identified in Book 1, Chapter 5, Section 5, it is recommended to adopt a simple "time slice" approach. Primarily there are three approaches able to be adopted:

- **Undertake risk assessment at the point in time of highest overall risk (T(max))** : Typically, this may be at the end of the project's ESL. By applying the risk assessment at T(max), and determining appropriate design criteria for this point, the proponent will effectively design its infrastructure to be acceptable at all points of its ESL. This is considered to be the most conservative approach and will lead to relative over-engineering of infrastructure at some points of its life, particularly early in its life.

- **Undertake risk assessment based on the existing environment (T(0))** : This approach accepts that non-stationary components have impacts on flood behaviour, risks will rise. This approach assumes that this growth in risk will be acceptable and therefore it will lead to under-engineering relative to current minimum design requirements as risks rise. This is considered the least conservative approach and the most likely to result in higher long term risks.

- **Undertake risk assessment based on the existing environment (T(0)a) and commit to managing residual risk as it arises** : This approach will require periodic reassessment of risks associated with the project at agreed points in time. This approach may lead to under-engineering towards the end of the re-evaluation period and is considered the second least conservative approach.

- **Undertake risk assessment at a representative point in the projects ESL (T(x))** and commit to managing the residual risk: This approach will likely lead to over-engineering in the initial (pre – T(x)) period, after which it will require periodic reassessment of risks associated with the project at agreed points in time.

Each of these four options revolve around the choice between conservatively over-engineering to ensure risk levels are satisfied, against programs of continuous upgrades in which changes in risk may be responded to through adaptation in design.

In general the T(max) approach may be identified as the preferred approach where:

- The magnitude of change in risk is well known and likely to be small;

- A project's effective service life is certain;

- The costs of over-engineering are low; or

- The potential for retro-fitting / incorporating adaptability is low.

In contrast, The T(0)a and T(x) approaches are more likely to be favourable where:

- The potential change is risk is high or uncertain;

- The projects effective service life is uncertain;

- The costs of over-engineering are high; or

- The potential for incorporating adaptability is high.

There may be thresholds or tipping points at which the frequency of flooding or the consequences (e.g. loss of life, damage to residential property) of that flooding or the associated risks are considered unacceptable to the community. If these can be identified they can provide a basis for considering the limit of tolerability (LoT). With knowledge of the anticipated rate of change in the likelihood or consequence of flood events over time, it may be possible to approximate the time at which the LoT will be exceeded for any one consequence. Based upon current understanding of any such limits, beyond this point, flooding in association with a given project may be considered generate unacceptable risks.

This concept is basically a trigger based concept. For example, residential properties may be designed to a 1% AEP level, which is acceptable to the local authority. However, they are willing to tolerate a 2% AEP level if unavoidable. Currently, a residential development is built to a 1% AEP level. However, due to changes in the catchment (increase in impervious areas in the catchment) and climate change, this is expected to reduce to a 2% AEP level after 30 years. This represents a trigger point at which time a mitigation measure may need to be implemented.

Such points may be utilised as the minimum points at which risks, and the appetite for risk, are reassessed (T(0)a). Where current planning allows for adaptation, these can be considered the protected timeframe where this adaptation may be necessary based upon current knowledge and projections (T(x) Figure 1.5.20).

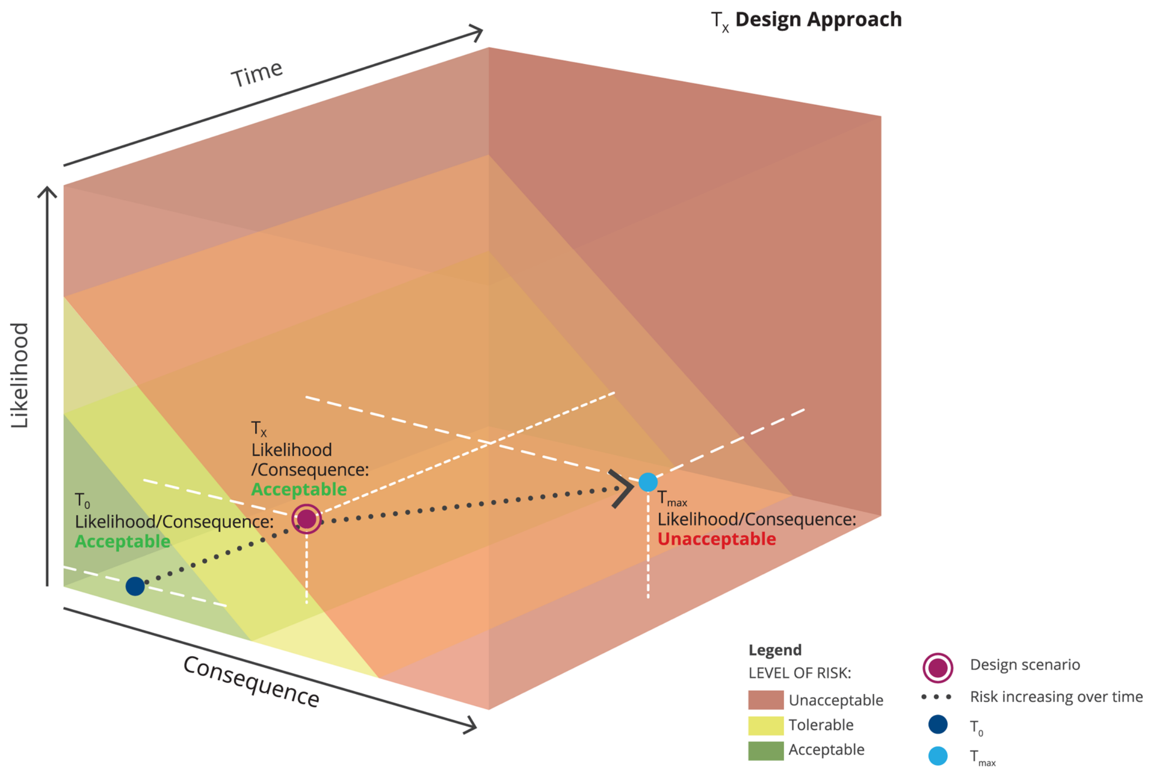


Figure 1.5.20. Change in Realised Risk Through Adopting a T(x) Design Approach.

From a practical perspective, consideration of the potential need for and likely methods for mitigating future growth in risk as part of the original decision can enable this work to be incorporated into upfront decisions (DECC, 2007). For example, land can be set aside or easements established to enable construction and maintenance of future mitigation measures. If this does not occur the mitigation measure may not be able to be implemented when required. Consideration of the costs of future mitigation in current decisions may in some cases influence the original decision on protection levels.

It is also noted that any option based around the future upgrade of infrastructure poses potential legal and commercial risks to both proponents and approval authorities. Given the extent of timeframe over which the infrastructure may be in place, the responsibility (and cost) of re-evaluation and upgrade in the future may change between individuals and there is the potential that the decision to upgrade at that time is not viable. In such circumstances the project may be decommissioned (these costs should be considered in any supporting economic analysis).

5.10.6. Economic Assessment

For non-stationary risk analysis, design options may be assessed through standard CBA evaluation of options. However, it is recommended that the changes over time be incorporated into the analysis. For example, the reduction in protection provided by a levee protecting a coastal town as flood levels change due to the increasing influence of coastal flooding as sea level rises due to climate change.

A simple approach for incorporating the changes over time is to take two or more time slices. For example, the flood inundation damages are calculated in year 0 and at the end of the Effective Service Life. Then the damages are assumed to change linearly between these two points. If the change is expected to occur differently, the more time slices may be required.

Adopting the methodology in Book 1, Chapter 5, Section 5, the key change is that the Annual Average Damage will progressively change over the Effective Service Life of the project. This can then be included into the economic assessment as described in Book 1, Chapter 5, Section 5.

There are several key considerations in undertaking economic analysis in non-stationary conditions:

Analysis Period

The inclusion of non-stationarity results in benefits and costs that vary through time. The important implication of this is that an economic outcome can be dependent on when the economic analysis period commences. For example, if we pay to construct a levee now to protect against climate change, then the cost is incurred in the present, but no real benefit may be realised for some time, for example twenty years. When the time value of money is considered in this scenario, it may be worth not investing in the levee until twenty years have passed.

This introduces a complication when comparing a number of alternative projects. Traditionally, you would be able to prioritise economically between projects based upon the larger BCR. However, the impact of non-stationarity introduces the dimension of time to the analysis. In other words, a project may not be viable now but may become viable in 10 years time. This type of assessment was applied in Thomson et al. (2011) and Thomson et al. (2012) for studies in the Solomon Islands.

An economic analysis under non-stationary conditions should be expanded to incorporate variable option implementation timing (ie. run CBA scenarios in which the project is developed in different phases over time). A CBA analysis could assess a range of scenarios (including staged scenarios) that capture this changing nature of costs and benefits over time.

Discount Rate

A challenge of non-stationary factors like climate change is that impacts which are experienced further into the future are diluted by standard discount rates. There is significant research that has been undertaken on appropriate discount rates for very long time periods. This is based on the argument that intergenerational equity should be considered, and that future generations should not be unfairly weighted compared with existing generations (Stern, 2006). This becomes particularly important for projects which are expected to have a relatively long effective service life, and therefore the benefits will span across multiple generations. Some recent studies, such as the Stern Review (Stern, 2006) have adopted very low discount rates (in the case of the Stern Review, 1.4%).

Some economists argue for zero discount rates when applied to environmental impacts. Quiggin (1993) goes further and suggests that the use of zero discount rates is in agreement with sustainability, where

"The interests of future generations should be given equal weight with our own in making decisions affecting the long term future"

Farber and Bradley (1996) suggest

"those changes that would enhance or degrade the human life support capacity of the ecosystem…. would not be discounted at all"

Philbert (2003) reviewed a number of approaches, and discussed that the discount rate may be derived not from the consumer perspective, but on a more general society perspective, based on "interpersonal comparisons". There are inherent difficulties with this analysis. Philbert (2003) argues that the current generation will incur costs for the benefit of future generations, but there will reach a point at which perceived gain for the next generation is not seen as worth the cost for the present generation. Therefore, a zero discount rate should not apply, as it would imply "very high investment by the present generation". Philbert (2003) suggests a discount rate that decreases over time, approaching the "lowest reasonably foreseeable rate" of economic growth. Philbert (2003) also suggests potentially increasing the value of environmental assets, as these are progressively consumed moving into the future.

Most of the discussion in relation to varying of discount rates focuses in on climate change and environmental policies. These policies, such as emission trading schemes, can result in costs now, in order to off-set potential significant costs in the future and impacts that may not be possible to be rectified by future generations.

The challenge with varying the discount rate is that it results in difficulties in comparing across projects. For example, if one project adopts a discount rate of 7% and another a discount rate of 4%, then it is difficult to directly compare the BCR results of such studies without thoroughly understanding the underlying assumptions.

For the purposes of the majority of water related infrastructure, and to ensure consistency across projects, it is recommended to adopt typical discount rates. Treasury guidance may recommend discount rates to provide consistency across competing projects. However, the above should be kept in mind for policies and planning that is targeting impacts well into the future.

Alternative Economic Models

Alternatively, a more robust economic analysis approach would be the use of Real Options Analysis (ROA). ROA is a recognised approach to address the uncertainties of future conditions in flood risk management by accounting for flexibility in investment decision (World Bank, 2010; Short et al., 2012; Park et al., 2014; HM Treasury and DEFRA, 2009).

The standard CBA approach is a relatively coarse mechanism in which costs and benefits assessed are considered as a whole and do not allow for discernment of the manner in which they accrue (e.g. changes in the rate at which benefits are received may not justify development of all project components as part of initial project construction). Neither does the analysis recognise that estimates of cost, benefit and risk into the future are inherently uncertain. As such, deferring decisions on infrastructure investment until a later date when more information is available may be the preferred approach. ROA allows the value of deferring investment decisions to be assessed.

In ROA, “options” represent predefined choices over a project’s Effective Service Life that strategically or operationally affects the course of the project. For example, a project may be to build a levee. If we build the levee in such a way that it is possible for it to be upgraded at a later date, a real option may be to increase levee height by 0.5m. The analysis defines decision points at which these choices are made (e.g. T(x)). The decision points can be points may be fixed in time or variable and triggered by internal or external events (e.g. occurrence of a 1% AEP event). Based on the Black-Scholes model utilised in financial options analysis, ROA utilises estimates of volatility (ie. the likelihood of a particular flood event or level of damage occurring in one year) to evaluate expected values/damages that are likely to be incurred over the Effective Service Life. The establishment of appropriate volatility measures is critical to ROA, and not all systems will have readily discernible volatilities.

For example, a flood levee may be required and it is known that currently it needs to be built to a 20% AEP in order to maintain an acceptable level of risk. However, it is also forecast that due to climate change, over 50 years, the magnitude of the 20% AEP event will be equivalent to the magnitude of the current 10% AEP event and that levee would need to be increased by one metre in height to provide the same level of risk. Assume, also that the volatility is such that there was 50% chance that the cost of flooding would increase by 5% per year and a 50% chance that the cost of flooding would increase by 1% per year. Utilising a numerical method for the pricing of options (e.g. the Binomial Method, Black-Scholes Model), and based on this volatility year on year, it would be possible to estimate the value of implementing an option in a given year (ie. the value of waiting to make a decision on investing until more information is available). For example, if it turns out that by year 10 that there have been 10 consecutive 5% increases, then the benefits of increasing the height of the levee at that point will be significantly greater than if there had been 10 consecutive 1% increases per year.

Other common decision-under-uncertainty making tools which may be utilised include Laplace’s Principle of Insufficient Reason or Walds Maximin Model.

5.11. Further Reading

SCARM (2000) Floodplain Management in Australia: Best Practice Principles and Guidelines. Agriculture and Resource Management Council of Australia and New Zealand, Standing Committee on Agriculture and Resource Management (SCARM). Report No 73. CSIRO Publishing, 2000.

Shand, T. D., Cox, R. J., Blacka, M. J. and Smith, G. P. (2011). Australian Rainfall and Runoff Revision Project 10: Appropriate Safety Criteria for Vehicles - Literature Review. Australian Rainfall and Runoff Revision Project 10. Stage 2 Report. Prepared by the Water Research Laboratory. P10/S2/020. February 2011.

Opper S, Emergency Management Planning for the Hawkesbury Nepean Valley, 40th Floodplain Management Authorities Conference, Parramatta, 9th – 12th May 2000.

NSW Government (2005). Floodplain Development Manual. Department Of Infrastructure Planning and Natural Resources. DIPNR 05_020.

NFRAG (2008) Flood Risk Management in Australia. The Australian Journal of Emergency Management. Vol 23, No. 4, November 2008.

McLuckie, D., Babister M., (2015). Improving national best practice in flood risk management, Floodplain Management Association National Conference, Brisbane, May 2015.

McLuckie, D., Babister M., Smith G., Thomson R., (2014). Updating National best practice guidance in flood risk management, Floodplain Management Association National Conference, Deniliquin, May 2014.

McLuckie, D., (2013). A guide to best practice in flood risk management in Australia, Floodplain Management Association National Conference, Tweed Heads, May 2013.

McLuckie, D., (2012). Best Practice in Flood Risk Management in Australia. Presentation to Engineers Australia Queensland Water Panel, Brisbane, May 2012.

COST 2013 - A review of applied methods in Europe for flood-frequency analysis in a changing environment: WG4 – Flood Frequency estimation methods and environmental change.

Babister, M., Retallick, M., 2013. Benchmarking Best Practice in Floodplain Management, Floodplain Management Association National Conference, Tweed Heads, May 2013

Babister, M., McLuckie, D., Retallick, M., Testoni, I. (2014). Comparing Modelling Approaches to End User needs, Floodplain Management Association Conference, Deniliquin, May 2014

5.12. Examples

5.12.1. Calculation of Average Annual Damages for a Community

$$\mathrm{AAD} = \left[\text{Sum from 1 to n of, } \frac{d_1 + d_2}{2}(p_2 - p_1) + \frac{d_2 + d_3}{2}(p_3 - p_2) + \ldots, \frac{d_{n-1} + d_n}{2}(p_n - p_{n-1})\right]$$

Where:

AAD = Annual Average Damages in $

d = damage in $

p = probability

points 1 to n are the different ordinates on the damage curve

Example:

Determine the damage under the curve for Figure 1.5.3. The table below provides the points on the curve.

AEP	Flood Damage for Event ($)	n
0.001%	$28,000,000	1
0.2%	$22,000,000	2
0.5%	$20,000,000	3
1.0%	$17,000,000	4

2.0%	$9,000,000	5
5.0%	$3,000,000	6
10%	$1,000,000	7
20%	$0	8

$$\text{AAD} =$$

$$\left[\begin{array}{l} (\frac{28,000,000+22,000,000}{2}x(0.002-0.0001))+(\frac{22,000,000+20,000,000}{2}x(0.005-0.002)) \\ +(\frac{20,000,000+17,000,000}{2}x(0.01-0.005))+(\frac{17,000,000+9,000,000}{2}x(0.02-0.01)) \\ +(\frac{9,000,000+3,000,000}{2}x(0.05-0.02))+(\frac{3,000,000+1,000,000}{2}x(0.1-0.05)) \\ +(\frac{1,000,000+0}{2}x(0.2-0.1)) \end{array}\right]$$

$$\text{AAD} = [49,750+63,000+92,500+130,000+180,000+100,000+50,000]$$

$$\text{AAD} = \$665,250$$

When it comes to examining treatment options it can be important to understand which portion of the curve is contributing significantly to damage and therefore a table such as follows can be useful. In this case events up to a 1% AEP event contribute $460,000 out of the $665,250 toward Annual Average Damages.

AEP n	AEP n-1	Flood Damage n ($)	Flood Damage n-1 ($)	Contribution to AAD ($)
0.001%	0.2%	$28,000,000	$22,000,000	$49,750
0.2%	0.5%	$22,000,000	$20,000,000	$63,000
0.5%	1.0%	$20,000,000	$17,000,000	$92,500
1.0%	2.0%	$17,000,000	$9,000,000	$130,000
2.0%	5.0%	$9,000,000	$3,000,000	$180,000
5.0%	10%	$3,000,000	$1,000,000	$100,000

10%	20%	$1,000,000	$0	$50,000
Total ($)	n/a	n/a	n/a	$665,250

5.12.2. Calculating Average Annual Benefits of a Treatment Measure

$$\mathrm{AAB} = \mathrm{AAD}_{case0} - AAD_{case1}$$

Where:

AAB = annual average benefits in $

AAD = annual average damages in $ - calculations see Book 1, Chapter 5, Section 12

Case 0 is the current situation or base case

Case 1 is the case with treatment in place

Example:

Determine the Average Annual Benefits

Step 1

Determine Annual Average Damages for the base case – undertaken in Book 1, Chapter 5, Section 12

Step 2

Determine Annual Average Damages for the treated case. See table below.

AEP n	AEP n-1	Flood Damage n ($)	Flood Damage n-1 ($)	Contribution to AAD ($)
0.001%	0.2%	$28,000,000	$22,000,000	$49,750
0.2%	0.5%	$22,000,000	$15,000,000	$55,500
0.5%	1.0%	$15,000,000	$2,000,000	$42,500
1.0%	2.0%	$2,000,000	$750,000	$13,750
2.0%	5.0%	$750,000	$500,000	$18,750
5.0%	10%	$500,000	$200,000	$17,500
10%	20%	$200,000	$0	$10,000
Total ($)	n/a	n/a	n/a	$207,750

Step 3

Determine Annual Average Damages for the base case – undertaken in Book 1, Chapter 5, Section 12

Step 4

Determine Annual Average Damages for the treated case. See table below.

AEP n	AEP n-1	Base Case Contribution to AAD From Book	Treated Case Contribution to AAD ($)	Contribution to AAB ($)

		1, Chapter 5, Section 12		
0.001%	0.2%	$49,750	$49,750	$0
0.2%	0.5%	$63,000	$55,500	$7,500
0.5%	1.0%	$92,500	$42,500	$50,000
1.0%	2.0%	$130,000	$13,750	$116,250
2.0%	5.0%	$180,000	$18,750	$161,250
5.0%	10%	$100,000	$17,500	$82,500
10%	20%	$50,000	$10,000	$40,000
Total ($)	n/a	$665,250	$207,750	$457,500

5.12.3. Calculating Net Present Value of Benefits

$$\mathrm{NPV} = \left[\text{Sum 1 to } n-1 \text{ of, } \frac{\mathrm{AAB}}{(1+\mathrm{dr})^{1}} + \frac{\mathrm{AAB}}{(1+\mathrm{dr})^{2}} + \ldots, \frac{\mathrm{AAB}}{(1+\mathrm{dr})^{(n-1)}}\right]$$

Where:

AAB = Annual Average Benefits in $ - calculation see Book 1, Chapter 5, Section 12

dr = discount rate

n= design life in years. Assumes no benefits during construction.

Note:

A range of discount rates may be used to give a range of NPVs which can in turn be used to determine a range of Benefit Cost Ratios (see Book 1, Chapter 5, Section 12) to test how financial benefits may vary with different financial situations.

Example:

Calculating Net Present Value of benefits

Step 1

Determine the Average Annual Benefits – undertaken in Book 1, Chapter 5, Section 12

Step 2

Assess Net Present Value of benefits. See table below.

$\mathrm{NPV}_{\mathrm{Benefits}} = \$6,099,257$ for a discount rate (dr) of 7%. This could vary from $4,473,916 for a dr of 10% to $9,055,194 for a dr of 4%.

Year N	Benefit in Future Year AAB ($)	Benefit in future year in current year for dr_1 = 4% ($)	Benefit in future year in current year for dr_2 = 7% ($)	Benefit in future year in current year for dr_3 = 10% ($)
0	0	0	0	0
1	457,500	439,904	427,570	415,909
2	457,500	422,984	399,598	378,099
3	457,500	406,716	373,456	343,727

4	457,500	391,073	349,025	312,479
5	457,500	376,032	326,191	284,072
6	457,500	361,569	304,852	258,247
7	457,500	347,662	284,908	234,770
8	457,500	334,291	266,269	213,427
9	457,500	321,433	248,850	194,025
10	457,500	309,071	232,570	176,386
11	457,500	297,183	217,355	160,351
12	457,500	285,753	203,135	145,774
13	457,500	274,763	189,846	132,521
14	457,500	264,195	177,426	120,474
15	457,500	254,034	165,819	109,522
16	457,500	244,263	154,971	99,565
17	457,500	234,868	144,833	90,514
18	457,500	225,835	135,358	82,285
19	457,500	217,149	126,503	74,805
20	457,500	208,797	118,227	68,004
21	457,500	200,766	110,492	61,822
22	457,500	193,045	103,264	56,202
23	457,500	185,620	96,508	51,093
24	457,500	178,481	90,195	46,448
25	457,500	171,616	84,294	42,225
26	457,500	165,015	78,779	38,387
27	457,500	158,669	73,626	34,897
28	457,500	152,566	68,809	31,725
29	457,500	146,698	64,307	28,841
30	457,500	141,056	60,100	26,219
31	457,500	135,631	56,169	23,835
32	457,500	130,414	52,494	21,668
33	457,500	125,398	49,060	19,698
34	457,500	120,575	45,850	17,908
35	457,500	115,938	42,851	16,280
36	457,500	111,478	40,047	14,800
37	457,500	107,191	37,428	13,454
38	457,500	103,068	34,979	12,231
39	457,500	99,104	32,691	11,119
40	457,500	95,292	30,552	10,108

Total ($)	$9,055,194	$6,099,257	$4,473,916

5.12.4. Calculating Net Present Value (NPV) of Lifecycle Costs

$$\mathrm{NPV} = \text{Capital Cost} + \left[\text{Sum 1 to } n-1 \text{ of}, \frac{\mathrm{AMC}}{(1+\mathrm{dr})^1} + \frac{\mathrm{AMC}}{(1+\mathrm{dr})^2} + \ldots, \frac{\mathrm{AMC}}{(1+\mathrm{dr})^{(n-1)}}\right]$$

Where:

Capital cost is all relevant upfront costs.

AMC = Annual operation and Maintenance Costs

dr = discount rate

n= design life in years. Assumes no benefits during construction.

Note:

A range of discount rates may be used to give a range of NPVs and in turn a range of Benefit Cost Ratios (see Book 1, Chapter 5, Section 12) to test how financial benefits may vary with changing financial situation.

Example:

Calculating Net Present Value of lifecycle costs

Step 1

Determine capital costs and Annual operation and Maintenance Costs

Step 2

Calculate Net Present Value of lifecycle costs. See table below.

Example:

Capital cost is $4,000,000 and Annual operation and Maintenance Costs is $100,000.

$\mathrm{NPV_{costs}} = \$5,333,171$ for a discount rate (dr) of 7%. This could vary from $4,977,105 for a dr of 10% to $5,979,277 for a dr of 4%.

Year N	Cost in Future Year AAB ($)	Cost in future year in current year for dr_1 = 4% ($)	Cost in future year in current year for dr_2 = 7% ($)	Cost in future year in current year for dr_3 = 10% ($)
0	4,000,000	4,000,000	4,000,000	4,000,000
1	100,000	96,154	93,458	90,909
2	100,000	92,456	87,344	82,645
3	100,000	88,900	81,630	75,131
4	100,000	85,480	76,290	68,301
5	100,000	82,193	71,299	62,092
6	100,000	79,031	66,634	56,447
7	100,000	75,992	62,275	51,316
8	100,000	73,069	58,201	46,651

9	100,000	70,259	54,393	42,410
10	100,000	67,556	50,835	38,554
11	100,000	64,958	47,509	35,049
12	100,000	62,460	44,401	31,863
13	100,000	60,057	41,496	28,966
14	100,000	57,748	38,782	26,333
15	100,000	55,526	36,245	23,939
16	100,000	53,391	33,873	21,763
17	100,000	51,337	31,657	19,784
18	100,000	49,363	29,586	17,986
19	100,000	47,464	27,651	16,351
20	100,000	45,639	25,842	14,864
21	100,000	43,883	24,151	13,513
22	100,000	42,196	22,571	12,285
23	100,000	40,573	21,095	11,168
24	100,000	39,012	19,715	10,153
25	100,000	37,512	18,425	9,230
26	100,000	36,069	17,220	8,391
27	100,000	34,682	16,093	7,628
28	100,000	33,348	15,040	6,934
29	100,000	32,065	14,056	6,304
30	100,000	30,832	13,137	5,731
31	100,000	29,646	12,277	5,210
32	100,000	28,506	11,474	4,736
33	100,000	27,409	10,723	4,306
34	100,000	26,355	10,022	3,914
35	100,000	25,342	9,366	3,558
36	100,000	24,367	8,754	3,235
37	100,000	23,430	8,181	2,941
38	100,000	22,529	7,646	2,673
39	100,000	21,662	7,146	2,430
40	100,000	20,829	6,678	2,209
Total ($)		$5,979,277	$5,333,171	$4,977,905

5.12.5. Calculating Benefit Cost Ratio (BCR)

$$\mathrm{BCR} = \frac{\mathrm{NPV}_{\mathrm{benefits}}}{\mathrm{NPV}_{\mathrm{costs}}}$$

Where:

$\text{BCR}=\text{Benefit Cost Ratio}$

$\text{NPV}_{\text{benefits}} = \text{Net Present Value of Benefits}$ - see Book 1, Chapter 5, Section 12

$\text{NPV}_{\text{costs}} = \text{Net Present Value of Costs}$ - see Book 1, Chapter 5, Section 12

Note:

A range of discount rates may be used to give a range of NPVs which can in turn be used to determine a range of Benefit Cost Ratios (see below) to test how financial benefits may vary with different financial situations.

Example:

Calculating BCR

Step 1

Calculate $\text{Net Present Value of Benefits} = \text{NPV}_{\text{benefits}}$

Step 2

Calculate $\text{Net Present Value of Costs} = \text{NPV}_{\text{costs}}$

Step 3

Calculate BCR. BCR = 1.14 for dr 7%, BCR = 1.51 for dr 4%, BCR = 0.9 for dr 10%.

Discount Rate dr (%)	NPV benefits ($)	NPV costs ($)	BCR
4	9,055,194	5,979,277	1.51
7	6,099,257	5,333,171	1.14
10	4,473,916	4,977,905	0.90

5.13. References

AEMI (Australian Emergency Management Institute) (2013), Australian Emergency Management Handbook 7: Managing the Floodplain Best Practice in Flood Risk Management in Australia AEMI, Canberra.

AEMI (Australian Emergency Management Institute) (2015), Australian Emergency Management Handbook 10: National Emergency Risk Assessment Guidelines AEMI, Canberra.

AEMI (Australian Emergency Management Institute) (2014a), Technical flood risk management guideline: Flood emergency response classification of the floodplain, AEMI, Canberra.

AEMI (Australian Emergency Management Institute) (2014b), Technical flood risk management guideline: Flood hazard, AEMI, Canberra.

AEMI (Australian Emergency Management Institute) (2014c), Guideline for using the national generic brief for flood investigations to develop project-specific specifications, AEMI, Canberra.

ANCOLD (Australian National Committee on Large Dams) (2003), Guidelines on Risk Assessment. Australian National Committee on Large Dams.

AUSTROADS (1994), Waterway Design: A Guide to the Hydraulic Design of Bridges, Culverts and Floodways, Ed: D. Flavell.

Condon, L.C., Gangopadhyay, S. and Pruitt, T. (2014), Climate change and non-stationary flood risk for the Upper Truckee River Basin. Hydrological Earth System Sciences Discussions, 11: 5077-5114.

DECC (NSW Department of Environment and Climate Change), (2007), Floodplain Risk Management Guideline, Residential Flood Damage, Sydney.

Farber, S. and Bradley, D. (1996), Ecological Economics. USDA Forest Service, available at: http://www.fs.fed.us.

Floodplain Regulations Committee (2010), A Guide for Higher Standard in Floodplain Management. Association of State Floodplain Managers.

HM Treasury and DEFRA (2009), Accounting for the effects of climate change: Supplementary Green Book Guidance.

ISO (International Standards Organisation). (2009a), Guide 73:2009 Risk Management Vocabulary, International Standards Organisation.

ISO (International Standards Organisation). (2009b), International Standard on Risk Management, ISO31000:2009, International Standards Organisation.

McLuckie, D., Toniato A., Askew E., Babister M. and Retallick M. (2016), Supporting Land Use Planning by providing improved information from the Floodplain Management Process, Floodplain Management Association Conference, Nowra, May 2016.

Ng, M. and Vogel, R.M. (2010), Multivariate non-stationary stochastic streamflow models for two urban watersheds. Proc., Environmental and Water Resources Institute World Congress, ASCE, Reston, VA, pp: 2550-2561.

Park, T., Kim, C. and Kim, H. (2014), Valuation of drainage infrastructure improvement under climate change using real options. Water Resource Management, 28: 445-457.

Philbert C. (2003), Discounting the Future. In: Internet Encyclopaedia of Ecological Economics, The International Society of Ecological Economics. Available at: http://www.ecologicaleconomics.org.

Quiggin J. (1993), Discounting and Sustainability. Review of Marketing and Agricultural Economics, 6(1), 161-64.

Rootzen, H. and Katz, R. (2013), Design Life Level: Quantifying risk in a changing climate. Water Resources Research, 49: 5964-5972.

Salas, J. and Obeysekera, J. (2013), Revisiting the concepts of return period and risk for nonstationary hydrologic extreme events. Journal of Hydologic Engineering, 19(3), 554-568.

Short, M., Peirson, W., Peters, G. and Cox, R. (2012), Managing adaptation of urban water systems in a changing climate. Water Resources Management, 26(7), 1953-1981

Stern, N. (2006), The Economics of Climate Change - The Stern Review, Cabinet Office - HM Treasury, Cambridge University Press. Available at: http://webarchive.nationalarchives.gov.uk/+/http://www.hm-treasury.gov.uk/sternreview_index.htm.

Thomson, R., Knee, P., Telford, T., Virivolomo, M. and Drynan, L. (2011), Climate Change Adaptation & Economics - A Case Study in the Solomon Islands, Coast and Ports Conference, Perth, Australia.

Thomson, R., Knee, P., Telford, T., Virivolomo, M. and Drynan, L. (2012), Incorporating Climate Change into Economic Analysis - A Case Study of Emergency Flood Repairs in the Solomon Islands, Practical Responses to Climate Change, Canberra, Australia.

Vogel, R.M., Yaindl, C. and Walter, M. (2011), Nonstationarity: Flood magnification and recurrence reduction factors in the United States, J. Am. Water Resour. Assoc., 47: 464-474.

Woodward, M., Gouldby, B., Kapelan, Z., Khu, S. and Townend, I. (2011), Incorporating real options into flood risk management decision making. 14th annual international real options conference.

World Bank (2010), Economics of adaptation to climate change: synthesis report. Washington D.C.

Åström, H., Friis Hansen, P., Garré, L. and Arnbjerg-Nielsen, K. (2013), An influence diagram for urban flood risk assessment through pluvial flood hazards under non-stationary conditions. NOVATECH.

Chapter 6. Climate Change Considerations

Conrad Wasko, Seth Westra, Rory Nathan, Dörte Jakob, Chris Nielsen, Jason Evans, Simon Rodgers, Michelle Ho, Mark Babister, Andrew Dowdy, Wendy Sharples

Chapter Status	Final
Date last updated	27/08/2024

6.1. Introduction

Key messages

Because our climate is changing, unadjusted historical observations are no longer a suitable basis for design flood estimation: they must be adjusted to reflect the impacts of rising global temperatures. This chapter provides guidance on how to do this. It is based on a systematic review and meta-analysis of recent peer-reviewed science.

There is unequivocal evidence that greenhouse gas emissions have caused global warming. The Intergovernmental Panel on Climate Change's sixth assessment report concluded that global surface temperatures have significantly increased above pre-industrial levels, with significant further warming expected (IPCC, 2023). This warming is causing an increase in many drivers of flood risk, including an intensification of extreme rainfall events and the elevation of average and extreme sea levels (IPCC, 2023).

Traditionally, design flood estimation has assumed that historical observations are representative of current and future conditions. This is no longer the case. Records on historical flooding or flood drivers (such as extreme rainfall or sea level) can no longer be assumed to provide a direct analogue of current or future flood risk. To account for significant observed and projected changes to the drivers of flooding, it is necessary to account for the non-stationarity of flooding in the assessment of current and future flood risks.

This chapter provides practitioners, designers, and decision makers with guidance on how to assess the impact of climate change on design flood characteristics. As with the rest of *Australian Rainfall and Runoff* (ARR) the content is advisory, so additional guidance or requirements may need to be sought from relevant government regulation in the jurisdiction of interest.

This chapter replaces the 2019 Climate Change Considerations chapter (Bates et al., 2019) in Book 1, Chapter 6 of ARR Version 4.1. This chapter provides guidance that reflects the contemporary science, is applicable across the range of design flood approaches, and facilitates consistent application of climate change in design flood estimation. This guidance is based on an extensive and rigorous systematic review and meta-analysis of over 300 distinct peer-reviewed scientific studies published largely from 2011 onwards. For further information on the scientific basis for this guidance, readers are referred to Wasko et al. (2024).

Key differences from the 2019 Climate Change Considerations chapter (Version 4.1) include:

- recognition that global temperatures have increased over the historical period used to derive design rainfall information
- a recommendation to adjust 2016 Intensity-Frequency-Duration (IFD) curves to present (i.e. current climate) conditions
- provision of information to support a range of approaches to decision-making
- provision of guidance across the range of Annual Exceedance Probability (AEP) considered in ARR up to and including the Probable Maximum Precipitation (PMP)
- provision of uncertainty estimates
- consideration of additional factors that influence design flood estimates; that is, changes in rainfall losses, temporal patterns, and sea level rise.

This chapter is structured as follows:

- Book 1, Chapter 6, Section 2 discusses the 2 primary types of uncertainty exacerbated by climate change that are relevant to flood design.
- Book 1, Chapter 6, Section 3 covers how climate change affects design flood estimation methods more broadly.
- Book 1, Chapter 6, Section 4 describes how to account for climate change in the key aspects of event-based design flood estimation, with the recommendations linked to the worked examples in Book 1, Chapter 6, Section 6.
- Book 1, Chapter 6, Section 5 recommends an approach for future updates to the climate change considerations chapter.
- Book 1, Chapter 6, Section 6 provides worked examples of the guidance presented in Book 1, Chapter 6, Section 4.

6.2. Decison Contexts for Flood Guidance

Key messages

Climate variability heavily influences the risks of flooding from one time period to the next ('aleatory uncertainty') and these risks will shift under climate change. There is considerable uncertainty associated with future global emissions and the physical effects of these emissions on flood drivers increases the uncertainty of design flood estimates ('epistemic uncertainty'). A user needs to choose their approach to design flood estimation in the context of these two sources of uncertainty.

ARR provides guidance to support the estimation of design floods that inform either standards-based design criteria (such as designing to the 1% AEP standard) or risk-based approaches that seek to understand flood behaviour across a range of flood magnitudes up to and including the Probable Maximum Flood (PMF) (see Book 1, Chapter 5 for more detail). Guidance in ARR is largely focused on dealing with the inherent randomness of factors (the aleatory uncertainties) that influence the exceedance probability of a given event (Book 4, Chapter 3). In rainfall-based estimates this most often involves consideration of Intensity-Frequency-Duration (IFD) rainfall bursts (Book 2, Chapter 3), variability in rainfall temporal patterns (Book 2, Chapter 5), and the influence of antecedent and event losses

(Book 5, Chapter 3). In techniques based on the direct analysis of flood data (Book 3), the aleatory uncertainty of these factors is implicitly accounted for in the sampling variability represented in the available flood observations. Climate change is now impacting the frequency and behaviour of many of the factors that influence flood behaviour resulting in non-stationarity in design flood estimates, shifting estimates of the aleatory uncertainty (Figure 1.6.1).

While much of the emphasis of climate science is to understand the likely historical and future changes to key flood risk drivers, another important consequence of climate change is an increase in the epistemic uncertainty of design flood estimates (Figure 1.6.1). This additional uncertainty arises from limitations in data and knowledge, where in the context of climate change this includes the uncertainty associated with future global emissions and mitigation strategies, resultant uncertainty in future global and regional temperature changes, and uncertainties on the implication of changing temperatures and associated processes on flood risk drivers. Within risk-based decision-making frameworks (where risk is commonly defined as 'the effect of uncertainty on objectives', ISO 31000), considerations of the projected changes in flood risk drivers as well as the increasing level of uncertainty associated with those drivers are both relevant in assessment of overall risk.

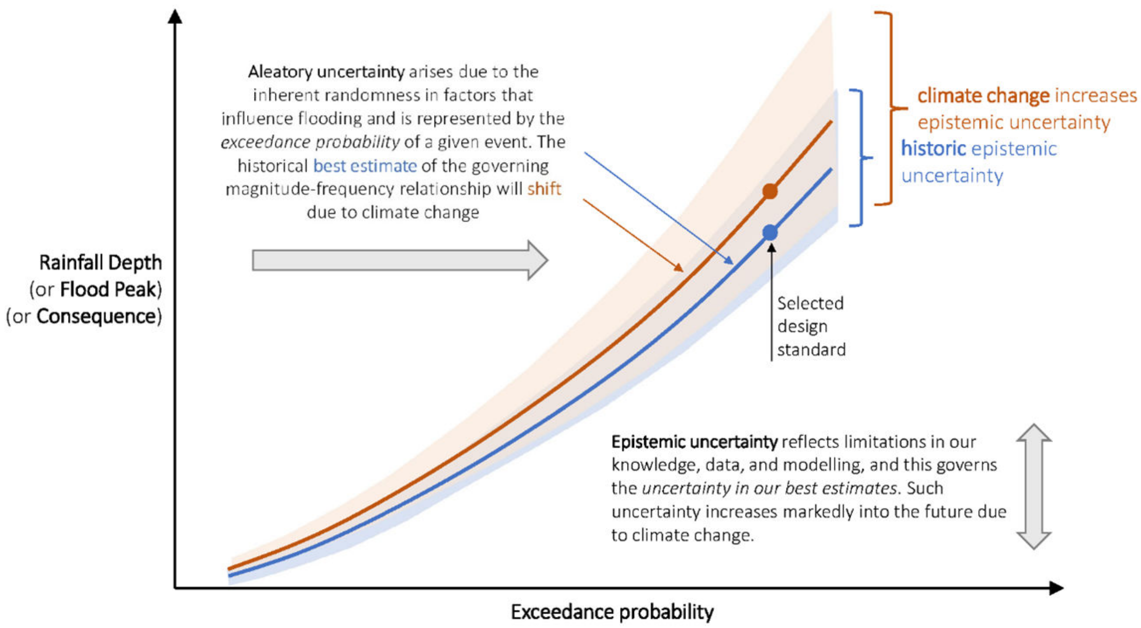


Figure 1.6.1. Climate Change is shifting our best estimate of the relationship between event magnitude and frequency and increasing the inherent uncertainty in such estimates

In addition to standards-based and risk-based decision-making approaches, there has recently been interest in decision-making frameworks that are less reliant on annualised probability-based estimates of flood magnitude. Examples include robust design approaches that can adapt to changes in design flood estimates over time (adaptive management), sensitivity and stress-testing approaches that explore the effect of alternative assumptions on decision-making ('bottom-up' and 'decision-scaling' approaches), and 'storyline' methods that often draw on historical flood events, but which are modified to account for potential impacts of climate change.

This chapter provides guidance for future changes to design floods recognising the requirements of current design procedures (and the need to capture the shifts in the magnitude-frequency relationships relevant to both standards-based and risk-based decision-making approaches), while also providing information to assist in estimating the uncertainty associated with those best estimates. This allows for consistency in the guidance here and the above decision-making frameworks. For example, a standards-based approach may adopt the median values presented in Book 1, Chapter 6, Section 4 and incorporate the uncertainty in a Monte Carlo framework. A bottom-up approach may use the uncertainty as a guide for sensitivity testing to develop system response surfaces for the hazard of interest. A risk-based approach may consider the probabilities and consequences of flooding across a range of exceedance probabilities, accounting for both aleatory and epistemic uncertainties.

The reader is referred to Book 1, Chapter 5 for further information on risk-based decision making, with Book 1, Chapter 5, Section 10 detailing a possible process of undertaking a non-stationary risk assessment. Further information on approaches for representing epistemic uncertainty as part of the design flood estimation process are presented in Book 7, Chapter 9, with the different approaches to address aleatory uncertainty described in Book 1, Chapter 3.

6.3. Selection of a Design Flood Estimation Method Under Climate Change

Key messages

There are two broad classes of design flood estimation methods: direct flood-based procedures and rainfall-based procedures. Rainfall-based procedures can be further divided into continuous and event-based approaches. The impacts of climate change are most readily incorporated into event-based methods, and hence this guidance focusses on generic advice that is relevant to these methods. Direct flood-based procedures (e.g. flood frequency) and continuous simulation approaches remain important tools for the assessment of flood risk, though there is no clear consensus on how such methods should be adjusted to account for climate change.

Although frequency analysis will continue to be used for design flood estimation and the calibration of models to historic data, climate change influences a range of flood drivers which impact on flood magnitude in a highly non-linear fashion. It is thus difficult to directly adjust flood information without reference to the causal processes by which climate change influences flood magnitude (Wasko et al., 2024). This means that extrapolating historical trends into the future or scaling historical flood magnitudes by a fixed amount may not adequately capture how changes in the causal drivers propagate through a catchment to influence changes in flood behaviour. Accordingly, it is recommended that any adjustment of historical flood frequency estimates to account for climate change be undertaken in a manner that it is consistent with the impact of changes in the flood drivers (e.g. extreme rainfall, antecedent conditions, sea level, etc) as described in this chapter.

In contrast to flood-based procedures, rainfall-based procedures provide a mechanism to relate information on climate projections to design flood estimates using either event-based or continuous simulation approaches. For continuous simulation to be suitable for flood estimation under a non-stationary climate, the timeseries need to reflect the complexities of future changes that are relevant to the design flood estimation problem. As highlighted in Wasko et al. (2024), extreme rainfall is likely to change at a different rate to annual average rainfall. Similarly short-duration extremes (sub-daily rainfall) and longer-duration extremes (multi-day accumulations) are likely to be experiencing differing rates of change in both frequency and intensity, leading to complex changes in the temporal patterns of rainfall. There may also be changes to the seasonality of heavy rainfall events, and to the sequencing of wet and dry periods.

Although continuous simulation approaches can be adapted to use climate-adjusted input timeseries (rainfall and potential evapotranspiration), further research is required to develop robust and practical methods to generate these climate-adjusted inputs into continuous simulation models, and applications will generally require the development of individualised solutions. As a pragmatic minimum, it is recommended that extreme rainfalls in the timeseries used for continuous simulation be scaled to reflect the recommendations for incorporating climate change into Intensity-Frequency-Duration curves (Book 1, Chapter 6, Section 4), with the remaining rainfalls adjusted to reflect projections of the mean seasonal or annual rainfall for the location of interest. Suitable account will also need to be given to adjusting evaporative demand inputs.

Due to the above considerations, and because event-based procedures are commonly applied in design flood estimation, event-based procedures are recommended as the primary class of procedures for incorporating climate change into design flood estimates. In the context of climate change assessments, the advantages of this class of procedures are the ability to clearly map climate projections (including those related to extreme rainfall and/or sea levels) to the inputs of event-based rainfall-runoff and hydraulic models, together with their general applicability across both gauged and ungauged catchments and for a wide range of annual exceedance probabilities from the 1 exceedance per year (EY) event through to the Probable Maximum Precipitation (PMP) (Figure 1.3.2 of Book 1, Chapter 3).

6.4. Incorporating Climate Change into Event-Based Design Flood Estimates

Key messages

Changes in extreme rainfall are likely to represent the primary mechanism for increases in flood risk across most Australian catchments. This section provides information on how to adjust design rainfall estimates as well as temporal patterns, loss parameters, and sea level rise. Uncertainties are presented for each of the aspects of the design flood estimate for which climate change guidance is provided. The reader is referred to supporting guidance for sea level rise. Examples are presented in Book 1, Chapter 6, Section 6.

Event-based procedures are summarised in Book 4, Chapter 3, and as highlighted in that chapter, they generally take into consideration the following climate-related factors:

1. A design storm of a given AEP and duration, usually derived from published IFD data (Book 2, Chapter 2).
2. Temporal patterns to distribute the design rainfall over the duration of the event (Book 2, Chapter 5).
3. Spatial patterns to represent rainfall variation over a catchment (Book 2, Chapter 4).
4. Loss parameters that represent soil moisture conditions in the catchment antecedent to the event and the capacity of the soil to absorb rainfall during the event (Book 5, Chapter 5).

This list of climate-related factors is not necessarily exhaustive. For example, in low-lying (such as estuarine) catchments, the above is combined with information on sea levels that are influenced by both astronomical tides and storm surges (Book 6, Chapter 5). In the following sections, guidance is provided on accounting for climate change associated with each of the 4 aspects of event-based design flood modelling described above. The section closes with a discussion of the approach to estimating the epistemic uncertainty.

6.4.1. Intensity-Frequency-Duration Curves

For most Australian catchments, changes in extreme rainfall are likely to represent the primary mechanism for increases in flood risk with Wasko et al. (2024) identifying over 40 studies that quantify changes of extreme rainfall over Australia. For consistency with the Intergovernmental Panel on Climate Change (IPCC) projections, and to be representative of the climatic drivers of changes in moisture sources, a scaling approach is recommended whereby design rainfalls are factored at a rate proportional to global surface (land and sea) temperature increase.

The current IFD curves that are included in the 2016 IFD portal[1] were derived using historical data (Book 2, Chapter 3). The data from individual gauges varies, but a good estimate for the midpoint of the data period used in estimating the 2016 IFDs is 1961-1990. Figure 1.6.2 presents the latest Intergovernmental Panel on Climate Change (IPCC) temperature projections based on Shared Socioeconomic Pathways (SSPs) that cover a broad range of potential future development options often referred to as very low (SSP1-1.9), low (SSP1-2.6), medium (SSP2-4.5), high (SSP3-7.0) and very high (SSP5-8.5) emissions pathways. The best estimate for the mid-point of the data used for the generation of the 2016 IFDs is shaded. As global temperatures have risen since this period, design rainfall estimates require factoring to account for this temperature increase.

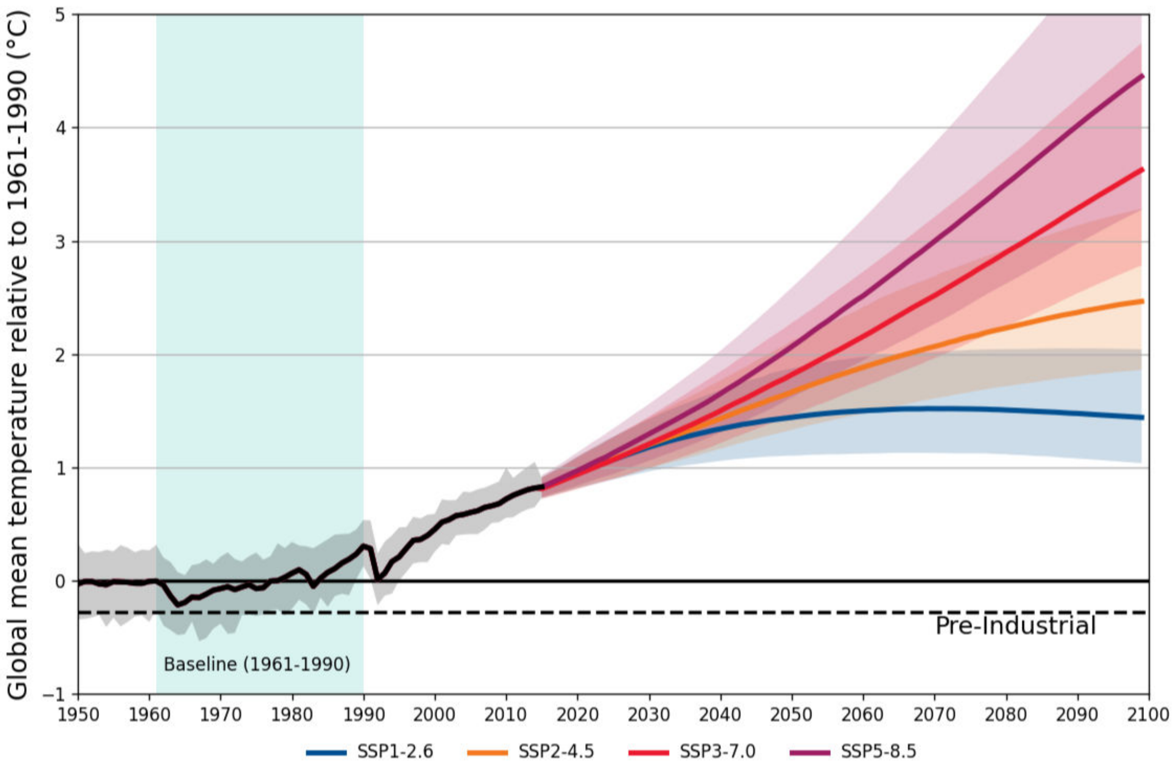


Figure 1.6.2. Projected temperature increases associated with AR6 socioeconomic pathways relative to 1961-1990 and their associated uncertainty[2]

[1]http://www.bom.gov.au/water/designRainfalls/ifd/

[2]Figure based on Figure SPM.8 from the Summary for Policymakers (SPM) of the Working Group I (WGI) Contribution to the Intergovernmental Panel on Climate Change (IPCC) Sixth Assessment Report (AR6) (IPCC, 2021; Fyfe et al., 2021). Data can be obtained from the associated references.

To account for changes since the period represented by the IFD curves in the 2016 IFD portal, it is recommended that IFD information as well as estimates of the PMP should be adjusted using Equation (1.6.1) and the relevant rate of change in Table 1.6.1:

$$I_p = I \times \left(1 + \frac{\alpha}{100}\right)^{\Delta \mathrm{T}} \tag{1.6.1}$$

where

- I_p the projected (current or future) design rainfall depth or intensity
- α is the rate of change from Table 1.6.1
- I is the historical design rainfall depth or intensity (e.g. from the 2016 IFD portal or historical PMP estimates)
- ∆T is the most up-to-date estimate of global (land and ocean) temperature projection for the design period of interest and selected climate scenario relative to a baseline time period. When used in conjunction with the 2016 IFD curves the baseline is recommended to be the 1961-1990 period (see Table 1.6.2 and text below).

The rates of change in Table 1.6.1 apply to exceedance probabilities from the 1EY event through to the PMP and have been developed for application across mainland Australia and Tasmania. There is some evidence for heterogeneity in the rate of change across space as well as by event severity. However, there is insufficient information to quantitatively describe these differences, and/or the magnitude of difference was deemed to be small relative to the associated uncertainty. The information provided in Table 1.6.1 relates to storm durations for which most published evidence is available. Guidance on factors for use with burst durations between 1 and 24 hours is given in Appendix A. As pre-burst rainfalls are defined as a fixed proportion of the burst rainfall depth (Book 2, Chapter 5), these rates of change also apply to pre-burst rainfalls.

The differing rates of change with storm duration reflect the fact that the mechanisms that cause extreme rainfall at these 2 durations are often distinct. Note that short duration extremes are often embedded in longer duration extremes. It is possible that application of the scaling factors provided in Table 1.6.1 and Table 1.6.5 may yield inconsistencies in the resulting design rainfall frequency curves. Such inconsistencies are not unexpected given the uncertainties involved in their derivation. Accordingly, the IFD curves should be adjusted as minimally as possible to ensure the curve of one duration does not cross the curve of another.

Table 1.6.1. Recommended rates of change (α) and associated uncertainty derived in Wasko et al. (2024), presented per degree global temperature change (%/°C). The factors in this table are applicable for exceedance probabilities from 1EY up to and including the PMP and are designed for application across mainland Australia and Tasmania.

	≤ 1 hr	> 1 hr and < 24 hr	≥ 24 hr
Central (median) estimate (%/°C)	15	Interpolation zone (see Table 1.6.5)	8
‘Likely’ range (corresponding to ~66%[a] range) (%/°C)	7-28		2-15

[a]Consistent with terminology used by the IPCC the 66% range corresponds to an uncertainty range of +/- 33%.

To ensure consistency with IPCC projections and be representative of the climatic drivers of change in extreme rainfall, the rates of change in Table 1.6.1 are presented relative to global temperature changes. Hence, to adjust IFDs for climate change, global temperature projections ∆T are required. The current global temperature projections are provided in Table 1.6.2 are derived from the IPCC Sixth Assessment Report (AR6). An example application of Equation (1.6.1) is provided in Example 1a in Book 1, Chapter 6, Section 6.

A range of factors are likely to influence selection of SSP(s) that are specific to the design problem of interest. For more information on how to include SSP uncertainty as part of a holistic assessment of flood risk uncertainty, refer to Book 1, Chapter 6, Section 6. Users should seek the most up to date relevant guidance or information in relation to the selection of timeframes and climate scenarios.

Table 1.6.2. Global mean surface temperature projections (∆T) for four socio-economic pathways relative to 1961-1990. The 90% uncertainty interval is provided in parentheses[a]

Climate Scenario	SSP1-2.6	SSP2-4.5	SSP3-7.0	SSP5-8.5
Current and near-term (2021-2040) (°C)	1.2 (0.9-1.5)	1.2 (0.9-1.5)	1.2 (0.9-1.5)	1.3 (1.0-1.6)
Medium-term (2041-2060) (°C)	1.4 (1.0-1.9)	1.7 (1.3-2.2)	1.8 (1.4-2.3)	2.1 (1.6-2.7)
Long-term (2081-2100) (°C)	1.5 (1.0-2.1)	2.4 (1.8-3.2)	3.3 (2.5-4.3)	4.1 (3.0-5.4)

[a]Projections adapted from the Summary for Policymakers (SPM) of the Working Group I (WGI) Contribution to the Intergovernmental Panel on Climate Change (IPCC) Sixth Assessment Report (AR6)[b]. The projected temperature increases in the table above use a different baseline to the global warming levels referenced in relation to the Paris Agreement which uses a pre-industrial period (represented by the 1850-1900 period). There are approximately 0.3 degrees of global warming between the pre-industrial reference and 1961-1990, and this amount needs to be added to the temperatures in the above if comparing to the 1850-1900 period. The IPCC offers a range of summary projections information including in the assessment reports and the interactive atlas. We have selected the global climate projections from AR6 as they represent a balanced assessment of future global climate based on multiple lines of evidence, rather than a simple average of the global climate model output. This data is only available at the global level, not the regional level.

[ab] Refer to Table SPM.1 from the Summary for Policymakers (SPM) of the Working Group I (WGI) Contribution to the Intergovernmental Panel on Climate Change (IPCC) Sixth Assessment Report (IPCC, 2021)

The IPCC AR6 projections use climate model simulations from the Coupled Model Intercomparison Project Phase 6 (CMIP6) based on a range of assumptions including physical climate science, socio-economic factors and mitigation efforts. These projections are neither predictions nor forecasts but instead describe plausible scenarios that represent future climate uncertainty. It is noted that future Assessment Reports by the IPCC may potentially move away from the current SSPs. If a user wishes to design based on Global Warming Levels – the global temperature change relative to the pre-industrial period of 1850-1900 – then an adjustment for the additional warming of 0.3°C between 1850-1900 and 1961-1990 is required (see Example 1b). Further, it may be that IFD curves will be updated in the future. If a user has reason to believe a different baseline (refer Figure 1.6.2) is applicable to the adjustment of design rainfalls, then the warming since the baseline needs to be calculated by the user (see Example 1c). Readers should always refer to the latest global climate projections and Global Warming Levels.

The projections in Table 1.6.2 are provided up to 2081-2100. For projections up to the year 2300, the reader is referred to Section 4.7.1.2 in Chapter 4 of the IPCC AR6 Report[3], noting that for SSP2-4.5, SSP3-7.0 and SSP5-8.5, temperatures are projected to continue increasing beyond 2100. Although there may be interest in using projections beyond 2100 for flood designs with a long effective service and/or design life, the available evidence to

[3]https://www.ipcc.ch/report/ar6/wg1/ (Lee et al., 2021)

support such projections is more speculative than that used to develop projections out to 2100. The defensibility of designing for specific climate change projections beyond 2100 is therefore questionable, and a more flexible and adaptive approach may be advisable. No formal guidance on adaptive approaches has yet been developed for flood design in Australia, but adaptive design generally involves building in flexibility to adapt to changing conditions. That is, rather than build a structure to withstand loading conditions as they might exist beyond 2100, structures with long service lives might be designed to cater for conditions over the time horizons described in Table 1.6.2, while also ensuring that the structure can be augmented in the future to cater for more severe conditions as necessary. A simple illustration of this concept is provided in Example 2 (Book 1, Chapter 6, Section 6).

6.4.2. Temporal Patterns

Evidence presented in Wasko et al. (2024) found that due to climate change, temporal patterns may become slightly more front-loaded with a greater proportion of the storm volume falling towards the start of the storm. While the shift towards more front-loaded storms based on the analysis of historical data is statistically significant, the magnitude of these changes is small and the impact of temporal pattern changes on design flood estimates may be of little practical significance, particularly for those systems where flood levels are largely dependent on flood volume.

Currently there is no published methodology for quantifying the effect of changing temporal patterns on design flood estimates, nor published literature on the implications on their impacts on design flood estimates. If a user believes their design flood estimate may be sensitive to small changes in the temporal pattern, a sensitivity analysis can be undertaken (Book 7, Chapter 9). This can be performed using a non-uniform sampling (or weighting) approach in either an Ensemble Event averaging (Book 4, Chapter 3, Section 2) or Monte Carlo scheme (Book 4, Chapter 3, Section 2) whereby temporal patterns are preferentially selected based on projected future changes. If Ensemble Event averaging is used, then it may also be possible to directly alter the shape of the temporal patterns.

The projected change can be assessed by applying a reduction in the percentage of the event duration at which 50% of the cumulative event precipitation total is reached. This reduction is to be applied relative to the global temperature change ΔT (Table 1.6.2) as per Equation (1.6.1). For storm durations less than or equal to 6 hours, this change is no greater than a 2.5% per degree global temperature change. For events between 6 and 24 hours in duration the reduction is no greater than 1% per degree global temperature change. For longer durations, the shift may be assumed to be zero. More detailed information on the variation and uncertainty in these factors can be found in Figure 9 in Visser et al. (2023).

6.4.3. Spatial Patterns

Although there is some evidence that climate change will influence spatial patterns of extreme rainfall, there are considerable uncertainties around such changes for localised regions. As such, there is insufficient justification for amending spatial patterns or areal reduction factors.

6.4.4. Loss Parameters

There is evidence that historical changes in antecedent moisture conditions – the expected conditions prior to an extreme rainfall event – have impacted on frequent flood peaks, with smaller proportional impacts for rarer events. The review of Wasko et al. (2024) found multiple studies that presented evidence of drying antecedent moisture conditions across

Australia, particularly in regions that are experiencing decreases in annual and/or seasonal rainfall. Ho et al. (2023) linked projected changes in soil moisture to loss model parameters, allowing for the projection of loss parameters in design flood estimation under climate change. Respecting that regional differences will exist due to the differential wetting/drying of the continent with climate change, particularly with greater drying projected in southern Australia, Table 1.6.3 presents rates of change per degree Celsius for the National Resource Management Regions clusters (Figure 2.5.7) (CSIRO and Bureau of Meteorology, 2015). These clusters are the same as those used for temporal patterns (Book 2, Chapter 5, Section 3) and can be obtained from the ARR datahub (Babister et al., 2016). As there is limited data for the Rangelands, a pragmatic solution is to apply the changes recommended for the East Flatlands and West Flatlands.

The rates of change for initial loss (IL) and continuing loss (CL) are generally positive (losses will generally increase with higher temperatures), which reduces the impact of increased rainfall intensities, particularly for frequent floods or for systems whose performance is dependent on the volume of a flood as well as its peak. While it is perhaps surprising that adjustment factors are applied to continuing loss, it needs to be recognised that the IL/CL model is a gross simplification of the governing catchment processes. For example, it ignores differences in the travel times of runoff response across the catchment, differences in partially saturated areas due to antecedent conditions, and spatial heterogeneity of soil properties. As such, it is to be expected that the tendency for soils to be drier in warmer conditions will impact on estimates of both initial and continuing loss.

To adjust the loss parameters for climate change, the rates of change can be applied as per Equation (1.6.1) relative to the 1961-1990 baseline, as this represents a similar data baseline for the derivation of loss parameters in Book 5, Chapter 3 (see Example 2). As these represent storm losses, if applied to bursts they can be applied with reference to Book 2, Chapter 5, Section 9. The uncertainty range represents pooling across individual sites, climate change projections, different climate models and bias corrections methods. Given this pooling, and the non-linear response of soil moisture to changes in rainfall and temperature, the uncertainty presented here is likely to underestimate the true uncertainty. A sensitivity analysis to these uncertainties can be undertaken following the methodology in Book 7, Chapter 9.

Table 1.6.3. Rates of change for initial loss (IL) and continuous loss (CL) parameters per degree global temperature change (%/°C) for Natural Resource Management Regions clusters (CSIRO and Bureau of Meteorology, 2015), adapted from Ho et al. (2023). The 'likely' range (corresponding to ~66% range) is presented in parenthesis. These rates of change should be applied relative to a 1961-1990 baseline global temperature date unless a reasonable alternative is justified

Natural Resource Management Cluster	IL (%/°C)	CL (%/°C)
East Flatlands, and West Flatlands, Rangelands, and Rangelands West	4.5 (2.0-7.1)	5.6 (2.5-8.7)
Murray Basin	3.1 (1.0-5.7)	6.7 (1.5-12.1)
Southern Slopes Mainland, and Southern Slopes Tasmania	3.9 (1.5-7.2)	8.5 (2.9-15.7)
East Coast North, and East Coast South	2.0 (0.6-4.3)	3.8 (1.1-8.0)

Natural Resource Management Cluster	IL (%/°C)	CL (%/°C)
Central Slopes	1.1 (0.4-2.2)	2.0 (-0.5-7.5)
Wet Tropics	0.8 (-0.4-2.0)	1.4 (-0.1-4.8)
Monsoonal North	2.4 (1.0-5.4)	4.4(3.1-9.5)

6.4.5. Sea Levels and Sea Level Interaction

There is significant evidence that sea levels are increasing and will continue to increase due to climate change. Acknowledging that sea level rise and changes in storm surges vary regionally, the reader is referred to the 4th Edition of the Guidelines for Responding to the Effects of Climate Change in Coastal and Ocean Engineering (Harper, 2017) (and future updates) as well as their own jurisdictional guidance on changes in the sea level. The change in the sea level, jointly with a change in the extreme rainfall, can then be considered using the steps outlined in Book 6, Chapter 5, Section 5.

Changes to the interaction (or 'joint probability') between high sea levels (due to the combination of high astronomic tides and storm surges) and heavy rainfall events, remain poorly understood. The approach to calculating the joint probability of extreme rainfall and elevated sea levels described in Book 6, Chapter 5 does not account for possible changes in the interaction between those drivers. One approach to evaluate the importance of changes to the interaction between extreme rainfall and storm surge would be to conduct a sensitivity analysis on the dependence parameter recommended in Book 6, Chapter 5.

6.4.6. Increased Uncertainty Due to Climate Change

As discussed in Book 1, Chapter 6, Section 1, design flood estimates need to account for climate change and the associated increase in uncertainty. Epistemic uncertainties (due to lack of knowledge) affect:

- temperature projections, due to future emission pathways and feedbacks within the earth system, as described in Table 1.6.2
- future precipitation extremes for a given temperature change, as described in Table 1.6.1
- loss parameters, as described in Table 1.6.3.

Although not provided here, epistemic uncertainties also impact:

- sea level rise
- other inputs to the rainfall-runoff model, including temporal and spatial patterns
- rainfall-runoff transformation, recognising that potential changes to catchment properties (such as vegetation changes and/or bushfire) may mean that calibration of rainfall-runoff models to historic information may not reflect future conditions.

Each of these epistemic uncertainties affect the overall confidence in the design flood estimate. Epistemic uncertainties are not unique to the consideration of climate change. For example, IFD curves have inherent uncertainties associated with finite sample sizes. There are also other knowledge-based uncertainties associated with translating rainfall to estimates of runoff, flood level and velocity. When applied in the context of risk assessments, there are also uncertainties in estimating consequences (such as damages and/or loss of life) that reflect current and future exposure and vulnerability of assets and populations. It is recommended that approaches to characterising uncertainty associated with design floods should recognise that both climate change factors and non-climate change factors influence the design flood uncertainty. Various approaches for representing uncertainty within design flood estimation methods are provided in Book 7, Chapter 9.

6.5. Future Updates to the Climate Change Considerations

Key messages

While information in this update is the best available at the time of publication, more and better information will become available over the coming years. Future updates should consider the same approach of drawing on peer-reviewed science, considering multiple lines of evidence, synthesising these lines of evidence and publishing the results in a reputable peer-reviewed journal.

Climate change projections provided in this guidance are based on the review of the science by Wasko et al. (2024) and represent best-available information at the time of publication. The science review adopted a rigorous systematic review process, together with a meta-analysis to quantify the scaling relationships presented in Book 1, Chapter 6, Section 4. Information from multiple lines of evidence was sought, including the instrumental records (such as daily and sub-daily rainfall datasets and radar rainfall records) and modelling studies (including general circulation models and several classes of regional climate models together with statistical downscaling approaches).

It is recommended that future best practice updates of this chapter should consider:

- drawing on peer-reviewed studies in reputable scientific journals
- adopting a multiple-lines-of-evidence approach that recognises potential limitations with any single line of evidence
- synthesising available evidence using protocols that ensure rigour and avoid potential for biases
- ensuring that any synthesis is peer-reviewed and published in a reputable scientific journal.

6.6. Worked Examples

Key messages

This section comprises 4 worked examples. Examples 1a, 1b and 1c present a simple end-of-life design. They aim to familiarise the practitioner with the use of temperature projections in the context of design rainfall adjustment. The choice of climate scenario in these first examples is solely adopted for the purposes of demonstrating design rainfall adjustment.

Example 2 focuses on design flood estimation using adaptive management, presenting a use case that demonstrates how a practitioner could adopt climate scenarios in their design.

The user should also consider the decisions their modelling is informing and seek relevant jurisdictional guidance or information in relation to the consideration of climate change, including the selection of timeframes and climate scenarios.

The examples are:

1a - Design rainfall adjustment for a given emissions scenario.

1b - Design rainfall adjustment for a given global warming level.

1c - Design rainfall adjustment using a non-standard baseline.

2 - Adaptive management under uncertainty in the future climate.

Example 1a - Design rainfall adjustment for a given emissions scenario

This example presents a common use case where a practitioner wishes to estimate the impact of climate change on the existing 2016 IFDs for a time horizon corresponding to the end of the century (2081 to 2100 time horizon, Table 1.6.2). For this example, the practitioner requires an estimate of the 1% AEP 24-hour rainfall depth.

Using the 2016 IFD portal (Book 2, Chapter 3) the rainfall depth for the 24 hour, 1% AEP is 300 mm.

Table 1.6.1 provides an 8%/°C rate of change for a duration of 24 hours. The practitioner is testing the impact of the SSP2-4.5, which corresponds to a 2.4°C increase by the end of the century (Table 1.6.2) relative to the baseline adopted for the 2016 IFDs.

Using Equation (1.6.1) the factored design rainfall is:

$$300 \times \left(1 + \frac{8}{100}\right)^{2.4} = 361\text{mm}$$

Example 1b - Design rainfall adjustment for a given global warming level

The Shared Socioeconomic Pathways (SSPs) used in the sixth Assessment Report by the IPCC are likely to be superseded. Global Warming Levels (GWLs), which are derived from climate models run using SSPs, may be used more widely in the future to describe temperature projections. GWLs are reported relative to the pre-industrial period which has a different baseline to that used in the derivation of the 2016 IFDs. This example presents an approach to estimating the impact of climate change on design rainfall using global warming levels.

For comparison, we follow Example 1a. A practitioner wishes to estimate the 1% AEP 24 hour rainfall depth for the end of the century (2081 to 2100 time horizon, Table 1.6.2). Here, the practitioner is testing a scenario under which the world will warm by 3.0°C[4] by the end of the century relative to pre-industrial levels.

Using the 2016 IFD portal (Book 2, Chapter 3) the rainfall depth for the 24 hour, 1% AEP is 300 mm.

Table 1.6.1 provides an 8%/°C rate of change for a duration of 24 hours. The data used to derive the IFD curves can be approximated to correspond to the baseline of 1961-1990, but Global Warming Levels are calculated from pre-industrial periods. Hence the additional

[4]This value corresponds closely to the projected warming for SSP2-4.5 by the end of the century.

warming is 3.0°C-0.3°C = 2.7°C where 0.3°C is the difference between the pre-industrial period and the period used for derivation of the 2016 IFD curves.

Using Equation (1.6.1) the factored design rainfall is:

$$300 \times \left(1 + \frac{8}{100}\right)^{2.7} = 369\text{mm}$$

Example 1c - Design rainfall adjustment using a non-standard baseline

It is likely that IFD curves may be updated from time to time and hence a different baseline will be relevant. Furthermore, a practitioner may have reason to believe that a different baseline applies to the data for a particular region or location. This example presents how temperature projections can be adjusted to a non-standard baseline.

Following Example 1a, a practitioner wishes to estimate the 1% AEP 24-hour rainfall depth for the end of the century (2081 to 2100 time horizon, Table 1.6.2). Here however, the practitioner has estimated the 24-hour 1% AEP design rainfall using a record length from 1951 to 2020.

Table 1.6.1 provides an 8%/°C rate of change for a duration of 24 hours. The practitioner needs to choose an emissions scenario and chooses to test the impact of SSP2-4.5 which corresponds to a 2.4°C increase by the end of the century (Table 1.6.2) relative to the baseline adopted for the 2016 IFDs. But the temperature projections in Table 1.6.2 are estimated relative to 1961-1990. Hence the practitioner needs to check if there was additional warming between these 2 baselines.

There are several reputable sources of global temperature anomalies[5]. Examples include:

- Met Office, in collaboration with the Climatic Research Unit (CRU) at the University of East Anglia (UK).[6]
- Goddard Institute for Space Studies (GISS), which is part of the National Aeronautics and Space Administration (NASA) (USA).[7]
- National Climatic Data Center (NCDC), which is part of the National Oceanic and Atmospheric Administration (NOAA) (USA).[8]

Using the data from NOAA, the difference between the mean global temperature for the 1951-2020 period is approximately 0.2°C warmer than the 1961-1990 baseline baseline (i.e. 0.5°C above the pre-industrial baseline 1850-1900). The practitioner can proceed using the temperatures provided in Table 1.6.2 by adjusting for this increase. The long-term climate projected temperature increase is 2.4°C-0.2°C = 2.2°C above the new 1951-2020 baseline.

Using Equation (1.6.1), the factored design rainfall is:

$$300 \times \left(1 + \frac{8}{100}\right)^{2.2} = 355\text{mm}$$

Example 2 - Adaptive management under uncertainty around future climate

[5]A summary is provided at https://www.metoffice.gov.uk/weather/climate/science/global-temperature-records ; with links provided at https://climate.metoffice.cloud/temperature.html.
[6]See also https://crudata.uea.ac.uk/cru/data/temperature/
[7]See also https://data.giss.nasa.gov/gistemp/
[8]See also https://www.ncei.noaa.gov/access/monitoring/global-temperature-anomalies

A local council is considering construction of a levee to protect an adjacent community from flooding. A range of options were considered to find a solution that best balances the trade-offs between construction costs and ongoing savings in avoided damages and flood hazards. It was decided that the levee should provide protection from 1% AEP flooding impacts.

In this example, SSP2-4.5 and SSP3-7.0 are adopted as the moderate and high-warming scenarios, respectively, and the SSP5-8.5 scenario is adopted to aid the stress testing of decisions under a lower risk tolerance. Although SSP5-8.5 is not consistent with expected emission trends (Schwalm et al., 2020), using this scenario in a suite is one method of considering the uncertainty in each of the relationships between emissions, temperatures and resulting climate impacts. That is, use of the SSP5-8.5 scenario is used here to notionally represent a plausible upper bound on the projected climate impacts.

In practice, given the deep uncertainties involved, it was decided to take an adaptive approach whereby the levee was initially designed to provide protection from 1% AEP flooding out to the mid-term (2041-2060) under the mid-range emission scenario (SSP2-4.5). The design incorporated additional capacity in the levee foundations, and an extended riverbank corridor width to allow for a wider and taller embankment. This feature could be added at a time in the future associated with a high-range emission scenario (SSP3-7.0) with consideration given to the very-high (SSP5-8.5) emission scenario.

Accordingly, designs were prepared to provide for 2 combinations of emissions scenarios and time horizons (SSP2-4.5 in the mid-term, 2041-2060, and SSP3-7.0 in the long-term, 2081-2100), with consideration given to SSP5-8.5).

A summary of the design inputs used to develop the adaptive approach is provided in Table 1.6.4 for an event with a critical duration of 24 hours and historical design rainfall from the 2016 IFD portal of 125 mm, with initial loss of 15.0 mm and continuing loss of 2.5 mm/hr.

Table 1.6.4. Design inputs factor for climate change for Example 2. Design inputs assume an event critical duration of 24 hours duration and historical design rainfall of 125 mm, with initial loss of 15.0 mm and continuing loss of 2.5 mm/hr

Design consideration	SSP2-4.5 (2041-2060)	SSP3-7.0 (2081-2100)	SSP5-8.5 (2081-2100)
Temperature increase (°C)	1.7	3.3	4.1
24 hr rainfall (mm)	$125 \times \left(1 + \frac{8}{100}\right)^{1.7} = 142$	$125 \times \left(1 + \frac{8}{100}\right)^{3.3} = 161$	$125 \times \left(1 + \frac{8}{100}\right)^{4.1} = 171$
Initial loss (mm)	$15.0 \times \left(1 + \frac{3.9}{100}\right)^{1.7}$ $= 16.0$	$15.0 \times \left(1 + \frac{3.9}{100}\right)^{3.3}$ $= 17.0$	$15.0 \times \left(1 + \frac{3.9}{100}\right)^{4.1}$ $= 17.6$
Continuing loss (mm/hr)	$2.5 \times \left(1 + \frac{8.5}{100}\right)^{1.7} = 2.9$	$2.5 \times \left(1 + \frac{8.5}{100}\right)^{3.3} = 3.3$	$2.5 \times \left(1 + \frac{8.5}{100}\right)^{4.1} = 3.5$

6.7. Appendix

Table 1.6.5. Interpolated rate of change for rainfall depth with associated uncertainty range. Values have been interpolated from the values provided in Table 1 which was derived in Wasko et al. (2024). Rates of change are presented per degree global temperature change

(%/°C). The factors in this table are applicable for exceedance probabilities from 1EY up to and including the PMP and are designed for application across mainland Australia and Tasmania. If applied to the 2016 IFD curves these rates of change should be applied relative to a 1961-1990 baseline global temperature date unless a reasonable alternative is justified. Less information is available for rates of change for storm bursts between 1 and 24 hours, and hence estimates for these durations are obtained by a simple non-linear interpolation that represents the pragmatic interpretation of results obtained from Visser et al. (2021)

Duration	Rate of change (%/°C) and estimates of 'likely' range (corresponding to ~66% range) in parentheses.
1.5 hour	13.7 (6.1-25.6)
2 hour	12.8 (5.5-24.0)
3 hour	11.8 (4.7-22.0)
4.5 hour	10.8 (4.0-20.3)
6 hour	10.2 (3.6-19.2)
9 hour	9.5 (3.1-17.8)
12 hour	9.0 (2.7-16.9)
18 hour	8.4 (2.3-15.7)

6.8. References

Babister, M., Trim, A., Testoni, I., Retallick, M. 2016. The Australian Rainfall and Runoff Datahub, 37th Hydrology and Water Resources Symposium Queenstown NZ

Bates, B., McLuckie D.,Westra S., Johnson F., Green J., Mummery J., Abbs D. (2019). Climate Change Considerations in Australian Rainfall and Runoff, A Guide to Flood Estimation, 2019 Version 4.1, pp. 159-172

CSIRO and Bureau of Meteorology (2015), Climate Change in Australia, Projections for Australia's NRM Regions. Technical Report, CSIRO and Bureau of Meteorology, Australia. Retrieved from www.climatechangeinaustralia.gov.au/en [http://www.climatechangeinaustralia.gov.au/en].

Fyfe, J., Fox-Kemper, B., Kopp, R., Garner, G. (2021). Summary for Policymakers of the Working Group I Contribution to the IPCC Sixth Assessment Report - data for Figure SPM.8 (v20210809). NERC EDS Centre for Environmental Data Analysis, doi: 10.5285/98af2184e13e4b91893ab72f301790db.

Harper, B. (2017) Guidelines for responding to the effects of climate change in coastal and ocean engineering. Engineers Australia, National Committee on Coastal and Ocean Engineering.

Ho, M., Wasko, C., O'Shea, D., Nathan, R., Vogel, E., Sharma, A. (2023). Changes in flood-associated rainfall losses under climate change. J. Hydrol. 625, 129950.

IPCC (2023). Climate Change 2023: Synthesis Report. Contribution of Working Groups I, II and III to the Sixth Assessment Report of the Intergovernmental Panel on Climate Change [Core Writing Team, H. Lee and J. Romero (eds.)]. IPCC, Geneva, Switzerland.

IPCC (2021). Summary for Policymakers. In: Climate Change 2021: The Physical Science Basis. Contribution of Working Group I to the Sixth Assessment Report of the

Intergovernmental Panel on Climate Change [Masson-Delmotte, V., P. Zhai, A. Pirani, S.L. Connors, C. Péan, S. Berger, N. Caud, Y. Chen, L. Goldfarb, M.I. Gomis, M. Huang, K. Leitzell, E. Lonnoy, J.B.R. Matthews, T.K. Maycock, T. Waterfield, O. Yelekçi, R. Yu, and B. Zhou (eds.)]. Cambridge University Press, Cambridge, United Kingdom and New York, NY, USA, pp. 3−32, doi:10.1017/9781009157896.001.

Lee, J.-Y., J. Marotzke, G. Bala, L. Cao, S. Corti, J.P. Dunne, F. Engelbrecht, E. Fischer, J.C. Fyfe, C. Jones, A. Maycock, J. Mutemi, O. Ndiaye, S. Panickal, and T. Zhou, 2021: Future Global Climate: Scenario-Based Projections and Near-Term Information. In Climate Change 2021: The Physical Science Basis. Contribution of Working Group I to the Sixth Assessment Report of the Intergovernmental Panel on Climate Change [Masson-Delmotte, V., P. Zhai, A. Pirani, S.L. Connors, C. Péan, S. Berger, N. Caud, Y. Chen, L. Goldfarb, M.I. Gomis, M. Huang, K. Leitzell, E. Lonnoy, J.B.R. Matthews, T.K. Maycock, T. Waterfield, O. Yelekçi, R. Yu, and B. Zhou (eds.)]. Cambridge University Press, Cambridge, United Kingdom and New York, NY, USA, pp. 553–672, doi:10.1017/9781009157896.006.

Schwalm, C.R., Glendon, S., Duffy, P.B. (2020). RCP8.5 tracks cumulative CO 2 emissions. Proc. Natl. Acad. Sci. 117, 19656–19657. https://doi.org/10.1073/pnas.2007117117

Visser, J.B., Wasko, C., Sharma, A., Nathan, R. (2023). Changing Storm Temporal Patterns with Increasing Temperatures across Australia. J. Clim. 36, 6247–6259.

Visser, J.B., Wasko, C., Sharma, A., Nathan, R. (2021). Eliminating the “hook” in Precipitation-Temperature Scaling. J. Clim. 34, 9535–9549. https://doi.org/10.1175/JCLI-D-21-0292.1

Wasko, C., Westra, S., Nathan, R., Pepler, A., Raupach, T., Dowdy, A., Johnson, F., Ho, M., McInnes, K., Jakob, D., Evans, J., Villarini, G., and Fowler, H. (2024) A systematic review of climate change science relevant to Australian design flood estimation, Hydrol. Earth Syst. Sci. 28, 1251–1285. https://doi.org/10.5194/hess-28-1251-2024

BOOK 2

Rainfall Estimation

Rainfall Estimation

Table of Contents

1. Introduction 1
    1.1. Scope and Intent 1
    1.2. Application of these Guidelines 1
    1.3. Climate Change 1
    1.4. Terminology 2
    1.5. References 2
2. Rainfall Models 3
    2.1. Introduction 3
    2.2. Space-Time Representation of Rainfall Events 4
    2.3. Orographic Enhancement and Rain Shadow Effects on Space-Time Patterns 7
    2.4. Conceptualisation of Design Rainfall Events 8
        2.4.1. Event Definitions 8
        2.4.2. Rainfall Event Duration 9
        2.4.3. Event Rainfall Depth (or Average Intensity) 9
        2.4.4. Temporal Patterns of Rainfall 9
        2.4.5. Spatial Patterns of Rainfall 9
    2.5. Spatial and Temporal Resolution of Design Rainfall Models 9
    2.6. Applications Where Flood Estimates are Required at Multiple Locations 11
    2.7. Climate Change Impacts 11
    2.8. References 12
3. Design Rainfall 13
    3.1. Introduction 13
    3.2. Design Rainfall Concepts 13
    3.3. Climate Change Impacts 14
    3.4. Frequent and Infrequent Design Rainfalls 15
        3.4.1. Overview 15
        3.4.2. Rainfall Database 17
        3.4.3. Extraction of Extreme Value Series 28
        3.4.4. Regionalisation 33
        3.4.5. Gridding 35
        3.4.6. Outputs 37
    3.5. Very Frequent Design Rainfalls 37
        3.5.1. Overview 37
        3.5.2. Rainfall Database 38
        3.5.3. Extraction of Extreme Value Series 39
        3.5.4. Ratio Method 41
        3.5.5. Gridding 41
        3.5.6. Outputs 42
    3.6. Rare Design Rainfalls 42
        3.6.1. Overview 42
        3.6.2. Rainfall Database 43
        3.6.3. Extraction of Extreme Value Series 44
        3.6.4. Regionalisation 45
        3.6.5. Gridding 45
        3.6.6. Outputs 45
    3.7. Probable Maximum Precipitation Estimates 46
        3.7.1. Overview 46
        3.7.2. Estimation of PMPs 46
        3.7.3. Generalised Methods for Probable Maximum Precipitation Estimation 46
        3.7.4. Generalised Method of Probable Maximum Precipitation Estimation 48

3.8. Uncertainty in Design Rainfalls 49
3.9. Application 50
3.9.1. Design Rainfalls 50
3.9.2. Frequent and Infrequent Design Rainfalls (IFDs) 52
3.9.3. Very Frequent Design Rainfalls 53
3.9.4. Rare Design Rainfalls 54
3.9.5. Probable Maximum Precipitation Estimates 56
3.10. Acknowledgements 57
3.11. References 57
4. Areal Reduction Factors 61
4.1. Introduction 61
4.2. Derivation of Areal Reduction Factors 61
4.3. Areal Reduction Factor Recommendations 62
4.3.1. Areal Reduction Factors for Catchments up to 30 000 km^2, Durations up to 7 days and Events More Frequent than 0.05% AEP 62
4.3.2. Events That are Rarer than 0.05% Annual Exceedance Probability 66
4.3.3. Catchments with Areas Greater than 30 000 km^2 66
4.4. Worked Example 66
4.5. Limitations and Recommended Further Research 66
4.6. Recommended Further Research 67
4.6.1. Areal Reduction Factors 67
4.7. References 68
5. Temporal Patterns 70
5.1. Introduction 70
5.2. Temporal Pattern Concepts 70
5.2.1. Storm Components 70
5.2.2. Pattern Variability 72
5.2.3. History of Design Temporal Pattern Development 74
5.3. Storm Database 77
5.3.1. Data Quality 79
5.3.2. Event Selection and Analysis 79
5.3.3. Regional Characteristics 80
5.4. Pre Burst Rainfall and Antecedent Conditions 83
5.5. Design Point Temporal Patterns 85
5.6. Temporal Patterns for Areal Rainfall Bursts 88
5.6.1. Areal Rainfall Time Series Grid for All Australia 88
5.6.2. Average Areal Rainfall Calculation 88
5.6.3. Areal Temporal Pattern Selection 89
5.6.4. Design Areal Temporal Patterns 90
5.7. Other Temporal Pattern Options 91
5.7.1. Use of Historical Temporal Patterns 91
5.7.2. Complete Storm Patterns 91
5.7.3. Continuous Data 92
5.8. Climate Change Impacts 92
5.9. Temporal Pattern Application and Pre-burst 93
5.9.1. General 93
5.9.2. Ensemble Considerations 93
5.9.3. Upscaling of Patterns 94
5.9.4. Dealing with Inconsistencies and Smoothing of Results 94
5.9.5. Practical Issues 94
5.9.6. Point and Areal Temporal Pattern Meta-Data 96
5.9.7. Very Rare Point Temporal Patterns 96

5.9.8. Region Considerations 96
5.9.9. Pre-burst 97
5.10. Example 97
5.11. References 99
6. Spatial Patterns of Rainfall 103
6.1. Introduction 103
6.2. Methods for Deriving Spatial Patterns of Rainfall for Events 103
6.2.1. Precipitation Observation Methods and Uncertainties Associated with Reconstructing Space-Time Rainfall Patterns 103
6.2.2. Data Availability 104
6.2.3. Construction of Space-Time Patterns from Rainfall Gauge Networks 105
6.2.4. Space-Time Patterns for Calibration 108
6.3. Spatial and Space-Time Patterns for Design Flood Estimation 108
6.3.1. Guidance for Catchments up to and Including 20 km^2: Single Uniform Spatial Pattern 109
6.3.2. Guidance for Catchments Greater than 20 km^2: Single Non-Uniform Spatial Pattern 109
6.3.3. Alternative Approach: Monte Carlo Sampling from Separate Populations of Spatial and Temporal Patterns 110
6.3.4. Alternative Approach: Monte Carlo Sampling from Single Population of Space-Time Patterns 111
6.3.5. Spatial Patterns for Pre-Burst and Post-Burst Rainfall 111
6.3.6. Spatial Patterns for Continuous Rainfall Series 111
6.4. Potential Influences of Climate Change on Areal Reduction Factors, Spatial and Space-Time Patterns 113
6.5. Worked Examples 113
6.5.1. Catchment Used for Worked Examples 113
6.5.2. Worked Example 1: Interpolation of Spatial Patterns for an Event Using Various Methods 114
6.5.3. Worked Example 2: Calculation of Catchment Average Design Rainfall Depths and Areal Reduction Factors 119
6.5.4. Worked Example 3: Calculation of Spatial Pattern for Design Flood Estimation 123
6.5.5. Worked Example 4: Application to Design Flood Estimation 127
6.6. Recommended Further Research 128
6.6.1. Deriving Spatial and Space-Time Patterns of Rainfall for Events 128
6.6.2. Space-Time Patterns for Calibration of Rainfall Runoff Models to Historical Floods 129
6.6.3. Spatial and Space-Time Patterns for Design Flood Estimation 129
6.6.4. Potential Influences of Climate Variability and Climate Change 130
6.7. References 130
7. Continuous Rainfall Simulation 132
7.1. Use of Continuous Simulation for Design Flood Estimation 132
7.2. Rainfall Data Preparation 134
7.2.1. Errors in Rainfall Measurements 134
7.2.2. Options for Catchments with no Rainfall Records 137
7.2.3. Missing Rainfall Observations 138
7.3. Stochastic Rainfall Generation Philosophy 140
7.4. Rainfall Generation Models 140
7.4.1. Daily Rainfall Generation 140
7.4.2. Sub-daily Rainfall Generation 153
7.4.3. Identifying ‘Nearby’ Stations - Application to Sydney Airport 162

7.4.4. Modification of Generated Design Rainfall Attributes 163
7.4.5. Example of Daily and Sub-Daily Rainfall Generation 166
7.5. Implications of Climate Change 170
7.6. References 170

List of Figures

2.2.1. Conceptual Diagram of Space-Time Pattern of Rainfall 5

2.2.2. Conceptual Diagram of the Spatial Pattern and Temporal Pattern Temporal and Spatial Averages Derived from the Space-Time Rainfall Field 6

2.2.3. Conceptual Diagram Showing the Temporal Pattern over a Catchment and the Spatial Pattern Derived over Model Subareas of the Catchment 7

2.3.1. Classes of Design Rainfalls 14

2.3.2. Frequent and Infrequent (Intensity Frequency Duration) Design Rainfall Method 16

2.3.3. Daily Read Rainfall Stations and Period of Record 20

2.3.4. Continuous Rainfall Stations and Period of Record 21

2.3.5. Daily Read Rainfall Stations Used for ARR 1987 and ARR 2016 Intensity Frequency Duration Data 22

2.3.6. Continuous Rainfall Stations Used for ARR 1987 and ARR 2016 Intensity Frequency Duration Data 23

2.3.7. Length of Available Daily Read Rainfall Data 24

2.3.8. Length of Available Continuous Rainfall Data 24

2.3.9. Number of Long-term Daily Read Stations Used for ARR 1987 and ARR 2016 Intensity Frequency Duration Data 25

2.3.10. Length of Record of Continuous Rainfall Stations Used for ARR 1987 and ARR 2016 Intensity Frequency Duration Data 26

2.3.11. Analysis Areas Adopted for the BGLSR 32

2.3.12. Daily Read Rainfall Stations and Continuous Rainfall Stations Used for Very Frequent Design Rainfalls 39

2.3.13. Procedure to Derive Very Frequent Design Rainfall Depth Grids From Ratios 42

2.3.14. Daily Read Rainfall Stations with 60 or More Years of Record 44

2.3.15. Generalised Probable Maximum Precipitation Method Zones 47

2.3.16. Design Rainfall Point Location Map Preview 51

2.3.17. IFD Outputs 52

2.3.18. Very frequent Design Rainfall Outputs 53

2.3.19. Rare design rainfall outputs 54

2.3.20. Design Rainfall Output Shown as Table 55

2.3.21. Design Rainfall Output Shown as Chart 56

2.4.1. Area Reduction Factors Regions for Durations 24 to 168 Hours 65

2.5.1. Typical Storm Components 71

2.5.2. Two Different Storm Events with Similar Intensity Frequency Duration Characteristics (Sydney Observatory Hill) – Two Hyetographs plus Burst Probability Graph 73

2.5.3. Ten 2 hr Dimensionless Mass Curves 74

2.5.4. Decay Curve of Ten Dimensionless Patterns and AVM Patterns 75

2.5.5. Pluviograph Stations Record Lengths 77

2.5.6. Pluviograph Stations used Throughout South-Eastern Australia with Record Lengths 78

2.5.7. Temporal Pattern Regions 81

2.5.8. Example of Front, Middle and Back Loaded Events 82

2.5.9. Pre-burst Rainfall 84

2.5.10. Pre-burst to burst ratio 84

2.5.11. Standardised pre-burst to burst ratio distributions 85

2.5.12. Temporal Pattern Ranges 86

2.5.13. Combinations of Aspect Ratio and Rotation for Hypothetical Catchments 89

2.5.14. Comparison of Point Temporal Patterns and Areal Temporal Patterns - East Coast South Region - 1 Day 90

2.5.15. Comparison of Areal Temporal Patterns and the Temporal Pattern of the Closest Pluviograph for the Same Event 91

2.5.16. Tennant Creek Catchment 98

2.5.17. Duration Box plot for the 1% AEP 99

2.6.1. Stanley River Catchment, Showing Runoff-routing Model Subcatchments and the Locations of Daily rainfall and pluviograph gauges 114

2.6.2. Rainfall totals (mm) Recorded at Rainfall Gauges for the January 2013 Event in the Vicinity of the Stanley River Catchment 115

2.6.3. Application of Thiessen Polygons- Rainfall Totals for the January 2013 Event - Stanley River Catchment 116

2.6.4. Application of Inverse Distance Weighting - Rainfall Totals for the January 2013 Event - Stanley River Catchment 117

2.6.5. Application of Ordinary Kriging - Rainfall Totals for the January 2013 Event - Stanley River Catchment 118

2.6.6. Observed Semi-variogram and Fitted Linear Semi-variogram for the January 2013 Rainfall Event for Stanley River catchment, Applied in the Ordinary Kriging Algorithm 119

2.6.7. Non-dimensional Spatial Pattern (percentage of catchment average design rainfall depths) for Events with AEP of 1% and more Frequent for Stanley River to Woodford (top panel) and Stanley River to Somerset Dam (bottom panel) 125

2.6.8. Design Spatial Pattern of Design Rainfall Depths 1% AEP 24 hour Event for Stanley River to Woodford (top panel) and Stanley River to Somerset Dam (bottom panel) 126

2.6.9. Flood Frequency Curves for Stanley River at Somerset Dam Inflow Derived from Analysis of Estimated Annual Maxima and from RORB Model Simulations 128

2.7.1. Flood Events for a Typical Australian Catchment - Scott Creek, South Australia ... 133

2.7.2. Double Mass Curve Analysis for Rainfall at Station A (from World Meteorological Organisation (1994)) 136

2.7.3. Total Rainfall Amounts for Rainfall Station 009557 over the Period 1956-1962 (from Viney and Bates (2004)) 137

2.7.4. Rainfall Stations used in Table 2.7.1 for the Transition Probability Model (Srikanthan et al., 2003) 146

2.7.5. Identification of “Similar” Locations for Daily Rainfall Generation using RMMM 151

2.7.6. Generation of Daily Rainfall Sequences using the Regionalised Modified Markov Model Approach ... 152
2.7.7. Disaggregated Rectangular Intensity Pulse Model (extracted from Heneker et al. (2001)) ... 155
2.7.8. Schematic of Non-dimensional Random Walk used in DRIP disaggregate pulses . 156
2.7.9. Heneker et al. (2001) Model Fitted to Monthly Inter-event Time Data for Melbourne in January ... 157
2.7.10. Heneker et al. (2001) Model Fitted to Monthly Storm Duration Data for Melbourne in May ... 158
2.7.11. State-based Method of Fragments Algorithm used in the Regionalised Method of Fragments Sub-daily Rainfall Generation Procedure ... 159
2.7.12. Sydney Airport and nearby pluviograph stations ... 163
2.7.13. Main Steps Involved in the Adjustment of Raw Continuous Rainfall Sequences to Preserve the Intensity Frequency Duration relationships ... 165
2.7.14. Annual Rainfall Simulations for Alice Springs using 100 Replicates ... 167
2.7.15. Intensity Frequency relationship for 24 hour Duration. ... 167
2.7.16. 6 minute (left column) and 6 hour (right column) Annual Maximum Rainfall against Exceedance Probability for Alice Springs. ... 169
2.7.17. Intensity Duration Frequency Relationships for Target and Simulated Rainfall before and after Bias Correction at Alice Springs ... 170

List of Tables

2.3.1. Classes of Design Rainfalls ... 14

2.3.2. Frequent and Infrequent (Intensity Frequency Duration) Design Rainfall Method ... 17

2.3.3. Rainfall Reporting Methods ... 18

2.3.4. Restricted to Unrestricted Conversion Factors ... 29

2.3.5. Intensity Frequency Duration Outputs ... 37

2.3.6. Very Frequent Design Rainfall Method ... 38

2.3.7. Very Frequent Design Rainfall Outputs ... 42

2.3.8. Rare Design Rainfall Method ... 43

2.3.9. Rare Design Rainfall Outputs ... 45

2.4.1. ARF Procedure for Catchments Less than 30 000 km^2 and Durations up to and Including 7 Days ... 62

2.4.2. ARF Equation (2.4.2) Coefficients by Region for Durations 24 to 168 hours Inclusive ... 65

2.5.1. Number of Pluviographs by Decade ... 78

2.5.2. Regions- Number of Gauges and Events ... 80

2.5.3. Burst Loading by Region and Duration ... 82

2.5.4. Regional Temporal Pattern Bins ... 86

2.5.5. Temporal Pattern Durations ... 86

2.5.6. Temporal Pattern Selection Criteria ... 87

2.5.7. Areal Rainfall Temporal Patterns - Catchment Areas and Durations ... 88

2.5.8. Minimum Number of Pluviographs Required for Event Selection for Each Catchment Area ... 89

2.5.9. Areal Temporal Pattern Sets for Ranges of Catchment Areas ... 93

2.5.10. Alternate Regions Used for Data ... 95

2.5.11. Flows for the 1% Annual Exceedance Probability for Ten Burst Events ... 99

2.6.1. Calculation of Weighted Average of Point Rainfall Depths for the 1% AEP 24 hour Design Rainfall Event for the Stanley River at Woodford ... 120

2.6.2. Stanley River Catchment to Woodford: Calculation of Catchment Average Design Rainfall Depths (bottom panel) from Weighted Average of Point Rainfall Depths (top panel) and Areal Reduction Factors (middle panel) ... 121

2.6.3. Stanley River Catchment to Somerset Dam: Calculation of Catchment Average Design Rainfall Depths (bottom panel) from Weighted Average of Point Rainfall Depths (top panel) and Areal Reduction Factors (middle panel) ... 122

2.6.4. Calculation of Design Spatial Pattern for Stanley River at Woodford ... 123

2.6.5. RORB Model Scenarios Run for Worked Example on Stanley River Catchment to Somerset Dam ... 127

2.7.1. Alternative Methods for Stochastic Generation of Daily Rainfall ... 141

2.7.2. Number of States used for Different Rainfall Stations in the Transition Probability Model (Srikanthan et al., 2003) 145

2.7.3. State Boundaries for Rainfall Amounts in the Transition Probability Model 146

2.7.4. Daily Scale Attributes used to Define Similarity between Locations 149

2.7.5. Commonly used Sub-daily Rainfall Generation Models 153

2.7.6. Sub-daily Attributes used to Define Similarity between Locations 160

2.7.7. Logistic Regression Coefficients for the Regionalised Method of Fragments Sub-daily Generation Model 161

2.7.8. Statistical Assessment of Daily Rainfall from RMMM for Alice Springs using 100 Replicates 67 years Long 166

2.7.9. Performance of extremes and representation of zeroes (for 6 minute time-steps) from the sub-daily rainfall generation using RMOF for at-site generation using observed sub-daily data (option 1), at-site disaggregation using observed daily data (option 2), and the purely regionalised case (option 3). 168

List of Equations

2.4.1. Short duration ARF Equation 64

Chapter 1. Introduction

Mark Babister, Monique Retallick

Chapter Status	Final
Date last updated	14/5/2019
Minor edits	27/08/2024

1.1. Scope and Intent

Nearly all design flood estimation techniques rely on rainfall inputs to estimate flood quantiles. These methods use catchment modelling techniques to estimate the flood quantiles that would be derived from Flood Frequency Analysis if a long-term gauge record was available. While simple methods just use rainfall intensity frequency duration data more complex approaches require temporal and spatial rainfall information and continuous simulation approaches require long-term rainfall sequences. Irrespective of the approach, it is important to understand how the design rainfall inputs were derived and how they vary from observed events.

Despite the advances in flood estimation many design inputs are assumed to be much simpler than real or observed events. The more complex methods continue to make assumptions including the use storm burst instead of a complete storm and spatial uniform temporal patterns. For these reasons actual rainfall events tend to show considerably more variability than design events and often have different probabilities at different locations.

This book describes the different rainfall inputs can be derived and how they can be used. Book 2, Chapter 2 provides an introduction to rainfall models. Book 2, Chapter 3 details the development of the design rainfalls (Intensity Frequency Duration data) by the Bureau of Meteorology. Book 2, Chapter 4 and Book 2, Chapter 5 discuss the spatial and temporal distributions of rainfall respectively. Book 2, Chapter 7 covers the development of continuous rainfall time series for use in continuous simulation models.

1.2. Application of these Guidelines

The application of the design inputs discussed in this Book to Very Rare and Extreme floods is discussed in Book 8.

1.3. Climate Change

These guidelines apply to the current climate. Statistically significant increases in rainfall intensity have been detected in Australia for short duration rainfall events and are likely to become more evident towards the end of the 21st century (Westra et al., 2013). Changes in long duration events are expected to be smaller and harder to detect, but projections analysed by CSIRO and Australian Bureau of Meteorology (2007) show that an increase in daily precipitation intensity is likely under climate change. It is worth noting that a warming climate can lead to decreases in annual rainfall along with increases in flood producing rainfall.

The IFD's presented in this chapter can be adjusted for future climates using the method outlined in Book 1, Chapter 6[1]. Scaling based on temperature is recommended, as climate models are much more reliable at producing temperature estimates than individual storm events.

The impact of climate change on storm frequency, mechanism, spatial and temporal behaviour is less understood. Work by (Abbs and Rafter, 2009) suggests that increases are likely to be more pronounced in areas with strong orographic enhancement. There is insufficient evidence to confirm whether this result is applicable to other parts of Australia. Work by (Wasco and Sharma, 2015) analysing historical storms found that, regardless of the climate region or season, temperature increases are associated with patterns becoming less uniform, with the largest fractions increasing in rainfall intensity and the lower fraction decreasing.

1.4. Terminology

The terminology for frequency descriptor described in Figure 1.2.1 applies to all chapters of this book other than Book 2, Chapter 3 Design Rainfall.

1.5. References

Abbs, D. and Rafter, T. (2009), Impact of Climate Variability and Climate Change on Rainfall Extremes in Western Sydney and Surrounding Areas: Component 4 - dynamical downscaling, CSIRO.

CSIRO and Australian Bureau of Meteorology (2007), Climate Change in Australia, CSIRO and Bureau of Meteorology Technical Report, p: 140. www.climatechangeinaustralia.gov.au

Wasko, C. and Sharma, A. (2015), Steeper temporal distribution of rain intensity at higher temperatures within Australian storms, Nature Geoscience, 8(7), 527-529.

Westra, S., Evans, J., Mehrotra, R., Sharma, A. (2013), A conditional disaggregation algorithm for generating fine time-scale rainfall data in a warmer climate, Journal of Hydrology, 479: 86-99

[1] This section was written before the latest climate change guidance in Book 1, Chapter 6 (2024). A minor change to the text has been made to reflect the change in guidance.

Chapter 2. Rainfall Models

James Ball, Phillip Jordan, Alan Seed, Rory Nathan, Michael Leonard, Erwin Weinmann

Chapter Status	Final
Date last updated	14/5/2019

2.1. Introduction

The philosophical basis for use of a catchment modelling approach is the generation of data that would have been recorded if a gauge were present at the location(s) of interest for the catchment condition(s) of interest. For reliable and robust predictions of design flood estimates with this philosophical basis, there is a need to ensure that rainfall characteristics as one of the major influencing factors are considered appropriately.

There are many features of rainfall to consider when developing a rainfall model for design flood prediction; exploration of these features can be undertaken using historical storm events as a basis. In using this approach, there is a need to acknowledge that consideration of historical events is an analysis problem and not a design problem. Nonetheless, insights into the characteristics of rainfall events for design purposes can be obtained from this review.

Rainfall exhibits both spatial and temporal variability at all spatial and temporal scales that are of interest in flood hydrology. High resolution recording instruments have identified temporal variability in rainfall from time scales of less than one minute to several days (Marani, 2005). Similarly, observations of rainfall from high resolution weather radar and satellites have demonstrated spatial variability in rainfall at spatial resolutions from 1 km to more than 500 km (Lovejoy and Schertzer, 2006).

While it is important to be aware of this large degree of variability, for design flood estimation based on catchment modelling it is only necessary to reflect rainfall variability at space and time scales that are influential in the formation of flood events. The main focus is generally on individual storms or bursts of intense rainfall within storms that cover the catchment extent. However, it needs to be recognised that, depending on the design problem (e.g. flood level determination in a system with very large storage and small outflow capacity), the relevant 'event' to be considered may consist of rainfall sequences that include not just one storm but extend over several months or even years.

Rainfall models are designed to capture in a simplified fashion those aspects of the spatial and temporal variability of rainfall that a relevant to specific applications. A broad distinction between different rainfall models can be made on the basis of their scope. Commonly rainfall models consider only the temporal dimension by neglecting the spatial dimension. Inclusion of the spatial dimension together with the temporal dimension results in an alternative form of a rainfall model. This leads to the following categorisation of rainfall models:

- Models that concentrate on significant rainfall events (storms or intense bursts within storms) at a point or with a typical spatial pattern that have the potential to produce floods;
- Models that attempt to simulate rainfall behaviour over an extended period at a point, producing essentially a complete (continuous) rainfall time series incorporating flood

producing bursts of rainfall, low intensity bursts of rainfall and the dry periods between bursts of rainfall(Book 2, Chapter 7); and

- Models that attempt to replicate rainfall in both the spatial and temporal dimensions. Currently, models in this category are being researched and are not in general usage. There are, however, many problems where rainfall models of this form may be applicable.

Rainfall models that concentrate on the flood producing bursts of rainfall have the inherent advantage of conciseness (from a flood perspective, only the interesting bursts of rainfall are considered). Hence, there is great potential to consider interactions of rainfall with other influential flood producing factors but they also need to allow for the impact of varying initial conditions.

Continuous rainfall models (Book 2, Chapter 7) have the inherent advantage of allowing the initial catchment conditions (e.g. soil moisture status and initial reservoir content) at the onset of a storm event to be simulated directly. However, the need to model the rainfall characteristics of both storm events (intense rainfall) and inter-event periods (no rainfall to low intensity rainfall) adds significant complexity to continuous rainfall models. The greater range of events these models cover tends to be achieved at the cost of reduced ability to represent rarer, higher intensity rainfall events. Additionally, very long sequences of rainfall observations are required to properly sample rarer events. These issues make continuous rainfall models more suitable for simulation of frequent events.

Rainfall data are mostly obtained from individual gauges (daily read gauges or pluviographs) and only provide data on point rainfalls. However, for catchment simulation the interest is on rainfall characteristics over the whole catchment. Rainfall models thus are needed to allow extrapolation of rainfall characteristics from the point scale to the catchment scale. In extrapolating rainfall characteristics from a point to a catchment or subcatchment, there is a need to ensure that the extrapolation does not introduce bias into the predictions. This applies to both continuous rainfall models and event rainfall models.

2.2. Space-Time Representation of Rainfall Events

When combined, the spatial and temporal variability of rainfall will be referred to as the space-time variability of rainfall. The space-time pattern of rainfall over a catchment or study area is therefore defined in three dimensions: two horizontal dimensions, which are normally latitude and longitude (or easting and northing in a projected coordinate system) and one temporal dimension. In practice, the space-time pattern of rainfall will often be described as a three dimensional matrix, with the value in each element of the matrix representing either the accumulated rainfall or the mean rainfall intensity for a grid cell over the catchment and a specified period of time within the event, as shown in Figure 2.2.1.

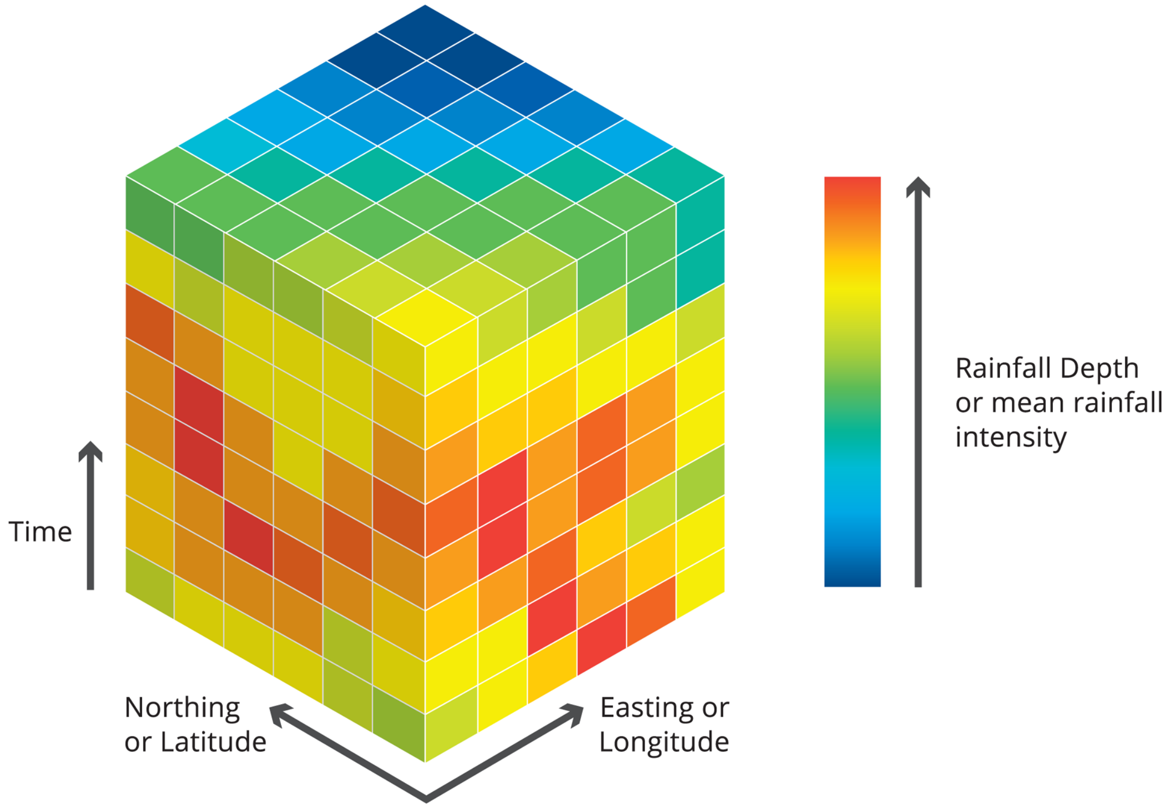


Figure 2.2.1. Conceptual Diagram of Space-Time Pattern of Rainfall

If the space-time pattern of rainfall is considered as a field defined in three dimensions, then the temporal and spatial patterns of rainfall that have conventionally been used in hydrology can be considered as convenient statistical means of summarising that field. The temporal pattern of rainfall over a catchment area is derived by taking an average in space (over one or more grid elements) of the rainfall depth (or mean intensity) over each time increment of the storm. The spatial pattern of rainfall for an event is defined by taking an average in time (over one or more time periods) of the rainfall depth (or mean intensity) over each grid cell of the catchment. Derivation of spatial and temporal patterns is demonstrated with the conceptual diagram in Figure 2.2.2. Commonly, the spatial pattern is defined by averaging over each subarea to be used in a model of the catchment or study area as shown in Figure 2.2.3. The application of some catchment modelling systems (for example, rainfall-on-grid models commonly used to simulate floods in urban areas), however, require grid based spatial patterns of rainfall. In these situations, each grid element can be considered as a subarea or subcatchment.

The space-time pattern of rainfall varies in a random manner between events and within events influenced by spatial and temporal correlation structures that are an inherent observed property of rainfall. The random space-time variability may make it difficult to specify typical or representative spatial patterns for some catchments. Umakhanthan and Ball (2005) in a study of the Upper Parramatta River Catchment in NSW showed the variation in the temporal and spatial correlation between storm events on that catchment.

However, there are often hydrometeorological drivers, as discussed in Book 2, Chapter 4 that cause some degree of similarity in spatial and space-time patterns of flood producing rainfall between events for a particular catchment. This similarity increases for the rarer events and decreases for the more frequent events.

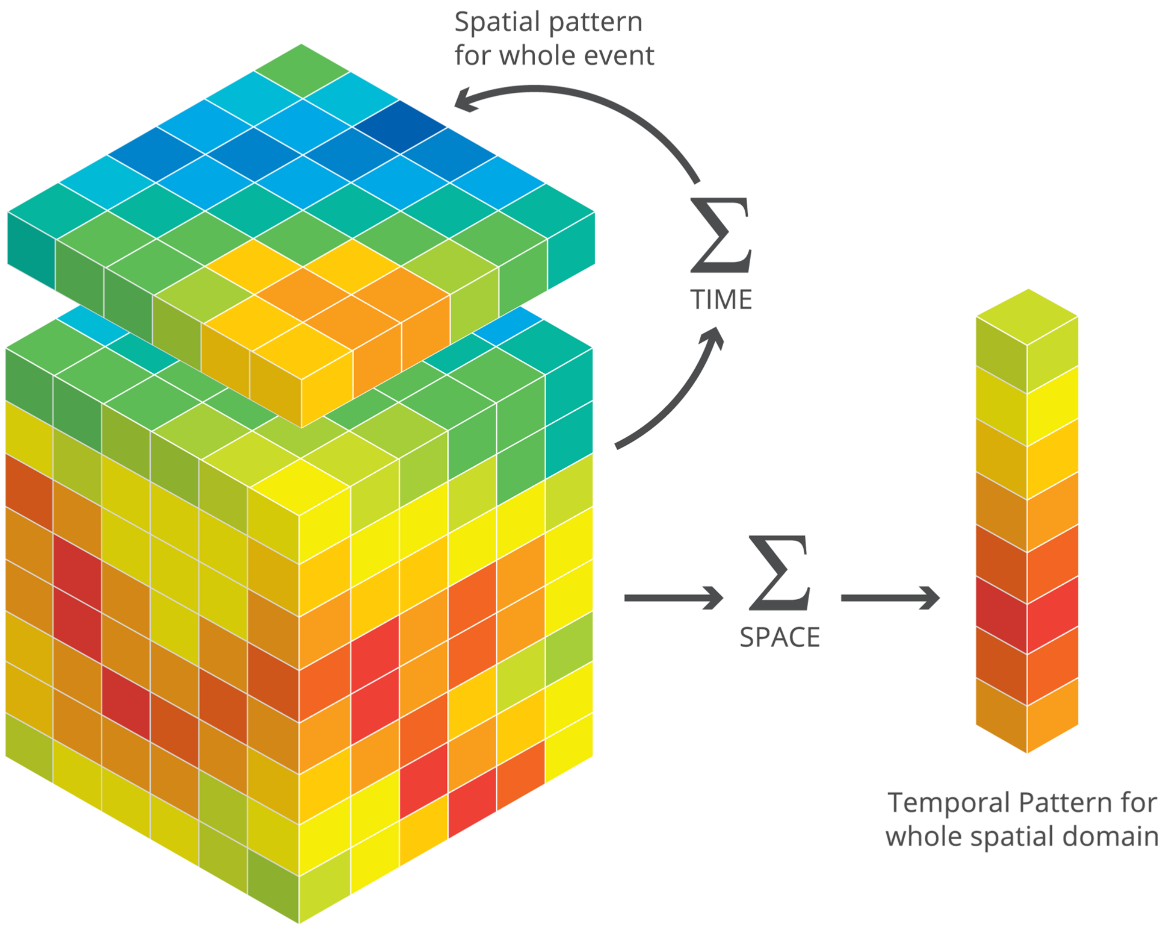


Figure 2.2.2. Conceptual Diagram of the Spatial Pattern and Temporal Pattern Temporal and Spatial Averages Derived from the Space-Time Rainfall Field

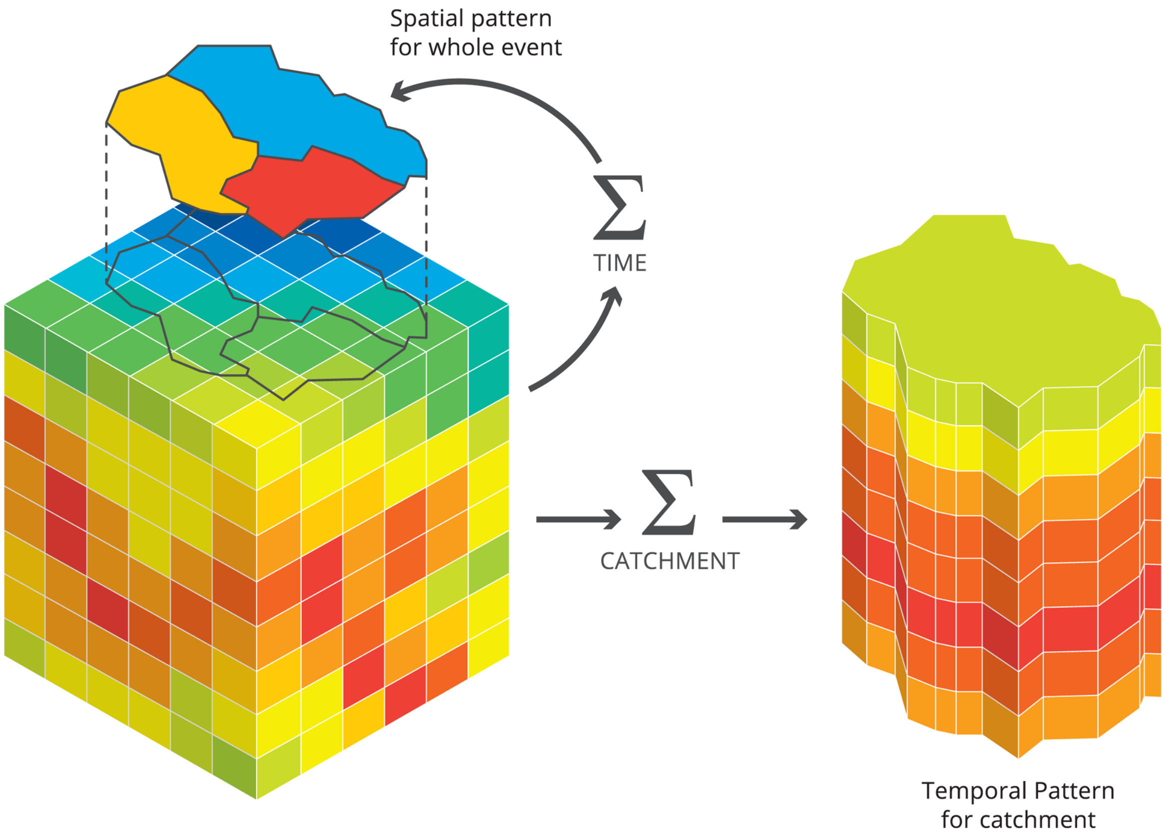


Figure 2.2.3. Conceptual Diagram Showing the Temporal Pattern over a Catchment and the Spatial Pattern Derived over Model Subareas of the Catchment

2.3. Orographic Enhancement and Rain Shadow Effects on Space-Time Patterns

Orographic precipitation, also known as relief precipitation, is precipitation generated by a forced upward movement of air upon encountering a physiographic upland. This lifting can be caused by two mechanisms:

- Upward deflection of large scale horizontal flow by the topography; or
- Anabatic or upward vertical propagation of moist air up an orographic slope caused by daytime heating of the mountain barrier surface.

Upon ascent, the air that is being lifted will expand and cool. This adiabatic cooling of a rising moist air parcel may lower its temperature to its dew point, thus allowing for condensation of the water vapour contained within it, and hence the formation of a cloud. Rainfall can be generated from the cloud through a number of physical processes (Gray and Seed, 2000). The cloud liquid droplets grow through collisions with other droplets to the size where they fall as rain. Rain drops from clouds at high altitude may fall through the clouds near the surface that have formed because of the uplift due to topography and grow as a result of collisions with the cloud droplets. Air may also become unstable as it is lifted over higher areas of terrain and convective storms may be triggered by this instability. These influences combine to typically produce a greater incidence of rainfall on the upwind side of hills and mountains and also typically larger rainfall intensities on the upwind side than would otherwise occur in flat terrain.

The space-time pattern will vary between every individual rainfall event that occurs in a catchment. In catchments that are subject to orographic influences, there will commonly be similarity in the space-time pattern of rainfall between many of the different events that are observed over the catchment. This will typically be the case for catchments that are subject to flood producing rainfall events that have similar hydrometeorological influences. For example, the spatial patterns of rainfall for different events may often demonstrate similar ratios of total rainfall depth in the higher elevations of the catchment to total rainfall depth at lower elevations.

The spatial patterns of rainfall in catchments that are influenced by orographic effects represent a systematic bias away from a completely uniform spatial pattern. The influence of this systematic bias in spatial pattern of rainfall should be explicitly considered in design flood estimation. Other hydrometeorological influences, such as the distance from a significant moisture source like the ocean may also give rise to systematic bias in the spatial pattern of rainfall.

2.4. Conceptualisation of Design Rainfall Events

Ideally, the space-time variation of rainfall over a catchment would be represented as a moving space-time field of rainfall at the appropriate spatial and temporal resolution. However, while current developments are progressing in that direction (for example, stochastic-space-time rainfall models developed by (Seed et al., 2002; Leonard et al., 2008), the rainfall models widely used in practice are based on a more reductionist approach, dealing separately with the spatial and temporal variability of rainfall. A result of this reductionist approach is that rainfall bursts are assumed to be stationary; in other words, the storm does not move during the period of rainfall.

For ease of modelling, storm events can be conceptualised and represented by four main event characteristics that are analysed and modelled separately:

- Duration of storm or burst event;
- Total rainfall depth (or average intensity) over the event duration, at a point or over a catchment;
- Spatial distribution (or pattern) of rainfall over the catchment during the event; and
- Temporal distribution (or pattern) of rainfall during the event.

These rainfall event characteristics are discussed in Book 2, Chapter 2, Section 4. to Book 2, Chapter 2, Section 4

2.4.1. Event Definitions

The modelling of rainfall events first requires a clear definition of what constitutes an event (Hoang et al., 1999). Given the variation of rainfall in time and space, it is not immediately apparent when an event starts and ends. Start and end points of rainfall events need to be defined by rainfall thresholds or separation in time from preceding/subsequent rainfall. For an event to be significant, it may also need to exceed a total event rainfall threshold.

Two different types of rainfall events are relevant for design flood estimation: complete storm events and internal bursts of intense rainfall. While complete storm events are the theoretically more appropriate form of event for flood simulation, the internal rainfall bursts of given duration, regardless of where they occur within a storm event, lend themselves more readily for statistical analysis. The Intensity Frequency Duration (IFD) data covered in Book 2, Chapter 3 are thus for rainfall burst events.

2.4.2. Rainfall Event Duration

Actual storm events vary in their duration, from local thunderstorms lasting minutes to extended rainfall events lasting several days. This variation occurs in a random fashion, and rainfall event duration for a particular region can be characterised by a probability distribution. However, for practical design flood estimation, the occurrence of rainfalls of different durations within an appropriate range is generally assumed to have equal probability, and the 'critical rainfall duration' is then determined as the one that maximises the value of the design flood characteristic of direct interest.

The design rainfall data provided in ARR covers the range of rainfall burst durations from 1 minute to 7 days.

2.4.3. Event Rainfall Depth (or Average Intensity)

The basic methods for estimating design rainfall depths (or average intensities) for different durations are discussed in Book 2, Chapter 3 for both point rainfalls . The principal modelling approach used is to fit a probability distribution to series of rainfall depth observations (annual maximum or peak over threshold) for the selected event duration at sites with long, reliable rainfall records. The results of these at-site analyses are then generalised over regions with similar rainfall characteristics and mapped over the whole of Australia. The conversion of point design rainfalls to average catchment design rainfalls is modelled through rainfall areal reduction factors is discussed in Book 2, Chapter 4.

2.4.4. Temporal Patterns of Rainfall

There are two distinct model representations of the temporal variability of rainfall within events (for complete storms or internal bursts), depending on whether the model only reflects the central tendency of different observed patterns or the variability of patterns for different events is also modelled. These differences in modelling approach are further discussed in Book 2, Chapter 2, Section 5 and Book 2, Chapter 5.

2.4.5. Spatial Patterns of Rainfall

In larger catchments and where there is a consistent spatial trend in observed rainfall depths, (see Book 2, Chapter 2, Section 4) this needs to be represented by a non-uniform spatial rainfall pattern. . The models for representing the typical spatial variability of rainfall are based either on the analysis and generalisation of historical storms or on spatial trends derived from analysis of design rainfall depths (IFD maps). The application of these models is explained in Book 2, Chapter 6, Section 4

In the following, a number of different approaches to model the space-time characteristics of event rainfall for design flood estimation are introduced briefly.

2.5. Spatial and Temporal Resolution of Design Rainfall Models

The space-time pattern of rainfall for an individual flood event will often have an appreciable influence on the flow hydrograph generated at the outlet of a catchment. Two rainfall events may have identical total volumes over a defined catchment area and duration but differences in their space-time patterns may produce very different hydrographs at the outlet of the catchment. Both the runoff generation and runoff-routing processes in catchments are typically non-linear, so a space-time pattern that exhibits more variability will normally generate a higher volume of runoff and larger peak flow at the catchment outlet than a space-time pattern that is more uniform.

Variability in hydrographs introduced by space-time variability in rainfall will be accentuated in catchments that have spatial and temporal variability in runoff generation and routing processes. For example, in a partly urbanised catchment, a rainfall event with a spatial pattern that has larger depths on the urban part of the catchment than the rural part would normally produce both a larger volume of runoff and flood peak at the catchment outlet than a storm of the same depth and duration that has a spatially uniform rainfall pattern. Other factors in catchments that may accentuate the influence of the space-time rainfall pattern on the variability in hydrographs produced at the catchment outlet include:

- the presence of reservoirs and lakes, for which all rainfall on the water surface is converted to runoff;
- the presence of dams, weirs, drains and other flow regulating structures;
- significant variations in soil type;
- significant variations in vegetation type, such as forested and cleared areas;
- the arrangement of the drainage network of the catchment the dependency of alternative flow paths on event magnitude and differences in contributing area with length of network;
- significant variations in stream channel and floodplain roughness;
- significant variations in slope of stream channels and floodplains;
- significant variations in antecedent climatic conditions across the catchment prior to the events; and
- variations in elevation, snowpack depth, density and temperature in those catchments subject to rain-on-snow flood events.

The required resolution of rainfall models to adequately reflect the variability of rainfall in historical rainfall events has been investigated by (Umakhanthan and Ball, 2005) for the Upper Parramatta River catchment. (Umakhanthan and Ball, 2005) categorised the variability of recorded storm events in the spatial and temporal domains and confirmed that the degree of spatial and temporal resolution of rainfall inputs to flood estimation models can have a significant impact on resulting flood estimates. A range of other studies have come to similar conclusions but have found it difficult to give more than qualitative guidance on the required degree of spatial and temporal resolution of rainfall for different modelling applications. The conclusions can be summarised in qualitative terms as:

- “Spatial rainfall patterns are understood to be a dominant source of variability for very large catchments and for urban catchments but for other hydrological contexts, results vary. Much of this knowledge is either site specific or is expressed qualitatively” (Woods and Sivapalan, 1999).
- Where short response times are involved in urban catchments, inadequate representation of temporal variability of rainfall can lead to significant underestimation of design flood peaks (Ball, 1994). More generally, the importance of temporal variability of rainfall in flood

modelling depends on the degree of ‘filtering’ of shorter term rainfall peaks through catchment routing processes (ie. the amount of storage in the catchment system) and the interaction of flood contributions from different parts of a catchment system.

Sensitivity analyses can be applied to determine for a specific application the influence of the adopted spatial and temporal resolution of design rainfalls on flood estimates and their uncertainty bounds.

2.6. Applications Where Flood Estimates are Required at Multiple Locations

Design flood estimates are often required at multiple locations within a catchment or study area. Ideally, flood simulation (e.g. using Monte Carlo approaches) should consider a large number of complete storm events that cover the whole AEP spectrum of interest and have internal characteristics which automatically reproduce the critical rainfall bursts over a range of temporal and spatial scales. Unfortunately, such comprehensive ensembles of synthetic storm events are not currently available, and combined system wide analysis is thus not yet feasible. Instead, separate analysis at the different locations (subcatchments) of interest is required, using design rainfall events for the relevant space and time scales. To this end, it is necessary to derive design rainfall inputs for the catchment upstream of each required location. This involves:

- Deriving average values of the point design rainfalls for the total catchment upstream of each location;
- Conversion of average point design rainfall values to areal estimates by multiplying by the ARF applicable to the total catchment area upstream of each location; and
- Adoption of space-time patterns of rainfall relevant to the total catchment area upstream of each location.

It is commonly found that design flood estimates are required at one or more locations in a catchment where flow gauges are not located. If so, it will be necessary to use the above procedure to derive design rainfalls for the catchment upstream of each gauge location so that the rainfall-based design floods estimates can be verified against estimates derived from Flood Frequency Analysis at each flow gauge. Different sets of design rainfall intensities, ARF and space-time patterns should be calculated for the each of the catchments draining to the other locations of interest, which are not at flow gauges.

2.7. Climate Change Impacts

Statistically significant increases in rainfall intensity have been detected in Australia for short duration rainfall events and are likely to become more evident towards the end of the 21st century (Westra et al., 2013). Changes in long duration events are expected to be smaller and harder to detect, but projections analysed by CSIRO and Australian Bureau of Meteorology (2007) show that an increase in daily precipitation intensity is likely under climate change. It is worth noting that a warming climate can lead to decreases in annual rainfall along with increases in flood producing rainfall.

The impact of climate change on storm frequency, mechanism, spatial and temporal behaviour is less understood.

Work by Abbs and Rafter (2009) suggests that increases are likely to be more pronounced in areas with strong orographic enhancement. There is insufficient evidence to confirm whether this result is applicable to other parts of Australia. Work by Wasco and Sharma (2015) analysing historical storms found that, regardless of the climate region or season, temperature increases are associated with patterns becoming less uniform, with the largest fractions increasing in rainfall intensity and the lower fraction decreasing.

The implications of these expected climate change impacts on the different design rainfall inputs to catchment modelling are discussed further in the relevant sub-sections of the following chapters.

2.8. References

Abbs, D. and Rafter, T. (2009), Impact of Climate Variability and Climate Change on Rainfall Extremes in Western Sydney and Surrounding Areas: Component 4 - dynamical downscaling, CSIRO.

Ball, J.E. (1994), 'The influence of storm temporal patterns on catchment response', Journal of Hydrology, 158(3-4), 285-303.

CSIRO and Australian Bureau of Meteorology (2007), Climate Change in Australia, CSIRO and Bureau of Meteorology Technical Report, p: 140. www.climatechangeinaustralia.gov.au

Gray, W. and Seed, A.W. (2000), The characterisation of orographic rainfall, Meteorological Applications, 7: 105-119.

Hoang, T.M.T., Rahman, A., Weinmann, P.E., Laurenson, E.M. and Nathan, R.J. (1999), Joint Probability Description of Design Rainfalls. 25th Hydrology and Water Resources Symposium, Brisbane, 2009, Brisbane.

Leonard, M., Lambert, M.F., Metcalfe, A.V. and Cowpertwait, P.S.P. (2008), A space-time Neyman-Scott rainfall model with defined storm extent. Water Resources Research, 44(9), 1-10, http://doi.org/10.1029/2007WR006110

Lovejoy, S. and D. Schertzer, 2006: Multifractals, cloud radiances and rain, J. Hydrology, 322: 59-88.

Marani, M., 2005: Non-power-law-scale properties of rainfall in space and time, Water Resour. Res., 41, W08413, doi:10.1029/2004WR003822.

Seed, A.W., Srikanthan, R., and Menabde, M. (2002), Stochastic space-time rainfall for designing urban drainage systems. Proc. International Conference on Urban Hydrology for the 21st Century, pp: 109-123, Kuala Lumpur.

Umakhanthan, K. and Ball, J.E. (2005), Rainfall models for catchment simulation. Australian Journal of Water Resources, 9(1), 55-67.

Wasko, C. and Sharma, A. (2015), Steeper temporal distribution of rain intensity at higher temperatures within Australian storms, Nature Geoscience, 8(7), 527-529.

Westra, S., Evans, J., Mehrotra, R., Sharma, A. (2013), A conditional disaggregation algorithm for generating fine time-scale rainfall data in a warmer climate, Journal of Hydrology, 479: 86-99

Woods and Sivapalan (1999), A synthesis of space-time variability in storm response: rainfall, runoff generation and routing. Water Resources Research, 35(8), 2469-2485.

Chapter 3. Design Rainfall

Janice Green, Fiona Johnson, Catherine Beesley, Cynthia The

Chapter Status	Final
Date last updated	14/5/2019
Minor edits	27/08/2024

3.1. Introduction

Obtaining an estimated rainfall depth for a specified probability is an essential component of the design of infrastructure including gutters, roofs, culverts, stormwater drains, flood mitigation levees, retarding basins and dams.

If sufficient rainfall records are available, at-site frequency analysis can be undertaken to estimate the rainfall depth corresponding to the specified design probability in some cases. However, limitations associated with the spatial and temporal distribution of recorded rainfall data necessitates the estimation of design rainfalls for most projects.

The purpose of this chapter is to outline the processes used to derive temporally and spatially consistent design rainfalls for Australia by the Bureau of Meteorology. The classes of design rainfall values for which estimates have been developed are described in Book 2, Chapter 3, Section 2. The practitioner is advised that this chapter uses different frequency descriptors (Table 2.3.1) used to describe events to other the rest of this Guideline (which use Figure 1.2.1).

Book 1, Chapter 6 summarises the current recommendations on how climate change should be incorporated into design rainfalls for those situations where the design life of the structure means that it could be affected by climate change.

Book 2, Chapter 3, Section 4 summarises the steps involved in deriving the frequent and infrequent design rainfalls (also known as the Intensity Frequency Duration (IFD) design rainfalls) for Australia. Book 2, Chapter 3, Section 5 and Book 2, Chapter 3, Section 6 describe how the very frequent and rare design rainfalls were estimated. The methods adopted are only briefly outlined in these sections, with additional references provided to facilitate access to further technical information for interested readers. More detail on each of the methods is provided in Bureau of Meteorology (2016).

In Book 2, Chapter 3, Section 7 a summary of the methods adopted for the estimation of Probable Maximum Precipitation is provided. Book 2, Chapter 3, Section 8 provides information on the uncertainties associated with the design rainfalls and Book 2, Chapter 3, Section 9 explains how to access estimates of each of the design rainfall classes.

3.2. Design Rainfall Concepts

Design rainfalls are a probabilistic or statistically-based estimate of the likelihood of a specific rainfall depth being recorded at a particular location within a defined duration. This is generally classified by an Annual Exceedance Probability (AEP) or Exceedances per Year (EY) (as defined in Book 1, Chapter 2, Section 2). Design rainfalls are therefore not real (or observed) rainfall events; they are values that are probabilistic in nature.

There are five broad classes of design rainfalls that are currently used for design purposes, generally categorised by frequency of occurrence. These are summarised below and presented graphically in Figure 2.3.1. However, it should be noted that there is some overlap between the classes. Different methods and data sets are required to estimate design rainfalls for the different classes and these are discussed in the following sections. The practitioner is advised that this chapter uses different frequency descriptors (Table 2.3.1) used to describe events to other the rest of this Guideline (which use Figure 1.2.1).

Table 2.3.1. Classes of Design Rainfalls

Design Rainfall Class	Frequency of Occurrence	Probability Range
Very Frequent Design Rainfalls	Very frequent	12 EY to 1 EY
Intensity Frequency Duration (IFD	Frequent	1 EY to 10% AEP
	Infrequent	10% to 1% AEP
Rare Design Rainfalls	Rare	1 in 100 AEP to 1 in 2000 AEP
Probable Maximum Precipitation (PMP)	Extreme	< 1 in 2000 AEP

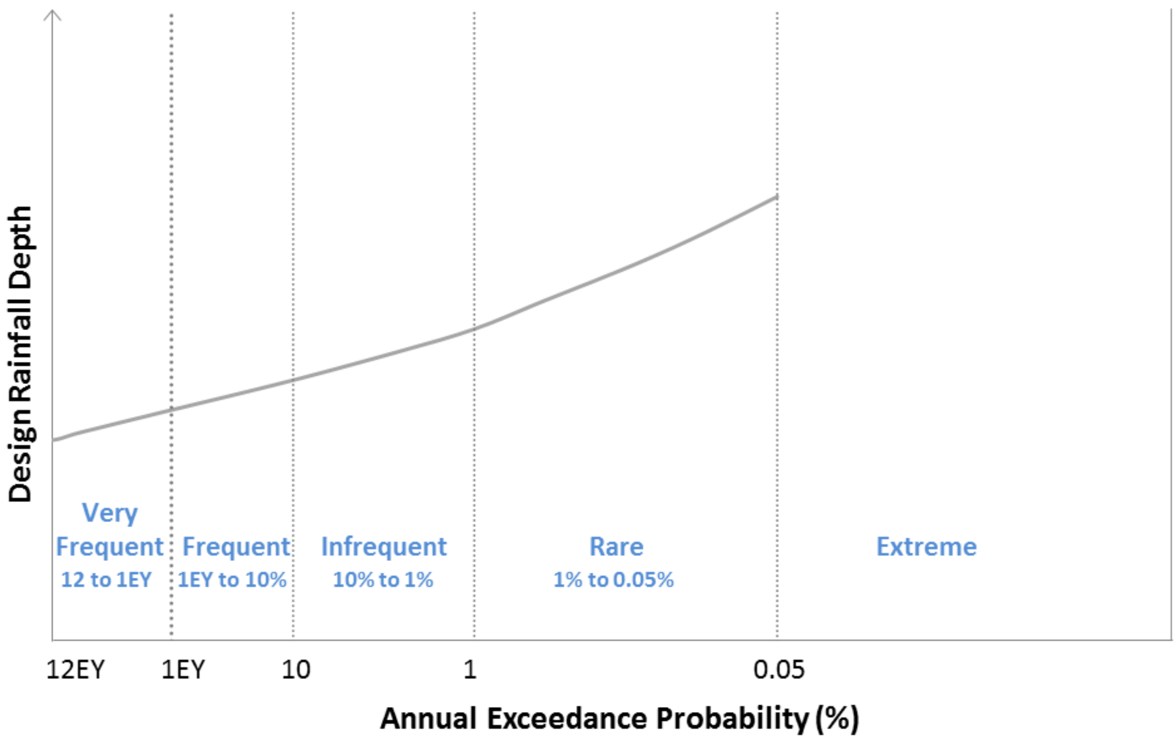


Figure 2.3.1. Classes of Design Rainfalls

3.3. Climate Change Impacts

The design rainfalls provided as part of these guidelines are based on observed rainfall data that represent, primarily, the climate of the 20th century. In order to assess the impact of future climates an adjustment must be made to the design rainfalls provided in this chapter. As part of the ARR revision projects a summary of the scientific understanding of how projected changes in the climate may alter the behaviour of factors that influence the estimation of the design floods was undertaken. Advice on how to adjust design rainfalls for climate change is detailed in Book 1, Chapter 6[1].

3.4. Frequent and Infrequent Design Rainfalls

3.4.1. Overview

This section summarises the steps involved in deriving frequent and infrequent designs rainfalls (Intensity Frequency Duration (IFDs)) for the probabilities from 1EY to 1% AEP. These classes of design rainfalls constitute the traditional IFD design rainfalls (Table 2.3.1). The main steps involved in the derivation of the frequent and infrequent design rainfalls include the collation of a quality controlled database, extraction of the extreme values series, frequency analysis, regionalisation and gridding processes. These steps are discussed in more detail in Book 2, Chapter 3, Section 4 to Book 2, Chapter 3, Section 4 and summarised in Figure 2.3.2 and in Table 2.3.2. The Sections in which each of the steps is discussed are shown in Figure 2.3.2.

[1] This section was written before the latest climate change guidance in Book 1, Chapter 6 (2024). A minor change to the text has been made to reflect the change in guidance.

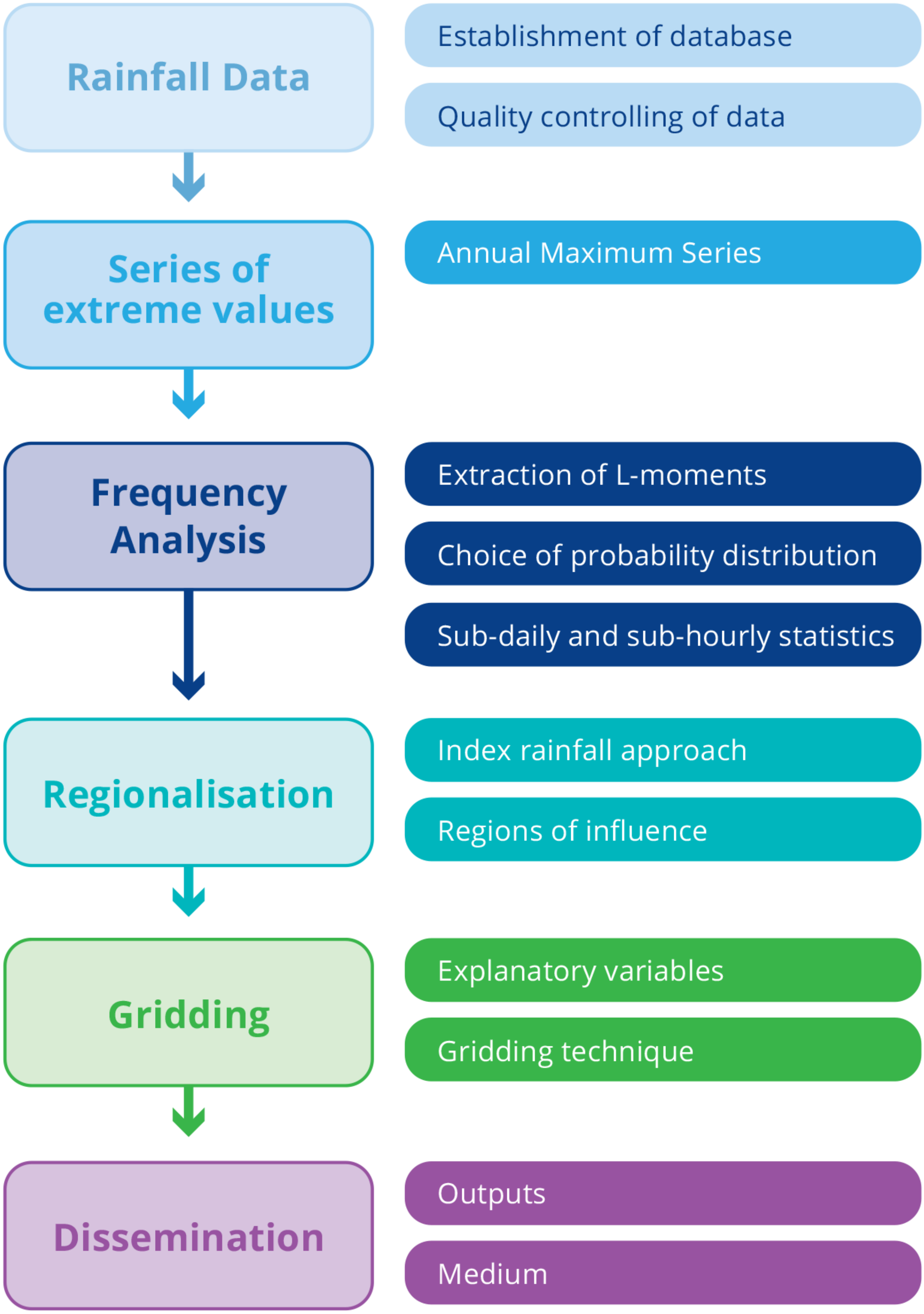


Figure 2.3.2. Frequent and Infrequent (Intensity Frequency Duration) Design Rainfall Method

Table 2.3.2. Frequent and Infrequent (Intensity Frequency Duration) Design Rainfall Method

Step	Method/Data
Number of rainfall stations	Daily read - 8074 gauges
	Continuous – 2280 gauges
Period of record	All available records up to 2012
Length of record used in analyses	Daily read ≥ 30 years
	Continuous > 8 years
Source of data	Organisations collecting rainfall data across Australia
Series of Extreme values	Annual Maximum Series (AMS)
Frequency analysis	Generalised Extreme Value (GEV) distribution fitted using L-moments
Extension of sub-daily rainfall statistics to daily read stations	Bayesian Generalised Least Squares Regression (BGLSR)
Regionalisation	Region of Influence (ROI)
Gridding	Regionalised at-site distribution parameters gridded using ANUSPLIN

3.4.2. Rainfall Database

Integral to the estimation of design rainfalls was the creation of a database containing data from all available rainfall stations across Australia. These rainfall data were collected at rainfall stations operated by various organisations using a range of types of collecting methods and instrumentation. Further information on the collection and archiving of rainfall data can be found in Book 1, Chapter 4, Section 9.

3.4.2.1. Types of Rainfall Data

Rainfall data are collected using a number of different types of instrumentation. These provide different temporal and spatial resolutions of rainfall data, depending on the instrument type and reporting method used. A brief summary of each of the main types of rainfall data are provided below. Table 2.3.3 summarises the types of rainfall reporting methods used and indicates their use in the estimation of the design rainfalls. Most of the rainfall data used to derive the design rainfalls were recorded by a daily read, pluviograph or Tipping Bucket Rain Gauge (TBRG) (Table 2.3.3).

3.4.2.1.1. Daily Read Rainfall Gauges

Daily read rainfall gauges are read at 9:00 am each day and a total rainfall depth for the previous 24 hours is reported (refer to Book 1, Chapter 4, Section 9).

3.4.2.1.2. Continuous Rainfall Gauges

Continuous rainfall stations measure rainfall depth at much finer time intervals. In Australia there have been two main types of continuous rainfall stations as discussed below.

i. Dines Tilting Syphon Pluviographs

Dines Tilting Syphon Pluviographs (DINES) record rainfall on a paper chart which is then digitised manually. Due to the limitations in the digitisation process, the minimum interval at which rainfall data could be accurately provided was 5 or 6 minutes.

ii. Tipping Bucket Raingauges (TBRG)

Since the 1990s the majority of Dines pluviographs have been replaced by Tipping Bucket Raingauges (TBRG) which typically have a 0.2 mm bucket capacity. Each time the bucket is filled the gauge tips creating an electrical impulse which is logged. Rainfall data from TBRGs can be accurately provided for intervals of less than one minute (refer to Book 1, Chapter 4, Section 9).

3.4.2.1.3. Event-Reporting Radio Telemetry Systems

The Event-Reporting Radio Telemetry Systems (ERTS) network consists of over 1000 stations across Australia, operated by the Bureau of Meteorlogy, local government and other water agencies. As the purpose of these gauges is to provide information for use in flood forecasting and warning, the location and calibration of these gauges is not necessarily in accordance with the procedures adopted for the main rainfall station networks. However, the data from the ERTS stations do provide an additional source of information on large rainfall events.

3.4.2.1.4. Radio Detection and Ranging

The Bureau of Meteorology's network of Radio Detection and Ranging (RADAR) provides near-real time estimates of rainfall accumulations which can be used in weather forecasting, flood modelling and flash flood warning. It also provides information on the spatial extent of rainfall events and can be used in combination with rainfall station measurements to improve estimates of rainfall in areas between rainfall stations.

However, because of the relatively sparse spatial distribution of the RADAR network across Australia and the short period of record, data from RADAR were not able to be used in the estimation of the design rainfalls.

3.4.2.1.5. Meta-data

Meta-data provides essential information about the rainfall station such as the location of the rainfall station, the type of instrumentation and data collection method. It therefore provides context for the rainfall data collected at a station and an indication of its quality. At a minimum, meta-data relating to location in terms of latitude and longitude were collated for each rainfall station. However, any additional meta-data that were available including elevation, details on siting and clearance and photographs were also collated.

Table 2.3.3. Rainfall Reporting Methods

Type	Reporting	Recording Resolution	Reporting Interval	Reporting Method	Used for Design Rainfalls
Daily	Daily	24 hour totals (9:00 am – 9:00 am)	Daily to monthly	Paper	Yes
DINES Pluviograph	Continuous	5-6 minutes	Daily to weekly	Digitised	Yes

Type	Reporting	Recording Resolution	Reporting Interval	Reporting Method	Used for Design Rainfalls
TBRG	Continuous	On occurrence	Hourly to six monthly	Logger	Yes
ERTS	Event	On occurrence	On occurrence	Electronic	Some
RADAR	Spatial	10 minutes	10 minutes	Digital	No

3.4.2.2. Sources of Data

The rainfall database used for the estimation of the design rainfalls included rainfall data collected at rainfall stations operated by organisations around Australia. There were two main sources of data; the Australian Data Archive for Meteorology and Australian Water Resources Information System (more detail on these data sources can be found in Book 1, Chapter 4, Section 9).

Rainfall data collected by the Bureau of Meteorology are stored in the Australian data Archive for Meteorology (ADAM) which contains approximately 20 000 daily read rainfall stations (both open and closed) starting in 1800; and nearly 1500 continuous rainfall stations – using both DINES and TBRG instrumentation.

Under the terms of the Water Regulations 2008, water information (including rainfall data) collected by organisations across Australia are required to be provided to the Bureau of Meteorology. The rainfall data collected by organisations including local and state government water agencies, hydropower generators and urban water utilities are stored in the Australian Water Resources Information System (AWRIS) together with other water information. At present, AWRIS contains:

- approximately 350 daily read rainfall stations; and
- approximately 2500 continuous rainfall stations.

Of particular importance to design rainfall estimation are the dense networks of continuous rainfall stations operated by urban water utilities which provide data in areas of steep rainfall gradients and urban areas.

3.4.2.3. Spatial Distribution of Rainfall Data

3.4.2.3.1. Daily Read Rainfall Stations

The location and period of record of the daily read rainfall stations operated by the Bureau of Meteorology are shown in Figure 2.3.3.

Figure 2.3.3 depicts the spatial coverage of the daily read rainfall stations across Australia is reasonably good, especially over the eastern states and around the coast. Gaps in the spatial coverage of the daily read rainfall station network occur in the eastern half of Western Australia; the western and north eastern parts of South Australia; and the parts of the Northern Territory that are removed from the road and rail networks.

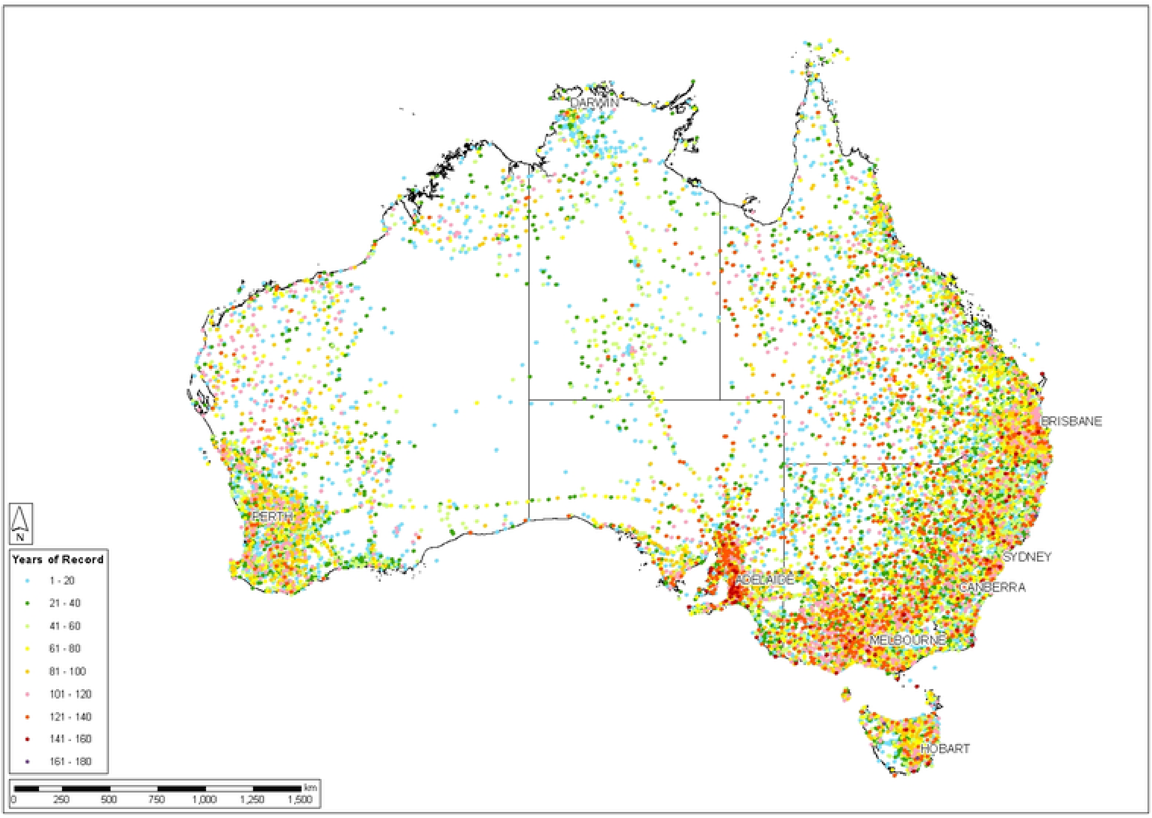


Figure 2.3.3. Daily Read Rainfall Stations and Period of Record

3.4.2.3.2. Continuous Rainfall Stations

The location and period of record of the continuous rainfall stations operated by the Bureau of Meteorology and other organisations are shown in Figure 2.3.4. The sparseness of the network of continuous rainfall stations across Australia can be seen from Figure 2.3.4, especially when compared to the spatial distribution of the daily read rainfall stations. In spite of the significant improvement in the spatial coverage of the continuous rainfall stations by the inclusion of rainfall stations operated by other organsiations, there are still large areas of Australia with either no or very few continuous rainfall stations.

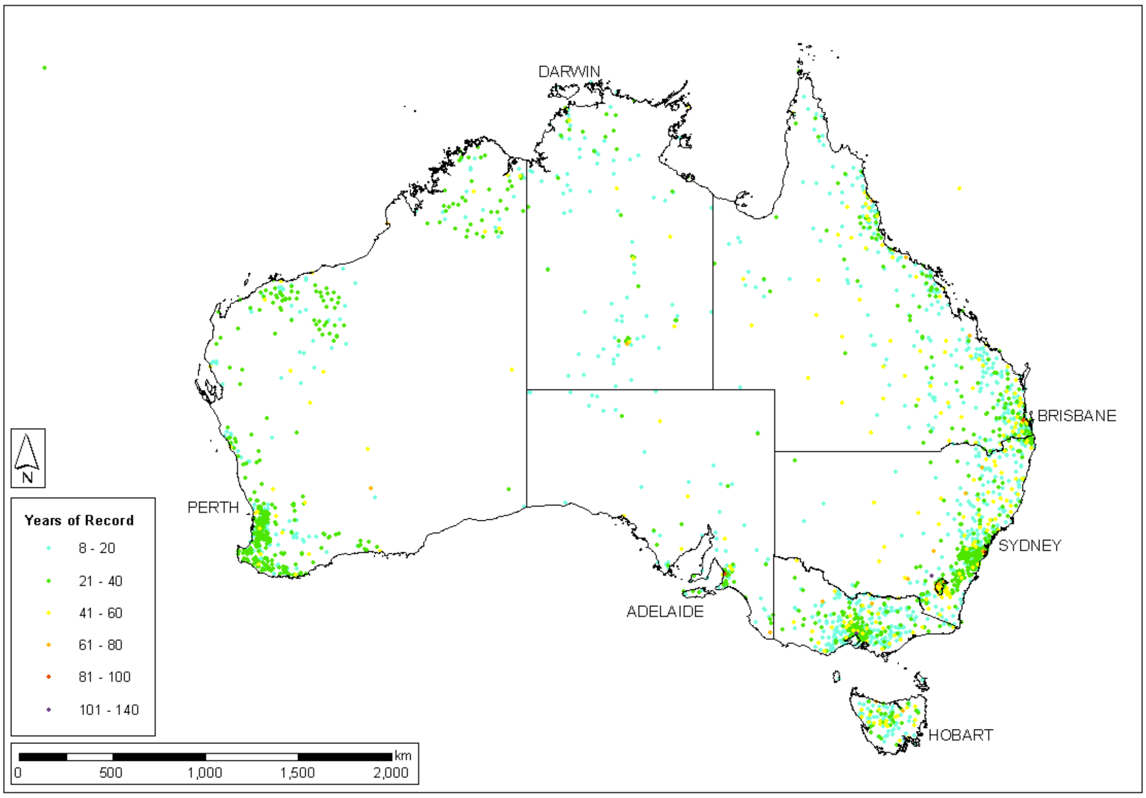


Figure 2.3.4. Continuous Rainfall Stations and Period of Record

3.4.2.3.3. Increase in Spatial Coverage

The increase in the spatial coverage of the daily read and continuous rainfall stations used for the design rainfalls in this edition of ARR compared to the spatial coverage available for the IFDs provided in ARR 1987 (Pilgrim, 1987) are shown in Figure 2.3.5 and Figure 2.3.6.

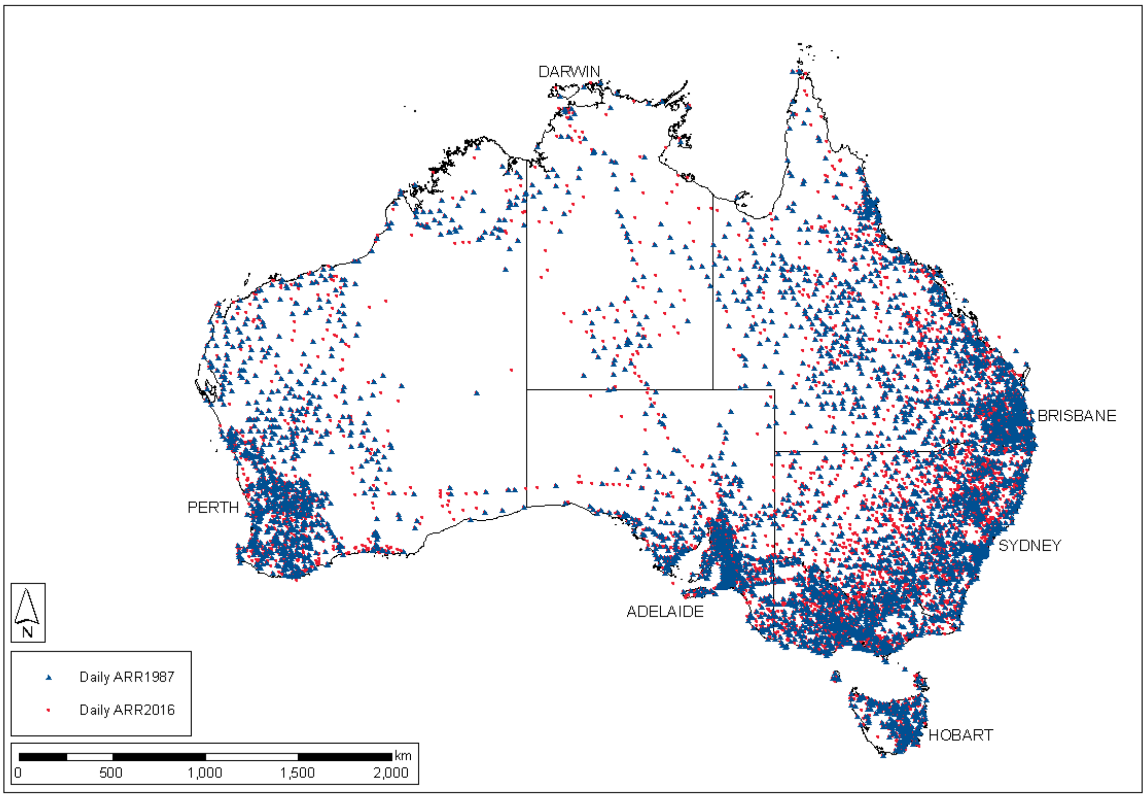


Figure 2.3.5. Daily Read Rainfall Stations Used for ARR 1987 and ARR 2016 Intensity Frequency Duration Data

The increase in daily read rainfall stations is due to the increased number of stations that met the minimum period of record criterion.

Figure 2.3.6 shows the inclusion of data from continuous rainfall stations operated by other organisations has resulted in a significant increase in the spatial coverage of these data. In particular, the spatial coverage along the east coast of Australia; the west coast of Tasmania; and large areas in Western Australia has been improved.

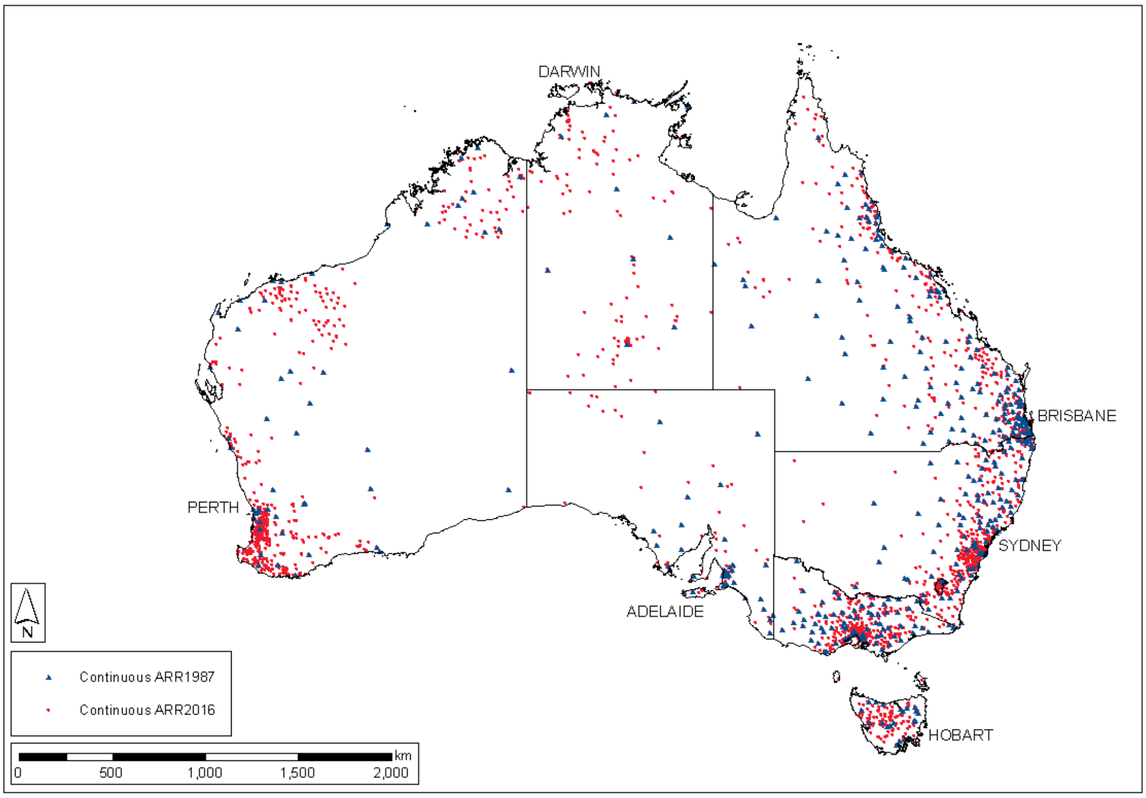


Figure 2.3.6. Continuous Rainfall Stations Used for ARR 1987 and ARR 2016 Intensity Frequency Duration Data

3.4.2.4. Temporal Distribution of Rainfall Data

3.4.2.4.1. Daily Read Rainfall Stations

Official daily read rainfall data are available from the early 1800s with the longest rainfall records in Australia being approximately 170 years. Some of these early rainfall stations are still open. The Bureau of Meteorology's ADAM database contains approximately 3000 daily read rainfall stations with more than 100 years of record.

In Figure 2.3.7 the distribution of record lengths for daily read rainfall stations is shown. It can be seen that, although there are a reasonable number of long term stations, approximately half of the daily read rainfall stations have less than 10 years of record.

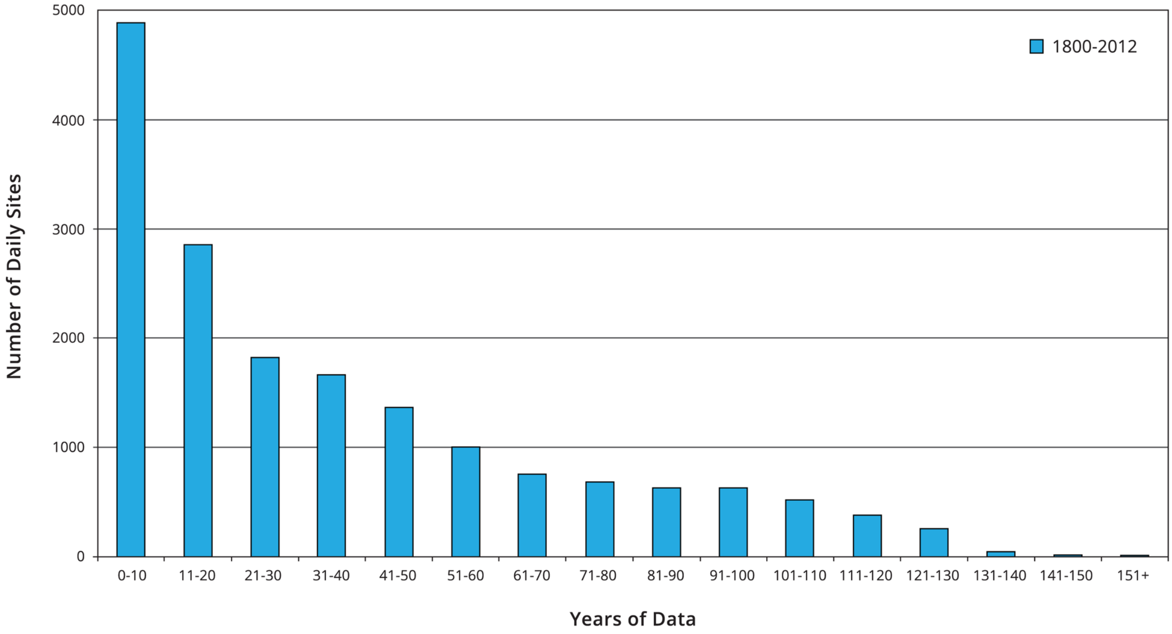


Figure 2.3.7. Length of Available Daily Read Rainfall Data

3.4.2.4.2. Continuous Rainfall Stations

While there are a small number of continuous rainfall stations with more than 70 years of record, the majority of stations have less than 40 years of record and a high proportion have less than 10 years of record. Figure 2.3.8 shows the distribution of available length of record for the continuous stations.

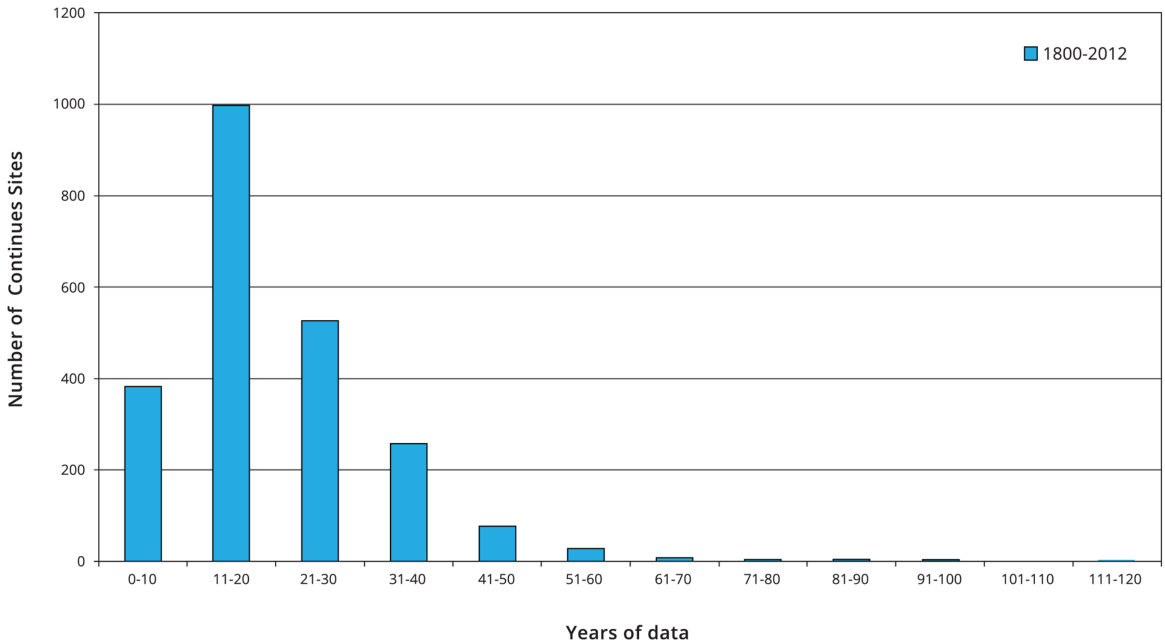


Figure 2.3.8. Length of Available Continuous Rainfall Data

3.4.2.4.3. Increase in Length of Available Record Compared to ARR 1987

The inclusion of nearly 30 years additional daily read rainfall data since the estimation of the ARR 1987 IFDs has increased both the amount of data available for the frequent and infrequent design rainfalls (IFDs) as well as the number of daily read rainfall stations which now met the minimum length of record criterion as shown in Figure 2.3.9.

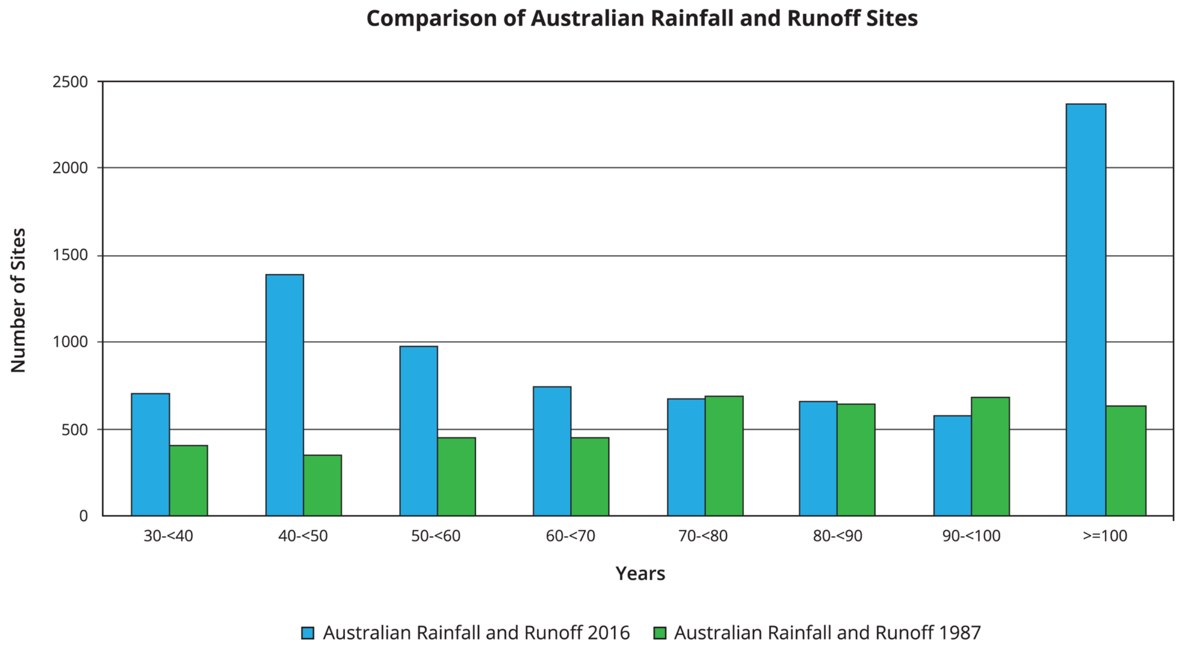


Figure 2.3.9. Number of Long-term Daily Read Stations Used for ARR 1987 and ARR 2016 Intensity Frequency Duration Data

For the continuous rainfall data, the inclusion of stations operated by other organisations and the nearly 30 years of additional data resulted in a significant increase in both the length of record available and the number of rainfall stations that met the minimum record length criterion (Figure 2.3.10).

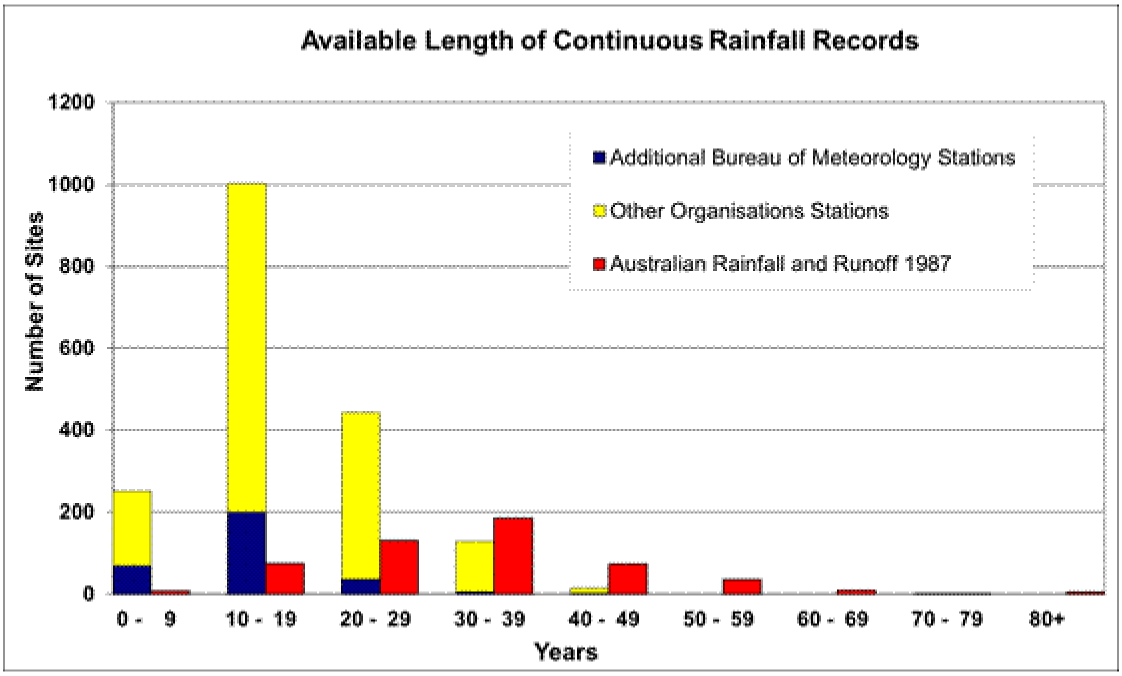


Figure 2.3.10. Length of Record of Continuous Rainfall Stations Used for ARR 1987 and ARR 2016 Intensity Frequency Duration Data

3.4.2.5. Quality Controlling Data

In addition to ensuring that as much data as possible was used in estimating the design rainfalls, it was also necessary that the rainfall data be quality controlled. In light of the volume of data that needed to be quality controlled, automated procedures were developed for the identification of suspect data and, as far as possible, the correction of these data. However, the quality controlling of the data could only be automated so far and a significant amount of data was required to be manually checked.

The quality controlling undertaken of both the daily read and continuous rainfall data is summarised below (refer to Green et al. (2011) for more information). The quality controlled database prepared for the estimation of the design rainfalls will be archived in AWRIS and made available from the Bureau of Meteorology's website via the Water Data Online product.

3.4.2.5.1. Daily Read Rainfall Stations

For the daily read rainfall data automated quality controlling procedures were developed in order to:

- infill missing data;
- disaggregate flagged accumulated daily rainfall totals;
- detect, identify and correct suspect data;
    - unflagged accumulated totals; and
    - time shifts.
- identify gross errors - data inconsistent with neighbouring records but not captured by either of the above two categories.

Manual correction of gross errors identified during the automated quality controlling procedures was facilitated through the use of the Bureau of Meteorology's Quality Monitoring System. The Bureau of Meteorology's Quality Monitoring System is a suite of programs that has functionalities to map the suspect value in relation to nearby stations and to link to Geographic Information System (GIS) data from other systems including RADAR, Satellite Imagery and Mean Sea Level Pressure Analysis.

3.4.2.5.2. Continuous Rainfall Stations

The automated quality control procedures for continuous rainfall data used comparisons with other data sources including the Australian Water Availability Project (AWAP) gridded data, daily read rainfall stations, automatic weather stations, and synoptic stations to identify spurious and missing data.

In order to reduce the amount of continuous rainfall data that needed to be quality controlled to a manageable volume, only a subset of the largest rainfall events was quality controlled. The subset was created by extracting the number of highest rainfall records equal to three times the number of years of record at each site for each duration being considered.

Each continuous rainfall value in the data subset that was flagged as being spurious by the automated quality controlling procedures was subjected to manual quality controlling. The manual quality controlling of the data was undertaken in order to determine whether the flagged value was correct or not. The manual quality controlling procedure adopted involved comparing 9:00 am to 9:00 am continuous rainfalls with daily (also 9:00 am to 9:00 am) rainfalls at the co-located daily read rainfall station. For continuous rainfall sites with no co-located daily site, the continuous rainfall record was compared with the daily rainfall record of the nearest site. The continuous rainfall value was not modified in any way - the comparison with daily values was made in order to assess whether it was valid or not. Where it was assessed that the flagged value was definitely incorrect it was excluded from the analyses, otherwise values were retained in the continuous rainfall database.

3.4.2.5.3. Meta-data

The meta-data associated with each of the rainfall stations were also checked. For the Bureau of Meteorology operated rainfall stations, the Bureau of Meteorology's meta-data database, SitesDB, includes details of the station's location in latitude and longitude, and elevation. For rainfall stations operated by other organisations, meta-data were provided with the rainfall data and stored in AWRIS. Gross error checks on station locations and elevation were performed by comparing elevations derived using a Digital Elevation Model (DEM) to those recorded in the station's meta-data. Checks of latitude and location were also carried out by plotting the latitudes and longitudes in GIS. Revisions to station locations or elevations were carried out using Google Earth and information on the station provided in the Bureau of Meteorology's station meta-data catalogue.

For the limited number of closed stations for which an elevation was not included in the meta-data, the station elevation was extracted from the Geoscience Australia 9 second DEM[2] based on the latitude and longitude.

3.4.2.6. Stationarity Assessment

The quality controlled database that was established contained rainfall data for the period extending from the 1800s to the present. However, if climate change has caused non-

[2] http://www.ga.gov.au/metadata-gateway/metadata/record/gcat_66006

stationarity in the recorded rainfalls, then possibly only a portion of the observed record should have been used in deriving the design rainfalls. This is because a key assumption in the statistical methods adopted for the derivation of the design rainfalls is that the data are stationary. In order to determine whether the complete period of available rainfall records could be adopted in estimating the design rainfalls, it was necessary to assess the degree of non-stationarity present in the historic record at rainfall stations across Australia (Green and Johnson, 2011).

Two methods were used to establish if there are trends in the Annual Maximum Series of rainfalls for Australia. The first examined the records at individual stations which were tested to assess trends in the time series of the annual maximum rainfalls and changes in the probability distributions fitted to the annual maxima to estimate design rainfall quantiles. The second method used an area averaged approach to check for regional trends in the number of exceedances of pre-determined thresholds. The approach was based on that carried out by Bonnin et al. (2010) to assess trends in large rainfall events in the USA as part of the revisions by the National Oceanic and Atmospheric Administration to design rainfalls.

It was concluded that although some stations showed strong trends in the annual maximum time series, particularly for short durations and more frequent events, the magnitude of these changes was within the expected accuracy of the fitted design rainfall relationships. It was therefore considered appropriate to assume stationarity and use the complete period of record at all stations in the estimation of the design rainfalls.

3.4.3. Extraction of Extreme Value Series

Rainfall frequency analysis was an integral part of the estimation of the design rainfalls as it enabled rainfall depths corresponding to a probability quantile to be ascertained. In estimating the frequent and infrequent design rainfalls it is large rainfalls that were being considered and therefore it was the extreme value series that was of interest.

The extreme value series can be defined using the Annual Maximum Series (AMS) or the Partial Duration Series (PDS) (also known as Peak over Threshold) (more information can be found in Book 3, Chapter 2). For the frequent and infrequent design rainfalls, the AMS was used to define the extreme value series because of its lack of ambiguity in defining the series; its relatively simple application and the problem of bias associated with the PDS for less frequent AEPs.

It should be noted that in extracting the AMS, the focus was on obtaining the largest rainfall depth in each year for each of the durations considered. Therefore the extracted depths comprised both total storm depths and bursts within storms.

In order to reduce the uncertainty in the design rainfall estimates, minimum station record lengths were adopted. The criteria were:

- 30 or more years of record for daily read rainfall stations; and
- More than 8 years of record for continuous rainfall stations.

These criteria were selected on the basis of optimising the spatial coverage of the rainfall stations while ensuring that there were sufficient AMS values at each site to undertake frequency analysis.

The daily read rainfall data are for the restricted period from 9:00 am to 9:00 am rather than for the actual duration of the event. As this may not lead to the largest rainfall total, it was necessary to convert these 'restricted' daily read rainfall depths to unrestricted rainfall depths. In order to do this, 'restricted' to unrestricted conversions factors were estimated using co-located daily read and continuous rainfall gauges at a number of locations around Australia of differing climatic conditions. The resultant factors are shown in Table 2.3.4.

Table 2.3.4. Restricted to Unrestricted Conversion Factors

Duration	1 Day	2 Days	3 Days	4 Days	5 Days	6 Days	7 Days
Factor	1.15	1.11	1.07	1.05	1.04	1.03	1.02

3.4.3.1. At-Site Frequency Analysis

In order to assess the most appropriate distribution to adopt across Australia for the AMS, a range of distributions was trialled using single site analysis. Five distributions – Generalised Extreme Value (GEV), Generalised Logistic (GLO), Generalised Normal (GNO), Log Pearson III (LP III) and Generalised Pareto (GPA) – were fitted to the AMS extracted from the available long-term continuous rainfall stations for a range of durations. The goodness of fit of each distribution was assessed using the approach recommended by Hosking and Wallis (1997). It was found that the Generalised Extreme Value (GEV) distribution produced the best fit to the AMS on an at-site analysis. The comparison of distributions was subsequently repeated for regional estimates with the same results (Green et al., 2012b).

3.4.3.2. Estimation of L-moments

The linear combinations of the data (L-moments) (Hosking and Wallis, 1997) of mean, variation (L-CV) and skewness (L-Skewness) were used to summarise the statistical properties of the extreme value series data at each station location. L-moments are commonly used in rainfall and Flood Frequency Analysis (Hosking and Wallis, 1997) due to their efficiency in fitting the data and lack of bias in the sample estimates, particularly in the higher order moments, when compared to ordinary moments.

While for durations of one day and longer this was a fairly straightforward approach, for sub-daily durations the scarcity of long-term continuous rainfall records meant that an alternative approach was needed to supplement the available data. For the IFD revision project, a Bayesian Generalised Least Squares Regression (BGLSR) approach was adopted, a summary of which is provided in Book 2, Chapter 3, Section 4; more details can be found in Johnson et al. (2012a) and Haddad et al. (2015).

3.4.3.2.1. Daily Durations

For daily read rainfall stations with 30 or more years of record, the mean, L-CV and L-Skewness, were determined from the at-site extreme value series for each duration.

3.4.3.2.2. Sub-Daily Durations – at Continuous Rainfall Stations

For continuous rainfall stations with more than eight years of record, the mean, L-CV and L-Skewness, were determined from the at-site extreme value series for each duration.

The spatial coverage of sub-daily rainfall stations is considerably less than that of the daily read stations (refer to Figure 2.3.4 and Figure 2.3.5). Therefore, a method was needed to improve the spatial coverage of the sub-daily data. This is most commonly done using information from the daily read stations with statistics of sub-daily data being inferred from those of the daily data. Previously, adopted techniques for predicting rainfall depths at durations below 24 hours from those for the 24, 48 and 72 hour durations have been factoring of the 24 hour IFDs; principal component analysis followed by regression; and Partial Least Squares Regression. However, a major weakness of these previously adopted approaches is their inability to account for variation in record lengths from site to site and inter-station correlation.

The approach adopted for the frequent and infrequent design rainfalls was Bayesian Generalised Least Squares Regression (BGLSR) as it accounts for possible cross-validation and unequal variance between stations by constructing an error co-variance matrix and can explicitly account for sampling uncertainty and inter-site dependence. Details of the BGLSR approach can be found in Reis et al. (2005), Madsen et al. (2002) and Madsen et al. (2009). In Australia, Haddad and Rahman (2012a) used BGLSR to obtain regional relationships to estimate peak streamflow in ungauged catchments and for pilot studies for the design rainfall project (Haddad et al., 2009; Haddad et al., 2011; Haddad et al., 2015).

3.4.3.2.3. Bayesian Generalised Least Squares Regression – Overview

BGLSR is an extension of Ordinary Least Squares (OLS) regression such that the predictor and (dependent variable) is calculated from a linear combination of a number of predictor variables (independent variables) with a suitable error model. In general the predictions for the rainfall statistic, *y*, of interest for site *i*, are made according to Equation (2.3.1).

$$y_i = \beta_0 + \sum_{\mathrm{j-1}}^{k} \beta_j X_{\mathrm{ij}} + \varepsilon_i + \delta_i \tag{2.3.1}$$

Where X_{ij} (j = 1,…,k) are the k predictor variables, β_j are the parameters of the model that must be estimated, ε is the sampling error and δ is the model error.

A further advantage of the BGLSR is that the Bayesian formulation allows for the separation of sampling and statistical modelling errors. This is important because it was found that the sampling errors dominate the total error in the statistical model. The BGLSR produces estimates of the standard error in:

- the regression coefficients;
- the predicted values at-site used in establishing the regression equations; and
- the predicted values at new sites (that is, sites not used in deriving the regression). In the application of the BGLSR these are the daily rainfall stations where the predictions of sub-daily rainfalls statistics are required.

The error variances for the predictions are comprised of the regional model error and the sampling variance.

The errors in the BGLSR model are assumed to have zero mean and the co-variance structure described in Equation (2.3.2).

$$Cov\left\{\varepsilon_i, \varepsilon_j\right\} = \begin{cases} \sigma^2_{\varepsilon_i}, & i = j \\ \sigma_{\varepsilon_i}\sigma_{\varepsilon_j}\rho_{\varepsilon_{ij}}, & i \neq j \end{cases}; Cov\left\{\delta_i, \delta_j\right\} = \begin{cases} \sigma^2_{\delta}, & i = j \\ 0, & i \neq j \end{cases} \tag{2.3.2}$$

Where $\sigma^2_{\varepsilon_i}$ is the sampling error variance at site *i*, $\rho_{\varepsilon_{ij}}$ is the correlation coefficient between sites *i* and *j*, $\sigma^2_{\delta i}$ is the model error variance. For the Bayesian framework introduced by Reis et al. (2005), the parameters of the model (β) are modelled with a multivariate normal distribution using a non informative prior. A quasi analytic approximation to the Bayesian formulation of the GLSR has been developed by Reis et al. (2005) to solve for the posterior distributions of the mean and variance for β.

3.4.3.2.4. Application of Bayesian Generalised Least Squares Regression

The aim of the BGLSR is to predict sub-daily rainfall statistics at the location of daily rainfall stations. As discussed previously, L-moments have been used to summarise the statistical properties of the AMS data (Hosking and Wallis, 1997) because L-moments are relatively robust against outliers in the datasets. The statistics that are required for the project are:

- Mean of the AMS (also called the index rainfall);
- L-coefficient of variation (L-CV); and
- L-Skewness.

These three statistics can then be used to define the parameters of any appropriate probability distribution which in the case of the design rainfalls had been shown to be the GEV distribution.

The initial work required to apply the BGLSR was to determine the appropriate predictors (i.e. X from Equation (2.3.1)) to estimate the three rainfall statistics listed above. A review of literature and meteorological causative mechanisms selected a number of site and rainfall characteristics for use as possible predictors as reported in Johnson et al. (2012a). These predictors were:

- Latitude and longitude;
- Elevation;
- Slope;
- Aspect;
- Distance from the coast;
- Mean annual rainfall; and
- Rainfall statistics (mean, L-CV and L-Skewness) for the 24, 48 and 72 hour duration events.

Haddad and Rahman (2012b) provide extensive details of the cross-validated predictor selection process for each of the study areas. It was found that the most important predictor is the 24 hour rainfall statistic. However performance of the model was not changed significantly by including all predictors so this approach was adopted.

As well as determining the optimum combination of predictor variables, the testing for the BGLSR needed to determine the number of stations to contribute to each regression equation. Ideally, the number of stations in each analysis area would be maximised to improve the accuracy of the regression equations. However the number of stations is limited to approximately 100 by the requirement for the error co-variance matrices to be invertible. The delineation of the analysis areas thus needed to balance these two competing requirements.

It was also important that stations were grouped into analysis areas where the causative mechanisms for large rainfall events are similar. The rainfall stations were grouped primarily according to climatic zones by considering the seasonality of rainfall events and mean annual rainfalls. Australian drainage divisions were also used to guide the division of larger climatic zones into smaller areas over which the BGLSR calculations are tractable, such as in the northern tropics where three analysis areas have been adopted (NT, GULF and NORTH_QLD). The final analysis areas are shown in Figure 2.3.11. The South East Coast and South Western WA areas are considered Regions of Interest. A 0.2 degree buffer has been used in assigning stations to each analysis area to provide a smooth transition between adjacent areas.

For each analysis area, a regression relationship was developed which could be applied to all stations within the analysis area. Where the density of stations was high, a Region Of Influence (ROI) approach (Burn, 1990) was adopted such that each station has its own ROI. This allowed the regression equations to smoothly vary across the data dense analysis areas. For sparser analysis areas, a clustering, or fixed region, approach was adopted such that stations were grouped by spatial proximity into analysis areas with rigid boundaries. All stations in each analysis area were used to derive one regression equation that was then adopted for the predictions at those stations.

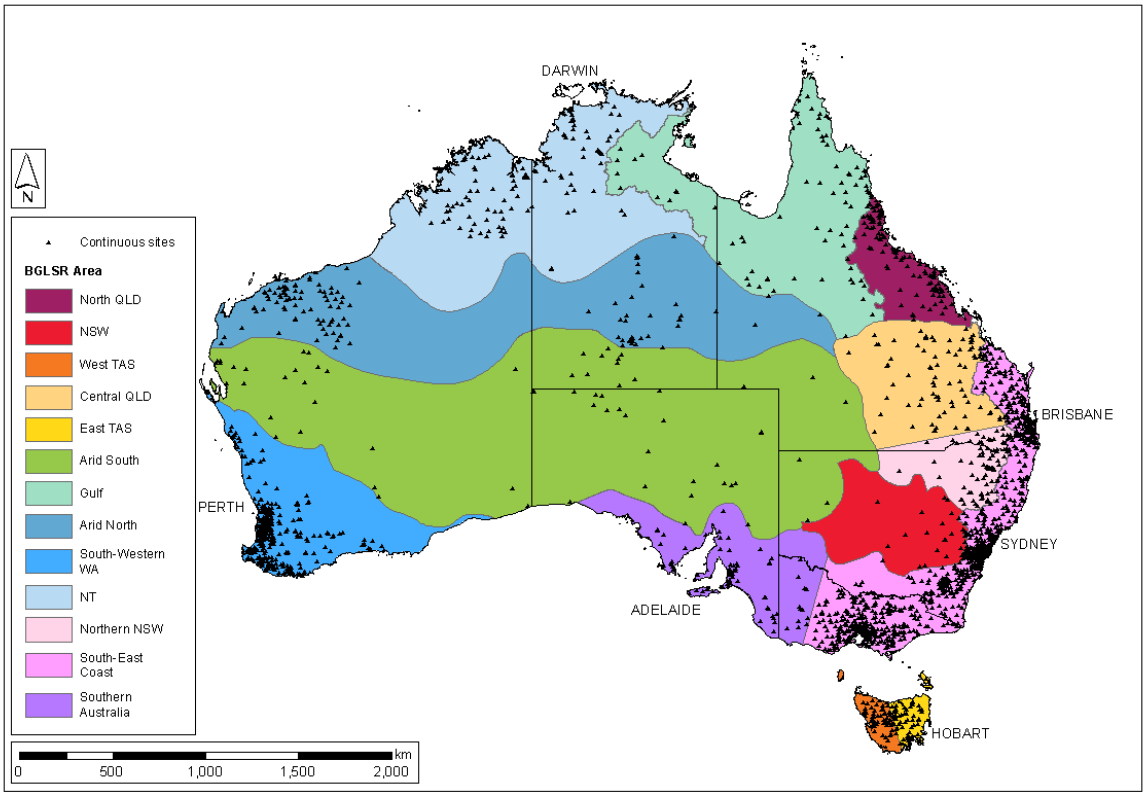


Figure 2.3.11. Analysis Areas Adopted for the BGLSR

To improve the predictions from the BGLSR it was desirable that the distribution of each predictor variable was relatively symmetric and preferably approximately normally distributed. For each analysis area the distribution of the predictor variables from all sites in the area were examined using histograms and quantile-quantile plots. For predictors that appeared to be strongly skewed, a range of transformations were trialled to attempt to reduce the skewness of the variable. The transformations included a natural logarithm, square root transformation and Box-Cox (i.e. power) transformation. In general the log transformation and the Box-Cox transformation were successful in reducing the skewness of the predictors.

After determining the regression coefficients for the analysis areas, these coefficients were combined with the set of predictors for the daily station locations to produce the estimates of the sub-daily rainfall statistics. There are no observations of the sub-daily rainfall statistics to which these predictions at daily sites can be compared. However “sanity” checks on the values were carried out by comparing the estimates to the 24 hour rainfall statistics and to the possible range of values for L-CV and L-Skewness (both limited to -1 to 1).

The result of using estimated sub-daily rainfall statistics was that the number of locations with sub-daily information was increased from approximately 2300 to approximately 9700 when both the daily and continuous rainfall stations locations are used. This substantially increased density of sub-daily rainfall data assisted in the subsequent gridding of the rainfall quantiles across Australia described in Book 2, Chapter 3, Section 4.

3.4.4. Regionalisation

Regionalisation recognises that for stations with short records, there is considerable uncertainty when estimating the parameters of probability distributions and short records can bias estimates of rainfall statistics. To overcome this, it is assumed that information can be combined from multiple stations to give more accurate estimates of the parameters of the extreme value probability distributions. One approach that is widely used to reduce the uncertainty and overcome bias in estimating rainfall quantiles is regional frequency analysis, also known as regionalisation.

For the design rainfalls, regionalisation was used to estimate the L-CV and L-Skewness with more confidence. The regionalisation approach adopted is generally called the “index flood procedure” (Hosking and Wallis, 1997). This approach assumes that sites can be grouped into homogenous regions, such that all sites in the region have the same probability distribution, other than a scaling factor. The scaling factor is termed the index flood or in this case, since the regionalisation is of rainfall data, the “index rainfall”. The index rainfall is the mean (that is, first L-moment) of the extreme value series data at the station location.

The homogenous regions for the frequency analysis can be defined in a number of ways. Cluster and partitioning methods divide the set of all stations into a fixed number of homogenous groups (Hosking and Wallis, 1997) where generally every site is assigned to one group. Alternatively, a ROI approach (Burn, 1990) can be adopted, such that for each station an individual homogenous region is defined. Each ROI will contain a potentially unique set of sites, with each site possibly contributing to multiple ROIs.

For the design rainfalls, the station point estimates were regionalised using a ROI as the advantage of this approach is that the region sizes can be easily varied according to station density and the available record lengths. The assumptions of the approach are, firstly, that the specified probability distribution (GEV in the case of the AMS) is appropriate; that the region is truly homogenous; and, finally, that sites are independent or that their dependence is quantified.

In the application of the ROI method, it was first necessary to establish how big the ROIs should be. The size of the ROI can be defined in two ways; either using the number of stations included in the region or alternatively by calculating the total number of station-years in the region as the sum of the record lengths of the individual stations included in the ROI.

Region sizes from 1 to 50 stations and from 50 to 5000 station-years were investigated to establish the optimum ROIs for estimating rainfall quantiles across Australia using a simple circular ROI and the Pooled Uncertainty Measure (PUM) (Kjeldsen and Jones, 2009). The minimum PUM values occur where there is an optimum size in the trade-off between bias and variance of generally lead to the minimum PUM value. When considering the region defined using the number of stations it was found that a region of 8 stations performed best. Given that the average record length for stations used in the analysis was 66 years, a region of 8 stations will have on average 528 years of data which is consistent with the region size using the station-year criteria. The findings were generally independent of rainfall event duration and frequency.

Defining regions in terms of station-years is attractive as this approach can adapt to different station densities and station record lengths. Given the similar results from both methods, the station-years definition for the region size was adopted.

After finalising the optimum region size, a number of geographic and non-geographic similarity measures were investigated as methods to define membership of each ROI. Three different alternatives for defining the ROIs using geographical similarity were investigated:

- Distance between sites (in kilometres) defined using latitude and longitude;
- Euclidean distance between sites where distance was defined using latitude, longitude and scaled elevation (Hutchinson, 1998); and
- Nearest neighbours defined using distance in kilometres inside an elliptical ROI.

Non-geographic characteristics were selected based on their potential influence on the properties of large rainfall events at a site (Johnson et al., 2012a). The site characteristics that were trialled were:

- Location (latitude and longitude);
- Elevation;
- Mean Annual Rainfall;
- Aspect;
- Slope;
- Distance from the coast;
- Mean date of AMS (seasonality); and
- Variability of AMS occurrence (seasonality).

The results of the trialling showed that the best results were provided using:

- a circular ROI; and
- distances defined in three dimensions using latitude; longitude and elevation.

3.4.4.1. Regionalisation - Application

To undertake the regionalisation the following procedure was followed initially using the 24 hour rainfall data:

- For each station location, a circular ROI was expanded until 500 stations years of record was achieved. The resultant region was tested for homogeneity using the H measure of (Hosking and Wallis, 1997);
- If the region was not homogenous the stations in the regions were checked according to the discordancy measures of Hosking and Wallis (1997) and the region membership revised where appropriate;
- The average L-CV for each region was calculated using a weighted average of the L-CV at all stations in the region, with the weights proportional to the station lengths. This was repeated for the L-Skewness; and
- The regionalised L-CV and L-Skewness were used to estimate the scale (α) and shape (κ) parameters of the growth curve (scaled GEV distribution) at each location.

The regions defined for the 24 hour duration rainfall data were used for all daily and sub-daily durations. More details on the regionalisation can be found in Johnson et al. (2012b).

3.4.5. Gridding

The regionalisation process resulted in estimates of the GEV parameters at all station locations, which were combined with the mean of the extreme value series at that site to estimate rainfall quantiles for any required exceedance probability. However frequent and infrequent design rainfall estimates are required across Australia, not just at station locations and therefore the results of the analyses needed to be extended in some way to ungauged locations.

3.4.5.1. Selection of Approach to be Adopted for Gridding

For the design rainfalls, the software package ANUSPLIN (Hutchinson, 2007) was chosen to grid the GEV parameters so that frequent and infrequent design rainfall estimates are available for any point in Australia. ANUSPLIN applies thin plate smoothing splines to interpolate and smooth multi-variate data. The degree of smoothing of the fitted functions was determined through generalised cross-validation. The splines are fitted using three independent variables; latitude, longitude and elevation. The elevation scale was exaggerated by a factor of 100 to represent the importance that elevation has on precipitation patterns (Hutchinson, 1998).

3.4.5.2. Selection of Parameters to be Gridded

The GEV parameters were gridded in ANUSPLIN rather than the rainfall depths, as testing showed little difference in the resulting quantile estimates irrespective of whether point GEV parameters or point rainfall depths were gridded. Gridding the GEV parameters provided more flexibility in the choice of exceedance probabilities that could be extracted and enables the provision of a greater number of AEPs.

3.4.5.3. Optimisation of Gridding

In undertaking the gridding, a considerable number of iterations was required to achieve an optimum outcome that represented the observed rainfalls but which did not place too much significance on short rainfall records or from poorly located rainfall stations. The appropriate degree of smoothing of the fitted functions was determined through generalised cross-validation with the number of knots and transformation adopted varied to achieve optimal results. In addition to the statistical tests to determine the appropriate degree of smoothing, qualitative assessments were also conducted by preparing maps which compared the index rainfall derived from at-site frequency analysis of rainfall records, the length of record available at each station, and the spatial density of the rainfall gauge network to the gridded index rainfalls produced by ANUSPLIN for daily durations.

The final IFD grids were produced by the application of ANUSPLIN using a 0.025 degree DEM resolution and adopting 3570 knots with no transformation of the data. More details on the gridding approach adopted can be found in The et al. (2012), The et al. (2014) and Johnson et al. (2015).

3.4.5.4. Calculation of Growth Factors and Rainfall Depths

The outputs of the ANUSPLIN analysis were grids across Australia of index rainfall and the GEV scale (α) and shape (κ) parameters for each duration. These were then processed to firstly estimate the growth factors for each grid location and then the rainfall depths for each exceedance probability, according to the following equations:

$$\xi = 1 - \alpha\{1 - \Gamma(1 + \kappa)\}/\kappa \tag{2.3.3}$$

where ξ is the location parameter for the regionalised growth curve and Γ represents the Gamma function.

$$q(F) = \xi + \alpha\{1 - (-\log(F))^{\kappa}\}/\kappa \tag{2.3.4}$$

where $q(F)$ is the quantile function of the growth curve for the cumulative probability F.

$$Q(F) = \mu q(F) \tag{2.3.5}$$

where $Q(F)$ is the quantile function of the scaled growth curve, which is multiplied by the index rainfall μ.

3.4.5.5. Derivation of Sub-Hourly Rainfall Depths

To derive frequent and infrequent design rainfalls for durations of less than one hour to one minute the ‘simple scaling’ model developed by Menabde et al. (1999) was adopted. The model was calibrated using the AMS from the Bureau of Meteorology’s continuous rainfall stations with more than eight years of data. For each continuous rainfall station the scaling factor, η, was determined and the at-site η values gridded to provide estimates for all grid locations. The model was then applied to the one hour duration rainfall depth grids to estimate the rainfall depths for the 1 minute to 30 minute rainfall events according to the following equation:

$$I_d = \left(\frac{d}{D}\right)^n I_D \tag{2.3.6}$$

Where I_d is the sub-hourly rainfall intensity for duration d , I_D is the 60 minute rainfall intensity (ie. duration D is 60 minutes).

3.4.5.6. Consistency Checking and Smoothing

In order to reduce inconsistencies across durations and smooth over discontinuities in the gridded data (unevenly spaced differences in design rainfall estimates at neighbouring durations) arising from application of the method, a smoothing process was undertaken. This was done by applying a sixth order polynomial to each grid point to all the standard durations from one minute up to seven days.

Although polynomials up to order 12 were investigated, a sixth order polynomial was adopted as investigations showed that this order polynomial gives adequate results.

Inconsistencies with respect to duration (rainfall depths at lower durations exceeding those at higher durations) were also found and were addressed.

Inconsistencies were detected by subtracting each grid from a longer duration grid at the same probability and checking for negative values. Inconsistencies were addressed by adjusting the longer duration rainfall upwards so that the ratio of shorter duration rainfall to the longer duration rainfall equals 0.99 or

$$\frac{\text{Rainfall depth at the shorter duration}}{\text{Rainfall depth at the longer duration}} = 0.99$$

The smoothing procedure was applied first to the original grids and the smoothed grids adjusted for inconsistencies. The grids were smoothed once again and a final adjustment for inconsistencies across durations was performed. The final grids were also checked for inconsistencies across AEP.

Grids of the polynomial coefficients were prepared in order to enable IFDs for any duration to be determined.

3.4.6. Outputs

The method described in Book 2, Chapter 3, Section 4 produced frequent and infrequent design rainfall estimates across Australia. The design rainfall estimates are provided both as rainfall depths in millimetres (mm) and rainfall intensities in millimetres per hour (mm/hr) for the standard durations and standard probabilities described in Table 2.3.5.

Table 2.3.5. Intensity Frequency Duration Outputs

Output	Values	Units
Standard durations	1, 2, 3, 4, 5, 10, 15, 30	Minutes
	1, 2, 3, 6, 12	Hours
	1, 2, 3, 4, 5, 6, 7	Days
Standard probabilities	1	EY
	63.2%, 50%, 20%, 10%, 5%, 2%, 1%	AEP

3.5. Very Frequent Design Rainfalls

3.5.1. Overview

The previous section summarised the steps involved in deriving the frequent and infrequent design rainfall values (IFDs) for probabilities from 1EY to 1% AEP. This range of probabilities is suitable for most design situations, however, many stormwater quality or Water Sensitive Urban Design guidelines recommend a flow threshold of Q_{3month} for the design of stormwater quality treatment devices.

Design rainfalls for the three month Average Recurrence Interval (or 4 EY) have not been previously available, with agencies giving their own advice on the approach for estimating very frequent design rainfalls. To address this need, estimates for probabilities more frequent than 1 EY have been derived.

To ensure consistency between the very frequent design rainfalls and the frequent and infrequent design rainfall, the overall approach adopted for the very frequent design rainfalls was very similar to that adopted for the frequent and infrequent design rainfall. However, some modifications to the approach were necessary because of the increased frequency of occurrence that was being considered. A summary of the method is presented in Table 2.3.6 and in Book 2, Chapter 3, Section 5 to Book 2, Chapter 3, Section 5. Further details can be found in The et al. (2015).

Table 2.3.6. Very Frequent Design Rainfall Method

Step	Method
Number of Rainfall Stations	Daily read – 15 364 Continuous – 2722
Period of Record	All available records up to 2012
Length of Record used in Analyses	Daily read > 5 years Continuous > 5 years
Source of Data	Organisations collecting rainfall data across Australia
Extreme Value Series	Partial Duration Series (PDS)
Frequency Analysis	Generalised Pareto (GPA) distribution fitted using L-moments
Ratios	Ratio X EY to 50% AEP
Gridding	Regionalised at-site distribution parameters gridded using ANUSPLIN

3.5.2. Rainfall Database

The data adopted consisted of the stations used for the frequent and infrequent design rainfalls (Green et al., 2011) and an additional 7290 stations with shorter periods of record, which have undergone the same rigorous quality controlling as the frequent and infrequent design rainfall database (Green et al., 2011; Green et al., 2012a). The locations of the stations used for the estimation of the very frequent design rainfalls are shown in Figure 2.3.12.

Additional stations could be used as the minimum number of years of record was reduced from 30 (for the frequent and infrequent design rainfalls) to five years for the very frequent design rainfalls. A threshold of five effective years was selected for daily and sub-daily sites as this was deemed to be statistically acceptable given the high frequency of the estimated exceedances compared to the previous 1 EY. The shorter record length ensures greater use of available sites but also ensures that there is sufficient information available to derive the more frequent probabilities from 12 EY to 2 EY.

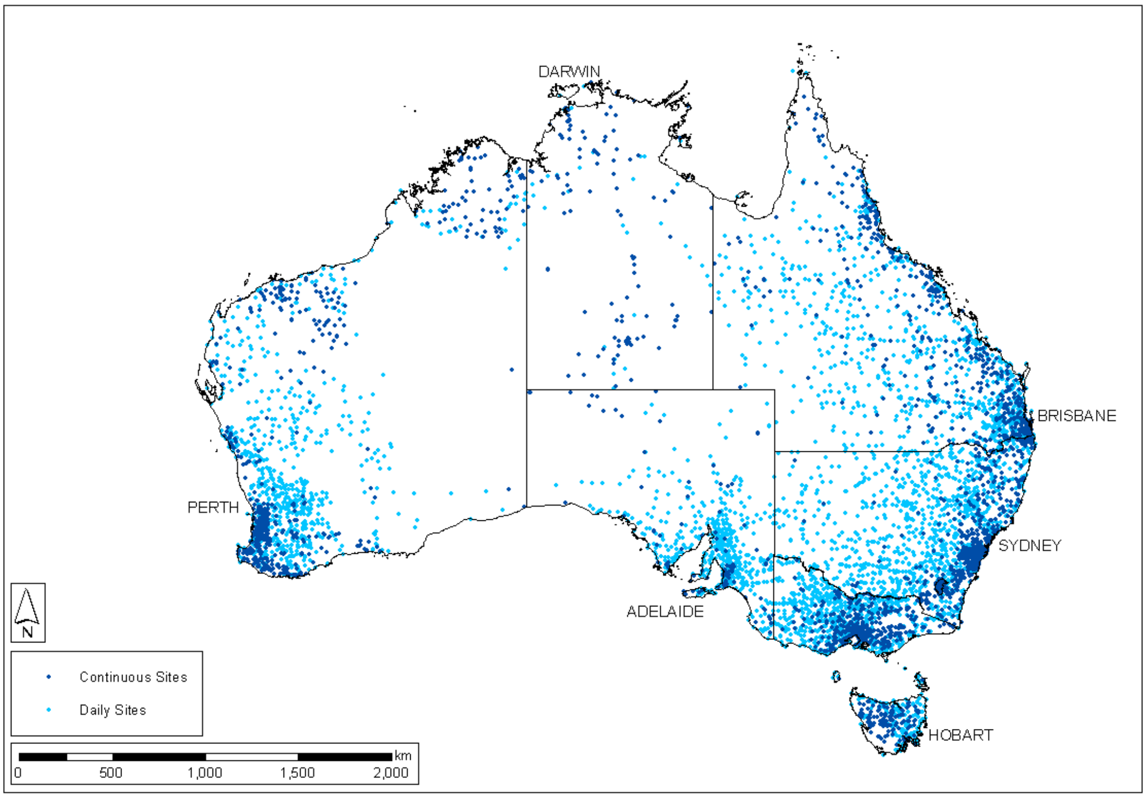


Figure 2.3.12. Daily Read Rainfall Stations and Continuous Rainfall Stations Used for Very Frequent Design Rainfalls

3.5.3. Extraction of Extreme Value Series

A Partial Duration Series (PDS) approach was adopted to estimate probabilities for events occurring more frequently than once a year. The advantage of using the PDS is that it extracts as much information as possible about large events and produces more accurate estimates for very frequent probabilities. As a PDS was used for the at-site series the selection of independent events was based on rank rather than temporal periods, thus completeness of record was not a consideration.

As a PDS approach was being adopted it was necessary to define the threshold above which all events will be included. It was important to identify the number of values per year that are required to accurately estimate the more frequent IFD's. Given that the most frequent probability is 12 EY, a minimum of 12 events per year was used to adequately represent the at-site distribution for these higher frequency events.

An assumption of the method, is that the events in the PDS are independent. In order to ensure that the events in the PDS were independent, a method that provided a consistent and meteorologically rigorous approach to defining independence of rainfall events across Australia was developed. The event independence testing criteria used were based on the Minimum Inter-event Time (MIT) approach (Xuereb and Green, 2012). The analyses suggested that a MIT that varied from two to six days with latitude across Australia was appropriate for event durations up to three days while, for durations longer than three days the MIT adopted was zero. For durations of less than one day, the MIT for the one day duration was adopted (Green et al., 2015).

3.5.3.1. At-Site Frequency Analysis

The approach adopted effectively treated the PDS as a Monthly Partial Duration Series or, more correctly for the current dataset, a Monthly Exceedance Series (MES) (PDS where number of values = number of effective months: 12 nE). While the extracted PDS is the same, the averaging duration is changed, representing the time in months rather than years. The selection of an MES rather than an annual series allowed significantly more records to be included from each site to establish the at-site rainfall distribution, capturing the more frequent rainfall patterns.

As discussed in Book 2, Chapter 3, Section 4 and Green et al. (2012b), testing of the most appropriate distribution to adopt for both the AMS and the PDS was undertaken as part of the derivation of the IFDs with results identifying the GEV distribution as the most appropriate for the AMS and the GPA distribution for the PDS. However, as a monthly exceedance data series was adopted for the very frequent design rainfalls there is some added uncertainty; to address this, a comparison was conducted of the GEV and GPA distributions. Twenty-four geographically distributed test sites with medium to long record lengths were selected for assessing the relative fit of the distributions to the at-site data. The test sites indicated that the GPA provides a closer fit to the site data in the majority of cases. On the basis of this, the very frequent design rainfalls used the GPA distribution fitted to the PDS for all stations which met the required record length.

3.5.3.2. Estimation of L-moments

Regional frequency analysis was undertaken using L-moments extracted from each of the at-site frequency distributions for sub-daily and daily data. The L-moments were used to estimate the parameters of the selected GPA distribution.

Extracting 12 independent events per year of record for the MES introduced the issue of zero values included in the PDS at some sites. This particularly occurred through the arid areas of central Australia to the west coast, where annual rainfall is highly variable and strong seasonality can occur. These areas have short wet seasons and can fail to have 12 rain events on average that are independent of one another for every year. However, given the previously defined minimum number of events being 12, these zero values events are considered as part of the distribution. To manage the occurrence of the zero values in the extreme value series, Hosking and Wallis (1997) suggest using a ‘mixed distribution’ or more correctly a conditional probability adjustment that gives a probability of a zero value, and cumulative distribution for the non-zero values as seen in Equation (2.3.7) (Guttman et al., 1993).

$$F(x) = p + (1 - p)G(x) \tag{2.3.7}$$

Where p is the probability of a zero rainfall value which is estimated by dividing the numbers of zeros by the total number of events and $G(x)$ is the cumulative distribution function of the non-zero rainfall events. Using this approach, if the Non-Exceedance Probability (NEP) of interest is less than p , then the quantile estimate is zero and if the NEP is greater than p, the quantile is estimated from $G(x)$ using the adjusted NEP shown in Equation (2.3.8).

$$NEP_{adj} = (NEP - p)/(1 - p) \tag{2.3.8}$$

For series with a small proportion of zeros, the impact on the distribution and resulting quantiles was negligible. For records with less than 10% zeros, there is very little difference and for up to nearly 20% zeros there is less than 10% average difference in the quantile depths. However, the differences become much more significant when the proportion of zeros increases.

3.5.4. Ratio Method

The parameters of the GPA distribution derived from the L-moments were used to calculate at-site quantiles. The parameters or quantiles could be gridded in a similar method to the frequent and infrequent design rainfalls and smoothed or rescaled to integrate them with the design rainfalls less frequent than 1 EY. Alternatively, as adopted, a ratio method was applied to derive the very frequent design rainfall estimates. A general ratio approach is currently used by various councils and authorities in Australia and internationally (Huff and Angel, 1992). It involves using the at-site data to determine the ratio of the various very frequent design rainfall values to either the 1 EY or 50% AEP gridded design rainfalls.

The ratio method adopted involves estimating at-site quantiles, using the at-site 50% AEP as the reference values for the ratios and gridding the calculated ratios. The advantage of this approach and using the at-site 50% AEP, was that it allows for the spatial variability in the ratios. In addition, the ratio was generally a more accurate representation of the *X* EY to 50% AEP ratio since it was calculated from the same dataset and resulted in a smooth spatial pattern. Consistency was also inherent since the ratios would always decrease with increasing probability. Since the ratios were spatially consistent, the final very frequent design rainfall depths follow the frequent and infrequent 50% AEP depths closely. These depth estimates were calculated using the gridded ratios, and multiplying by the 50% AEP design rainfall.

3.5.5. Gridding

As with the frequent and infrequent design rainfalls the ratios for all durations and EYs were gridded using the splining software ANUSPLIN (Hutchinson and Xu, 2013). To determine the most appropriate method to adopt for the gridding of the ratios a range of tests was undertaken of combinations of variates and different knot sets. The final case adopted was a spline that incorporated latitude, longitude and elevation using 4000 knots for the daily dataset and 1000 knots for the sub-daily dataset. The 0.025 degree Digital Elevation Model of Australia was used to provide the elevation data which were the same as that used in the derivation of the frequent and infrequent grids (The et al., 2014).

3.5.5.1. Depth Estimates

Very frequent design rainfall depth estimates for each duration and EY were calculated by multiplying the ratio grids with the corresponding 50% AEP design rainfall grids (Figure 2.3.13). As the frequent and infrequent design rainfall grids were based on AMS estimates, an AMS/PDS conversion factor was applied to account for the lower estimates (Green et al., 2012b).

The grids were smoothed to reduce any inconsistencies across durations and to smooth over discontinuities in the gridded data. A sixth order polynomial was applied to each grid point for all the standard durations from 1 minute up to 7 days. Grids were also checked for inconsistencies across EY.

Figure 2.3.13. Procedure to Derive Very Frequent Design Rainfall Depth Grids From Ratios

3.5.6. Outputs

The method described in Book 2, Chapter 3, Section 5 to Book 2, Chapter 3, Section 5 produced very frequent design rainfall estimates across Australia. The very frequent design rainfall estimates are provided both as rainfall depths in millimetres (mm) and rainfall intensities in millimetres per hour (mm/hr) for the standard durations and standard probabilities in Table 2.3.7:

Table 2.3.7. Very Frequent Design Rainfall Outputs

Output	Values	Units
Standard Durations	1, 2, 3, 4, 5, 10, 15, 30	Minutes
	1, 2, 3, 6, 12	Hours
	1, 2, 3, 4, 5, 6, 7	Days
Standard Probabilities	12, 6, 4, 3, 2, 1, 0.5, 0.2	EY

3.6. Rare Design Rainfalls

3.6.1. Overview

Rare design rainfalls (for 1 in 100 to 1 in 2000 AEP) are used by engineers, hydrologists, and planners for a range of purposes including:

- the design of dams that fall into the Significant and Low Flood Capacity Category where the Acceptable Flood Capacity is the 1 in 1000 AEP design flood (ANCOLD, 2000);
- the design of bridges, where the ultimate limit state adopted in the Australian bridge design code is defined as 'the capability of a bridge to withstand, without collapse, the design flood associated with a 2000 year return interval' (Austroads, 1992);
- the incorporation of climate change into IFDs in accordance with Book 1, Chapter 6 which recommends that if the design probability for a structure is 1% AEP, then the possible impacts of climate change could be assessed using 0.5% and 0.2% AEP (Bates et al., 2015); and
- the undertaking of spillway adequacy assessments of existing dams as the Dam Crest Flood (DCF) of many dams lies between the 1% AEP flood and the Probable Maximum Flood (as defined by the Probable Maximum Precipitation, PMP). Rare design rainfalls enable more accurate definition of the design rainfall and flood frequency curves between the 1% AEP and Probable Maximum Events.

Unlike the derivation of very frequent, frequent and infrequent design rainfalls which are based on observed rainfall events that lie within the range of probabilities being estimated, rare design rainfalls are an extrapolation beyond observed events. The longest period for which daily read rainfall records are available is around 170 years (Figure 2.3.3 and

Figure 2.3.7) however rare design rainfalls are required for probabilities much rarer than this. As a consequence it is difficult to validate the resultant rare design rainfalls and therefore the method adopted needs to be based on a qualitative assessment that the assumptions made in the method are reasonable and that the adopted approach is consistent with methods used to derive more frequent design rainfalls where the results can be validated.

The method adopted for deriving the rare design rainfalls was based on the data and method adopted for the more frequent design rainfalls but places more weight on the largest observed rainfall events which are of most relevance to rare design rainfalls. The adopted regional LH-moments approach is summarised in Table 2.3.8 and Book 2, Chapter 3, Section 6 to Book 2, Chapter 3, Section 6. More detail can be found in Green et al. (2015) and Bureau of Meteorology (2016).

Table 2.3.8. Rare Design Rainfall Method

Step	Method
Number of rainfall stations	Daily read – 8074 for index value Daily read – 3955 for LCV and LSK
Period of record	All available records up to 2012
Length of Record Used in Analyses	Daily read ≥ 30 years for index value Daily read ≥ 60 years for LCV and LSK
Source of Data	Bureau of Meteorology
Extreme Value Series	Annual Maximum Series (AMS)
Frequency Analysis	Generalised Extreme Value (GEV) distribution fitted using LH(2)-moments
Regionalisation	Region of Influence
Gridding	Regionalised GEV parameters gridded using ANUSPLIN

3.6.2. Rainfall Database

The quality controlled rainfall database established for the derivation of the more frequent design rainfalls was used as the basis for the database used for the rare design rainfalls. However, as the estimation of rare design rainfalls relies on long-term records, only those stations with more than 60 years of record were selected. This reduced to data set to approximately 4000 stations, the locations of which are shown in Figure 2.3.14. In order to ensure consistency with the more frequent design rainfalls, the index values were derived using the same dataset as the more frequent design rainfalls.

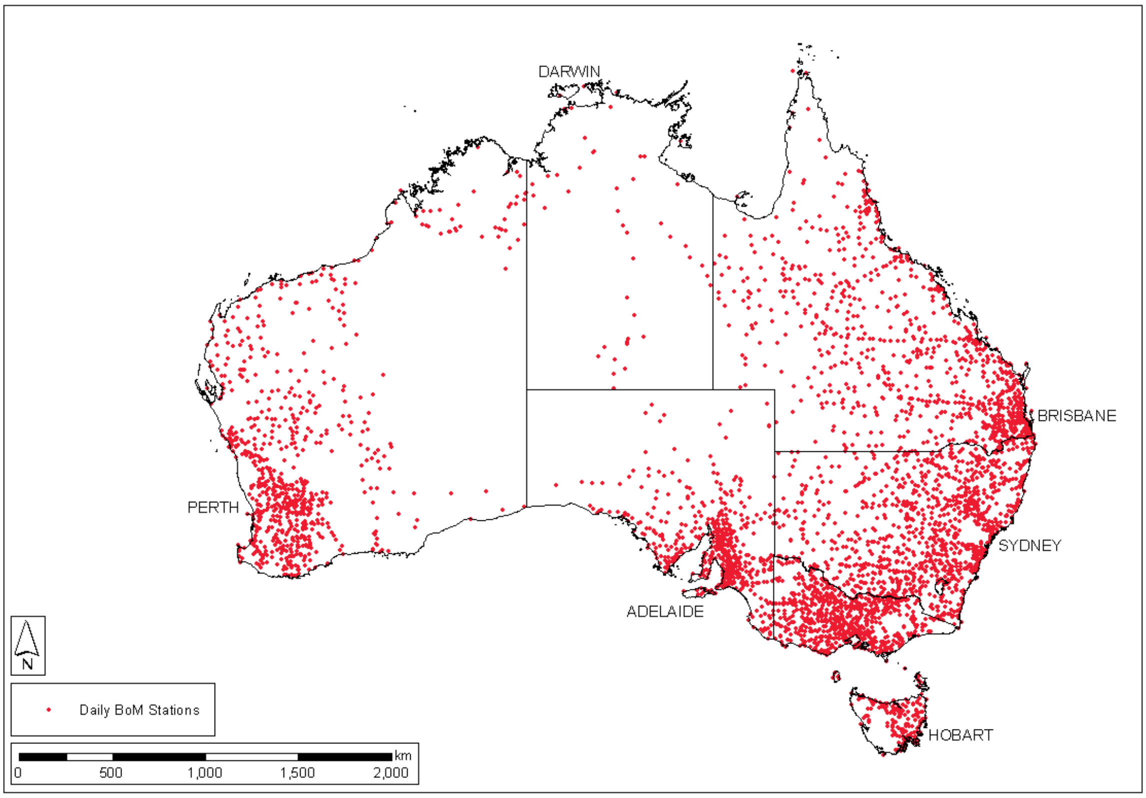


Figure 2.3.14. Daily Read Rainfall Stations with 60 or More Years of Record

3.6.3. Extraction of Extreme Value Series

The AMS was extracted from all daily read rainfall stations with 60 or more years of record. The AMS was used to define the extreme value series for the rare design rainfalls as the focus is on the largest recorded events.

As discussed in Book 2, Chapter 3, Section 4, the GEV distribution was adopted for AMS for the frequent and infrequent design rainfalls following extensive testing of a range of candidate distributions. On the basis of these trials and similar results found by Nandakumar et al. (1997) and Schaefer (1990), the GEV distribution was adopted for the rare design rainfall analyses.

In keeping with the approach adopted for the more frequent design rainfalls, the statistical properties of the at-site data were estimated and then translated into the relevant GEV distribution parameters. However, whereas L-moments were used for the more frequent design rainfalls, for the rare design rainfalls LH-moments were adopted (Wang, 1997). LH-moments were adopted as they more accurately fit the upper tail (rarer probabilities) of the distribution.

LH-moments are a generalisation of L-moments and allow the distribution to be increasingly focused on the larger data values depending on the value of η, where η=0 is equivalent to L-moments. The equations for deriving LH-moments and the associated GEV parameters are given in Wang (1998). LH-moments with a η=2 were selected as a compromise between providing a better fit to the tail of the at-site distribution without giving too much influence to the high outliers. LH-moments (η=2) were derived for all stations with greater than 60 years AMS.

3.6.4. Regionalisation

For the rare design rainfalls, the ROI approach adopted for the IFDs was used to reduce the uncertainty in the estimated LH-moments by regionalising the station point estimates. While 500 station years was found to be an optimum pool size for the IFDs, because the rare design rainfalls are provided for probabilities up to 1 in 2000 AEP, the ROI needed to be increased. The tradeoff between gaining improved accuracy from a larger pool of data was that the assumption of homogeneity may not be satisfied. Testing was conducted to find the pool size that reduced uncertainty without introducing significant homogeneity, with a minimum of 2000 station years adopted. However, where necessary, the number of pooled station years was increased above this number to maximize the available record used, while ensuring homogeneity.

The average LH-CV for each region was calculated using a weighted average of the LH-CV at all stations in the region, with the weights proportional to the station lengths. This was repeated for the LH-Skewness.

3.6.5. Gridding

The Index, regionalised LH-CV and LH-SK values for all durations and AEP's were gridded using the splining software ANUSPLIN (Hutchinson and Xu, 2013) that was adopted for the more frequent design rainfalls. To determine the most appropriate method to adopt for the gridding of the moments a range of tests was undertaken of combinations of different knot sets. The final case adopted was a spline that incorporated latitude, longitude and elevation using 3750 knots for the Index (as was adopted for the more frequent design rainfalls) and 2200 knots for the regionalised LH-CV and LH-SK values. The 0.025 degree Digital Elevation Model (DEM) of Australia was used to provide the elevation data which was the same as that used in the derivation of the frequent and infrequent grids (The et al., 2014) .

In order to provide consistent design rainfall estimates across all durations and probabilities, a suitable method was required to integrate the rare design rainfalls with the more frequent design rainfalls. After testing of various ‘anchor’ points, the rare design rainfalls were anchored to the more frequent design rainfalls at the 5% AEP as it was considered that the rare design rainfalls provide a better estimate of the upper tail of the distribution down to the 5% AEP.

3.6.6. Outputs

The method described in Book 2, Chapter 3, Section 6 to Book 2, Chapter 3, Section 6 produced rare design rainfall estimates across Australia for the standard durations and standard probabilities in Table 2.3.9.

Table 2.3.9. Rare Design Rainfall Outputs

Output	Values	Units
Standard Durations	1, 2, 3, 4, 5, 6, 7	Days
Standard Probabilities	1 in 100; 1 in 200; 1 in 500; 1 in 1000; 1 in 2000	AEP

3.7. Probable Maximum Precipitation Estimates

3.7.1. Overview

The design rainfalls classes described in Book 2, Chapter 3, Section 4 and Book 2, Chapter 3, Section 5 were derived using frequency analysis. However, extreme rainfalls events such as the Probable Maximum Precipitation (PMP), lie beyond both any directly observed events and the limit to which observed data can be extrapolated. As a result, estimation of extreme rainfall events is based on the broadest understanding of extreme events and the meteorological processes that produce them.

The Probable Maximum Precipitation (PMP) is defined as ‘the theoretical greatest depth of precipitation that is physically possible over a particular catchment’ (World Meteorological Organisation, 1986). The PMP assumes the simultaneous occurrence in one storm of maximum amount of moisture and the maximum conversion rate of moisture to precipitation (maximum efficiency).

3.7.2. Estimation of PMPs

The Bureau of Meteorology has been providing PMP estimates for over 70 years, however, the methods adopted have changed with time, as the understanding of extreme storms and the mechanisms which produce them have developed, and the databases of observed extreme storms have expanded. These methods include:

- **In Situ Maximisation Method:** During the 1950’s to 1970’s PMP estimates were based on the maximisation of the moisture content of storms which had been observed over the catchment of interest. The limitation of this method was that the differing lengths of rainfall records and occurrence or non-occurrence of an extreme storm led to inconsistent PMP estimates for catchments within the same region.

- **Storm Transposition Method:** During the late 1960’s and early 1970’s the size of the extreme storm sample for a specific catchment was increased by the transposition to the catchment of interest of extreme storms which had been observed over nearby catchments which had similar hydrometeorological and topographic features. Although this improved the within-region consistency of PMP estimates, the method was limited, as only storms from a similar topographic region could be transposed, and the selection of storms introduced a significant level of subjectivity.

- **Generalised Methods:** From the mid-1970’s generalised methods were introduced into Australia. Generalised methods make use of all available storm data for a large region by making adjustments for moisture availability and differing topographic effects. The generalised methods currently adopted in Australia are described in Book 2, Chapter 3, Section 7.

3.7.3. Generalised Methods for Probable Maximum Precipitation Estimation

There are three main generalised methods used for PMP estimation in Australia. These methods and their area of applicability are shown in Figure 2.3.15 and described in Book 2, Chapter 3, Section 7 to Book 2, Chapter 3, Section 7. There are two regional methods (Book 2, Chapter 3, Section 7, Book 2, Chapter 3, Section 7).

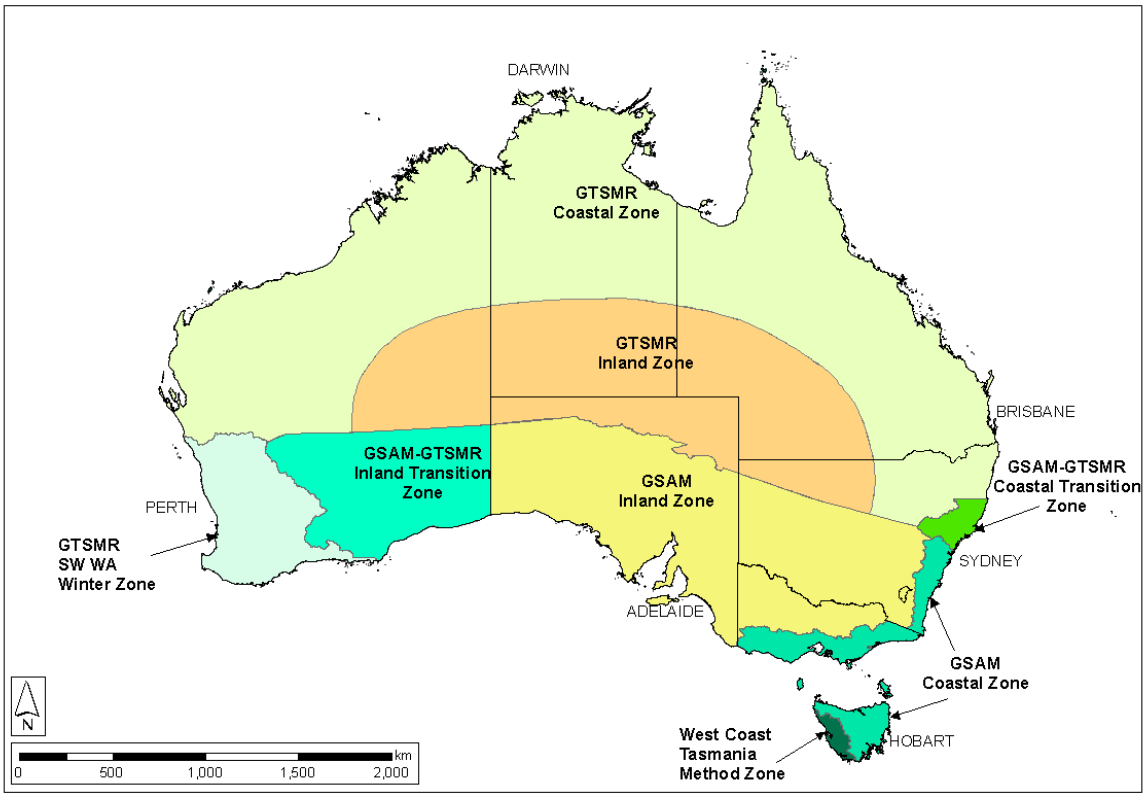


Figure 2.3.15. Generalised Probable Maximum Precipitation Method Zones

3.7.3.1. Generalised Short-Duration Method

The Generalised Short Duration Method (GSDM) is applicable across Australia for catchment areas less than 1000 km^2 and for durations up to three hours or six hours depending on the location within of Australia (Bureau of Meteorology, 2003).

3.7.3.2. Generalised South-east Australia Method

The Generalised South-east Australia Method (GSAM) is applicable to the southern third of Australia where it is assumed that the causative mechanism of the PMP would not be tropical. The GSAM method is applicable for durations from 12 hours to 120 hours (Bureau of Meteorology, 2006; Minty et al., 1996).

3.7.3.3. Revised General Tropical Storm Method

The Revised Generalised Tropical Storm Method (GTSMR) is applicable to the northern two-thirds of Australia where it is assumed that the causative mechanism of the PMP would be a tropical storm. The GTSMR method is applicable for durations from 12 hours to 120 hours (Bureau of Meteorology, 2005; Walland et al., 2003).

3.7.3.4. West Coast Tasmania Method Zone

The West Coast Tasmania Method Zone applies to the west coast region of Tasmania which is outside the region of applicability of the Tasmanian GSAM coastal zone. It is applicable for durations between 24 and 72 hours (Xuereb et al., 2001).

3.7.3.5. GSAM-GTSMR Coastal Transition Zone

The GSAM-GTSMR Coastal Transition Zone method to the coastal area in NSW where is it considered that PMPs could be caused by either tropical or non-tropical storms.

3.7.4. Generalised Method of Probable Maximum Precipitation Estimation

Although each of the three generalised methods has specific features, the generalised method can be summarised as follows:

3.7.4.1. Development of Storm Database

The ten highest one to seven day rainfalls which were common to a number of stations were selected and the storms prioritised according to the rarity of the event in order to identify the 100 or more largest storms. For each storm the following analyses were undertaken.

- The rainfall totals for the total storm duration were plotted on a topographic map and isohyets drawn to determine the spatial extent and distribution of each storm.
- To determine the storm temporal distribution, parallelograms were drawn around the storm centre for standard areas of 100; 500; 1 000; 2 500; 10 000; 40 000 and 60 000 km^2. The average daily rainfall depths within a parallelogram were determined using Thiessen weights. For each standard area, the percentage of the total storm that fell during each 24 hour period was determined. These daily data were supplemented by pluviograph and 3 hourly synoptic charts.
- The representative dew point temperature for each storm was determined using a number of sources including the Australian Region Mean Sea Level charts, National Climate Centre Archives and Observers' Logbooks.

3.7.4.2. Generalisation of Storm Database

The 'site specific' attributes of each storm were removed in order to attain a homogenous data set.

The effects of storm type were removed from the data set by the dividing of Australia into the GSAM and GTSMR regions on the basis of the type of storm that produces the largest observed rainfall depths. The two regions were further divided into Coastal and Inland Zones on the basis that different mechanisms produce the largest rainfall depths in each of the zones (refer Figure 2.3.15).

The specification of each storm in terms of depth-area-duration curves as done previously effectively removed the storm specific spatial distribution.

The removal of the site specific topographic effects was undertaken using 72 hour, 50 year ARI 'flat land' rainfall intensity field in order to produce the convergence component of each storm.

3.7.4.3. Removal of the Site-Specific and Storm-Specific Moisture Content

To remove the storm-specific moisture content from each storm and simultaneously to maximise the moisture content, the convergence depths were multiplied by a moisture maximisation factor. The moisture maximisation factor is defined as the ratio of extreme precipitable water associated with the extreme dew point temperature at the storm location.

The site-specific moisture content of each storm was removed by transposition to a single location which for the GSAM was chosen as Brisbane and for GTSMR as Broome. For each location, representative seasonal extreme 24 hour persisting dew point temperatures were selected and the moisture content for each storm standardised.

3.7.4.4. Determination of ‘Storm’ of Maximum Moisture Content

The moisture maximisation factor and the standardised convergence depths were combined to estimate the maximised standardised convergence depths. To determine a single hypothetical storm of maximum moisture content, an envelope curve was drawn to the set of maximised, standardised convergence depth-area curves.

3.7.4.5. Determination of Catchment Specific Probable Maximum Precipitation

The envelope curves represent the maximised convergence component of the PMP at the standardising locations (Brisbane for the GSAM and Broome for GTSMR). To obtain an estimate of the PMP for a specific catchment it is necessary to build in the moisture content and topographic influences specific to the catchment of interest. The moisture content of the standard PMP convergence depth is adjustment using a Moisture Adjustment Factor (MAF) such that:

$$\mathrm{MAF}=\mathrm{EPW}_{\mathrm{catchment}}/\mathrm{EPW}_{\mathrm{std}} \tag{2.3.9}$$

where MAF = Moisture Adjustment Factor; $\mathrm{EPW}_{\mathrm{catchment}}$ is the Extreme Precipitable Water associated with the catchment extreme dew point temperature; $\mathrm{EPW}_{\mathrm{std}}$ is the Extreme Precipitable Water associated with the standard extreme dew point temperature for appropriate season.

The Topographic Enhancement Factor (TEF) for the catchment PMP is estimated in the same manner as the topographic component of the storms in the database using the 72 hour 50 year ARI rainfall intensities.

The total PMP for a specific catchment for each of the standard durations is estimated as:

$$\mathrm{PMP}_{\mathrm{catchment}}=\mathrm{MCD}_{\mathrm{std}}*\mathrm{MAF}_{\mathrm{catchment}}*\mathrm{TEF}_{\mathrm{catchment}} \tag{2.3.10}$$

3.8. Uncertainty in Design Rainfalls

The design rainfalls described in this Chapter are the best estimates currently available and have been derived using an extensive rainfall database which has been analysed using the most appropriate techniques.

However, there are uncertainties associated with the design rainfalls which arise from various sources including:

- errors in the data due to short record length, instrumentation errors, gaps in the data, unidentified errors in the data;
- sampling errors including network sparsity, poorly placed gauges, non-representativeness of gauging networks; and

- limitations in the adopted methods including delineation of regions, lack of homogeneity in regions, selection of distribution, parameter and quantile estimation, and extrapolation of data to ungauged locations.

These uncertainties need to be taken into consideration when the design rainfalls are being used in conjunction with other design flood inputs. Quantification of the uncertainties associated with the design rainfalls is described in Bureau of Meteorology (2016) as well as advice on how to incorporate uncertainty when using the design rainfalls.

3.9. Application

3.9.1. Design Rainfalls

The very frequent, frequent, infrequent, and rare design rainfalls are available via the Bureau of Meteorology's website[3].

3.9.1.1. Input Data Requirements and Options

To estimate a design rainfall for a point location it is necessary to enter the co-ordinate of the location in one of three co-ordinate format options:

- Decimal degrees;
- Degrees, Minutes, Seconds; and
- Easting, Northing, Zone.

The location of the entered co-ordinate can be seen by using the map preview option and a location label can also be entered (see Figure 2.3.16).

[3]http://www.bom.gov.au/water/designRainfalls/ifd/index.shtml

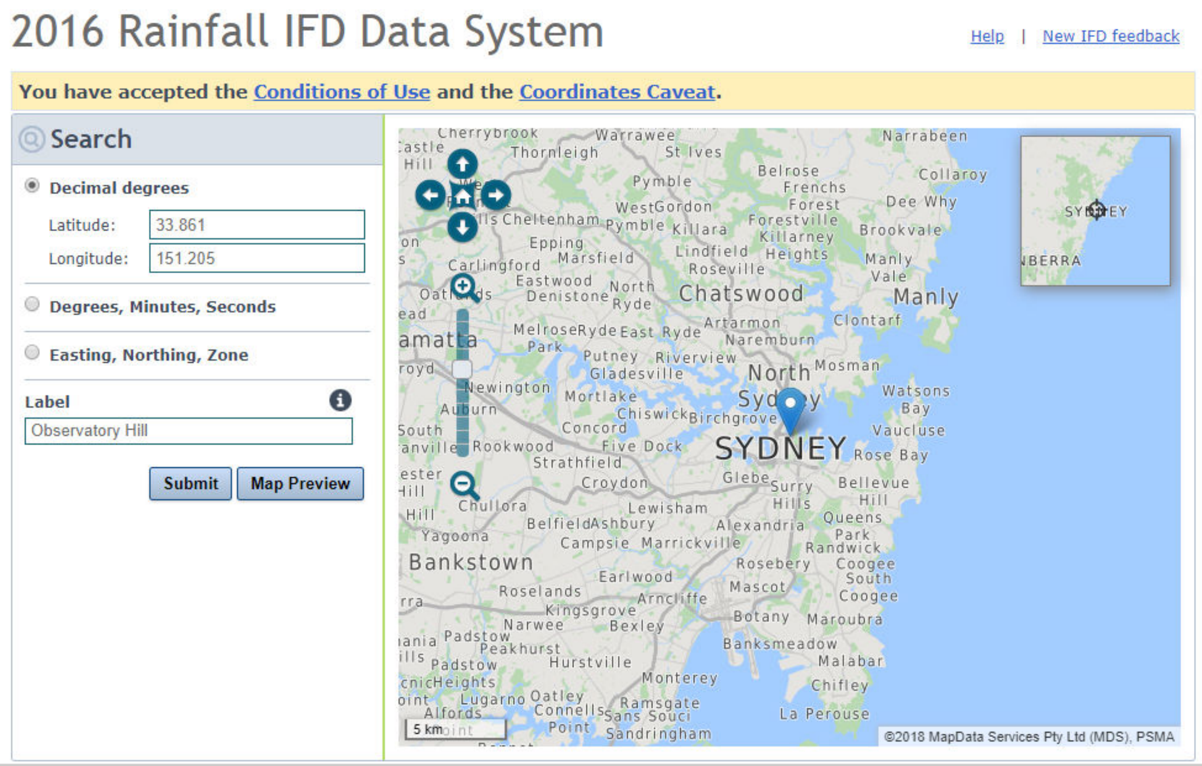


Figure 2.3.16. Design Rainfall Point Location Map Preview

Determine the design rainfall class for which design rainfalls are required:

- Very frequent
- Frequent and infrequent (IFDs)
- Rare

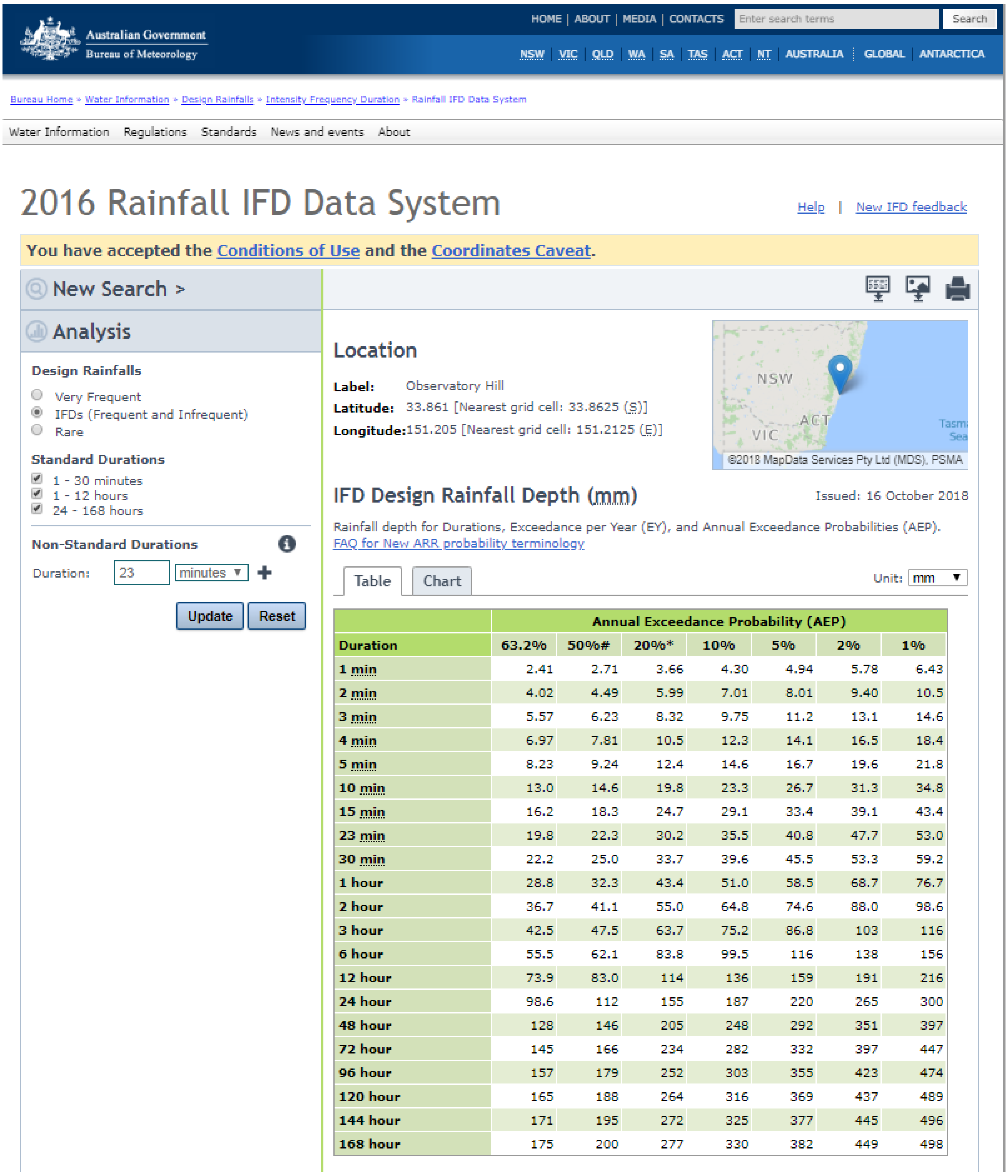
Determine the durations for which design rainfalls are required:

- Standard
- Non-standard

Determine the units in which the design rainfalls will be provided:

- Depths as millimetres (mm)
- Intensities as millimetres per hour (mm/hr)

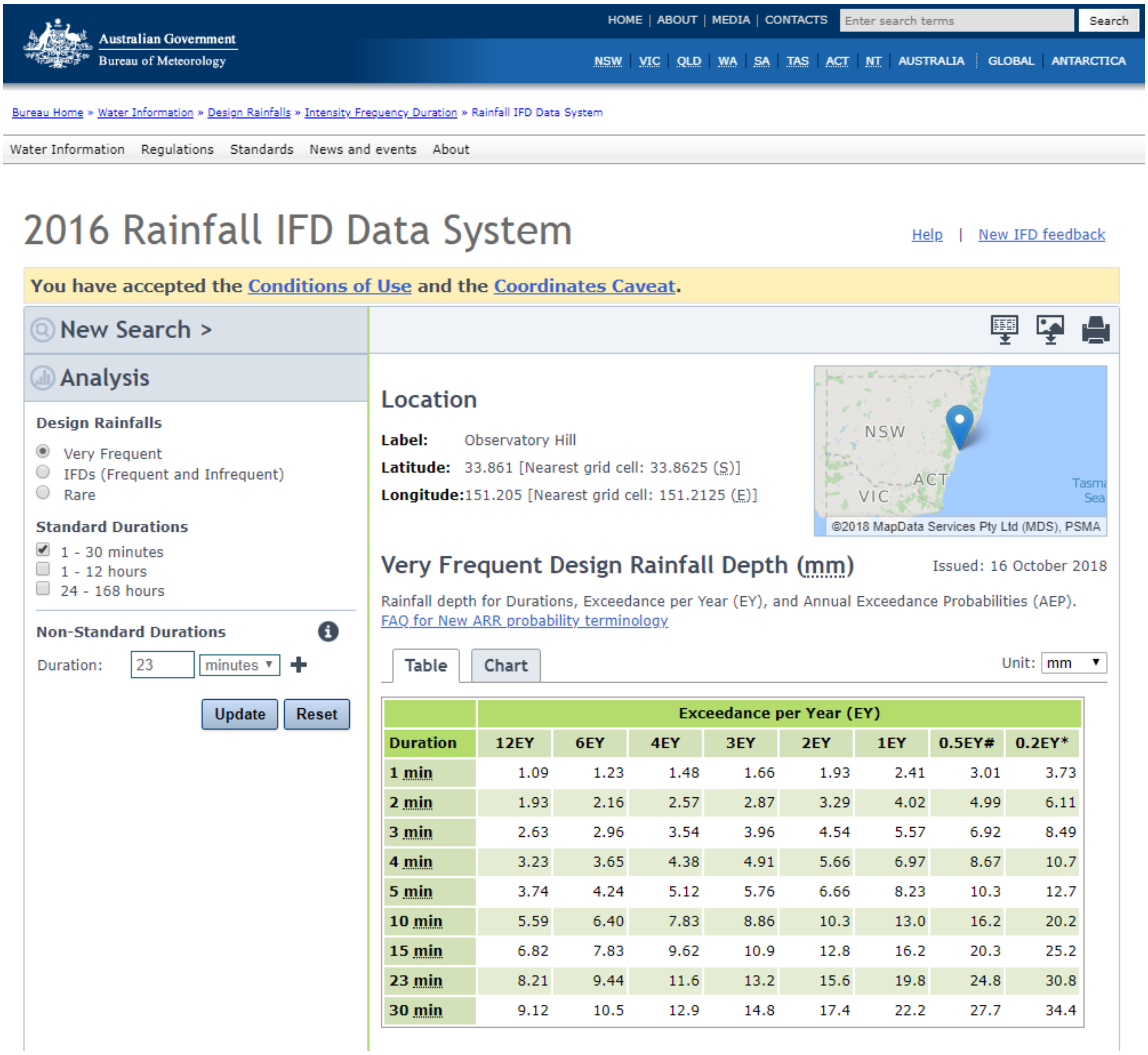
3.9.2. Frequent and Infrequent Design Rainfalls (IFDs)



	Annual Exceedance Probability (AEP)						
Duration	63.2%	50%#	20%*	10%	5%	2%	1%
1 min	2.41	2.71	3.66	4.30	4.94	5.78	6.43
2 min	4.02	4.49	5.99	7.01	8.01	9.40	10.5
3 min	5.57	6.23	8.32	9.75	11.2	13.1	14.6
4 min	6.97	7.81	10.5	12.3	14.1	16.5	18.4
5 min	8.23	9.24	12.4	14.6	16.7	19.6	21.8
10 min	13.0	14.6	19.8	23.3	26.7	31.3	34.8
15 min	16.2	18.3	24.7	29.1	33.4	39.1	43.4
23 min	19.8	22.3	30.2	35.5	40.8	47.7	53.0
30 min	22.2	25.0	33.7	39.6	45.5	53.3	59.2
1 hour	28.8	32.3	43.4	51.0	58.5	68.7	76.7
2 hour	36.7	41.1	55.0	64.8	74.6	88.0	98.6
3 hour	42.5	47.5	63.7	75.2	86.8	103	116
6 hour	55.5	62.1	83.8	99.5	116	138	156
12 hour	73.9	83.0	114	136	159	191	216
24 hour	98.6	112	155	187	220	265	300
48 hour	128	146	205	248	292	351	397
72 hour	145	166	234	282	332	397	447
96 hour	157	179	252	303	355	423	474
120 hour	165	188	264	316	369	437	489
144 hour	171	195	272	325	377	445	496
168 hour	175	200	277	330	382	449	498

Figure 2.3.17. IFD Outputs

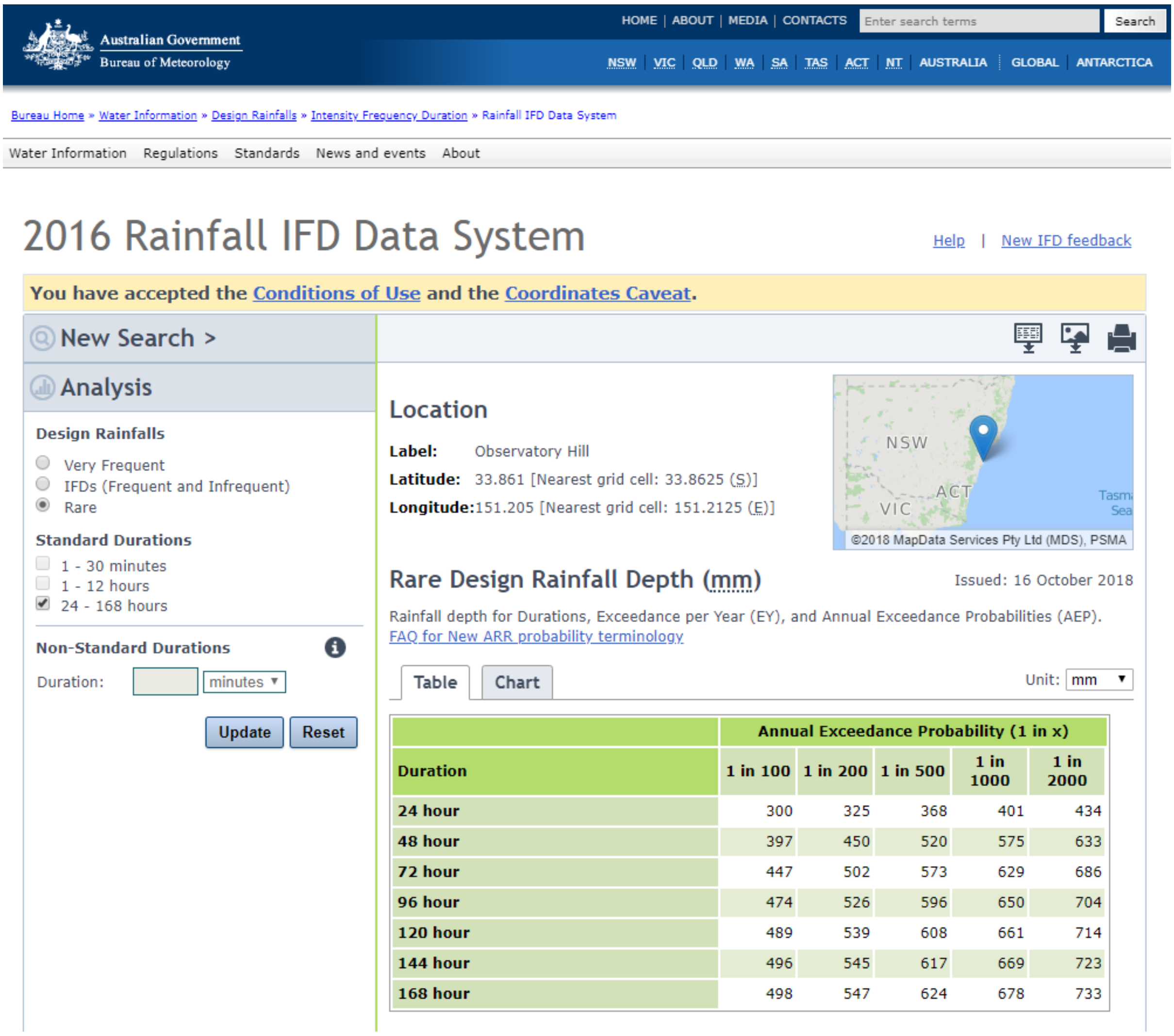
3.9.3. Very Frequent Design Rainfalls



	Exceedance per Year (EY)							
Duration	12EY	6EY	4EY	3EY	2EY	1EY	0.5EY#	0.2EY*
1 min	1.09	1.23	1.48	1.66	1.93	2.41	3.01	3.73
2 min	1.93	2.16	2.57	2.87	3.29	4.02	4.99	6.11
3 min	2.63	2.96	3.54	3.96	4.54	5.57	6.92	8.49
4 min	3.23	3.65	4.38	4.91	5.66	6.97	8.67	10.7
5 min	3.74	4.24	5.12	5.76	6.66	8.23	10.3	12.7
10 min	5.59	6.40	7.83	8.86	10.3	13.0	16.2	20.2
15 min	6.82	7.83	9.62	10.9	12.8	16.2	20.3	25.2
23 min	8.21	9.44	11.6	13.2	15.6	19.8	24.8	30.8
30 min	9.12	10.5	12.9	14.8	17.4	22.2	27.7	34.4

Figure 2.3.18. Very frequent Design Rainfall Outputs

3.9.4. Rare Design Rainfalls

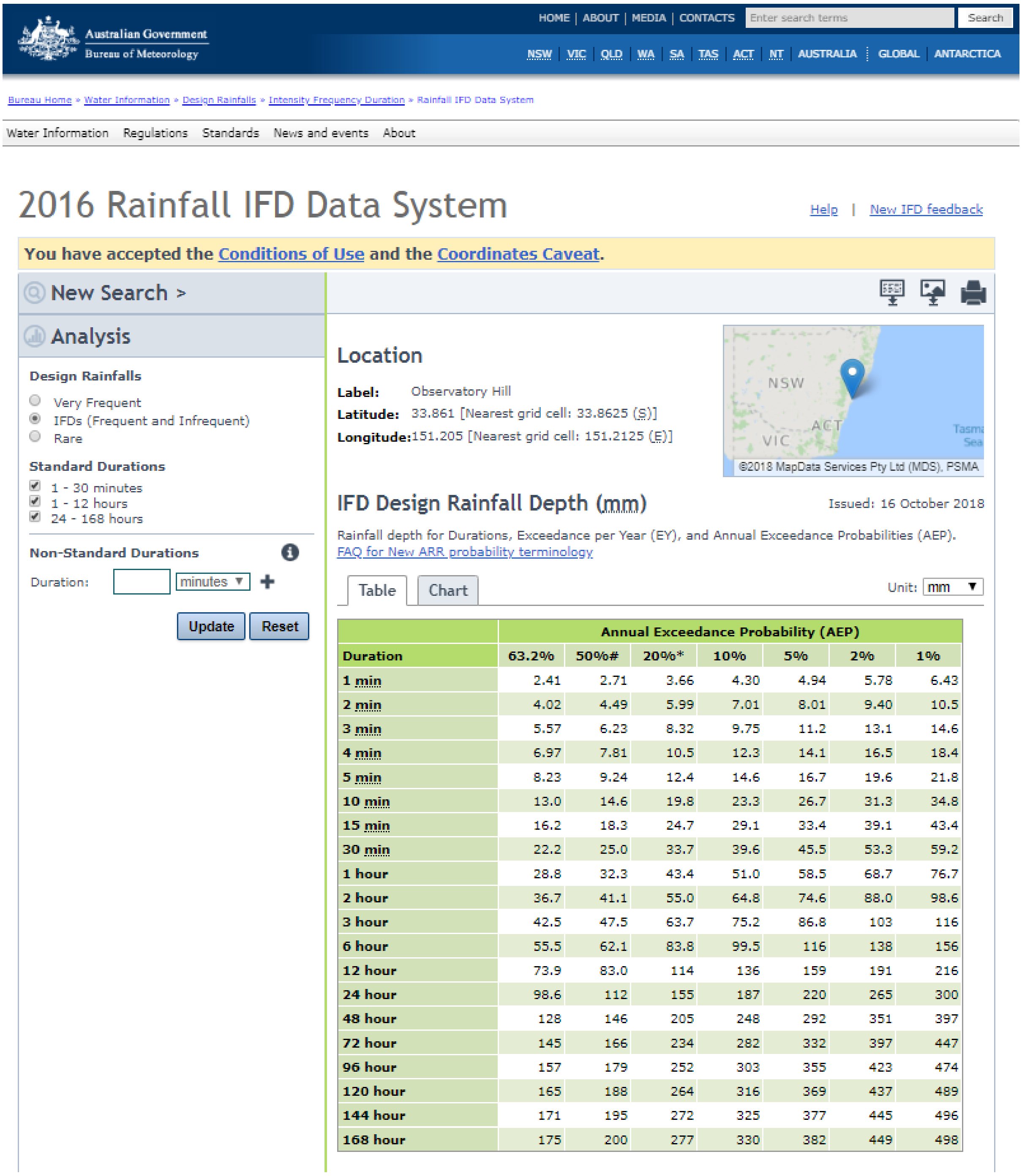


	Annual Exceedance Probability (1 in x)				
Duration	1 in 100	1 in 200	1 in 500	1 in 1000	1 in 2000
24 hour	300	325	368	401	434
48 hour	397	450	520	575	633
72 hour	447	502	573	629	686
96 hour	474	526	596	650	704
120 hour	489	539	608	661	714
144 hour	496	545	617	669	723
168 hour	498	547	624	678	733

Figure 2.3.19. Rare design rainfall outputs

3.9.4.1. Output Options

The design rainfall information for the selected location can be seen in either tabular form (Figure 2.3.20) or graphical form (Figure 2.3.21).


HOME | ABOUT | MEDIA | CONTACTS | Search

Australian Government
Bureau of Meteorology

NSW | VIC | QLD | WA | SA | TAS | ACT | NT | AUSTRALIA | GLOBAL | ANTARCTICA

Bureau Home » Water Information » Design Rainfalls » Intensity Frequency Duration » Rainfall IFD Data System

Water Information Regulations Standards News and events About

2016 Rainfall IFD Data System

Help | New IFD feedback

You have accepted the Conditions of Use and the Coordinates Caveat.

New Search >

Analysis

Design Rainfalls

- Very Frequent
- IFDs (Frequent and Infrequent)
- Rare

Standard Durations

- 1 - 30 minutes
- 1 - 12 hours
- 24 - 168 hours

Non-Standard Durations

Duration: minutes

Update Reset

Location

Label: Observatory Hill
Latitude: 33.861 [Nearest grid cell: 33.8625 (S)]
Longitude: 151.205 [Nearest grid cell: 151.2125 (E)]

IFD Design Rainfall Depth (mm)

Issued: 16 October 2018

Rainfall depth for Durations, Exceedance per Year (EY), and Annual Exceedance Probabilities (AEP).
FAQ for New ARR probability terminology

Table | Chart

Unit: mm

	Annual Exceedance Probability (AEP)						
Duration	**63.2%**	**50%#**	**20%***	**10%**	**5%**	**2%**	**1%**
1 min	2.41	2.71	3.66	4.30	4.94	5.78	6.43
2 min	4.02	4.49	5.99	7.01	8.01	9.40	10.5
3 min	5.57	6.23	8.32	9.75	11.2	13.1	14.6
4 min	6.97	7.81	10.5	12.3	14.1	16.5	18.4
5 min	8.23	9.24	12.4	14.6	16.7	19.6	21.8
10 min	13.0	14.6	19.8	23.3	26.7	31.3	34.8
15 min	16.2	18.3	24.7	29.1	33.4	39.1	43.4
30 min	22.2	25.0	33.7	39.6	45.5	53.3	59.2
1 hour	28.8	32.3	43.4	51.0	58.5	68.7	76.7
2 hour	36.7	41.1	55.0	64.8	74.6	88.0	98.6
3 hour	42.5	47.5	63.7	75.2	86.8	103	116
6 hour	55.5	62.1	83.8	99.5	116	138	156
12 hour	73.9	83.0	114	136	159	191	216
24 hour	98.6	112	155	187	220	265	300
48 hour	128	146	205	248	292	351	397
72 hour	145	166	234	282	332	397	447
96 hour	157	179	252	303	355	423	474
120 hour	165	188	264	316	369	437	489
144 hour	171	195	272	325	377	445	496
168 hour	175	200	277	330	382	449	498

Figure 2.3.20. Design Rainfall Output Shown as Table

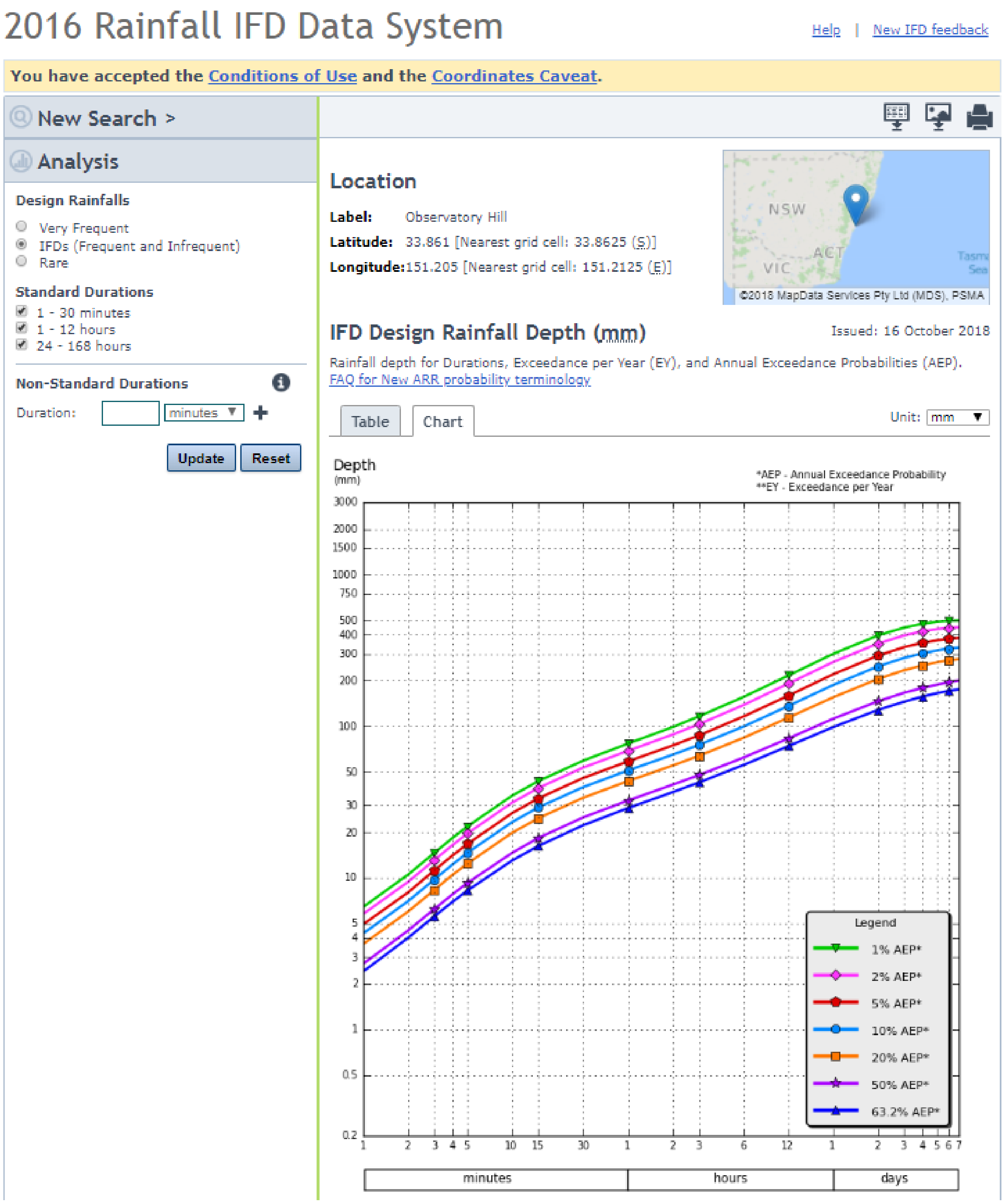


Figure 2.3.21. Design Rainfall Output Shown as Chart

3.9.4.2. Export Options

The table can be exported as a .csv file for ease of incorporation into software and the chart as a .png to facilitate integration into reports.

3.9.5. Probable Maximum Precipitation Estimates

Probable Maximum Precipitation estimates for the GSDM can be estimated using the method contained in Bureau of Meteorology (2003) which can be downloaded from the Bureau of Meteorology's website.

Guides for the application of the GSAM and GTSMR methods are available from the Bureau of Meteorology (Bureau of Meteorology, 2005; Bureau of Meteorology, 2006).

3.10. Acknowledgements

The authors would like to acknowledge the contributions made by the following people during the IFD Revision Project:

- Dr Louise Minty and Dr Ian Prosser for their direction of the project
- Andrew Frost, Khaled Haddad, Catherine Jolly, Garry Moore, Scott Podger, Ataur Rahman, Lionel Siriwardena and Karin Xuereb for their contribution during various stages of the project
- Professor Mike Hutchinson for his ongoing advice on the application of ANUSPLIN
- Erwin Jeremiah, Deacon McKay, Scott Podger and Michael Sugiyanto for their cheerful quality controlling of the data
- Dr George Kuczera, Dr Nanda Nandakumar; Dr Rory Nathan and Erwin Weinmann for their sage advice on the rare design rainfalls
- Martin Chan, Damian Chong, Ceredwyn Ealanta Murray Henderson, Chris Lee, Quentin Leseney, Maria Levtova, Max Monahon, Julian Noye, and William Tall for the webpages.

3.11. References

ANCOLD (Australian National Committee on Large Dams) (2000), Guidelines on Selection of Acceptable Flood Capacity for Dams. Australian National Committee on Large Dams.

AUSTROADS (1992) '92 AUSTROADS Bridge Design Code.

Bates, B.C., McLuckie, D., Westra, S., Johnson, F., Green, J., Mummery, J. and Abbs, D. (2015), Revision of Australian Rainfall and Runoff - The Interim Climate Change Guideline. Presented at National Floodplain Management Authorities Conference, Brisbane, Qld, May.

Bonnin, G., Maitaria, K. and Yekta, M. (2010), Trends in Heavy Rainfalls in the Observed Record in Selected Areas of the U.S. IN PALMER, R. N. (Ed.) World Environmental and Water Resources Congress 2010: Challenges of Change. Providence, Rhode Island, American Society of Civil Engineers.

Bureau of Meteorology (2016), Design rainfalls for Australia: data, methods and analyses. Bureau of Meteorology, Melbourne, VIC.

Bureau of Meteorology (2003), The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method, [Online] Bureau of Meteorology, Melbourne, Australia, June 2003, (p: 39). Available at: http://www.bom.gov.au/water/designRainfalls/pmp/gsdm_document.shtml.

Bureau of Meteorology (2006), Guidebook of the Estimation of Probable Maximum Precipitation: Generalised Southeast Australia Method, included on the compact disc 'Guide to the Estimation of Probable Maximum Precipitation: Generalised Southeast Australia Method', Hydrometeorological Advisory Service, Bureau of Meteorology, October.

Bureau of Meteorology (2005), Guidebook of the Estimation of Probable Maximum Precipitation: Generalised Tropical Storm Method, Included on compact disc 'Guide to the

Estimation of Probable Maximum Precipitation: Generalised Tropical Storm Method', Hydrometeorological Advisory Service, Bureau of Meteorology, September.

Burn, D.H. (1990), An appraisal of the 'region of influence' approach to flood frequency analysis, Hydrological Sciences - Journal-des Sciences Hydrologiques, 35(2), 149-165.

Green, J. and Johnson, F.J. (2011), Stationarity Assessment of Australian Rainfall, Internal Bureau of Meteorology Report.

Green, J.H., Xuereb, X. and Siriwardena, L. (2011), Establishment of a Quality Controlled Rainfall Database for the Revision of the Intensity-Frequency Duration (IFD) Estimates for Australia. Presented at 34th IAHR Congress, Brisbane, Qld, June.

Green, J.H., Beesley, C., Frost, A., Podger, S. and The, C. (2015), National Estimates of Rare Design Rainfall. Presented at Hydrology and Water Resources Symposium, Hobart, Tas, December 2015.

Green, J.H., Johnson, J., McKay, D., Podger, P., Sugiyanto, M. and Siriwardena, L. (2012a), Quality Controlling Daily Read Rainfall Data for the Intensity-Frequency Duration (IFD) Revision Rainfall Project. Presented at Hydrology and Water Resources Symposium, Sydney, NSW, November.

Green, J.H., Xuereb, K., Johnson, J., Moore, G. and The, C. (2012b), The Revised Intensity-Frequency Duration (IFD) Design Rainfall Estimates for Australia - An Overview. Presented at Hydrology and Water Resources Symposium, Sydney, NSW, November.

Guttman, N.B., Hosking, J.R.M. and Wallis, J.R. (1993), Regional precipitation quantile values for the continental U. S. computed from L-moments. Journal of Climate, 6: 2326-2340.

Haddad, K. and Rahman, A. (2012a), Regional flood frequency analysis in eastern Australia: Bayesian GLS regression-based methods within fixed region and ROI framework: quantile regression vs parameter regression technique. Journal of Hydrology, 430: 142-161.

Haddad, K. and Rahman, A. (2012b), A Pilot Study on Design Rainfall Estimation in Australia using L-moments and Bayesian Generalised Least Squares Regression: Comparison of Fixed Region, Facets and Region of Influence Approach, EnviroWater Sydney.

Haddad, K., Johnson, F., Rahman, A., Green, J. and Kuczera, G. (2015), Comparing three methods to form regions for design rainfall statistics: Two case studies in Australia. Journal of Hydrology, 527: 62-76.

Haddad, K., Pirozzi, J., McPherson, G., Rahman, A. and Kuczera, G. (2009),'Regional flood estimation technique for NSW: application of generalised least squares quantile regression technique', Proc. 32nd Hydrology and Water Resources Symp., Newcastle, pp: 829-840.

Haddad, K., Rahman, A. and Green, J. (2011), Design Rainfall Estimation in Australia: A Case Study using L moments and Generalized Least Squares Regression, Stochastic Environmental Research & Risk Assessment, 25(6), 815-825.

Hosking, J.R.M. and Wallis, J.R. (1997), Regional Frequency Analysis: An Approach Based on L-Moments. CambridgeUniversity Press, Cambridge, UK. p: 224.

Huff, F.A. and Angel, J.R. (1992), Rainfall Frequency Atlas of the Midwest. Illinois State Water Survey, Champaign, Bulletin 71, 1992.

Hutchinson, M.F. (1998), Interpolation of rainfall data with thin plate smoothing splines - part 2: Analysis of topographic dependence, Journal of Geographic Information and Decision Analysis, 2(2), 152-167.

Hutchinson, M.F. (2007), ANUSPLIN version 4.37 User Guide, The Australian National University, Centre for Resources and Environmental Studies, Canberra.

Hutchinson, M.F. and Xu, T. (2013), ANUSPLIN Version 4.4 User Guide, The AustralianNationalUniversity. FennerSchool of Environment and Society, Canberra, Australia.

Johnson, F., Hutchinson, M.F., The, C., Beesley, C. and Green, J. (2015), Topographic relationships for design rainfalls over Australia. Accepted for publication Journal of Hydrology.

Johnson, F., Xuereb, K., Jeremiah, E. and Green, J. (2012a), Regionalisation of Rainfall Statistics for the IFD Revision Project. Presented at Hydrology and Water Resources Symposium, Sydney, NSW, November 2012.

Johnson, F., Haddad, K., Rahman, A., and Green, J. (2012b), Application of Bayesian GLSR to Estimate Sub Daily Rainfall Parameters for the IFD Revision Project. Presented at Hydrology and Water Resources Symposium, Sydney, NSW, November 2012.

Kjeldsen, T.R. and Jones, D.A. (2009), A formal statistical model for pooled analysis of extreme floods, Hydrology Research, 40(5), 465-480.

Madsen, H., Mikkelsen, P.S., Rosbjerg, D. and Harremoes, P. (2002), Regional estimation of rainfall intensity duration curves using generalised least squares regression of partial duration series statistics. Water Resources Research, 38(11), 1239.

Madsen, H., Arnbjerg-Neilsen, K. and Mikkelsen, P.S. (2009), Update of regional intensity-duration-frequency curves in Denmark: Tendency towards increased storm intensities. Atmospheric Research, 92: 343-349.

Menabde, M., Seed, S. and Pegram, G. (1999), A simple scaling model for extreme rainfall. Water Resources Research, 35: 335-339

Minty, L.J., Meighen, J. and Kennedy, M.R. (1996), Development of the Generalised Southeast Australia Method for Estimating Probable Maximum Precipitation, [Online] HRS Report No. 4, Hydrology Report Series, Bureau of Meteorology, Melbourne, Australia, August 1996, p: 48. Available at: http://www.bom.gov.au/water/designRainfalls/pmp/gsam.shtml

Nandakumar, N., Weinmann, P.E., Mein, R.G. and Nathan, R.J. (1997), Estimation of Extreme Rainfalls for Victoria using the CRCFORGE Method, CRC for Catchment Hydrology.

Pilgrim, DH (ed) (1987) Australian Rainfall and Runoff - A Guide to Flood Estimation, Institution of Engineers, Australia, Barton, ACT, 1987.

Reis Jr., D.S., Stedinger, J.R. and Martins, E.S. (2005), Bayesian GLS regression with application to LP3 regional skew estimation. Water Resour Res. 41, W10419, (1) 1029.

Schaefer, M.G. (1990), Regional analysis of precipitation annual maxima in Washington State, Water Resoures Research, 26(1), 119-131.

The, C., Johnson, F., Hutchinson, M., and Green, J. (2012), Gridding of Design Rainfall Parameters for the IFD Revision Project for Australia. Presented at Hydrology and Water Resources Symposium, Sydney, NSW, November.

The, C., Hutchinson, M., Johnson, F., Beesley, C. and Green, J. (2014), Application of ANUSPLIN to produce new Intensity-Frequency-Duration (IFD) design rainfalls across Australia. Presentated at Hydrology and Water Resources Symposium, Perth, WA, February 2014.

The, C., Beesley, C., Podger, S., Green, J., Hutchinson, M. and Jolly, C. (2015), Very frequent design rainfalls - an enhancement to the new IFDs. Accepted for presentation at Hydrology and Water Resources Symposium, Hobart, TAS, December 2015.

Walland, D.J., Meighen, J., Xuereb, K.C., Beesley, C.A. and Hoang, T.M.T. (2003), Revision of the Generalised Tropical Storm Method for Estimating Probable Maximum Precipitation [Online], HRS Report No.8, Hydrology Report Series, Bureau of Meteorology Melbourne, Australia, available at: http://www.bom.gov.au/water/designRainfalls/hrs8.shtml.

Wang, Q.J. (1997), LH moments for statistical analysis of extreme events. Water Resources Research, 33(12), 2841-2848.

Wang, Q.J. (1998), Approximate goodness-of-fit tests of fitted generalised extreme value distributions using LH moments. Water Resources Research, 34(12), 3497-3502.

World Meteorological Organisation (1986), 'Manual for Estimation of Probable Maximum Precipitation'. Second Edition. Operational Hydrology Report No.1, WMO - No.332, Geneva.

Xuereb, K. and Green, J. (2012), Defining Independence of Rainfall Events with a Partial Duration Series Approach. Presented at Hydrology and Water Resources Symposium, Sydney, NSW, November 2012.

Xuereb, K.C., Moore, G.J., and Taylor, B.F. (2001), Development of the Method of Storm Transposition and Maximisation for the West Coast of Tasmania. HRS Report No.7, Hydrology Report Series, Bureau of Meteorology, Melbourne, Australia, January 2001.

Chapter 4. Areal Reduction Factors

Phillip Jordan, Rory Nathan, Scott Podger, Mark Babister, Peter Stensmyr, Janice Green

Chapter Status	Final
Date last updated	14/5/2019

4.1. Introduction

Design rainfall information for flood estimation generally is made available in the form of rainfall Intensity Frequency Duration (IFD) date (Book 2, Chapter 3) that relates to specific points in a catchment rather than to the whole catchment area. However, most flood estimates are required for catchments that are sufficiently large that design rainfall intensities at a point are not representative of the areal average rainfall intensity across the catchment. The ratio between the design values of areal average rainfall and point rainfall, computed for the same duration and Annual Exceedance Probability (AEP), is called the Areal Reduction Factor (ARF). This allows for the fact that larger catchments are less likely than smaller catchments to experience high intensity storms simultaneously over the whole of the catchment area.

It should be noted that the ARF provides a correction factor between the catchment rainfall depth (for a given combination of AEP and duration) and the mean of the point rainfall depths across a catchment (for the same AEP and duration combination). Applying an ARF is a necessary input to computation of design flood estimates from a catchment model that preserves a probability neutral transition between the design rainfall and the design flood characteristics. The ARF merely influences the average depth of rainfall across the catchment, it does not account for variability in the spatial and/or space-time patterns of its occurrence over the catchment.

Recommendations for ARF to be adopted are provided in Book 2, Chapter 4, Section 3.

4.2. Derivation of Areal Reduction Factors

The method adopted for the derivation of areal reduction factors is a modified version of Bell's method (Bell, 1976; Siriwardena and Weinmann, 1996). ARFs derived by this method have been widely used for some time and have been shown to provide effective estimates of areal design rainfall for deriving rainfall-based flood frequency curves. The method allows the dependence between ARF and catchment area to be determined, as well as the variation with AEP. The method has been applied with data collected and processed for the estimation of IFDs; they thus provide for a more comprehensive and consistent set of estimates than were prepared by Jordan et al. (2013).

The modified Bell's method involves defining hypothetical circular catchments in areas with sufficient data and creating an areal rainfall time series for each catchment by weighting point rainfall values based on Thiessen polygon areas (or an equivalent weighting method). The frequency quantiles calculated from the areal rainfall time series are divided by the weighted point frequency quantiles for the sites within the catchment, yielding an ARF estimates for the given catchment area and a range of AEPs. Once ARFs have been calculated for the required catchment areas, durations and AEPs for as many locations as possible, they are averaged across these attributes and an equation is fitted to provide a prediction model for the selected region.

The adopted methodology is described in more detail in Podger et al. (2015a), Podger et al. (2015b) and Stensmyr et al (2014).

4.3. Areal Reduction Factor Recommendations

4.3.1. Areal Reduction Factors for Catchments up to 30 000 km^2, Durations up to 7 days and Events More Frequent than 0.05% AEP

Areal Reduction Factors (ARF) for catchments with areas up to 30 000 km^2, for durations up to and including 168 hours (7 days) and for AEP more frequent than 0.05% (1 in 2000) Annual Exceedance Probability (AEP) are recommended based on the values derived by Podger et al. (2015a), Podger et al. (2015b) and Stensmyr et al (2014). This guidance should be adopted unless rigorous subsequent research or catchment-specific investigations have been conducted to define a more appropriate, locally specific, ARF.

The design areal rainfall to be applied in a design flood simulation is the average rainfall over the total catchment area to the point of interest. Consequently, the ARF should be computed for the total catchment upstream of each location of interest where a design flood estimate is required. The ARF should not be computed independently for each subarea in a runoff-routing model of the catchment of interest, as this would result in systematic overestimation of catchment rainfalls and simulated design flood hydrographs.

The ARF to be applied to design rainfall is a function of the total area of the catchment, the duration of the design rainfall event and it's AEP. The ARF should be computed using the relevant procedure described in Table 2.4.1.

If the duration of interest is greater than 12 hours, Equation (2.4.2) will be required as part of the calculation procedure and the coefficients of Equation (2.4.2) vary regionally across Australia. The applicable ARF region should be selected by referring to Figure 2.4.1. Where a catchment overlaps the boundary between regions, the ARF should be selected for the region that has the largest overlap with the boundary of the catchment. The coefficients to be applied with Equation (2.4.2) should be selected from the appropriate region from Table 2.4.2.

Table 2.4.1. ARF Procedure for Catchments Less than 30 000 km^2 and Durations up to and Including 7 Days

Catchment Area	Duration ≤ 12 hours	Duration Between 12 and 24 hours	Duration ≥ 24 Hours (1 Day) and ≤ 7 Days (168 hours)
≤ 1 km^2	ARF = 1		
Between 1 and 10 km^2	1. Compute ARF(10 km^2) using Equation (2.4.1) for area = 10 km^2 and selected duration 2. Interpolate ARF for catchment area and selected	1. Compute ARF(24 hr, 10 km^2) using Equation (2.4.2) for area = 10 km^2 and duration = 1440 min 2. Compute ARF(12 hr, 10 km^2) using Equation (2.4.1) for	1. Compute ARF(10 km^2) using Equation (2.4.2) for area = 10 km^2 2. Interpolate ARF for catchment area and selected duration using Equation (2.4.4)

<table>
<tr><th>Catchment Area</th><th>Duration ≤ 12 hours</th><th>Duration Between 12 and 24 hours</th><th>Duration ≥ 24 Hours (1 Day) and ≤ 7 Days (168 hours)</th></tr>
<tr><td></td><td>duration using Equation (2.4.4)</td><td>area = 10 km^2 and duration = 720 min
3. Interpolate ARF(10 km^2) for selected duration using Equation (2.4.4)
4. Interpolate ARF for catchment area and selected duration using Equation (2.4.3)</td><td></td></tr>
<tr><td>Between 10 and 1000 km^2</td><td>• Compute ARF using Equation (2.4.1) for catchment area and selected duration</td><td rowspan="2">1. Compute ARF(24 hr) using Equation (2.4.2) for catchment area and duration = 1440 min
2. Compute ARF(12 hr) using Equation (2.4.1) for catchment area and duration = 720 min
3. Interpolate ARF for selected duration using Equation (2.4.3)</td><td rowspan="2">• Compute ARF using Equation (2.4.2) for catchment area and selected duration</td></tr>
<tr><td>Between 1000 and 30 000 km^2</td><td>Generalised equations not applicable</td></tr>
<tr><td>>30 000 km^2</td><td colspan="3">Generalised equations not applicable. It is recommended that the practitioner should perform a frequency analysis of catchment rainfall data for the catchment of interest.</td></tr>
</table>

Notes on Table 2.4.1:

- Equation (2.4.1), Equation (2.4.2) and Equation (2.4.3) require the selected duration to be provided in minutes.

- There has been limited research on ARF applicable to catchments that are less than 10 km^2. The recommended procedure is to adopt an ARF of unity for catchments that are less than 1 km^2, with an interpolation to the empirically derived equations for catchments that are between 1 and 10 km^2 in area (refer to Equation (2.4.3)).

- The ARF equations derived by Podger et al. (2015a), Podger et al. (2015b) and Stensmyr et al (2014) were derived for the 50% to 1% AEPs. Although these have been recommended for use for a wider range of AEP, (out to 0.05% AEP), further verification is ongoing on the validity of this approach. As a result, the coefficients of Equation (2.4.2) (from Table 2.4.2) and/or the regional boundaries (refer to Figure 2.4.1) may be revised.

Short Duration ARF Equation

Equation (2.4.1)

$$ARF = Min\left\{1, \left[\begin{array}{c} 1 - 0.287(Area^{0.265} - 0.439\log_{10}(Duration))\,.\,Duration^{-0.36} \\ +2.26 \text{ x } 10^{-3} \text{ x } Area^{0.226}\,.\,Duration^{0.125}(0.3 + \log_{10}(AEP)) \\ +0.0141 \text{ x } Area^{0.213} \text{x } 10^{-0.021\frac{(Duration-180)^2}{1440}}\ (0.3 + \log_{10}(AEP)) \end{array}\right]\right\}$$

where Area is in km^2, Duration is in minutes and AEP is a fraction (between 0.5 and 0.0005).

Long Duration ARF Equation

$$ARF = Min\left\{1, \left[\begin{array}{c} 1 - a(Area^{b} - c\log_{10}Duration)Duration^{-d} \\ +eArea^{f}Duration^{g}(0.3 + \log_{10}AEP) \\ +h10^{iArea\frac{Duration}{1440}}(0.3 + \log_{10}AEP)] \end{array}\right]\right\} \quad (2.4.2)$$

where Area is in km^2, Duration is in minutes and AEP is a fraction (between 0.5 and 0.0005).

Interpolation Equation for Durations between 12 and 24 hours

$$ARF = ARF_{12hour} + (ARF_{24hour} - ARF_{12hour})\frac{(Duration - 720)}{720} \quad (2.4.3)$$

where Duration is in minutes.

ARF Equation for Areas between 1 and 10 km^2

$$ARF = 1 - 0.6614\left(1 - ARF_{10km^2}\right)\left(A^{0.4} - 1\right) \quad (2.4.4)$$

where Area is in km^2

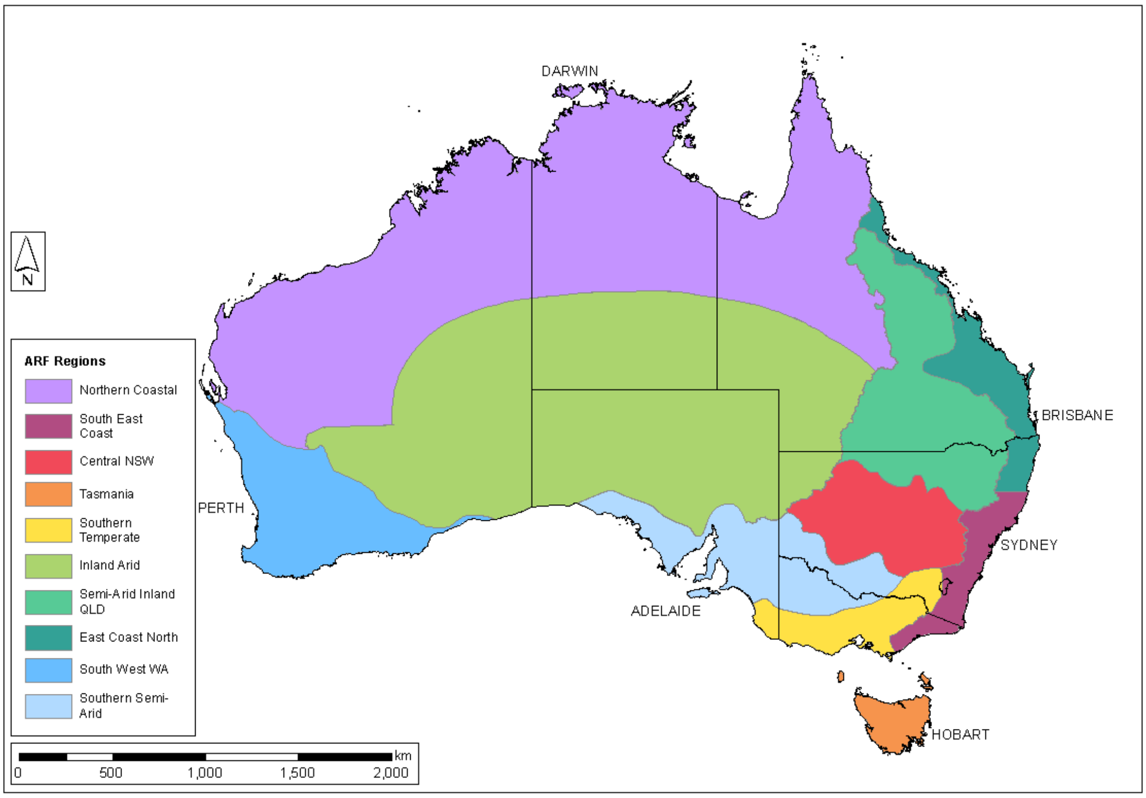


Figure 2.4.1. Area Reduction Factors Regions for Durations 24 to 168 Hours

Table 2.4.2. ARF Equation (2.4.2) Coefficients by Region for Durations 24 to 168 hours Inclusive

Region[1]	a	b	c	d	e	f	g	h	i
East Coast North	0.327	0.241	0.448	0.36	0.00096	0.48	-0.21	0.012	-0.0013
Semi-arid Inland Queensland	0.159	0.283	0.25	0.308	7.3E-07	1	0.039	0	0
Tasmania	0.0605	0.347	0.2	0.283	0.00076	0.347	0.0877	0.012	-0.00033
South-West Western Australia	0.183	0.259	0.271	0.33	3.85E-06	0.41	0.55	0.00817	-0.00045
Central New South Wales	0.265	0.241	0.505	0.321	0.00056	0.414	-0.021	0.015	-0.00033
South-East Coast	0.06	0.361	0	0.317	8.11E-05	0.651	0	0	0
Southern Semi-arid	0.254	0.247	0.403	0.351	0.0013	0.302	0.058	0	0
Southern Temperate	0.158	0.276	0.372	0.315	0.000141	0.41	0.15	0.01	-0.0027
Northern Coastal	0.326	0.223	0.442	0.323	0.0013	0.58	-0.374	0.013	-0.0015

Region[1]	a	b	c	d	e	f	g	h	i
Inland Arid	0.297	0.234	0.449	0.344	0.001 42	0.216	0.129	0	0

[1]These values are provided on the ARR Data Hub for the relevant region when queried (Babister et al. (2016), accessible at http://data.arr-software.org/)

4.3.2. Events That are Rarer than 0.05% Annual Exceedance Probability

The ARF equations are only recommended for use for events more frequent than 0.05% AEP. For more extreme events, the procedures recommended in Book 8, Chapter 3, Section 5 should be used to determine catchment average design rainfall depths. The interpolation procedure recommended in Book 8, Chapter 3, Section 5 uses the catchment average design rainfall depth for 0.05% AEP, which would be calculated using the average of the point design intensities across the catchment multiplied by the ARF estimates recommended above and the PMP depth, which is already estimated as a catchment average value.

4.3.3. Catchments with Areas Greater than 30 000 km^2

The largest (circular) catchments used by Podger et al. (2015a) to estimate ARF were 30 000 km^2 and which set the upper limit of applicability of the ARF equations. As the catchment area increases beyond 30 000 km^2, it becomes increasingly likely that storm events would only influence part of the overall catchment area, which increases the uncertainty associated with adjusting point design intensities using an ARF.

Design rainfall depths for catchments larger than 30 000 km^2 should be derived from frequency analysis of catchment average rainfall depths over the specific catchment. The design rainfall depths from the catchment-specific frequency analysis should be checked by dividing them by the average of the point rainfall depths from point IFD analysis for the catchment (Bureau of Meteorology, 2013) to infer the ARF for the catchment for each rainfall duration and AEP. It would be expected that for a catchment larger than 30 000 km^2, the ARF inferred from this check for each duration and AEP should be less than the ARF calculated from the regional method Equation (2.4.2) for the corresponding duration and AEP combination. It would also be expected that the inferred ARF (for a given AEP) should increase with rainfall duration.

For catchments larger than 30 000 km^2, it becomes increasingly likely that rainfall events that would give rise to flooding would be concentrated in one part of the catchment. For catchments larger than 30 000 km^2 it is strongly recommended that partial area storms are explicitly modelled (using Monte Carlo or other joint probability approaches). Explicit modelling of partial area storms should also be considered for catchments in the range between 5 000 km^2 and 30 000 km^2.

4.4. Worked Example

A worked example for the calculation of the ARF and the areal design rainfall for a catchment in Queensland is provided in Book 2, Chapter 6, Section 5.

4.5. Limitations and Recommended Further Research

The ARF equations developed in Australia have been derived using data driven and empirical methods, with limited theoretical underpinning. ARF values for a particular catchment would derive from a combination of the mixture of storm types causing heavy rainfall within a region, the direction and speed of movement of those storms and the spatial and temporal characteristics of those storms. Analysis by a hydrometeorologist of the prevalence of different storm types within different parts of Australia and the advection, temporal and spatial characteristics of those storms is likely to provide an understanding of the causes of variations in ARF. Such understanding is difficult to infer directly, on its own, from the empirically derived ARF equations that are currently recommended for use in Australia. It is recommended that hydrometeorologists are engaged to investigate the causes of variations in ARF.

Once the hydrometeological analysis recommended above has been undertaken, the outcomes of that work may enable further research and improvements in the following specific areas:

- Clarification of how well the ARFs derived using an empirical method such as Bell's method, compare with those derived from a suitable theoretical method that may better account for hydrometeorological understanding of the drivers of variability in ARFs.
- There are some areas within each of the regions where the ARF values determined empirically for the circular catchments demonstrated a trend toward being larger or smaller than obtained from the ARF equations fitted to the mean ARF values from all circular catchments within the region for a given area, duration and AEP. Hydrometeorological understanding may enable definition of smaller sub-regions, combining of existing regions (with the existing regions largely defined using state and territory boundaries), or definition of new regions in order to reduce the uncertainty introduced by this variability.
- Seasonality was found to be a significant driver of ARFs in Western Australia but has not been investigated for other parts of Australia. Hydrometeorological understanding may guide the regions where seasonal dependence in ARF would be likely, the start and end dates of seasons and how transition periods between seasons should be handled.

It is recommended that after an appropriate study has been undertaken to determine the hydrometeorological causes of variations in ARF that further studies are then scoped and prioritised according to areas where the hydrometeorological causes can be best exploited to reduce residual uncertainty in ARFs.

4.6. Recommended Further Research

4.6.1. Areal Reduction Factors

The ARF equations developed in Australia have been derived using data driven and empirical methods, with limited theoretical underpinning. ARF values for a particular catchment would derive from a combination of the mixture of storm types causing heavy rainfall within a region, the direction and speed of movement of those storms and the spatial and temporal characteristics of those storms. Analysis by a hydrometeorologist of the prevalence of different storm types within different parts of Australia and the advection, temporal and spatial characteristics of those storms is likely to provide an understanding of the causes of variations in ARF. Such understanding is difficult to infer directly, on its own, from the empirically derived ARF equations that are currently recommended for use in Australia. It is recommended that hydrometeorologists are engaged to investigate the causes of variations in ARF.

Once the hydrometeological analysis recommended above has been undertaken, the outcomes of that work may enable further research and improvements in the following areas:

- how well the ARFs derived using an empirical method such as Bell's method, compare with those derived from a suitable theoretical method that may better account for hydrometeorological understanding of the drivers of variability in ARFs.

- There are some areas within each of the regions where the ARF values determined empirically for the circular catchments demonstrated a trend toward being larger or smaller than the fitted ARF equations, which were fitted to the mean ARF values from all circular catchments within the region for a given area, duration and AEP. Hydrometeorological understanding may enable definition of smaller sub-regions, combining of existing regions (with the existing regions largely defined using state and territory boundaries), or definition of new regions in order to reduce the uncertainty introduced by this variability.

- Seasonality was found to be a significant driver of ARFs in Western Australia but has not been investigated for other parts of Australia. Hydrometeorological understanding may guide the regions where seasonal dependence in ARF would be likely, the start and end dates of seasons and how transition periods between seasons should be handled.

It is recommended that after an appropriate study has been undertaken to determine the hydrometeorological causes of variations in ARF that further studies are then scoped and prioritised according to areas where the hydrometeorological causes are best exploited to reduce residual uncertainty in ARFs.

4.7. References

Babister, M., Trim, A., Testoni, I. and Retallick, M. 2016. The Australian Rainfall and Runoff Datahub, 37th Hydrology and Water Resources Symposium Queenstown NZ

Bell, F.C. (1976), The areal reduction factors in rainfall frequency estimation. Natural Environmental Research Council (NERC), Report No. 35, Institute of Hydrology, Wallingford, U.K.

Bureau of Meteorology (2013), Revised Design Rainfall Intensity Frequency Duration Estimates for Australia. Available at: http://www.bom.gov.au/water/designRainfalls/revised-ifd/, Accessed [July 2015].

Jordan, P., Weinmann, P.E., Hill, P. and Wiesenfeld, C. (2013), Collation and Review of Areal Reduction Factors from Applications of the CRC-FORGE Method in Australia, Final Report, Australian Rainfall and Runoff Revision Project 2: Spatial Patterns of Rainfall, Engineers Australia, Barton, ACT.

Podger, S., Green, J., Jolly, C., The, C. and Beesley, C. (2015a),Creating long duration areal reduction factors for the new Intensity-Frequency-Duration (IFD) design rainfalls, Proc. Engineers Australia Hydrology and Water Resources Symposium, Hobart, Tasmania, Australia.

Podger, S., Green, J., Stensmyr, P. and Babister, M. (2015b), Combining long and short duration areal reduction factors, Proc. Engineers Australia Hydrology and Water Resources Symposium, Hobart, Tasmania, Australia.

Siriwardena, L. and P.E. Weinmann, (1996), Derivation of areal reduction factors for design rainfalls in Victoria, 96(4), 60.

Stensmyr, P., Babister, M. and Retallick, M. (2014), Australian Rainfall and Runoff Revision Project 2: Spatial Patterns of Rainfall: Stage 2 Report, Short Duration Areal Reduction Factors, ARR Report Number P2/S2/019, ISBN 978-085825-9614.

Chapter 5. Temporal Patterns

Mark Babister, Monique Retallick, Melanie Loveridge, Isabelle Testoni, Scott Podger

Chapter Status	Final
Date last updated	14/5/2019

5.1. Introduction

The majority of hydrograph estimation methods used for flood estimation require a temporal pattern that describes how rainfall falls over time as a design input. Traditionally a single burst temporal pattern has been used for each rainfall event duration. The use of a single pattern has been questioned for some time (Nathan and Weinmann, 1995) as the analysis of observed rainfall events from even a single pluviograph shows that a wide variety of temporal patterns is possible.

The importance of temporal patterns has increased as the practice of flood estimation has evolved from peak flow estimation to full hydrograph estimation. There has been a strong move toward storage-based mitigation solutions in urban catchments which require realistic temporal patterns that reproduce total storm volumes as well as the temporal distribution of rainfall within the event.

This chapter discusses use of temporal patterns for design flood estimation where a fixed temporal pattern is applied over the entire catchment. Book 2, Chapter 6 discusses the more complex case of space-time patterns.

This chapter is structured as follows:

- Book 2, Chapter 5, Section 2 – discusses fundamental temporal pattern concepts and how the concept of using ensembles of temporal pattern developed;
- Book 2, Chapter 5, Section 3 – discusses the storm database that was used to develop temporal patterns;
- Book 2, Chapter 5, Section 4 – describes the estimation of pre-burst rainfall;
- Book 2, Chapter 5, Section 5 – describes the development of ensembles of design temporal patterns;
- Book 2, Chapter 5, Section 6 – discusses areal temporal patterns;
- Book 2, Chapter 5, Section 8 – discusses possible effects of climate change on temporal patterns; and
- Book 2, Chapter 5, Section 9 – discusses the application of design temporal patterns;

5.2. Temporal Pattern Concepts

5.2.1. Storm Components

In order to properly understand the temporal patterns it is necessary to understand the components of a storm event and how they relate to Intensity Frequency Duration Data (IFD) and catchment response. Figure 2.5.1 depicts a typical storm pattern and how components of the storm can be characterised. It is important to note the components can be characterised either by IFD relationships or by catchment response and are highly dependent on the definitions used. The components of a storm include:

- *antecedent rainfall* - is rainfall that has fallen before the storm event and is not considered part of the storm but can affect catchment response. This is not considered in this chapter but is introduced for completeness.

- *pre-burst rainfall* - is storm rainfall that occurs before the main burst. With the exception of relatively frequent events, it generally does not have a significant influence on catchment response but is very important for understanding catchment and storage conditions before the main rainfall burst.

- *the burst* - represents the main part of the storm but is very dependent on the definition used. Bursts have typically been characterised by duration. The burst could be defined as the *critical rainfall burst* , the rainfall period within the storm that has the lowest probability, or *the critical response burst* that corresponds to the duration which produces the largest catchment response for a given rainfall Annual Exceedance Probability (AEP).

- *post-burst rainfall* - is rainfall that occurs after the main burst and is generally only considered when aspects of hydrograph recession are important. This could be for drawing down a dam after a flood event or understanding how inundation times affect flood recovery, road closures or agricultural land.

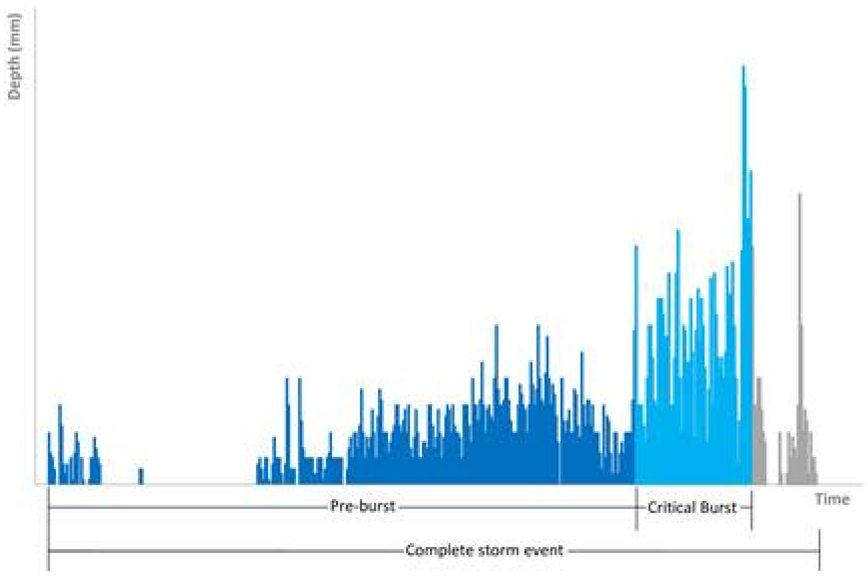


Figure 2.5.1. Typical Storm Components

If the critical response burst is not the same as the critical rainfall burst then the critical response burst is either:

- part of a longer critical rainfall burst; or

- a storm that contains rarer shorter duration bursts.

Rarer shorter duration bursts within a burst are typically called embedded bursts and can cause problems in modelling as, while the intention may be to assess the catchment response to a burst of a defined duration and probability, the response to a rarer shorter duration burst is also being assessed.

The distinction between a burst and a complete storm is important as complete storms are used for calibration and bursts are typically used for design. Though this difference is less important for catchments with long duration responses, as the bursts typically represent nearly the entire storm event.

5.2.2. Pattern Variability

It has been well recognised that temporal patterns exhibit significant variability between rainfall events of similar magnitude, and the adopted pattern can have a significant effect on the estimated peak flow. Askew (1975), Milston (1979), Brown (1982), Wood and Alvarez (1982), Cordery et al. (1984) give examples of differences of up to 50% in flood peaks resulting from different assumed temporal patterns. Ball (1994) discusses how catchment peak flow tends to increase as the variability of the pattern increases.

Pilgrim et al. (1969) describe the variability of patterns:

“In nature, a wide range of patterns is possible. Some storms have their period of peak intensity occur early, while other storms have the peak rainfall intensity occur towards the end of the storm period and a large number have a tendency for the peak to occur more or less centrally.”

Figure 2.5.2 depicts two very different storms from Sydney Observatory Hill that have similar IFD characteristics from 15 min to 12 hrs and a 2 hr critical rainfall burst, however on other criteria they are very different. The first pattern has 182 mm before the 2 hr burst, while the second pattern has 16 mm. For the 2nd event the commencement of runoff will be closely aligned with the main burst while the former event is likely to have significant runoff prior to the main burst. The rainfall burst also exhibits significant variability, Figure 2.5.3 presents the temporal patterns (as dimensionless mass curves) of ten 2 hr bursts of similar probability that show the variability described by Pilgrim et al. (1969).

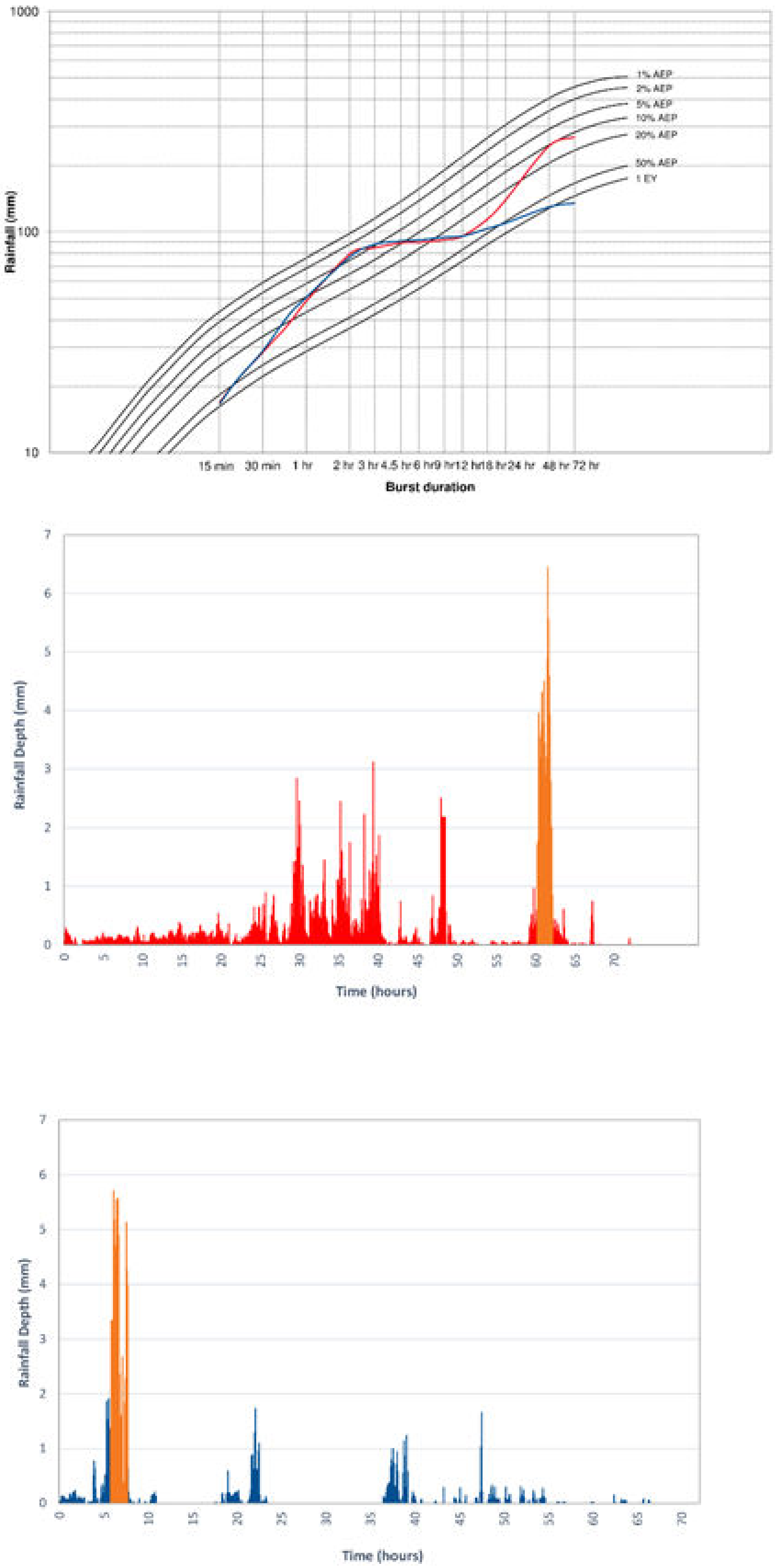


Figure 2.5.2. Two Different Storm Events with Similar Intensity Frequency Duration Characteristics (Sydney Observatory Hill) – Two Hyetographs plus Burst Probability Graph

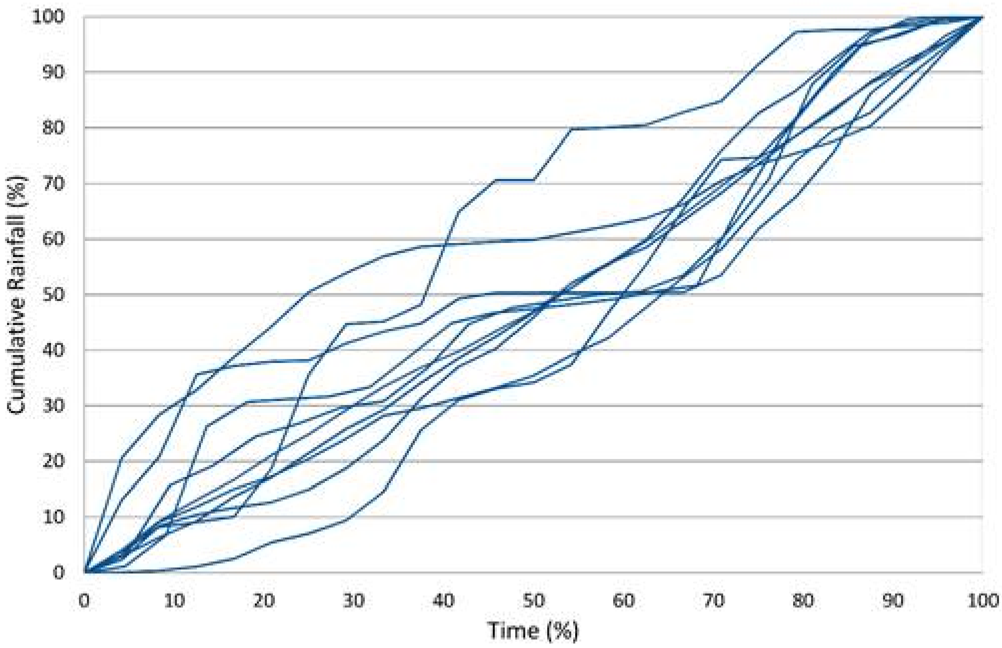


Figure 2.5.3. Ten 2 hr Dimensionless Mass Curves

Most of the historical research on temporal patterns has assumed that the central tendency of the pattern is more important than the variability, with the aim of producing a typical, representative or median pattern. French (1985) describes how a pattern can be considered a two dimensional quantity with most methods breaking the pattern into two manageable one dimensional quantities that describe the magnitude of the element and the order of the elements. This can be described as a rank order vector and a decay curve that describes how the magnitude decreases between ranks.

Monte Carlo and ensemble modelling techniques try to overcome the problems associated with this simplification by using an ensemble of temporal patterns. A short history of temporal patterns helps explain that, while the complexity of temporal patterns has been well understood for a long time, it has been difficult to produce a simple set of design patterns.

5.2.3. History of Design Temporal Pattern Development

Australian Rainfall and Runoff 1977 (Pattison, 1977; Institution of Engineers, Australia., 1977) notes that there has historically been a slow change in the application of temporal distributions of rainfall in hydrologic studies from more arbitrary approaches to approaches that reproduce the IFD characteristics (Keifer and Chu (1957) and Huff (1967)). The modern interpretation is the duration independent storm (Varga et al., 2009). These approaches reproduce the IFD loading and at best can reproduce aspects of a single storm. Such approaches have some applicability for peak flow estimation but cannot reproduce realistic hydrographs.

Other early approaches included in the development of patterns based on observed pluviograph records were for complete storms rather than bursts. Such approaches require the scaling of complete storms. Other methods have allowed the arbitrary rearrangement of patterns to maximise peak discharge. Such approaches are very far removed from concept of probability neutrality.

The development of the Average Variability Method by Pilgrim et al. (1969) and Pilgrim and Cordery (1975) analysed the variability of patterns by separating the analysis of the magnitude or the ranks from the analysis of rank order. This approach is only applicable to bursts and was applied in ARR 1977 and in a very detailed way in ARR 1987. A variation of this approach was developed by Hall and Kneen (1973). During the finalisation of ARR 1987 some problems where found with the use of the patterns and extensive testing was carried out which resulted in some changes to remove some embedded bursts, including some arbitrary rank order changes. It has been assumed that an AVM pattern would preserve probability neutrality. Single burst per duration AVM patterns have been extensively used since ARR 1987 and appear to have performed reasonable well for peak flow estimation.

By their very nature AVM patterns do preserve the average rank magnitudes. Figure 2.5.4 plots the magnitude or decay curve for the ten dimensionless curves in Figure 2.5.3 and the AVM curve from these events and all 10 events that would be considered in a AVM analysis.

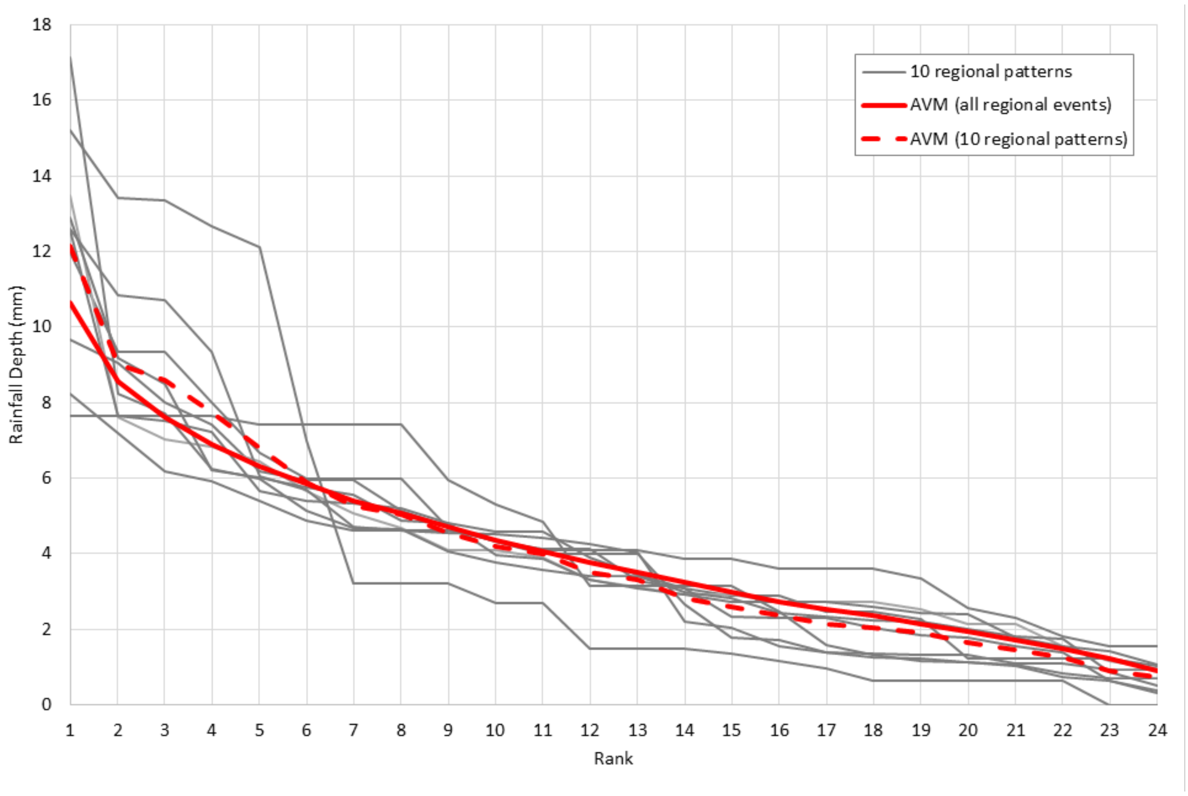


Figure 2.5.4. Decay Curve of Ten Dimensionless Patterns and AVM Patterns

The issues that have arisen have been discussed in Retallick et al. (2009):

- Specific durations tend to dominate;
- Big changes in pattern and corresponding peak flow occur at some region boundaries;
- The burst only approach has made conversion of observed losses to burst losses difficult;
- The design patterns often contain significant embedded bursts; and
- Filtering to remove embedded bursts that was recommended in ARR 1987 (Pilgrim, 1987) has had a mixed uptake and sometimes produces unrealistically long critical durations.

The AVM approach also works best when there is a dominant typical pattern shape. As part of the AVM approach the average and standard deviation of the rank of each period is calculated, when there is no dominant pattern these averages can be very similar and the resultant AVM pattern need not bare any resemblance to any of the observed patterns.

The complexity of producing a representative or median pattern has led many practitioners to question the concept and ask whether it is better to specifically account for this variability by modelling an ensemble of temporal patterns.

The problems with the AVM method and other median or representative patterns is that it assumes the variability of actual patterns is much less important than their central tendency. Such an approach does not account for how temporal patterns interact with catchments to produce peak flows and hydrographs. The response can be very catchment-specific, and there is no guarantee that a representative pattern will produce the medium response from an ensemble of patterns that properly captures the variability of observed patterns. These problems can become more pronounced when changes are made to the catchment response or storage characteristics. Phillips and Yu (2015) examined the impact of storage on an ensemble of events for the lot, neighbourhood and subcatchment scale.

It is unclear who first proposed the concept of running an ensemble of temporal patterns to account for the variability of patterns. Practitioners have a long history of comparing the peak flow estimates from design patterns to estimates from patterns extracted from real storms that occurred on that catchment of interest. The development of Monte Carlo methods (Hoang et al., 1999; Rahman et al., 2002; Weinmann et al., 2000) that stochastically sample from observed events based on complete storms and embedded bursts of maximum intensity ("storm cores") is well documented and has helped highlight the value of using ensembles. Others, such as Nathan et al. (2002) have used ensembles of storm bursts based on observed point and areal storms to facilitate their use directly with design IFD information. The development of the Bureau of Meteorology significant storm database (Meighen and Kennedy, 1995) enabled practitioners to easily test ensembles of areal patterns and how they affected other flood characteristics; examples include Webb McKeown and Associates (2003). While there is a long history of testing ensembles of temporal patterns, Sih et al. (2008) is the first example of detailed testing of ensemble patterns as a design input outside a Monte Carlo environment. This long development history of ensemble simulation can probably be explained by the fact that the concept could not be confirmed until rigorous Monte Carlo techniques became available for validation.

Parallel to the development of ensemble and Monte Carlo approaches, practitioners have become concerned with using burst patterns where complete storm volume is important. Rigby and Bannigan (1996) suggest the entire burst approach needed to be reviewed and design storms needed to replace design bursts. For the Wollongong area Rigby and Bannigan (1996) demonstrated that historically most short duration events were embedded in longer duration events. They recommended that short duration events could be embedded in a 24 hour event of the same probability. They particularly cautioned against using bursts on catchments with significant natural or man-made storages. Phillips et al. (1994) had found similar problems in the upper Parramatta River and suggested embedded storms were more realistic, and that basin storages would be underestimated with a burst approach unless the embedded nature of events was factored into the starting volumes. Rigby et al. (2003) extended the earlier work to include guidance on using the embedded design storms. Roso and Rigby (2006) recommended a storm based approach be used when there are significant storages or diversions present in the catchment. Kuczera et al. (2003), inspired by Rigby and Bannigan (1996), explored basin performance using a theoretical catchment at Observatory Hill, Sydney in a continuous simulation approach and found similar problems with peak flow being underestimated by a similar amount when storages were present. All of these studies were based on catchments less than 110 km^2 that are close to Sydney.

5.3. Storm Database

As part of the IFD revision (Book 2, Chapter 3) the Bureau of Meteorology produced a quality controlled pluviograph database, containing 2280 stations with more than 8 station years (refer toBook 2, Chapter 3 and Book 1, Chapter 4). The average station record length is 25 years, with a combined record length of 57 000 years. A total of 754 of the stations are owned by the Bureau of Meteorology and the other 1526 are owned by other data agencies throughout Australia. This data was provided by the Bureau of Meteorology, who undertook quality controlling to identify suspect data, such as missing data, accumulated totals and time shifts (Green et al., 2011). All station data was provided as time series data sampled at a 5 minute time step. In addition to the rainfall depths, each data point had a quality code indicating the quality of the data. Figure 2.5.5 below depicts the geographical distribution of the pluviographs across Australia. Each station is represented as a circle, with the size of the circle indicating the record length of the station.

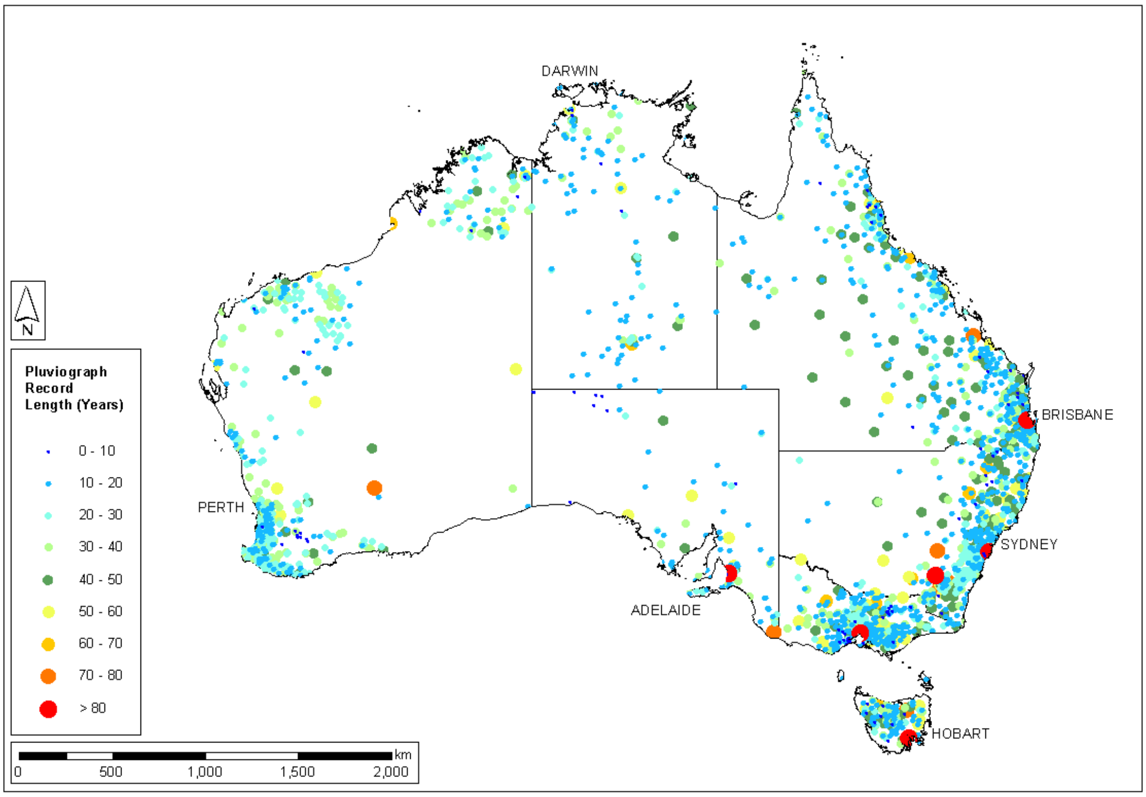


Figure 2.5.5. Pluviograph Stations Record Lengths

Table 2.5.1 presents the maximum number stations available in any year for decadal periods. Table 2.5.1 demonstrates that most of the pluviograph record is from 1960 onwards. Though overall the data comes from a period that starts before 1900, over half of the data is from the period from 1993 onwards.

Table 2.5.1. Number of Pluviographs by Decade

Decade	Maximum Number of Pluviographs in any Year
<1900	2
1900-1940	24
1940-1960	154
1960-1970	392
1970-1980	923
1980-1990	1273
1990 -2000	1926
2000-2010	1837[a]

[a]Not all data was available from 2009 onwards

As shown in Figure 2.5.5 the highest density of pluviograph stations is typically found along coastal areas of Australia around key population centres. In Figure 2.5.6, the pluviograph stations are seen to be clustered around urban areas, such as Sydney, Wollongong, Melbourne and Canberra. Less data is available in central Australia, with the exception of the Alice Springs area. Large areas of central Western Australia contain no data.

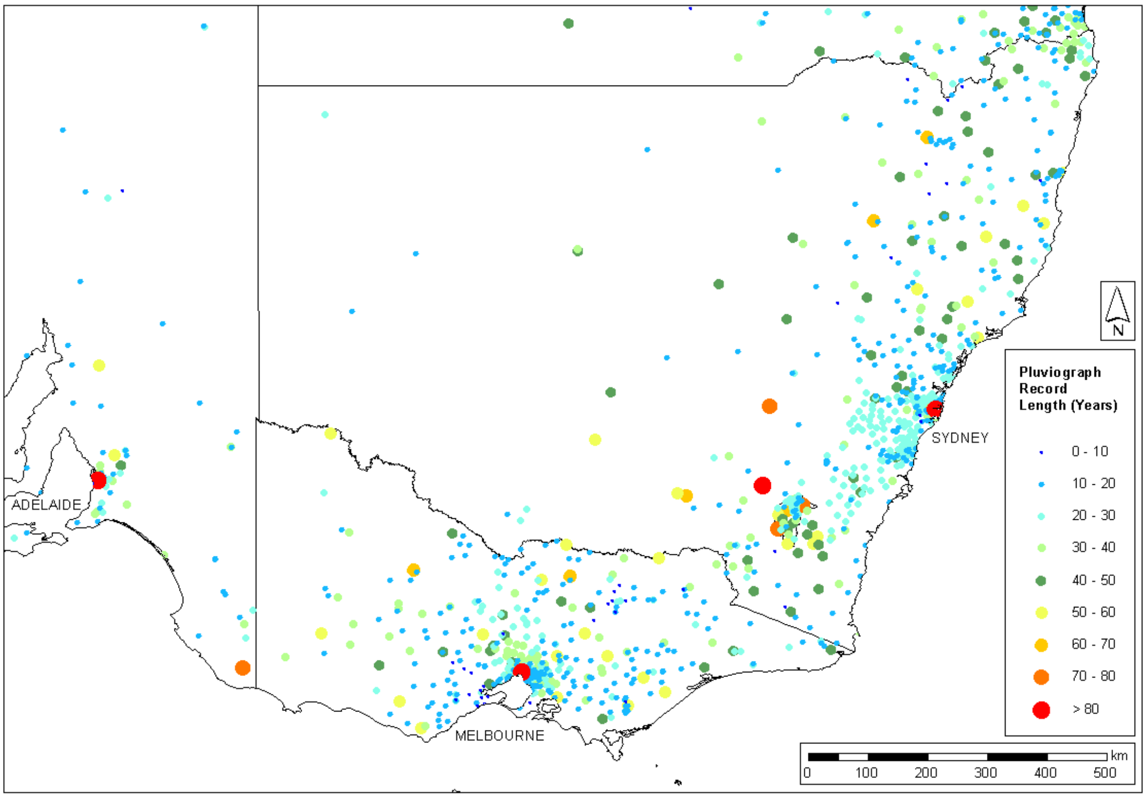


Figure 2.5.6. Pluviograph Stations used Throughout South-Eastern Australia with Record Lengths

5.3.1. Data Quality

Where a single station is being analysed it is important that it has a reasonably long record (preferably longer than 30 years) and only a small percentage of missing data. However, for studies that pool data the quality of the data is a bigger issue than record length. It is however, important to remember that most of the pluviograph data is from the last few decades.

The pluviograph database contains a significant number of events with long periods of apparently uniform rainfall. While some of this could be from events with relatively uniform rainfall, most appears to be the result of disaggregating accumulated rainfall totals uniformly. Even if rainfall is uniform, a tipping bucket rain gauge recording at 5 minutes will only record uniform rainfall over an extended period if the depth in each five minute period is equal or slightly larger than an integer multiple of the tipping bucket capacity. It was concluded that most of the periods of uniform rainfall are probably caused by digitisation and resampling at different time steps. Events with large periods of uniform rainfall were disregarded, while events were kept if the uniform period was only a small portion of the entire event.

In addition, several other issues with the data quality were found. Some records contained significant periods of missing data. There were also periods of interpolated data, where several data points were indicated as interpolated from a later point, presumably at the end of an event. Events where a significant part of the rainfall was interpolated were excluded from further analysis. There were also sections where the rainfall was uniform over many intervals within an event, indicating that the data points were interpolated, even though the quality control value did not specify that to be the case. Since storms generally do not have uniform rainfall at the local scale, events that had a significant part of their total rainfall depth occurring in consecutive identical intervals were also excluded.

5.3.2. Event Selection and Analysis

Once the data quality checking was complete, all events with a burst greater than 1 Exceedance per Year (EY) using the 2013 IFD and which were not flagged with problems were extracted for further analysis. For each event the start and finish of the event was defined using the methodology described in ARR Revision Project 3 - Temporal Patterns of Rainfall Report Part 1 (WMAwater, 2015a), which was consistent with the storm event definition used in the ARR Revision Project 6 - Losses (Hill et al., 2014). Once all events with a burst greater than 1EY were defined for every duration, the following properties where calculated:

- burst rainfall depth probabilities for each duration;
- start and finish time for each burst;
- time when 50% of the burst depth occurred (ie. burst loading);
- pre-burst rainfall depth for each burst duration;
- pre-burst to burst rainfall ratio for each duration; and
- Post-burst depth.

For each event the rarest or critical rainfall burst (of any duration and location within the overall event) was also calculated.

5.3.3. Regional Characteristics

To assess the regional characteristics, 12 temporal pattern regions were defined based on the 54 Natural Resource Management (NRM) sub-regions used for investigating the impacts of climate change (CSIRO and Australian Bureau of Meteorology, 2015). The adopted regions follow drainage basin boundaries. Figure 2.5.7 depicts the adopted temporal pattern regions. Table 2.5.2 summarises the number of rainfall gauges and storm events that exist in each region.

Table 2.5.2. Regions- Number of Gauges and Events

Region	Number of Gauges	Number of Station Years	Number of Events	Average Number of Events per Station Year
Southern Slopes (Tasmania)	110	2954	3477	1.18
Southern Slopes (mainland)	356	8536	20581	2.41
Murray Basin	233	6316	18399	2.91
Central Slopes	118	2767	7167	2.59
East Coast South	331	8067	19856	2.46
East Coast North	210	5187	12123	2.34
Wet Tropics	99	2474	5437	2.20
Monsoonal North	211	5054	12287	2.43
Rangelands West	93	2334	5391	2.31
Rangelands	226	5561	12618	2.27
Flatlands West	349	9113	26402	2.90
Flatlands East	56	1401	3450	2.46

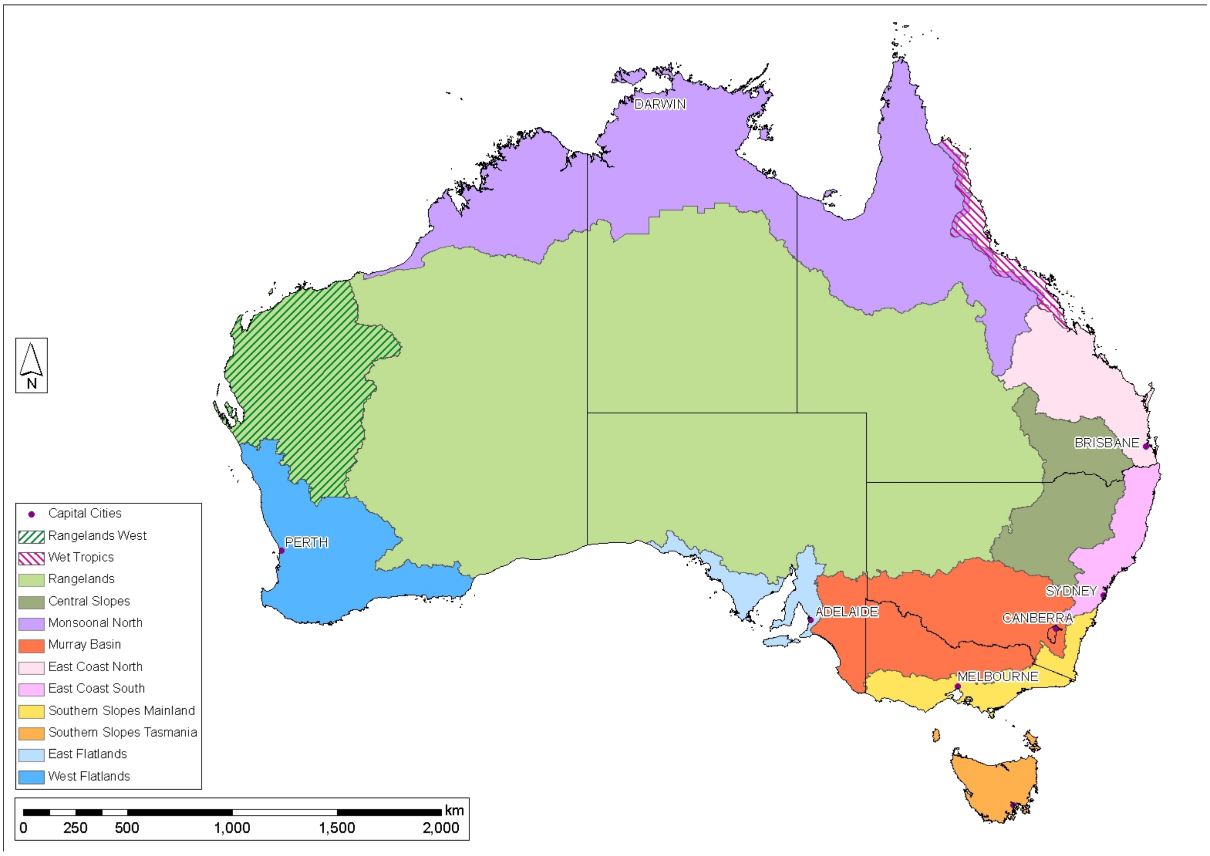


Figure 2.5.7. Temporal Pattern Regions

The term ‘burst loading’ refers to the distribution of rainfall within a burst and is a defining characteristic of a rainfall event. For each event the burst loading was calculated as the percentage of the time taken for 50% of the burst depth to occur. The burst loading can be used as a simple measure of when the heaviest part of the burst occurs and can be used to categorise events as ‘front’, ‘middle’ or ‘back’ loaded. Events where categorised into three groups, depending on where 50% of the burst rainfall occurs:

- front loaded – 0 to 40% of the time;
- middle loaded – 40 to 60% of the time; and
- back loaded – 60 to 100% of the time.

Figure 2.5.8 depicts a typical event from each category.

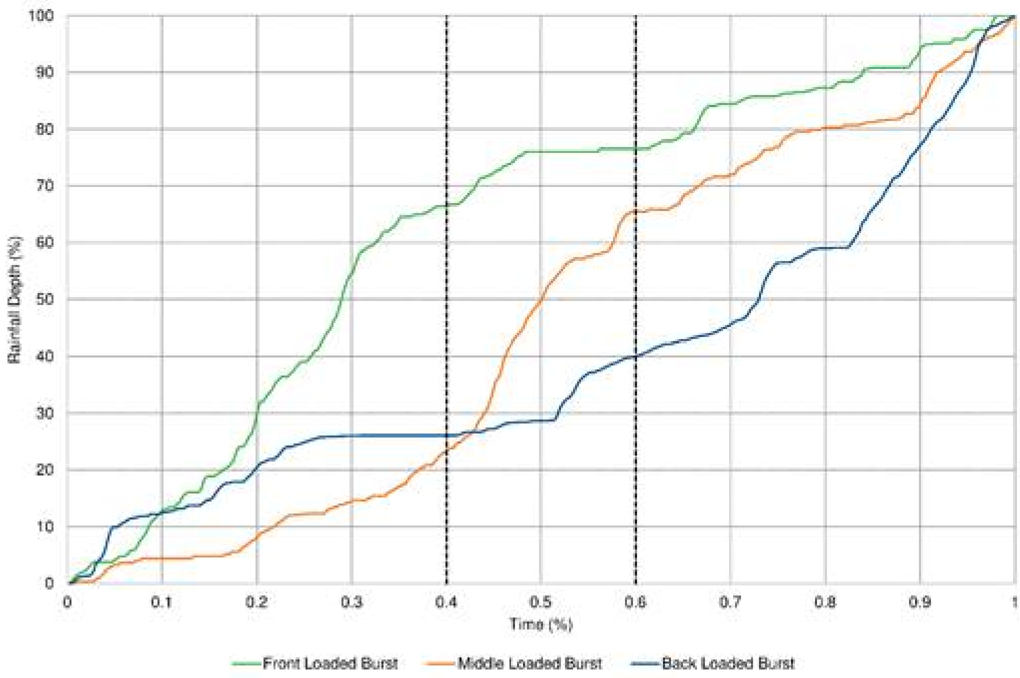


Figure 2.5.8. Example of Front, Middle and Back Loaded Events

This simple categorisation provides a pragmatic means of capturing when the peak loading occurs, as described by Pilgrim et al. (1969), though it is worth recognising that for a double peaked event, when most of the rainfall is in the early and later part of the burst, the loading can somewhat illogically fall into the middle category.

Each region was characterised by its burst loading distribution, which describes the percentage of front, middle and back loaded events for different durations. The proportion of front/middle/back loading for each region was determined for less than and greater than 6 hours duration (Table 2.5.3). The proportion was assumed to be constant across all AEPs.

Table 2.5.3. Burst Loading by Region and Duration

Region	Duration	Front Loaded (%)	Middle Loaded (%)	Back Loaded (%)
Southern Slopes (Tasmania)	≤ 6hr	21.5	64.1	14.4
	> 6hr	20.5	60.0	19.5
Southern Slopes (mainland)	≤ 6hr	30.1	53.0	16.9
	> 6hr	22.7	53.7	23.6
Murray Basin	≤ 6hr	28.3	53.8	17.9
	> 6hr	24.7	52.5	22.7
Central Slopes	≤ 6hr	31.0	53.3	15.7
	> 6hr	27.0	46.9	26.1
East Coast South	≤ 6hr	26.5	57.1	16.4

Region	Duration	Front Loaded (%)	Middle Loaded (%)	Back Loaded (%)
	> 6hr	17.1	58.6	24.3
East Coast North	≤ 6hr	28.9	56.5	14.6
	> 6hr	23.4	48.5	28.1
Wet Tropics	≤ 6hr	16.0	71.8	12.2
	> 6hr	18.7	58.1	23.2
Monsoonal North	≤ 6hr	27.6	63.7	8.8
	> 6hr	27.5	41.4	31.2
Rangelands West	≤ 6hr	23.7	62.5	13.8
	> 6hr	23.6	49.2	27.2
Rangelands	≤ 6hr	29.0	56.6	14.3
	> 6hr	24.4	49.2	26.4
Flatlands West	≤ 6hr	30.8	49.3	19.9
	> 6hr	31.4	48.9	19.7
Flatlands East	≤ 6hr	27.6	52.4	20.0
	> 6hr	17.4	54.4	28.2

5.4. Pre Burst Rainfall and Antecedent Conditions

The events database allowed the pre-burst behaviour of rainfall events to be characterised, regionalised and mapped. Temporal Patterns report 2 WMAwater (2015a) presents the detailed analysis, regionalisation and mapping of pre-burst behaviour for Australia. Due to the relatively short pluviograph records, the approach assumes that this behaviour can be pooled to develop reasonably sized samples by transferring storms from nearby locations with similar IFD characteristics. This was done on a Region of Influence basis and not a fixed region basis, as the ratio of pre-burst rainfall has some correlation with Intensity Frequency Duration characteristics. In many parts of Australia the pre-burst rainfall generally represents a very small amount of the event and generally does not contribute to the runoff response, so it can be treated in a relatively simply manner. However, in some parts of the country pre-burst rainfall can represent a significant part of the rainfall event and runoff response. Pre-burst can also be important in urban catchments with large directly connected impervious areas (Book 5). Storage strategies need to account for this additional runoff when sizing storage tanks and basins (Book 9). The pre-burst was characterised based on the rarest rainfall duration burst within the storm using the 2013 IFDs. Hill et al. (2015) found this approach gave a biased estimate of the average pre-burst, systematically underestimating the depth of the pre-burst. Following from this work and the expected update of the IFDs from the Bureau of Meteorology in 2016 this work will be updated and Figure 2.5.9, Figure 2.5.10 and Figure 2.5.11 will be updated accordingly.

Figure 2.5.9 and Figure 2.5.10 depict the median ratio of the pre-burst to burst and the depth of pre-burst in mm for the 6hr duration and probabilities mapped across Australia. The full data set of maps is available online at the ARR data repository (http://data.arr-software.org/).

Figure 2.5.11 shows the probability distribution of the pre-burst rainfall for each region.

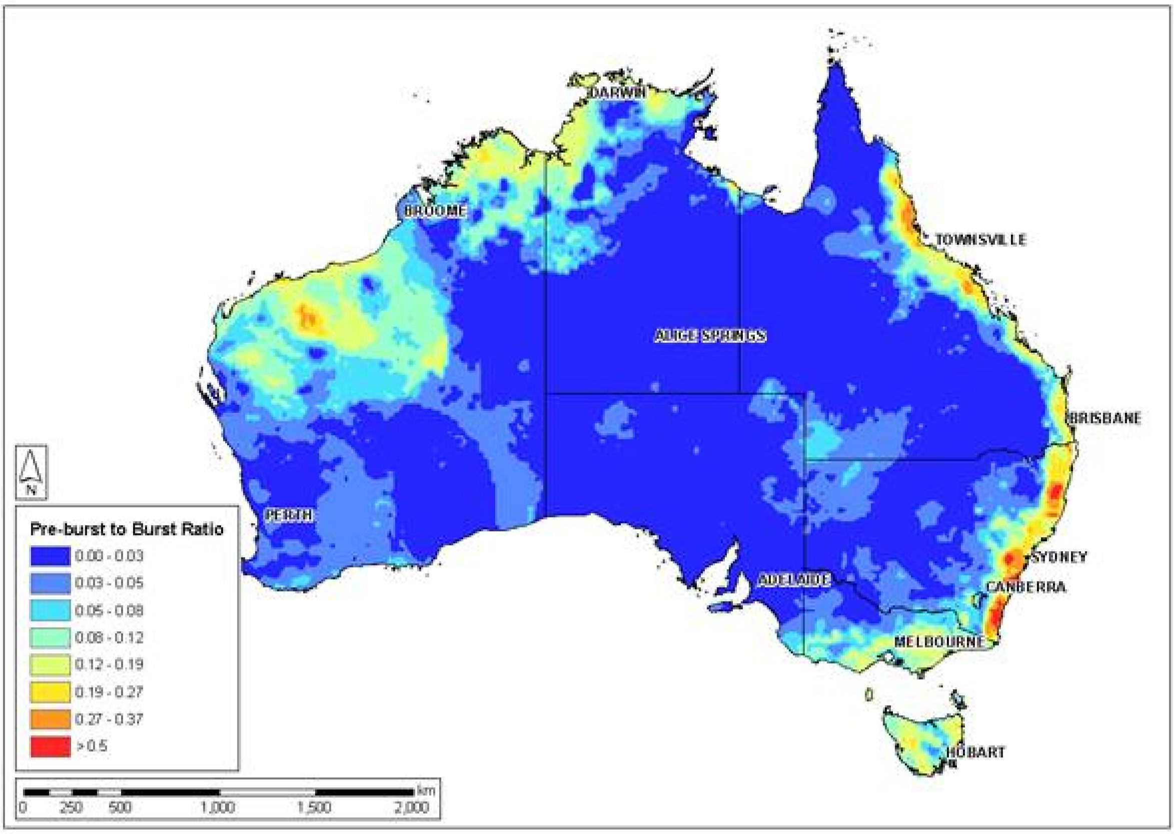


Figure 2.5.9. Pre-burst Rainfall

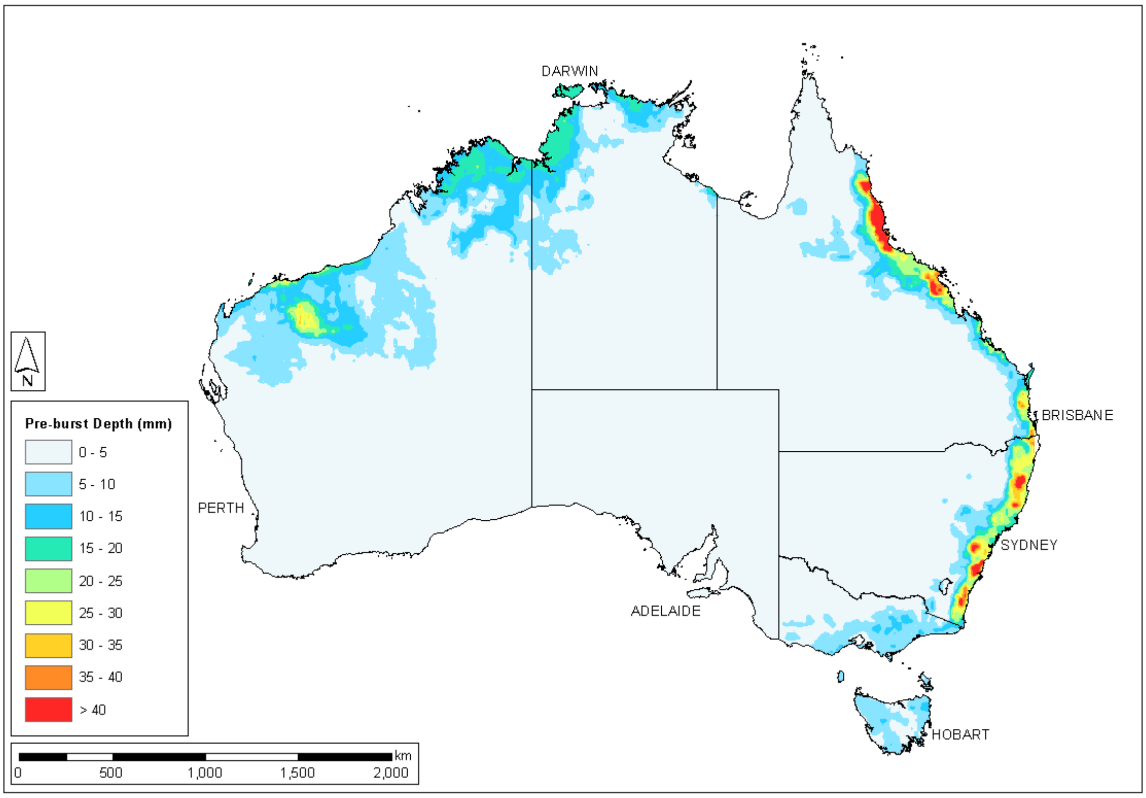


Figure 2.5.10. Pre-burst to burst ratio

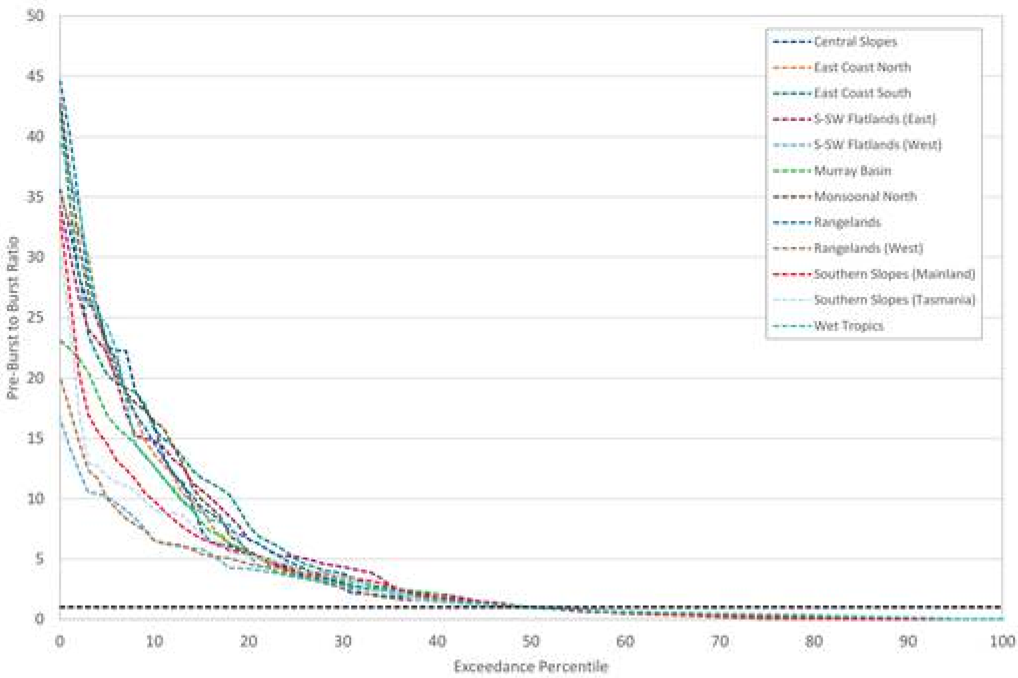


Figure 2.5.11. Standardised pre-burst to burst ratio distributions

The manner in which pre-burst rainfall is treated depends on the magnitude of the pre-burst rainfall, how this compares to losses and whether pre-burst runoff is likely and will have a significant effect on hydrograph volumes. As longer duration bursts tend to represent most of the storm events, it is generally only an issue for smaller catchments. If pre-burst rainfall is unlikely to affect the runoff responses, it is best treated in a simple manner with losses. For simple urban cases pre-burst rainfall can be used to condition storage starting conditions. Where pre-burst is influential for flood response, it can be sampled from its distribution and applied with a typical pre-burst temporal pattern.

5.5. Design Point Temporal Patterns

As part of the ARR Revision Project 3 - Temporal Patterns of Rainfall (WMAwater, 2015b) a series of temporal pattern techniques were trialled. A total of 35 test catchments were adopted across Australia, ranging in area from 30 to 80 km^2. The aims of the testing were to assess the performance of the design method using best estimates of the revised inputs and to determine what influence the temporal patterns have on design estimates. Temporal pattern ensembles were tested within a Monte Carlo framework and a simpler quantile ensemble framework. Two methods were trialled, the regional temporal patterns (as discussed in Book 2, Chapter 5, Section 3) along with a Region of Influence (ROI) approach. Three event types were trialled being burst only, burst plus pre-burst and complete storm events (for ROI only). A range of initial loss approaches were trailed, including burst and storm losses, combined with the median value or a distribution of values. The nature of regional temporal pattern ensembles requires AEP bins to be derived. The ensembles for each AEP bin were trialled across all AEPs to assess the sensitivity of the AEP bins.

Based on the results of this performance testing, the temporal patterns derived by theregional burst approach are recommended for general use. Temporal patterns for complete storms derived by the ROI approach are left as an alternative for small, volume-sensitive systems where pre- and post-burst rainfall is important, though it may be that such systems are better analysed using continuous simulation approaches.

Other major findings from ARR Revision Project 3 - Temporal Patterns of Rainfall (WMAwater, 2015b) were:

1. Irrespective of the method used to derive the temporal patterns, when using a representative ensemble of patterns all methods produce relatively similar results; and

2. Frequent patterns should not be used for rarer events; scaling a temporal pattern introduces more variability and produces higher design estimates.

Ensembles of 20 temporal patterns were initially envisaged, however a large number of regions had insufficient data available to warrant a sample of this size. Different ensemble sizes were tested to determine the sensitivity to ensemble size, with little bias found between the ensemble sizes. Samples of 10 temporal patterns were therefore adopted as an appropriate compromise between pattern variability and data availability. Ensembles were generated for each AEP bin, duration and region. The four AEP bins adopted (see Table 2.5.4 and Figure 2.5.12) were based on the burst AEP at the source of the event. Temporal Pattern were extracted for the following durations from 15 minutes to 7 days, as shown in Table 2.5.5.

Table 2.5.4. Regional Temporal Pattern Bins

AEP Group	AEP Range
Very Rare	Rarest 10 within region
Rare	Rarer than 3.2% AEP
Intermediate	Between 3.2% and 14.4% AEP
Frequent	More frequent than 14.4% AEP

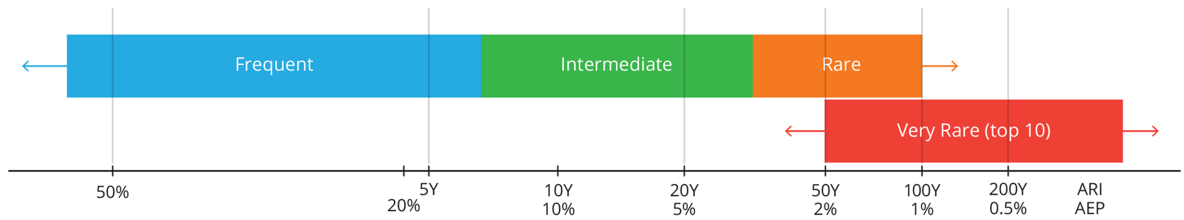


Figure 2.5.12. Temporal Pattern Ranges

Table 2.5.5. Temporal Pattern Durations

Duration		
Minutes	*Hours*	*Days*
15	0.25	0.010
30	0.5	0.021
60	1	0.042
120	2	0.083

Duration		
180	3	0.125
270	4.5	0.188
360	6	0.250
540	9	0.375
720	12	0.500
1080	18	0.750
1440	24	1
2160	36	1.5
2880	48	2
4320	72	3
5760	96	4
7200	120	5
8640	144	6
10080	168	7

When selecting a small ensemble of temporal patterns it is important to capture the typical variability of the observed events. A methodology was therefore adopted that samples from observed events with the intent of generating a representative ensemble in terms of the variability of actual events, with no obvious bias. Whilst it is difficult to quantify or verify the achievement of this objective, steps have been undertaken to ensure the samples are broadly representative, including a visual check of all ensembles.

There are a large number of frequent events from which to select, however, the choice is more limited for rarer events.

For each AEP and duration bin the sampling of an ensemble of 10 patterns used the preferred criteria in Table 2.5.6. The relaxed criteria were used where less than 10 suitable patterns could be found using the preferred criteria.

Table 2.5.6. Temporal Pattern Selection Criteria

Preferred Criteria (for Data Rich Regions)	Relaxed Criteria (for Data Sparse Regions)
No rarer embedded bursts	n/a
No events overlapping in time	n/a
Loading characteristics enforced	Loading characteristics ignored
Events from within a region	Sampled from within region and neighbouring region with similar climatic characteristics

While patterns containing embedded rarer bursts at their recording location were not selected, events can still have embedded bursts at other locations within the region. The presence of major embedded bursts may warrant filtering of patterns.

5.6. Temporal Patterns for Areal Rainfall Bursts

As part of ARR Revision Project 3 - Temporal Patterns of Rainfall a series of areal average temporal patterns have been produced for different sized hypothetical catchments. These patterns average the spatial variability of rainfall in each time step which does remove some of the variability of actual space-time rainfall fields. The process of calculating areal temporal patterns involves identifying high rainfall areal events and calculating areal temporal patterns. A brief summary is provided in this chapter (refer to Podger et al. (2016) for more detail). Areal rainfall temporal patterns were calculated for the area and duration combinations listed in Table 2.5.7.

Table 2.5.7. Areal Rainfall Temporal Patterns - Catchment Areas and Durations

Variable	
Catchment Area (km^2)	100, 200, 500, 1000, 2500, 5000, 10 000, 20 000, 40 000
Durations (Hours)	12, 18, 24, 36, 48, 72, 96, 120, 144, 168

5.6.1. Areal Rainfall Time Series Grid for All Australia

To create sets of areal temporal patterns an areal rainfall grid was derived for 30 minute time steps using the events database (described in Book 2, Chapter 5, Section 3, ie. 2280 pluviographs). Rainfall values from these pluviographs were further filtered for long disaggregations and erroneously high rainfall values. A grid resolution of 0.025° identical to the grid cell size used for the design rainfalls (Book 2, Chapter 3) was chosen for simplicity of use. A natural neighbours algorithm, which is a variant of Thiessen polygons, was used to interpolate rainfall values from the pluviograph stations to each grid cell with rainfall values for each time step from January 1960 to December 2010.

5.6.2. Average Areal Rainfall Calculation

The approach aims to identify the largest set of independent areal rainfall events within the Temporal Pattern Regions shown in Figure 2.5.7 by not restricting catchments to a particular shape. For each combination of duration and area, a set of rectangular hypothetical catchments were used to sample the areal rainfall time series and create catchment average rainfalls. At each catchment centroid, 12 hypothetical catchments were trialled using the combinations of 3 aspect ratios (1.5, 2.3 and 3.6) and 4 rotations (0, 45, 90 and 135 degrees) shown in Figure 2.5.13. To best represent Australian catchments, the aspect ratios were chosen by selecting the 20th, 50th and 80th percentiles of shape factors calculated for the 798 catchments that were adopted for the RFFE method described in Book 3, Chapter 3. Average areal rainfall was calculated for a hypothetical catchment's each time step simply by averaging the rainfall of all the grid cells with midpoints within the catchment area. To lessen computational requirements, larger time steps were used for larger catchment areas.

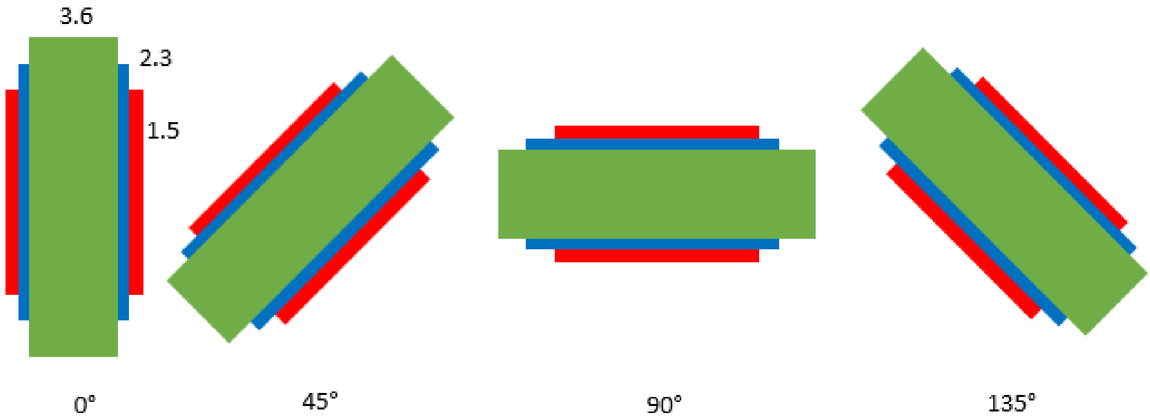


Figure 2.5.13. Combinations of Aspect Ratio and Rotation for Hypothetical Catchments

5.6.3. Areal Temporal Pattern Selection

Event rainfall totals were calculated for the sampled areal temporal patterns. For each of the temporal pattern regions (Figure 2.5.7) a set of events for all combinations of catchment area and durations (as per Table 2.5.7) with the largest depths and no space-time overlap were selected. The space-time filter required a minimum of 3 days between the end of one event and the start of another and for catchment centres to be a minimum of either twice their longest side or 100 km apart.

To ensure data quality, hypothetical catchments that did not contain the minimum number of pluviograph stations (Table 2.5.8) within their vicinity that were producing quality data at the time of the event were disregarded. An additional filter was then applied to remove events that had too much area assigned to a single gauge or erroneous/unrealistic rainfall values. Each space-time independent areal temporal pattern had a Pearson correlation coefficient derived between the areal pattern and all the corresponding temporal patterns of pluviograph stations within its vicinity. Patterns that had a very high correlation to a single gauge and no others or did not have enough stations with a reasonable correlation to the areal pattern were removed.

Table 2.5.8. Minimum Number of Pluviographs Required for Event Selection for Each Catchment Area

Catchment Areas (km^2)	Minimum Number of Pluviographs
100	3
200	3
500	3
1000	6
2500	6
5000	6
10 000	9
20 000	20
40 000	40

Given that events were chosen simply based on their total rainfall depth, many of the longer duration patterns selected represented a shorter duration. Using the same procedure implemented in defining event extents in the events database, events that had extents less than the duration shorter than the duration of interest were removed.

5.6.4. Design Areal Temporal Patterns

Due to the constraints on data quality, a full set of areal temporal patterns for every region, duration and area could not be generated. Therefore, to use areal temporal patterns for data sparse regions, it is necessary to sample patterns from climatically similar or nearby regions. This is especially a problem for longer durations, with some durations such as the 7 days not having enough patterns in any region. For these durations patterns must therefore be sampled from all of Australia, although data dense areas contain the majority of these events.

Like the point temporal patterns, meta-data on each pattern has been provided that allows practitioners to track the location and time of the event. Figure 2.5.14 compares the cumulative mass curves for 24 hour point and area temporal patterns. This demonstrates how the spatial averaging produced by areal temporal patterns reduces the variability. Figure 2.5.15 compares areal temporal patterns to the temporal patterns of the pluviograph closest to the area's centroid, further highlighting a reduction in variability with the areal temporal patterns.

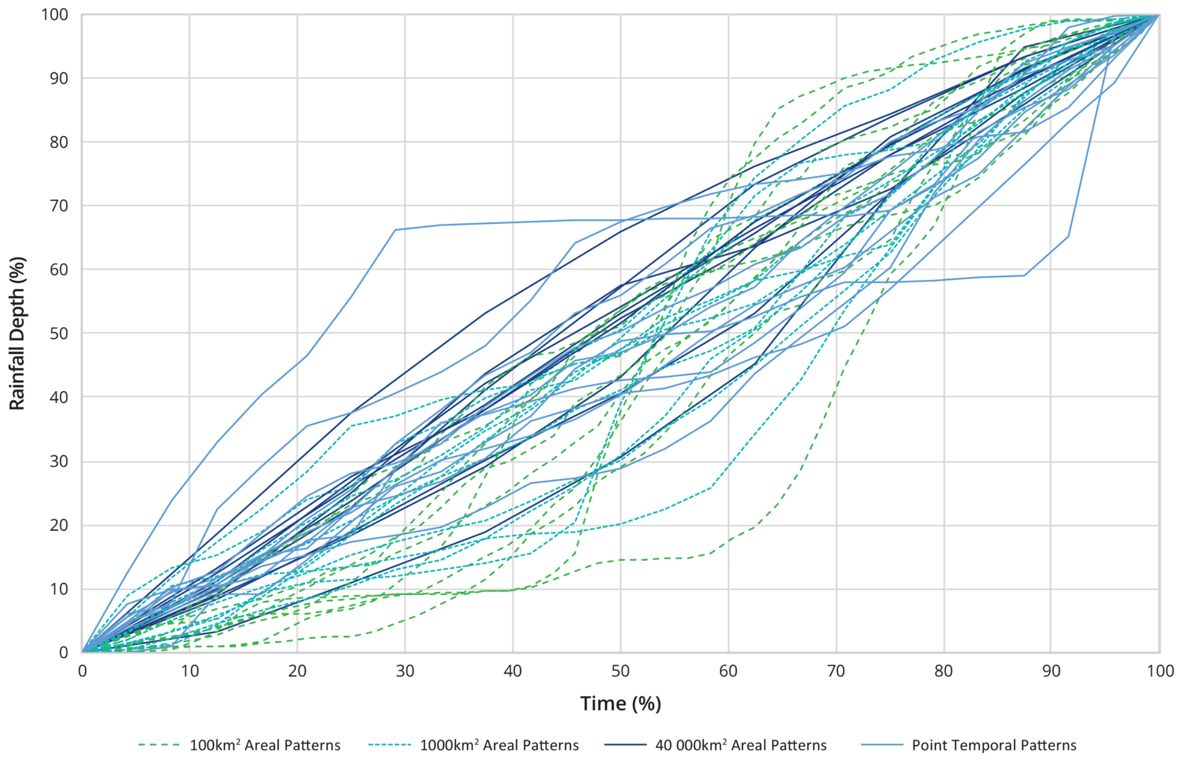


Figure 2.5.14. Comparison of Point Temporal Patterns and Areal Temporal Patterns - East Coast South Region - 1 Day

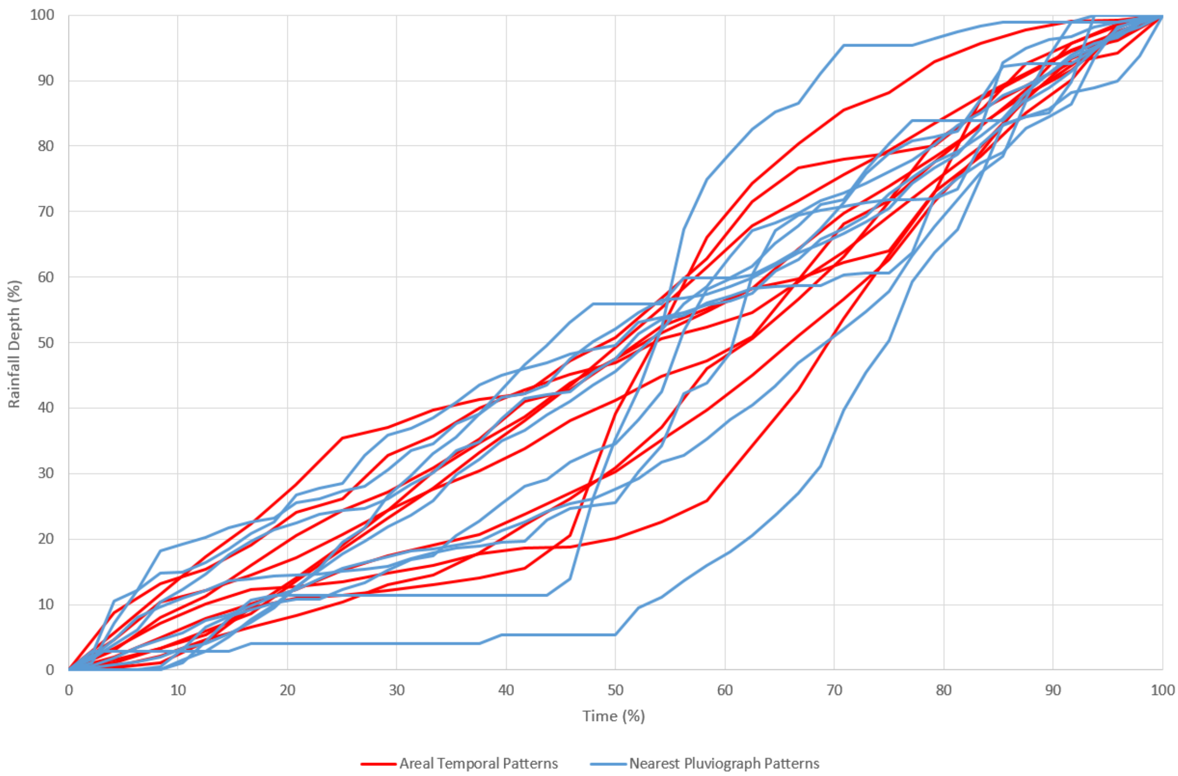


Figure 2.5.15. Comparison of Areal Temporal Patterns and the Temporal Pattern of the Closest Pluviograph for the Same Event

5.7. Other Temporal Pattern Options

5.7.1. Use of Historical Temporal Patterns

On a large catchment with good quality temporal rainfall data it is not uncommon to have a reasonable number of historical patterns for events rarer than the 20% AEP event. Where there is a reasonable amount of local temporal data of appropriate AEPs across the catchment, it is appropriate to use this local data in place of regional temporal patterns. On large catchments, local point patterns will exhibit more variability and generally produce higher flow estimates than locally derived areal temporal patterns and should be used with caution. Some guidance on the use of local temporal patterns is provided in Book 2, Chapter 6.

Book 2, Chapter 6 provides advice on the assignment of historical temporal patterns when modelling observed events for calibration or analysis.

5.7.2. Complete Storm Patterns

The methodology developed for the pre-burst estimation pooled storm events from nearby catchments based on distance and IFD similarity. This approach was extended to develop ensembles of complete storms based on their critical burst duration by selecting events that can be put in a site specific duration probability bin with minimal scaling. The approach is somewhat experimental and has performed as well as burst approaches in limited testing. While categorising events on the basis of critical storm burst solves a lot of event selection problems but it is different from more traditional burst approaches. Aspects of the approach are described in ARR Revision Project 3 – Temporal Patterns of Rainfall Report (WMAwater, 2015c).

The major differences are:

- Events are selected and duration binned on the basis of the critical rainfall duration;
- An event can only be used in one duration so a large storm sample is required;
- Duration bins represent a range of storm durations instead of a fixed duration;
- Event filtering is not required; and
- A new sample needs to be created at each location so a regional approach is not possible.

The process of binning events on the basis of their critical rainfall duration means that events that produce a catchment response because of rainfall over a certain duration can be placed in a bin of a very different duration. This means that under a critical duration approach these event are unlikely to influence the design estimate even if they produce the largest catchment response. This problem would not occur in a Monte Carlo framework that samples across durations.

Despite these limitations, a complete storm approach has the advantage of producing realistic rainfall storm events that have the correct burst IFD and storm volume characteristics. Coombes et al. (2015) showed considerable difference in basin performance on a small urban catchment between burst and complete storm approaches.

5.7.3. Continuous Data

While continuous simulation is generally more appropriate for modelling very frequent events there are some situations where event models might be appropriate. For these situations temporal patterns could be sampled from a local pluviograph or sampled from a generated continuous sequence using the techniques described in Book 2, Chapter 7.

5.8. Climate Change Impacts

Very little information is known about how climate change will affect storm or burst temporal patterns. The most definitive work is by Wasco and Sharma (2015) which analysed the relationship between burst patterns from 79 Australian rainfall gauges and temperature variations. This study found that, regardless of the climate region or season, temperature increases are associated with patterns becoming less uniform, with the largest fractions increasing in rainfall intensity and the lower fraction decreasing. The scaling of the largest fractions was more pronounced in short duration bursts, and rainfall gauges to the north of the country showed more pronounced changes. While this work is based on the present climate, it provides an insight into how patterns could change under a future warming. Such climate changes would also affect IFD characteristics.

Westra et al. (2013) proposed rainfall sequences for future climates could be constructed by sampling historical rainfall patterns corresponding to warmer days at the same location, or from locations which have an atmospheric profile more reflective of expected future climate. Such an approach could conceptually be applied in the selection of design burst patterns but would require significant testing.

It is currently not possible to provide practical advice to practitioners on this aspect of temporal patterns. It is recommended that until further studies are completed, for simulations applying projected climate change, the temporal patterns applied should be those derived for existing climatic conditions but recognising the additional uncertainty in simulation results.

5.9. Temporal Pattern Application and Pre-burst

5.9.1. General

For the majority of problems, the practitioner needs to select a temporal pattern based on the area and probability of the event they are modelling. For Very Rare and Extreme events specific advice is provided in Book 8. The derivation of long duration events (both point or areal temporal patterns) present a number of challenges. Many of the events selected in the IFD Annual Maxima Series were from shorter duration events and can not be selected as longer duration temporal patterns. Some long duration events are more like consecutive events with none or little rainfall in between shorter events. For this reason, in most locations, it was necessary to borrow temporal patterns (point and areal) from adjoining regions. Daily rainfall gauges located in the catchment of interest should give a good insight into the applicability of longer duration temporal patterns approaching 7 days.

Point temporal patterns should be used for catchments less than 75 km^2. Areal temporal patterns have been derived for a number of different catchment areas. Table 2.5.9 provides a guide to applying the areal patterns. Both point and areal temporal pattern sets can be downloaded from the ARR Data Hub (Babister et al. (2016), accessible at http://data.arr-software.org/)

Table 2.5.9. Areal Temporal Pattern Sets for Ranges of Catchment Areas

Range of Target Catchment Areas (km^2)	Catchment Area of Designated Areal Temporal Pattern Set (km^2)
75 – 140	100
140 – 300	200
300 – 700	500
700 – 1600	1000
1600 – 3500	2500
3500 – 7000	5000
7000 – 14,000	10,000
14,000 – 28,000	20,000
28,000 +	40,000

5.9.2. Ensemble Considerations

The use of an ensemble of 10 temporal patterns as discussed in Book 2, Chapter 5, Section 5 is recommended. The temporal patterns have been chosen to represent the variability in observed patterns. Given the run times of two dimensional hydraulic models it is not practical to run all 10 patterns (for multiple durations). A more practical approach would be to run a separate hydrological modelling process of the whole catchment of interest in order to determine the average pattern in terms of peak flow or volume depending on the problem (Book 2, Chapter 5, Section 10). It is not recommended that the temporal pattern that represents the worst (or best) case be used by itself for design. Testing has demonstrated that on most catchments large number of events in the ensemble patterns are clustered around the mean and median.

When selecting a single representative (average) pattern the practitioner needs to look at the whole catchment response hydrograph and not local inflow hydrographs.

The ensemble of 10 pattern provides a range of plausible answers. The practitioner should consider the benefits of investigating multiple temporal patterns or Monte Carlo for sensitive designs and solutions.

Running an ensemble of ten temporal patterns through a two dimensional model could be time consuming. One option is to double the grid cell size which will decrease run time 8 fold.

5.9.3. Upscaling of Patterns

Practitioners are cautioned to avoid or minimise upscaling patterns from frequent events to rare events. In some locations the dominate rainfall mechanism might change with probability and frequent events can exhibit more variability than rarer events. While there are good meteorological reasons for very rare events to exhibit less variability then frequent events (need reference) the causes are probably more complex than a single reason and somewhat region specific. Testing carried out as part of the ARR Revision Project 3 – Temporal Patterns of Rainfall has shown that in some locations that upscaling can be a significant issue (Loveridge et al., 2015) while in other locations results were insensitive. In nearly all testing locations results were relatively insensitive to down scaling of patterns.

5.9.4. Dealing with Inconsistencies and Smoothing of Results

Testing has demonstrated that inconsistent results can sometimes occur with AEP when the bin patterns are drawn from changes particularly between the frequent and intermediate bins. The problem was more pronounced in drier regions where a large part of the rainfall was taken up by losses. In some cases, flood magnitude decreased for rarer AEPs while in others the change with AEP was very stepped. On some catchments, the critical duration varied inconsistently with AEP. Where problems are mainly caused by the pattern bin changing two simple options are suggested, either:

1. Draw from both bins at the boundary effectively doubling the ensemble size which effectively smooths the results; or

2. Replacing the frequent probability bin with the intermediate bin to ensure a smooth catchment response with rainfall.

Consideration should be given to filtering out (or excluding) embedded bursts of lower AEP by re-distributing rainfalls of high intensity to other time increments proportionally to their magnitude (e.g. Herron et al. (2011)). In some situations results will still need to be smoothed.

5.9.5. Practical Issues

If modelling events greater than 5 day, the practitioner should look at the daily rainfall gauge totals for gauges within the catchment and confirm what the 7 day events look like in the catchment compared to the regional patterns.

If 10 temporal patterns are not available for a given region, duration and frequency bin then patterns were taken from other similar regions (Table 2.5.10). For 7 day events there was not enough patterns and all regions were pooled. Local knowledge can be used to add events to the ensemble that weren't selected in the regional sample.

Table 2.5.10. Alternate Regions Used for Data

Region	Alternate Region
Southern Slopes (Tasmania)	Southern Slopes (mainland), Murray Basin
Southern Slopes (mainland)	Murray Basin, Southern Slopes (Tasmania), Central Slopes, East Coast South
Murray Basin	Central Slopes, Southern Slopes (mainland), Flatlands East, East Coast South
Central Slopes	Murray Basin, East Coast South, East Coast North, Southern Slopes (mainland)
East Coast South	East Coast North, Central Slopes, Murray Basin, Southern Slopes (mainland)
East Coast North	East Coast South, Wet Tropics, Central Slopes, Murray Basin
Wet Tropics	Monsoonal North, East Coast North
Monsoonal North	Wet Tropics, East Coast North, Rangelands West
Rangelands West	Rangelands, Flatlands West

Region	Alternate Region
Rangelands	Flatlands West, Flatlands East, Flatlands West
Flatlands West	Flatlands East, Rangelands West
Flatlands East	Flatlands West, Flatlands West, Rangelands

5.9.6. Point and Areal Temporal Pattern Meta-Data

Meta-data is provided (accessed via http://data.arr-software.org/) which describes where the regional point and areal temporal patterns were originally located and when they occurred. This includes:

- *Event ID* - Unique ID given to each pattern;
- *Region* - The region in which the pattern applies;
- *Region (Source)* - The region the temporal pattern occurred in;
- *Burst Loading* - Classified events as front, middle or back (see Book 2, Chapter 5, Section 3);
- *Burst Loading (%)* - The percentage of the event that falls into the loading category;
- *Burst Depth* - Source temporal pattern burst depth. This is the total depth the original pattern had, patterns are given in percentages and not original depths;
- *AEP (source)* - The source temporal pattern AEP at the location it was recorded. This AEP is based of the 2016 IFDs and the burst depth of the event;
- *Burst Start* - The time and date at which the burst began;
- *Burst End* - The time and date at which the burst ended; and
- *Pluviograph number* - The ID of the pluviograph as from the Bureau of Meteorology.

There will be events that weren't picked up in the regional sample however, local experience will allow them to be used in the ensemble (note when doing this the front, middle and back loading of the regional needs to be considered).

5.9.7. Very Rare Point Temporal Patterns

For point temporal patterns 4 bins are provided (Figure 2.5.12). The very rare bin contains the rarest ten patterns within the region. These patterns may not feature in the rare bin for the region as the rare bin is sampled randomly.

5.9.8. Region Considerations

The regional point temporal patterns have been sourced from various locations throughout the region and due to IFD gradients within the regions embedded bursts may occur.

Depending on the severity of the embedded burst, filtering of rarer bursts is recommended. When minor embedded bursts occur in only a few of the 10 patterns in the ensemble filtering can be neglected. Testing has shown with the 2013 Intensity Frequency Duration data that there are some locations where all observed temporal patterns would have embedded bursts.

Locations close to region boundaries might experience storm mechanisms represented by both regions. In these cases it is recommended to run a larger ensemble of all patterns, ie. All 10 patterns from each neighboring region.

5.9.9. Pre-burst

The treatment is dependent on the approach taken to model losses and the magnitude of the pre-burst. In most cases practitioners will use just the median pre-burst and median storm initial loss. However, in some cases the practitioner may sample from distributions of pre-burst and initial loss. As pre-burst varies with location, duration and probability the adopted approach will vary, but it is probably sensible to adopt a consistent approach across durations and probabilities being assessed. Pre-burst for a specific location can be downloaded from the ARR Data Hub (Babister et al. (2016), accessible at http://data.arr-software.org/).

In locations and for durations that do not have significant pre-burst (Figure 2.5.10), the pre-burst depth can be ignored when applying a temporal pattern. Therefore the Burst IL (IL_b) can be taken as the Storm IL (IL_s). In those locations where the pre-burst is significant a number of approaches are possible:

- *Storm IL is greater than Pre-burst* – Pre-burst should be taken out of the storm IL

  ie. IL_s – Pre-burst = IL_b;

- *Storm IL is approximately equal to Pre-burst* – In the case where storm IL and pre-burst are close to equal IL is satisfied and no IL needs to be taken from the burst temporal pattern;

- *Pre-burst is greater than storm IL* - In the case where pre-burst is larger than the storm IL there are a number of options:

  - Apply a pre-burst temporal pattern after taking out the Storm IL. There is little research that has investigated pre-burst temporal patterns;

  - Test the sensitivity of pre-burst on the resultant flood estimate and determine if it can be ignored;

  - Apply a complete storm approach instead of a burst approach e.g. Coombes et al. (2015).

5.10. Example

The Tennant Creek Catchment is located in the Northern Territory and was used for the ARR research Projects 3 and 6. The catchment and catchment location is shown in Figure 2.5.16. It is located in the Rangeland Region for temporal patterns. The temporal pattern data for the region can be extracted at http://data.arr-software.org/. In this particular example a RORB model was set up for the catchment. The model was previously calibrated with an IL_b=0 and a CL=7 mm/hr. As the catchment area is 72.3 km^2 it was decided that the critical duration would be between 60 minutes and 1440 minutes (1 day).

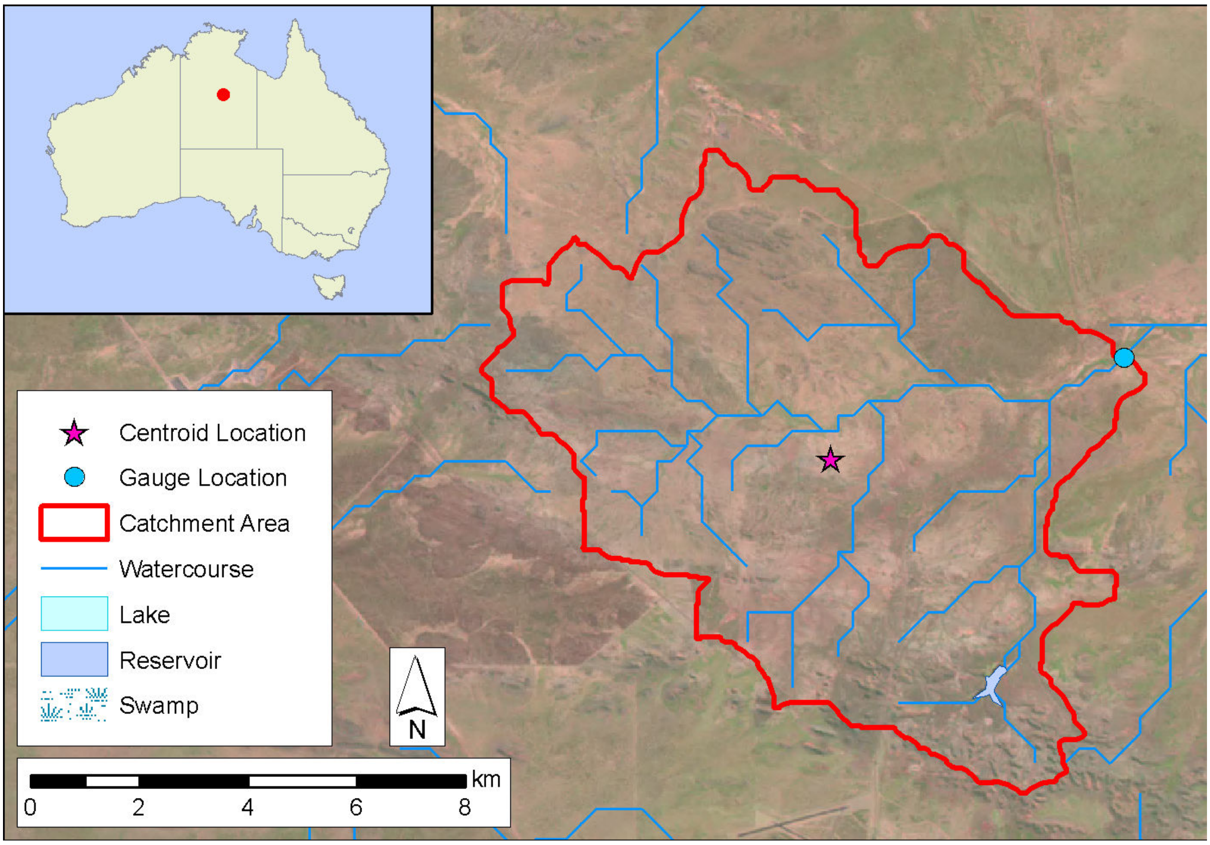


Figure 2.5.16. Tennant Creek Catchment

For this example only the 1% AEP patterns were run through the hydrologic model. 10 patterns from the rare AEP bin and the Rangelands region were run for each of the following durations:

- 60 minute
- 120 minute
- 180 minute
- 270 minute
- 360 minute
- 540 minute
- 720 minute
- 1080 minute
- 1440 minute

Figure 2.5.17 is a presentation of the results in a box plot. The results are also presented in Table 2.5.11, it should be noted that even though the columns are labelled bursts 1 to 10 they are not the same storms across the durations. The box plot presents clearly that the 180 minute duration is critical. The average peak flow of the 180 minute duration bursts (277.35 m^3/s) should then be taken as the 1% AEP flow. If a practitioner wanted to run a pattern through a hydraulic model and it was not practical to run all 10 patterns, the pattern closest to the average (shown in Table 2.5.11) is Burst number 1.

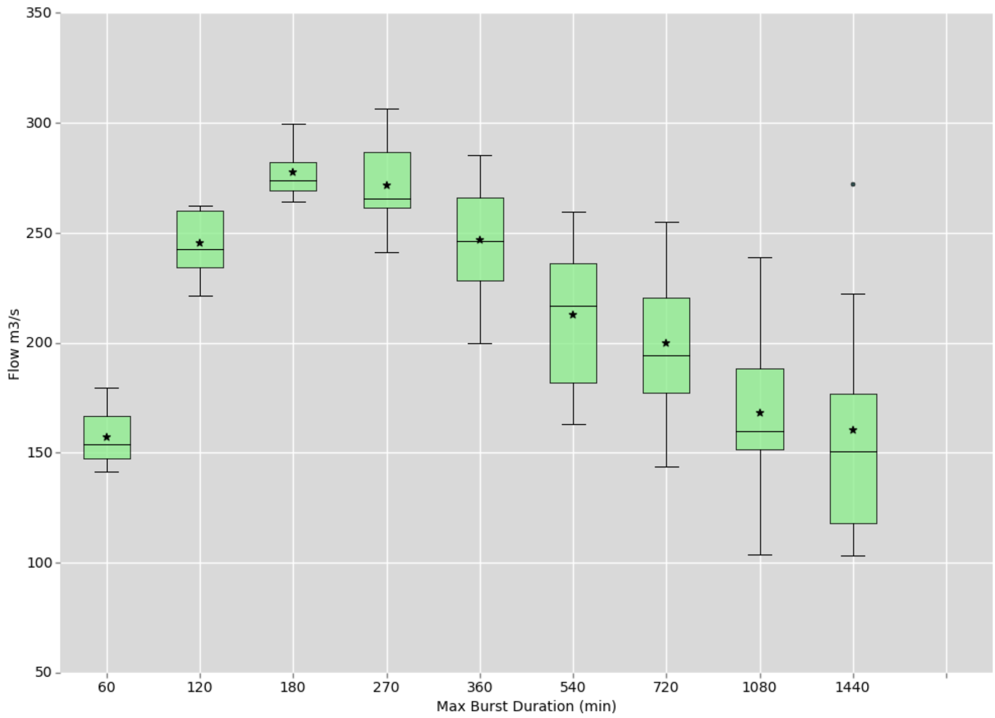


Figure 2.5.17. Duration Box plot for the 1% AEP

Table 2.5.11. Flows for the 1% Annual Exceedance Probability for Ten Burst Events

Duration	1% AEP Peak Flow (m^3/s)										
Burst 1	Burst 2	Burst 3	Burst 4	Burst 5	Burst 6	Burst 7	Burst 8	Burst 9	Burst 10	Average	Median	
60	146.9	173.8	179.6	145.7	157.2	159.6	169.4	141.6	148.8	150.2	157.28	153.7
120	221.5	230.2	261.9	238.1	260.3	259.5	233	240.2	244.9	262.6	245.22	242.55
180	279.4	264.8	279.6	274.4	263.5	273.5	264.1	272.8	273.1	268.3	277.35	273.95
270	262.5	241.1	284.1	288.5	306.6	264.7	266.8	261.2	287.8	254.4	271.77	265.75
360	255.5	285.5	214.4	243.9	199.7	223.2	246.3	246.3	269.8	283.9	246.85	246.3
540	230.9	163.1	168.7	195.4	254.7	203.2	236.6	235.6	177.5	259.7	212.54	217.05
720	193.7	173.5	188.1	143.8	254.9	243.6	195.2	210.3	172.3	224.2	199.96	194.45
1080	197.7	159.2	103.8	239	179.1	151.3	191.2	160.5	148.1	151.5	168.14	159.85
1440	177.9	113.5	144	108.8	222.5	103.4	173.2	156.9	272	130.5	160.27	150.45

5.11. References

Askew, A.J. (1975), Use of Risk Premium in Chance-Constrained Dynamic Programming. Water Resources Research, 11(6): 862-866.

Babister, M., Trim, A., Testoni, I. and Retallick, M. 2016. The Australian Rainfall and Runoff Datahub, 37th Hydrology and Water Resources Symposium Queenstown NZ

Ball, J.E. (1994), 'The influence of storm temporal patterns on catchment response', Journal of Hydrology, 158(3-4), 285-303.

Brown, J.A.H. (1982), A Review of flood estimation procedures, Proceedings of the workshop on spillway design, Department of National Development and Energy, AWRC Conference Series No.6.

CSIRO and Bureau of Meteorology (2015), Climate Change in Australia Information for Australia's Natural Resource Management Regions: Technical Report, CSIRO and Bureau of Meteorology, Australia.

Coombes, P., Babister, M. and McAlister, T. (2015).Is the Science and Data underpinning the Rational Method Robust for use in Evolving Urban Catchments, Proceedings of the 36th Hydrology and Water Resources Symposium Hobart 2015.

Cordery, I., Pilgrim D.H. and Rowbottom I.A. (1984), Time patterns of rainfall for estimating design floods on a frequency basis. Wat. Sci. Tech., 16(8-9), 155-165.

French, R.H. (1985), Open Channel Hydraulics, McGraw-Hill Book Company, New York.

Green, J.H., Xuereb, K. and Siriwardena, L. (2011), Establishment of a Quality Controlled Rainfall Database for the Revision of the (IFD) Estimates for Australia, Proceedings of 34th IAHR World Congress, Brisbane, 26 June-1st July 2011.

Hall, A.J. and Kneen, T.H. (1973), Design Temporal Patterns of Storm Rainfall in Australia, Proc. Hydrology Symposium, Inst. Eng. Aust., pp: 77-84

Herron, A., Stephens, D., Nathan, R. and Jayatilaka, L. (2011), Monte Carlo Temporal Patterns for Melbourne. In: Proc 34th World Congress of the International Association for Hydro- Environment Research and Engineering: 33rd Hydrology and Water Resources Symposium and 10th Conference on Hydraulics in Water Engineering. Barton, A.C.T.: Engineers Australia, pp: 186-193.

Hill, P.H., Graszkiewicz, Z., Taylor, M. and Nathan, R. (2014), Australian Rainfall and Runoff Revision Project 6: Loss Models for Catchment Simulation, Phase 4, October 2014, Report No. P6/S3/016B.

Hill, P.I., Graszkiewicz, Z., Loveridge, M., Nathan, R.J. and Scorah, M. (2015), Analysis of loss values for Australian rural catchments to underpin ARR guidance. Hydrology and Water Resources Symposium 2015, Hobart, 9-10 December 2015.

Hoang, T.M.T., Rahman, A., Weinmann, P.E., Laurenson, E. and Nathan, R. (1999) Joint Probability description of design rainfall, Water 99 Joint congress, Brisbane.

Huff, A. (1967), Time distribution in heavy storms, Water Resources Research, 3: 1007-1019.

IEAust. (1977), Australian Rainfall and Runoff: flood analysis and design. The Institution of Engineers Australia. Canberra.

Keifer, C.J. and Chu, H.H. (1957), Synthetic storm pattern for drainage design, ASCE Journal of the Hydraulics Division, 83(4), 1-25.

Kuczera, G., Lambert, M., Heneker, T., Jennings, S., Frost, A. and Coombes, P. (2003) Joint Probability and design storms at the cross roads, Proceedings of the 28th International Hydrology and Water Resources Symposium, Wollongong.

Loveridge, M., Babister, M., Retallick, M. and Testoni, I. (2015). Testing the suitability of rainfall temporal pattern ensembles for design flood estimation, Proceedings of the 36th Hydrology and Water Resources Symposium Hobart 2015.

Meighen, J. and Kennedy, M.R. (1995), Catalogue of Significant Rainfall Occurrences over Southeast Australia , HRS Report No. 3, Hydrology Report Series, Bureau of Meteorology, Melbourne, Australia, October.

Milston, A.K. (1979), The influence of temporal patterns of design rainfall in peak flood discharge, Thesis (M.Eng.Sci.), University of New South Wales.

Nathan, R.J. and Weinmann, P.E. (1995), The estimation of extreme floods - the need and scope for revision of our national guidelines. Aus J Water Resources, 1(1), 40-50.

Nathan, R.J., Weinmann, P.E. and Hill, P.I. (2002), Use of a Monte-Carlo framework to characterise hydrologic risk. Proc., ANCOLD conference on dams, Adelaide.

Nathan, R., Weinmann, E., Hill, P. (2003) Use of Monte Carlo Simulation to Estimate the Expected Probability of Large to Extreme Floods, Proc. 28th Int. Hydrology and Water Res. Symp., Wollongong, pp: 1.105-1.112.

Pattison, A, (ed) 1977, Australian Rainfall and Runoff: Flood Analysis and Design, The Institution of Engineers Australia

Phillips, P. and Yu, S. (2015), How robust are OSD and OSR Systems?, 2015 WSUD & IECA, Sydney.

Phillips, B.C., Lees, S.J. and Lynch, S.J. (1994), Embedded Design Storms - an Improved Procedure for Design Flood Level Estimation?. In: Water Down Under 94: Surface Hydrology and Water Resources Papers; Preprints of Papers. Barton, ACT: Institution of Engineers, Australia, 235-240. National conference publication (Institution of Engineers, Australia), No. 94/15.

Pilgrim, DH (ed) (1987) Australian Rainfall and Runoff - A Guide to Flood Estimation, Institution of Engineers, Australia, Barton, ACT, 1987.

Pilgrim, D.H. and Cordery, I. (1975), Rainfall temporal patterns for design floods, Journal of the Hydraulics Division, 101(1), 81-95.

Pilgrim, D.H., Cordery, I. and French, R. (1969), Temporal patterns of design rainfall for Sydney, Institution of Engineers, Australia, Civil Eng. Trans., CE11: 9-14.

Podger, S., Babister, M., Brady, P. (2016), Deriving Temporal Patterns for Areal Rainfall Bursts, Proceedings of the 37th Hydrology and Water Resources Symposium Queenstown 2016.

Rahman, A., Weinmann, P.E., Hoang, T.M.T. and Laurenson, E.M. (2002), Monte Carlo simulation of flood frequency curves from rainfall. Journal of Hydrology.

Retallick, M., Babister, M., Varga, C., Ball, J. and Askew, A. (2009), Do Filtered patterns resemble real patterns? Proceedings of the Hydrology and Water Resources Symposium, Newcastle 2009, Engineers Australia.

Rigby, E. and Bannigan, D. (1996), The Embedded design storm concept - A critical review, 23rd Hydrology and Water Resources Symposium Hobart.

Rigby, E., Boyd, M., Roso, S. and Van Drie, R. (2003), Storms, Storm bursts and flood estimation- A need for review of the AR&R procedures, Proceedings of the 28th International Hydrology and Water Resources Symposium, Wollongong.

Roso, S. and Rigby, E. (2006), The impact of embedded design storms on flows within a catchment, Proceedings of the 30th Hydrology and Water Resources Symposium Launceston 2006.

Sih, K., Hill, P. and Nathan, R. (2008), Evaluation of Simple Approaches to Incorporating Variability in Design Temporal Patterns. In: Lambert, Martin (Editor); Daniell, TM (Editor); Leonard, Michael (Editor). Proceedings of Water Down Under 2008. Modbury, SA: Engineers Australia; Causal Productions, pp: 1049-1059.

Varga, C., Ball, J., Babister, M. (2009), An Alternative for Developing Temporal Patterns, Proceedings of the Hydrology and Water Resources Symposium, Newcastle 2009, Engineers Australia

WMAwater (2015a), Australian Rainfall and Runoff Revision Project 3: Temporal Patterns of Rainfall, Part 1 - Development of an Events Database, Stage 3 Report, October 2015.

WMAwater (2015b), Australian Rainfall and Runoff Revision Project 3: Temporal Patterns of Rainfall, Part 3 - Preliminary Testing of Temporal Pattern Ensembles, Stage 3 Report, October 2015.

WMAwater (2015c), Australian Rainfall and Runoff Revision Project 3: Temporal Patterns of Rainfall, Part 3 - Preliminary Testing of Temporal Pattern Ensembles, Stage 3 Report, October 2015.

Wasko, C. and Sharma, A. (2015), Steeper temporal distribution of rain intensity at higher temperatures within Australian storms, Nature Geoscience, 8(7), 527-529.

Webb McKeown and Associates, (2003), Assessment of the variability of rates of rise for hydrographs on the Hawkesbury Nepean River, December 2003.

Weinmann, P., Rahman, A., Hoang, T., Laurenson, E., Nathan, R. (2000), Monte Carlo Simulation of flood Frequency Curves, Proceedings of Hydro 2000, 3rd International hydrology and water resources symposium, Perth, Inst. Of Engineers Australia, pp: 564-569.

Westra, S., Evans, J., Mehrotra, R., Sharma, A. (2013), A conditional disaggregation algorithm for generating fine time-scale rainfall data in a warmer climate, Journal of Hydrology, 479: 86-99

Wood, J.F. and Alvarez, K. (1982), Review of Pindari Dam Design Inflow Flood Estimate, Proceedings of the workshop on spillway design, Department of National Development and Energy, AWRC Conference Series No.6.

Chapter 6. Spatial Patterns of Rainfall

Phillip Jordan, Alan Seed, Rory Nathan

Chapter Status	Final
Date last updated	14/5/2019

6.1. Introduction

As discussed in Book 2, Chapter 2, the description of rainfall events used in most currently applied design flood estimation methods is based on a reductionist approach, where the temporal and spatial variations of rainfall within an event are represented separately by typical temporal patterns and spatial patterns of event rainfall.

This chapter provides practitioners with recommendations on the derivation and application of spatial patterns of rainfall for use in design flood estimation using representations of varying complexity. This includes recommendations for reconstructing the space-time patterns of rainfall for the observed events used in the calibration of hydrologic catchment models.

There are a number of items where the authors recognise that the guidance adopted in this chapter is uncertain and where benefit would be obtained from further research to better quantify and potentially reduce the impact of those uncertainties on design flood estimation practice. Section 6 lists and briefly discusses the residual uncertainties relating to various aspects of the derivation and application of spatial and space-time patterns of rainfall, and recommends potential areas of future investigation.

6.2. Methods for Deriving Spatial Patterns of Rainfall for Events

6.2.1. Precipitation Observation Methods and Uncertainties Associated with Reconstructing Space-Time Rainfall Patterns

There is no accepted method for determining the space-time pattern of rainfall that is not influenced by the resolution and accuracy of networks of rainfall observations.

Rainfall gauges provide data on the rainfall depths observed at the point location of the rainfall gauge over different periods of time. Daily reporting rainfall gauges provide rainfall depths recorded at the gauge over the preceding day period, with the Bureau of Meteorology's typical practice being that these gauges report at 9:00 am local time on each day. Pluviograph or tipping bucket rainfall gauges can provide rainfall depths observed at a point location for sub-daily temporal resolution. They can provide rainfall depths with temporal resolutions down to less than one minute.

Rainfall gauges are subject to some observational errors but they typically provide relatively accurate measurements of the time series of rainfall recorded at a point location. They can under-record rainfall during periods of high winds, particularly for snow or for when rainfall intensities are low. There can be errors associated with estimating rainfall rates from tipping bucket rainfall gauges over defined periods of time from the recorded times of the bucket tips. During periods of very high rainfall intensity, tipping bucket rainfall gauges can be subject to errors induced by the bucket failing to tip or tipping when it is partially full. Rainfall gauges can also be subject to errors in manual recording of the data or electronic transmission of data from telemetered rainfall sites.

The chief uncertainty introduced by a network of rainfall gauges is in accurately observing the space-time pattern of rainfall across an area because they cannot observe variations in rainfall patterns between the gauges (Seed and Austin, 1990; Barnston, 1991; Bradley et al., 1997).

Remote sensing approaches can provide estimates of rainfall intensity observed on a spatial grid across a wide observation domain for a given period of time. The two most commonly available remote sensing approaches for rainfall estimation are ground-based weather radar and satellite observing systems.

Weather radars measure the reflectivity returned by rain drops, hail stones or snow, which are converted into a rainfall intensity estimate. There are several different types of errors in this process that degrade the accuracy of the radar rainfall measurement (Joss and Waldvogel, 1990; Collier, 1996). The analysis of radar data to derive space-time patterns requires specialist expertise that lies outside the scope of these guidelines. However, such analysis could be considered for large or high risk studies which are able to secure the specialist expertise required. Weather radar approaches typically provide estimates of rainfall intensities that are more accurate on a relative basis within the space-time field of the event than in absolute magnitude terms.

6.2.2. Data Availability

Rainfall gauge observations are available at many locations across Australia for very long time periods, with data available at some sites since the middle of the 19th century (refer to Book 2, Chapter 3). By contrast, reliable archives of data from remote sensing (weather radar and satellite based instruments) are only available from about the mid 1990s. For many events, the only data that will be available to the practitioner to reconstruct the space-time pattern of rainfall will be from rainfall gauges.

The practitioner should make use of the available data to reconstruct the space-time pattern of rainfall across the catchment or study area for the events that are to be utilised in model calibration and design flood simulation. The practitioner should assess the suitability of the rainfall data that is available for the event for reconstructing the space-time pattern, including rainfall gauges and any data that is available from remote sensing.

The practitioner should consider the events that are to be used for constructing the space-time patterns of rainfall. Factors that should be considered in selecting events are the:

- Number of sites and locations, relative to the catchment, of daily rainfall gauges;
- Number of sites and locations, relative to the catchment, of continuous rainfall gauges;
- Existence or otherwise of remotely sensed data;
- Likely accuracy of quantitative rainfall estimates derived from remotely sensed data;
- Purpose of estimating the space-time rainfall pattern, whether it is for hydrological model calibration, deriving a space-time pattern or spatial pattern for inclusion in design flood simulation or both;
- Existence and quality of recorded flood levels, flood extents and gauged flows for the event, which make it a candidate for model calibration; and

- Estimated AEP of the flood event or the rainfall total for the event over the catchment of interest, relative to the AEP of the design floods that are to be estimated.

The practitioner may need to make judgements between using the space-time pattern for an older event, for which there is no remote sensing data and relatively poor coverage of rain gauge data but which produced higher flood levels and a more recent event that has remotely sensed data and/or better coverage of rain gauge data but which produced a smaller flood.

6.2.3. Construction of Space-Time Patterns from Rainfall Gauge Networks

When estimating the space-time pattern of rainfall from rainfall gauges there is an uncertainty introduced to the estimates in the interpolation of the unobserved rainfall at locations between the gauges (for example, Urbonas et al. (1992) and Ball and Luk (1998)).

The conventional approach applied in flood estimation for approximating the space-time pattern of rainfall from gauge networks has been:

1. To estimate the spatial pattern of rainfall for the whole rainfall event; and

2. To disaggregate the rainfall accumulation for each part of the spatial domain, often a model subarea or subcatchment, using the temporal pattern observed at a particular rainfall gauge.

The conventional approach is a valid method in most situations but it may be that a more sophisticated approach involving construction of different spatial patterns for different increments of the event are required when spatial or temporal variability of the rainfall pattern for the event is large (Umakhanthan and Ball, 2005). Considerations for application of each of the two steps are discussed in Book 2, Chapter 6, Section 2 and Book 2, Chapter 6, Section 2. Potential alternative approaches are discussed in Book 2, Chapter 6, Section 2.

6.2.3.1. Construction of Spatial Patterns

The spatial pattern should be constructed using rainfall totals from daily rainfall gauges and where available continuous rainfall gauges. Gauges should be obtained from both within the catchment or study area and for a region around the catchment. As an indicative value, the region used for constructing the spatial pattern should extend to include gauges that are within at least 10 km of the catchment or study area boundary or further if internal catchment gauges are further from the boundary.

There is no preferred technique for constructing a spatial pattern of rainfall for an event. Hand drawing of rainfall contours informed by the rainfall totals at the gauges remains a valid approach that will produce acceptable results for many rainfall events.

Spatial interpolation techniques using a computer usually involve interpolation between the point observations onto a grid, defined in either a geographic or projected Cartesian coordinate system. The grid resolution should be sufficiently fine to capture the spatial variability in the rainfall field at a meaningful scale for the catchment. It is recommended that the resolution of the grid should be 1 km (for a projected grid) or 0.01° (for a geographic grid) or finer. There are many potential approaches that have been developed for spatial interpolation (Verworn and Haberlandt (2011) and the references therein), including:

- Construction of Thiessen polygons, which is equivalent to adopting the rainfall depth from the nearest neighbour rainfall gauge when applied using a grid;
- Weighting of rainfall using the inverse of the square of the distances to the gauges;
- Natural neighbours;
- Spline interpolation algorithms;
- Ordinary Kriging; and
- Variants on Kriging, such as indicator Kriging, regression Kriging and Kriging with external drift.

Some Kriging and spline interpolation algorithms allow for the use of a covariate in the interpolation algorithm, which may improve the accuracy of the interpolation. Either elevation or design rainfall intensities for a relevant AEP and duration may provide appropriate covariates that improve the accuracy of the interpolation, particularly in catchments or study areas that are subject to appreciable and consistent orographic effects.

Gridded daily rainfall data sets are available from SILO and the Australian Water Availability Project (Jones et al., 2009). These data sets may be useful for providing spatial patterns of rainfall events but they should be used with caution as they were not derived with the intention of being used for design flood estimation (refer to Book 1, Chapter 4, Section 9 and Book 2, Chapter 7, Section 2).

Regardless of the approach that is used to produce the spatial pattern, the practitioner should check the spatial pattern produced by mapping it against the point values observed at rainfall gauges. If the mapping reveals anomalies in the interpolation approach, an alternative method should be adopted. Where large gaps in rainfall gauge coverage exist (particularly in mountainous areas) careful review of the simplifying assumptions made in the interpolation procedure should be undertaken by the practitioner to avoid unrealistic spatial patterns.

Rainfall totals for each model subarea should be estimated from the spatially interpolated rainfall field for the event by averaging the rainfall totals at all grid cells that intersect with the spatial extent of the model subarea. Mathematically, this is represented by:

$$S_s = \frac{\sum_{i,j} A_{s \cap i,j} \rho_{i,j}}{\sum_{i,j} A_{s \cap i,j}} \tag{2.6.1}$$

where S_s is the rainfall depth for the total event applied to model subarea, s, $A_{s \cap i,j}$ is the area of overlap between grid cell at coordinate location (i,j) in the interpolated grid and the model subarea s, and $\rho_{i,j}$ is the interpolated rainfall total for the grid cell.

For rainfall events that extend for a period longer than 24 hours, it may be useful to construct spatial patterns for separate time periods of the event to investigate the temporal evolution of the event.

6.2.3.2. Disaggregation Using Temporal Patterns from Observed Pluviograph Data

A conventional approach adopted is to disaggregate the total rainfall for each subarea of a rainfall runoff model using the temporal pattern recorded at a recording rain gauge, using the formula:

$$R_{s,i} = \frac{S_s r_{g,i}}{\sum_i r_{g,i}} \tag{2.6.2}$$

Where $R_{s,i}$ is the rainfall depth applied to model subarea, *s,* for model time increment, *i*; S_S is the total event rainfall for model subarea *s*; $r_{g,i}$ is the rainfall depth recorded at gauge, *g* , for model time increment, *i*; and the summation is formed for all time increments over the event.

Where data is available from more than one recording rainfall gauge, this provides options to the practitioner on which gauge to select to provide the temporal pattern for each subarea of the model. An assumption is often made that the most appropriate pattern would be provided by the recording rainfall gauge that is located closest, by horizontal distance, from the centroid of the model subarea. Whilst the closest gauge by physical distance makes intuitive sense, it is not necessarily the case that it must provide the most appropriate pattern for allocating the temporal pattern of a particular subarea. The practitioner may consult other information, such as catchment topography, data on wind velocities during the event or remote sensing data to guide the selection of an alternative to the nearest gauge for providing the temporal pattern. The practitioner may also use other information on the meteorology of the event to justify use of an adjusted temporal pattern for disaggregation. For example, if information was available that a particular rainfall event was moving in a particular direction at an average velocity of 15 km/h and a model subarea was located 30 km downwind of a rainfall gauge, it may be justified to adjust the temporal pattern recorded at this gauge by moving it backward in time by two hours, to represent the estimated travel time of the storm from the rainfall gauge to the model subarea.

The time series of rainfall at each recording gauge should be checked before it is used for disaggregation. The event total rainfall at each recording gauge should be checked, where available, against the event total rainfall at other daily recording and continuously rainfall gauges in the vicinity. A gauge should not be used if significant anomalies are identified in the recorded data for the site.

6.2.3.3. Alternative Approaches to Construction of Space-Time Patterns

A potential alternative approach to construct the space-time pattern of rainfall for an event from rainfall gauge data only is to construct a three dimensional space-time pattern grid. In this approach, the overall event spatial pattern would be interpolated onto a grid, using one of the potential approaches discussed in Book 2, Chapter 6, Section 2. The total rainfall for each grid cell would then be disaggregated using the temporal pattern from an assigned rainfall gauge, using a similar method as discussed in Book 2, Chapter 6, Section 2. For each time increment, a summation is formed for all of the grid cells that intersect spatially with each model subarea. Each subarea would then have its own, potentially unique, temporal pattern for the event.

In a catchment that is well instrumented with rainfall gauges, it is possible to perform a spatial interpolation on to a grid for each time increment. The gridded rainfall for each time increment would then be summed to produce a temporal pattern for each model subarea. If this approach is used, the total rainfall across the event should be calculated for each subarea and then compared to the rainfall computed from spatially interpolating the event total rainfall only. Adjustments should be considered to the approach if the totals for any subarea differ by more than 5%.

6.2.4. Space-Time Patterns for Calibration

Simulation of historical flood events for calibration purposes includes reconstruction of the space-time pattern of rainfall over the catchment by the practitioner. The values of the apparent optimum set of model parameters for a given event can be influenced by the approach taken to estimate the space-time pattern of rainfall for the event. For example, if a catchment is simulated using an initial loss-continuing loss runoff generation model, then if all other parameters and inputs are the same a lower continuing loss rate is likely to be required to generate the same volume of runoff if a uniform spatial pattern is adopted compared with a non-uniform spatial pattern. The practitioner should consider and articulate the influence of assumptions made in deriving the space-time pattern of rainfall for the event on the values of the runoff-routing model parameters calibrated for the event.

The spatial coverage of pluviographs around a catchment may be relatively sparse. This introduces uncertainty into the estimation of the actual temporal pattern of rainfall for any given subarea or grid cell of a model. Whilst a reasonable assumption may be that the temporal pattern for a given model subarea would be defined by the pluviograph that is nearest in horizontal distance, this may not necessarily produce the most accurate temporal pattern for the model subarea. As discussed in Book 2, Chapter 6, Section 2, a more accurate representation of the space-time rainfall pattern over a model subarea may be produced by judicious adjustment of the temporal pattern observed at a gauge or selection of a temporal pattern from a gauge that is not physically closest to the subarea centroid. Adjusting the assignment of temporal patterns to subareas in this manner may assist the practitioner in achieving a more robust calibration of the model parameters to the event. Adjustment of temporal patterns and temporal pattern assignment is therefore allowable, particularly if meteorological evidence is provided to support the decision.

The spatial or space-time pattern should be interpolated on to a regular grid that is constructed over the catchment. The resolution of the grid should be sufficiently fine to allow for spatial variations to be adequately represented. In most situations, the grid resolution should be selected so that there are at least 4 grid cells overlapping with the smallest subcatchment to be adopted in the model.

Remote sensing data, where available, may be used to estimate the space-time rainfall field of an event for catchment modelling system calibration. If used, the space-time rainfall field should be corrected using data from rainfall gauges, using a recommended approach for adjusting for the mean field bias.

6.3. Spatial and Space-Time Patterns for Design Flood Estimation

The aim of a design flood estimation should be to provide a probability neutral transformation between the design rainfall inputs and design flood characteristics

The space-time pattern or set of space-time patterns adopted for design flood estimation should be chosen in a manner that, when coupled with other aspects of the catchment modelling system, preserves the AEP of the design flood when derived from its causative rainfall.

6.3.1. Guidance for Catchments up to and Including 20 km^2: Single Uniform Spatial Pattern

Catchments with areas up to and including 20 km^2 are sufficiently small that there is little available data to derive a spatial pattern. For these catchments, it is usually acceptable to adopt a uniform spatial pattern.

If there is sufficient density of continuously rainfall gauges that have recorded a number of rainfall events, using this data to derive alternative (non-uniform) design spatial patterns may be considered.

6.3.2. Guidance for Catchments Greater than 20 km^2: Single Non-Uniform Spatial Pattern

As a minimum, it is recommended that a single non-uniform spatial pattern is applied to catchments with an area greater than 20 km^2. The non-uniform spatial pattern should be derived with the aim of replicating the systematic variation in spatial variability that would be expected across the catchment during rainfall events of similar AEP to the design floods that are being estimated.

For estimation of design flood events more frequent than and including the 1% AEP event, the spatial pattern should be estimated using the spatial pattern derived from the design rainfall grids (as discussed in Book 2, Chapter 3) across the catchment for the relevant IFD surface for the AEP and duration. In many cases there will be little relative variation in spatial distribution between probabilities or adjacent duration. Different spatial patterns could be applied for different durations. Alternatively, one spatial pattern may be estimated for the critical duration and this single spatial pattern may then be applied for all durations.

For estimation of design flood events rarer than 1% AEP with durations of 6 hours and less on catchment areas less than 1000 km^2, the spatial pattern should be derived in accordance with Woolhiser (1992) for the relevant duration. Use of different spatial patterns for different AEP ranges may introduce inconsistencies at the adjacent limits of each method, and if this is the case then any such inconsistencies should be smoothed in an appropriate fashion.

For estimation of design flood events rarer than 1% AEP with durations of 9 hours and greater or on catchment areas greater than 1000 km^2, the spatial pattern should be derived from the Topographic AdjustmentFactor (TAF) database derived from the generalised PMP method that is relevant for zone that the catchment is located in. Use of different spatial patterns for different AEP ranges may introduce inconsistencies at the adjacent limits of each method, and if this is the case then any such inconsistencies should be smoothed in an appropriate fashion.

For large studies and particularly for large catchments the practitioner should investigate and analyse the variability in spatial patterns between events. Where topography is dominant or large events are generally produced by a single rainfall mechanism there is likely to be only moderate variability between events but for some catchments there can be significant variations in space-time patterns between events. The practitioner should prepare and examine maps of the spatial pattern of rainfall for each event as a whole and for time slices, for example each 24 hour period, using an approach described in Book 2, Chapter 6, Section 2. These spatial patterns should be compared to rainfall accumulations from

Intensity Frequency Duration analysis for a relevant duration and AEP (refer to Jordan et al., 2015 for an example of this approach). Consistency in spatial patterns between events may reveal that it is acceptable to apply a single spatial pattern for all design flood estimates, particularly if it is consistent with the design rainfall analysis.

As discussed in Book 2, Chapter 4, Section 3, partial area storms should always be explicitly considered for catchments with an area exceeding 30 000 km^2 and it should be considered for catchments larger than 5000 km^2.

6.3.3. Alternative Approach: Monte Carlo Sampling from Separate Populations of Spatial and Temporal Patterns

A more advanced approach that may be justified would be Monte Carlo simulation by sampling from a set of space-time rainfall patterns across the catchment of interest. There are two potential options that may be considered for implementing this approach: (1) sampling from separate populations of spatial and temporal patterns for the catchment; or (2) sampling from a single set of “linked” space-time patterns for the catchment.

The first approach requires the assembly of:

- A set (or population) of spatial patterns across the catchment of interest from a number of observed rainfall events; and
- A set of temporal patterns from a number of observed rainfall events.

The spatial pattern of rainfall should be assembled for each event in the population, in accordance with the methods discussed in Book 2, Chapter 6, Section 2. The catchment average rainfall accumulation should be computed for each event and continuously rainfall gauges located in or near the catchment should be used to estimate the duration over which most of the total rainfall accumulation was likely to have fallen in the catchment. The estimated catchment average depth for the event and the estimated rainfall event duration should be used with the table of design rainfall estimates for the catchment, after application of the applicable ARF, to estimate the AEP of the rainfall for each event.

Similarly, the temporal pattern of rainfall should be assembled for each event in the population. The temporal pattern may be assembled at a single continuously rainfall gauge or from a combination of a number of continuously rainfall gauges located in the vicinity of the catchment or study area. The temporal pattern should be analysed to extract the maximum burst for a number of different durations. The rainfall accumulations over these bursts should be compared to the design rainfall estimates at the location of the rainfall gauge, without the application of the ARF, to estimate the AEP of the rainfall for each event.

A sample of patterns for use in the Monte Carlo simulation should be selected from the set of historical events that are available. A sufficient number of events should be selected to allow for a meaningfully large sample in the Monte Carlo simulation. It is expected that a minimum of five patterns would be required each of the sets of spatial and temporal patterns. However, events should only be selected for inclusion in the sample if they are relatively similar in terms of the AEP of the rainfall to the range of AEP that design flood estimates are being produced. Ideally, the spatial patterns and temporal patterns of events selected for the Monte Carlo sample should have an estimated AEP that is between 1/10 and ten times the AEP of the design flood event to be simulated. Adding more historic spatial patterns to an ensemble does not necessarily improve the simulation accuracy of a Monte Carlo model, as the additional patterns that are most likely to be added would be at the more common end of the AEP range. In many catchments, design floods of interest are caused by rainfall events with a specific hydrometeorological mechanism, which is then associated with a range of space-time rainfall patterns that are different to those observed in rainfall events caused by more commonly occurring hydrometeorological conditions.

Unless there is hydrometeorological evidence to the contrary, all potential spatial and temporal patterns in the sets available for sampling should be given equal probability of selection in the Monte Carlo simulation.

After the spatial and temporal patterns for the design rainfall burst have been selected stochastically, the patterns should be scaled so that the catchment average rainfall depth for the design rainfall burst matches the depth generated stochastically by the sampling scheme.

There has been limited assessment on methods for selection of space-time patterns for use in Monte Carlo simulation for design flood estimation. Further research should be conducted in this area to provide more robust guidance on the minimum number of temporal and spatial patterns in the sampling populations, the range of AEP represented by the populations of spatial and temporal patterns to be sampled compared to the AEP of the depth of the rainfall burst and the relative probabilities to be applied in the selection of spatial and temporal patterns.

6.3.4. Alternative Approach: Monte Carlo Sampling from Single Population of Space-Time Patterns

The foregoing approach ignores the potential dependency that exists between the temporal and spatial characteristics of storms. According, an alternative approach would involve sampling the space-time pattern for the event from a single population of space-time rainfall patterns over the catchment. The sample of space-time patterns may be assembled from space-time patterns of rainfall observed during historical rainfall events in the catchment. The space-time patterns should be assembled in accordance with the methods discussed in Book 2, Chapter 6, Section 2. The estimated catchment average depth for different burst durations within the event and the estimated rainfall event duration should be used with the table of design rainfall estimates for the catchment, after application of the applicable ARF, to estimate the AEP of the rainfall for each event.

It may be an option to transpose space-time rainfall patterns from an area with a good observational network for rainfall to a catchment with a poorer observational network. If this is done, the practitioner should only transpose (non-dimensional) space-time rainfall patterns from an area that is subject to rainfall events that are driven by similar hydrometeorological processes. The transposition region should be subject to similar orographic influences. In some cases, the space-time patterns may need to be rotated to maintain consistency between the spatial gradients in the space-time patterns and orographically influenced gradients in the design rainfall gridded data.

6.3.5. Spatial Patterns for Pre-Burst and Post-Burst Rainfall

If pre-burst or post-burst rainfall is to be applied, it is recommended that, unless there is evidence to the contrary, the spatial pattern applied to the pre-burst or post-burst rainfall should be the same as the spatial pattern applied for the design rainfall burst.

6.3.6. Spatial Patterns for Continuous Rainfall Series

Book 2, Chapter 7 discusses the production of continuous rainfall time series for production of design flood estimation using continuous simulation approaches. Book 2, Chapter 7 includes an approach to post-process the sequence of generated rainfall data for the catchment of interest so that the characteristics of the large and extreme rainfall events in the sequence reflect the Intensity Frequency Duration (IFD) statistics for the catchment of interest. If such an adjustment is conducted then it is recommended that the IFD statistics used as the basis of the adjustment are calculated for the catchment of interest after multiplying by the ARF that is applicable for the catchment area.

Book 2, Chapter 7 recommends that the IFD adjustment is applied for a set of target durations of either 6 minutes, 1 hour and 3 hours or for a set of durations of 6 minutes, 30 minutes, 1 hour, 3 hours, 6 hours and 12 hours. It is recommended therefore that the following procedure is adopted:

1. The IFD statistics for each of the target durations are calculated as the average of the point IFD statistics across the catchment, for each of the standard AEP (1EY to 1% AEP);

2. The ARF are computed for each of the target durations, at each of the standard AEP, for the total area of the catchment to be modelled;

3. The catchment IFD statistics are computed for each of the target durations, at each of the standard AEP, as the product of the point IFD statistics (from step 1) and the ARF (from step 2);

4. The catchment IFD statistics (from step 3) are applied in the modification procedure as the depths at the selected target durations.

The practitioner may be adopting a catchment model that allows for spatial distribution of the simulated rainfall sequence across the catchment. If this is the case, it is recommended that the generated sequence of rainfall is scaled for each portion of the model (subcatchment or grid cell as applicable to the particular model) to reflect the spatial distribution of rainfall that would be typically observed across the catchment. Parts of the catchment that are typically wetter would have rainfall depths applied in the model that are larger than the generated mean rainfall depth across the catchment but with the same timing and sequencing. Conversely, parts of the catchment that are typically drier would have rainfall depths applied in the model that are smaller than the generated mean rainfall depth across the catchment but with the same timing and sequencing.

Selection of an appropriate means of deriving the spatial pattern for a continuous simulation model that includes spatial distribution depends upon the AEP of the design events that are of most interest and the flood response characteristics of the catchment:

- When the focus is on estimation of floods with relatively frequent AEP (around 10% or more common) and for catchments with large moisture stores having significant relation between antecedent rainfall and the annual maximum flood, it is recommended that the spatial pattern applied in the model should be estimated from contours of mean annual rainfall;

- However when the focus is on estimation on floods with rarer AEP (5% or rarer) and for catchments where the influence of large moisture stores are less significant, it is recommended that the spatial pattern should be selected in a manner that is consistent with the recommendation for design event simulation (refer to Book 2, Chapter 6, Section 3 and Book 2, Chapter 6, Section 3 above).

6.4. Potential Influences of Climate Change on Areal Reduction Factors, Spatial and Space-Time Patterns

There is very little credible guidance on how climate change is projected to influence ARF, spatial patterns or space-time patterns of rainfall events used in design flood estimation. Abbs and Rafter (2009) used dynamic downscaling using a regional climate model to identify that increases in rainfall intensity are likely to be greater in those areas of south-east Queensland that are subject to orographic enhancement than those areas that are not. There is insufficient evidence to confirm whether this projection is an artefact of the downscaling approach and whether it would still apply if the dynamic downscaling model were forced with Global Circulation Model results from the more recent Intergovernmental Panel on Climate Change (2013). Even if it were proven for Southeast Queensland, it is not clear that the guidance would be more generally applicable to other parts of Australia.

It is recommended that until credible further studies are completed showing otherwise, for simulations applying projected climate change ARF, spatial patterns and space-time patterns of design rainfall should be the same as derived under existing climatic conditions.

6.5. Worked Examples

All of the worked examples in this chapter use data for the Stanley River catchment, which is in the upper part of the Brisbane River basin in Southeast Queensland. Worked example 1 demonstrates three different mathematical algorithms for estimating the spatial pattern of design rainfall for a particular rainfall event that occurred in January 2013. Worked examples 2 through 4 demonstrate the process for design flood estimation, from calculation of ARF and catchment average design rainfall estimates (Worked example 2), through calculation of a representative spatial pattern (Worked example 3), to production of design flood estimates using a runoff-routing model (Worked example 4).

6.5.1. Catchment Used for Worked Examples

The Stanley River catchment drains into Somerset Dam, which is in the upper part of the Brisbane River basin in South-east Queensland. Figure 2.6.1 shows a map of the 1324 km^2 shows the catchment area, with Somerset Dam located in the southwestern corner. For this worked example, the Stanley River catchment is modelled using a runoff-routing model with 76 subcatchments, with subareas as shown in Figure 2.6.1.

Design rainfall estimates are developed for the Stanley River at Woodford, which has a catchment area of 245 km^2. The catchment to Woodford is the north-eastern portion of the catchment to Somerset Dam and for this worked example it includes fifteen subcatchments in the runoff-routing model.

A significant feature of the Stanley River catchment is the appreciable gradient in rainfall that is typically observed during large rainfall events. Tropical cyclones, ex-Tropical Cyclones, East Coast Lows and other rainfall producing systems typically feed moisture into the catchment from the Pacific Ocean. Since the north-eastern part of the catchment is only 20 km from the coast but the western side of the catchment is almost 70 km from the coast, the typical direction of storm movement and typical direction of flow of warm moist air from the ocean results in a gradient of rainfall totals that reduce from east to west across the catchment in most rainfall events. The strength of the rainfall gradient is enhanced by orographic effects with the highest totals typically also occurring in the north-eastern part of the catchment.

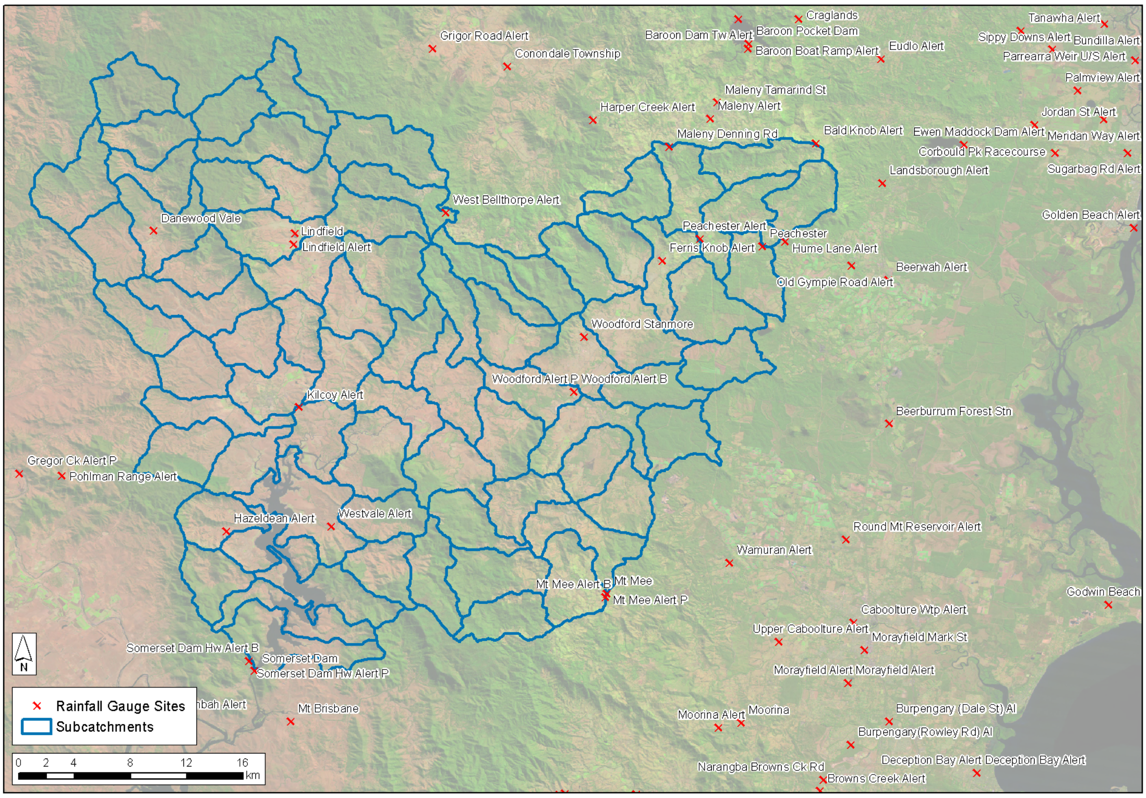


Figure 2.6.1. Stanley River Catchment, Showing Runoff-routing Model Subcatchments and the Locations of Daily rainfall and pluviograph gauges

6.5.2. Worked Example 1: Interpolation of Spatial Patterns for an Event Using Various Methods

Tropical Cyclone Oswald generated heavy rainfall in the Stanley River catchment between 23 and 29 January 2013, generating flooding in the catchment. Rainfall totals were observed at 20 continuous rainfall gauges around the Stanley River catchment, as shown in Figure 2.6.2 (rainfall data supplied by SeqWater). The blue circles are scaled in proportion to the rainfall depth recorded at the gauge.

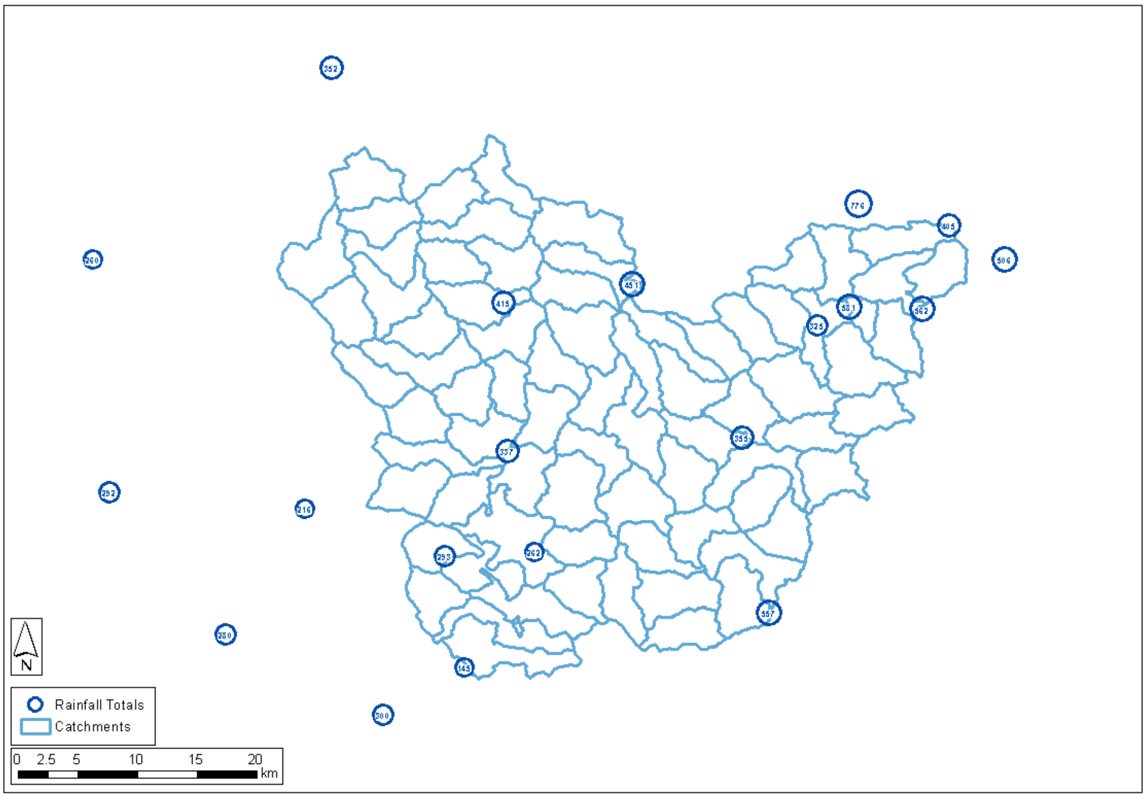


Figure 2.6.2. Rainfall totals (mm) Recorded at Rainfall Gauges for the January 2013 Event in the Vicinity of the Stanley River Catchment

The January 2013 rainfall event was used in this worked example to demonstrate various approaches to interpolation of spatial patterns of historical rainfall events, for the purpose of calibration of runoff-routing models. For all of the algorithms, the rainfall totals were first interpolated onto a 0.5 km resolution grid over the catchment. Rainfall totals for each of the 76 runoff-routing model subcatchments were then computed from the average of the rainfall totals at the grid cells that overlapped each subcatchment.

Rainfall totals were spatially interpolated using:

1. Thiessen polygons, as shown in Figure 2.6.3. Observed totals at gauges are shown as blue circles in both panels. The top panel shows interpolation to a 0.5 km grid (red shading), whilst the bottom panel shows calculated subcatchment average depths in mm (red circles);

2. Inverse distance weighting, as shown in Figure 2.6.4. Observed totals at gauges are shown as blue circles in both panels. The top panel shows interpolation to a 0.5 km grid (red shading), whilst the bottom panel shows calculated subcatchment average depths in mm (red circles); and

3. Ordinary Kriging, as shown in Figure 2.6.5. Ordinary Kriging was applied using a linear semi-variogram that was fitted to observed rainfall totals at the 20 gauges from the January 2013 event, as shown in Figure 2.6.6.Observed totals at gauges are shown as blue circles in both panels. The top panel shows interpolation to a 0.5 km grid (red shading), whilst the bottom panel shows calculated subcatchment average depths in mm (red circles).

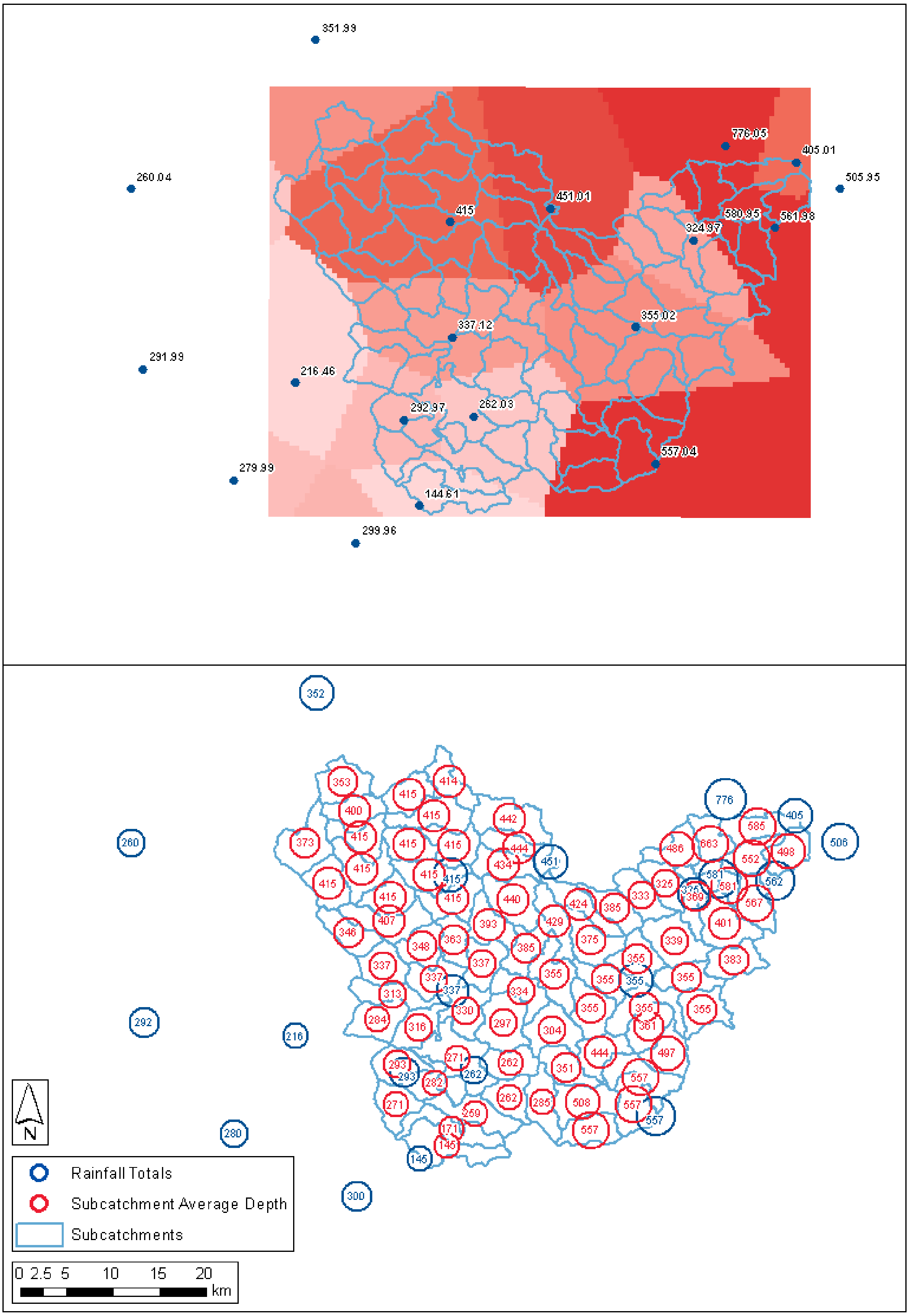


Figure 2.6.3. Application of Thiessen Polygons- Rainfall Totals for the January 2013 Event - Stanley River Catchment

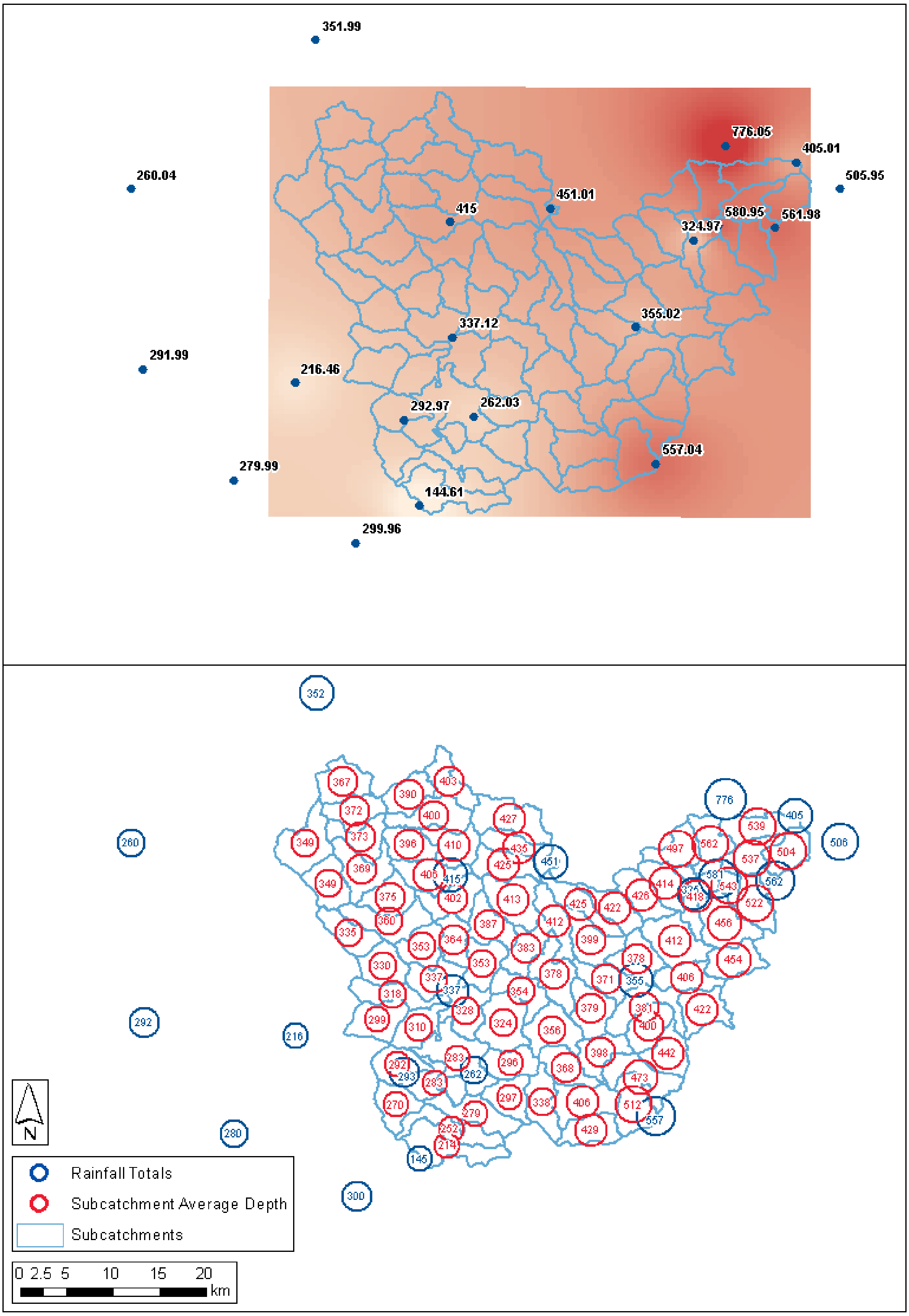


Figure 2.6.4. Application of Inverse Distance Weighting - Rainfall Totals for the January 2013 Event - Stanley River Catchment

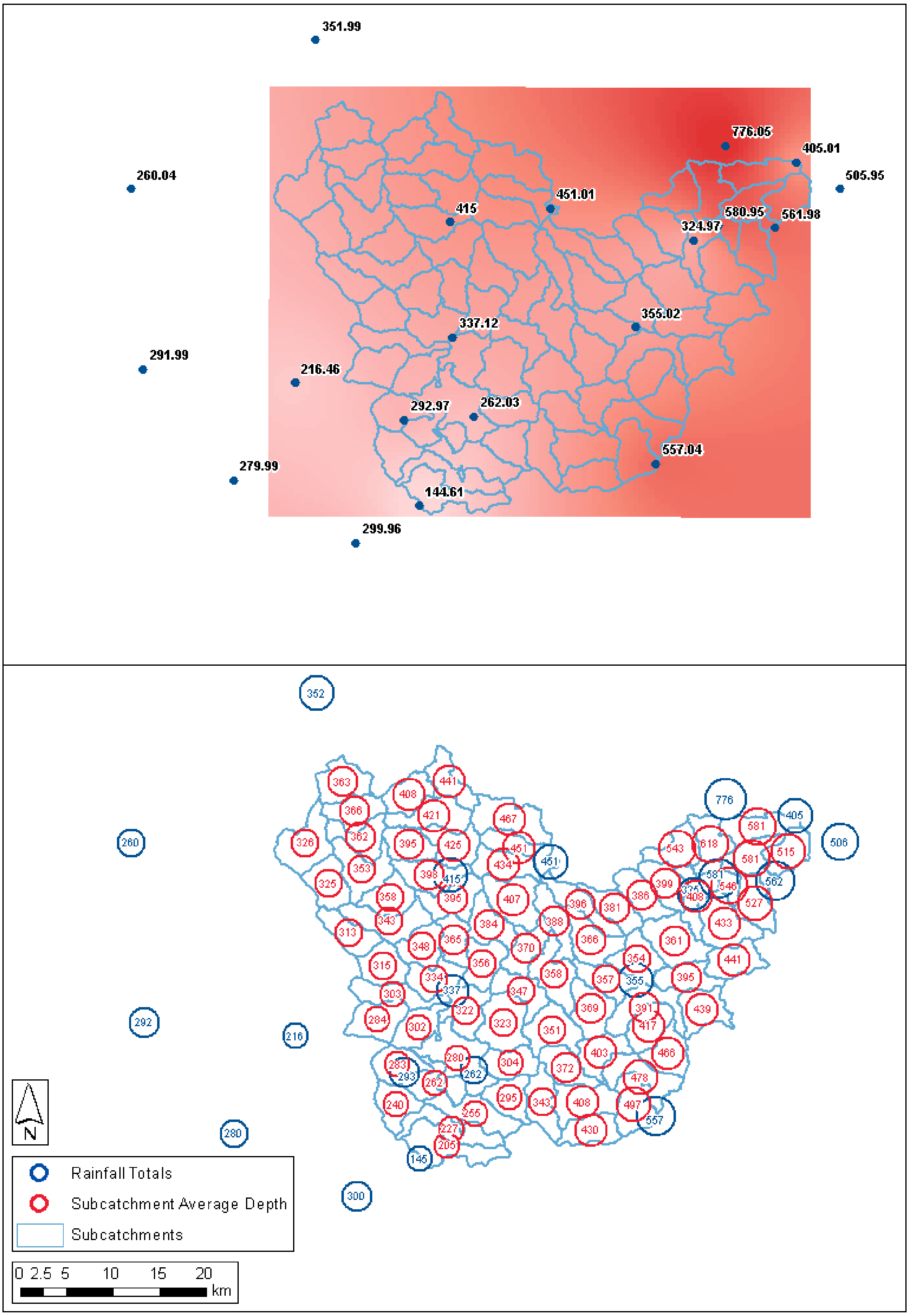


Figure 2.6.5. Application of Ordinary Kriging - Rainfall Totals for the January 2013 Event - Stanley River Catchment

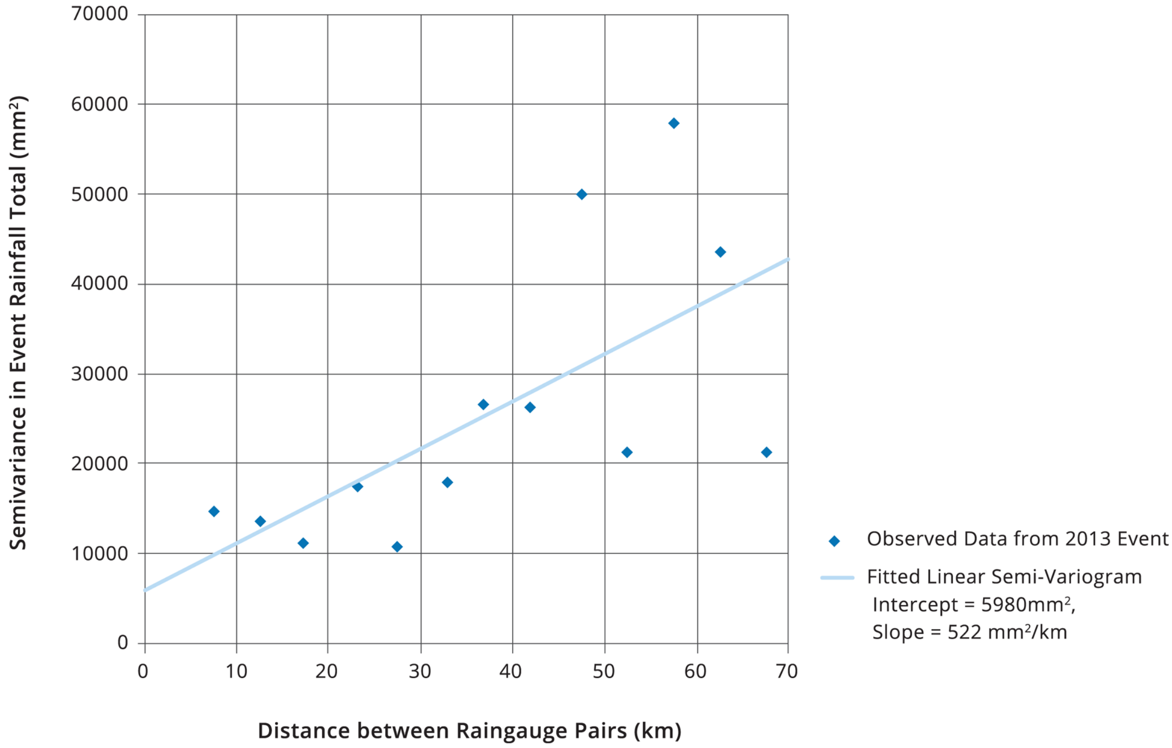


Figure 2.6.6. Observed Semi-variogram and Fitted Linear Semi-variogram for the January 2013 Rainfall Event for Stanley River catchment, Applied in the Ordinary Kriging Algorithm

6.5.3. Worked Example 2: Calculation of Catchment Average Design Rainfall Depths and Areal Reduction Factors

Design rainfall intensities were extracted at the centroids of each of the fifteen runoff-routing model subcatchments in the catchment of the Stanley River to Woodford from for the 1% AEP and 24 hour duration. The weighted average of the point rainfall depths was computed, as shown in Table 2.6.1.

The catchment area for the Stanley River to Woodford is 245.07 km^2. The ARF for 24 hour duration was computed by applying Equation (2.4.2), with the relevant coefficients for the East Coast North region (from the 1st row of Table 2.4.2). For the 1% AEP event the relevant ARF is given by:

$$\begin{aligned} &Aeral \quad reduction \quad factor \\ &= Min\Big\{1, [1 - a(Area^b - c\log_{10} Duration)Duration^{-d} \\ &+ eArea^f Duration^g (0.3 + \log_{10} AEP) \\ &+ h10^{iArea\frac{Duration}{1440}} (0.3 + \log_{10} AEP)]\Big\} \end{aligned} \qquad (2.6.3)$$

$$
\begin{aligned}
&Areal\quad reduction\quad factor\\
&= Min\{1, [1 - 0.327(245.07^{0.241} - 0.448\log_{10}(1440))1440^{-0.36}\\
&+0.00096 \text{ x } 245.07^{0.48}1440^{-0.21}(0.3 + \log_{10}(0.01))\\
&+0.012 \text{ x } 10^{-0.001 \text{ x } 245.07 \text{ x } \frac{1440}{1440}}(0.3 + \log_{10}0.01)]\Big\}
\end{aligned}
\tag{2.6.4}
$$

$$
Areal\, reduction\, factor_{(24hour)} = 0.929 \tag{2.6.5}
$$

Table 2.6.1. Calculation of Weighted Average of Point Rainfall Depths for the 1% AEP 24 hour Design Rainfall Event for the Stanley River at Woodford

Centroid Latitude (°)	Centroid Longitude (°)	Area (km^2)	1% AEP, 24 hour Design Point Rainfall Depth at Centroid (mm)	Design Depth x Area (ML)
-26.8467	152.8510	5.66	511.4	2896.0
-26.8060	152.8320	17.31	518.5	8973.2
-26.7895	152.8776	16.84	570.4	9605.7
-26.8201	152.8719	16.55	530.9	8787.9
-26.8631	152.8756	16.25	517.7	8414.3
-26.8110	152.8000	15.10	527.0	7956.5
-26.8135	152.9077	15.01	551.3	8276.7
-26.8998	152.7978	23.45	467.1	10954.8
-26.8828	152.8459	22.74	493.3	11218.6
-26.9171	152.7620	17.78	454.2	8076.1
-26.8557	152.7656	16.34	490.0	8007.1
-26.9190	152.8552	16.24	502.6	8164.3
-26.8568	152.8185	15.62	484.6	7571.4
-26.9354	152.8096	15.12	465.2	7031.9
-26.8445	152.7889	15.04	508.9	7654.0
Totals		245.07		123588.6
Weighted Average = 123588.6 / 245.07			504.3	

The catchment average design rainfall depth for 1% AEP, 24 hour duration for the Stanley River at Woodford was therefore computed by multiplying the ARF by the weighted average of the design point rainfall depths:

$$
\begin{aligned}
&\text{Catchment ave. design rainfall depth } (24\quad hour,\quad 1\%AEP)\\
&= \text{ARF } (24\quad hr) \times \text{Weighted ave. of design point rainfall depths } (24\quad hr,\quad 1\%AEP)
\end{aligned}
\tag{2.6.6}
$$

$$
\text{Catchment average design rainfall depth}(24hour, 1\%AEP) = 0.929 \times 504.3 \text{ mm} \tag{2.6.7}
$$

$$
\text{Catchment average design rainfall depth}(24hour, 1\%AEP) = 468.6 \text{ mm} \tag{2.6.8}
$$

The calculation was repeated for the catchment of the Stanley River to Woodford for each combination of standard durations between 3 and 72 hours and the 1 Exceedance per Year to the 1% AEP. These computations are shown for the Stanley River catchment to Woodford in Table 2.6.2.

The catchment area for the Stanley River to Somerset Dam is 1324 km^2. The calculation of catchment average design rainfall intensities, after application of areal reduction factors, is shown in Table 2.6.3. Comparing the top panels of Table 2.6.2 and Table 2.6.3, for the corresponding AEP and durations the weighted averages of the point rainfall depths for the catchment to Somerset Dam are less than those for Woodford, due to the gradient in the IFD grids. Comparing the middle panels of Table 2.6.2 and Table 2.6.3, for the corresponding AEP and durations the ARF catchment to Somerset Dam are less than those for Woodford because the catchment area to Somerset Dam is larger. Hence comparing the bottom panels of Table 2.6.2 and Table 2.6.3, for the corresponding AEP and durations the catchment average design rainfall depths to Somerset Dam are less than those for Woodford.

Table 2.6.2. Stanley River Catchment to Woodford: Calculation of Catchment Average Design Rainfall Depths (bottom panel) from Weighted Average of Point Rainfall Depths (top panel) and Areal Reduction Factors (middle panel)

Weighted Average of Point Rainfall Depths (mm)							
Duration (hours)	**1 Exceedance per Year**	**Annual Exceedance Probability**					
		50%	**20%**	**10%**	**5%**	**2%**	**1%**
3	54.1	61.6	85.7	102.8	120.0	143.5	162.1
6	70.5	81.5	117.3	142.8	168.7	204.6	233.3
12	94.4	110.8	164.9	203.8	243.8	299.5	344.5
24	128.1	152.0	231.4	289.5	349.8	434.8	504.3
48	170.2	202.2	310.5	391.2	476.1	598.0	699.3
72	195.5	231.7	355.3	448.5	547.6	691.4	812.1
Areal Reduction Factor							
Duration (hours)	**1 Exceedance per Year**	**Annual Exceedance Probability**					
		50%	**20%**	**10%**	**5%**	**2%**	**1%**
3	0.841	0.835	0.812	0.794	0.776	0.753	0.735
6	0.879	0.876	0.864	0.854	0.845	0.832	0.823
12	0.909	0.907	0.901	0.896	0.891	0.884	0.879
24	0.945	0.944	0.940	0.938	0.935	0.932	0.929
48	0.959	0.959	0.957	0.955	0.954	0.951	0.950
72	0.966	0.966	0.964	0.963	0.962	0.961	0.959
Catchment Average Design Rainfall Depth (mm)							

Duration (hours)	1 Exceedance per Year	Annual Exceedance Probability					
		50%	**20%**	**10%**	**5%**	**2%**	**1%**
3	45.5	51.4	69.6	81.6	93.1	108.0	119.2
6	62.0	71.4	101.3	122.0	142.5	170.2	191.9
12	85.8	100.5	148.5	182.6	217.1	264.8	302.9
24	121.1	143.4	217.6	271.5	327.1	405.2	468.6
48	163.3	193.9	297.1	373.6	454.0	569.0	664.2
72	188.8	223.7	342.6	432.0	526.8	664.2	779.2

Table 2.6.3. Stanley River Catchment to Somerset Dam: Calculation of Catchment Average Design Rainfall Depths (bottom panel) from Weighted Average of Point Rainfall Depths (top panel) and Areal Reduction Factors (middle panel)

Weighted Average of Point Rainfall Depths (mm)							
Duration (hours)	**1 Exceedance per Year**	**Annual Exceedance Probability**					
		50%	**20%**	**10%**	**5%**	**2%**	**1%**
3	48.3	54.8	75.7	90.2	104.7	124.2	139.5
6	61.0	70.0	99.2	119.7	140.3	168.5	190.9
12	79.0	91.9	133.9	163.9	194.4	236.6	270.4
24	103.8	121.9	182.2	226.1	271.4	335.1	387.0
48	134.5	158.7	240.6	301.7	366.0	458.6	535.5
72	153.0	180.4	274.2	345.2	420.9	531.1	624.0
Areal Reduction Factor							
Duration (hours)	**1 Exceedance per Year**	**Annual Exceedance Probability**					
		50%	**20%**	**10%**	**5%**	**2%**	**1%**
3	0.735	0.727	0.694	0.669	0.644	0.611	0.586
6	0.796	0.792	0.774	0.761	0.748	0.731	0.718
12	0.843	0.841	0.832	0.826	0.826	0.811	0.804
24	0.900	0.899	0.896	0.894	0.892	0.889	0.887
48	0.924	0.924	0.921	0.920	0.918	0.916	0.914
72	0.936	0.935	0.933	0.932	0.930	0.928	0.926
Catchment Average Design Rainfall Depth (mm)							
Duration (hours)	**1 Exceeda**	**Annual Exceedance Probability**					

	nce per Year						
		50%	**20%**	**10%**	**5%**	**2%**	**1%**
3	35.5	39.9	52.5	60.4	67.4	75.9	81.8
6	48.6	55.4	76.8	91.1	105.0	123.2	137.0
12	66.6	77.3	111.4	135.3	159.3	191.8	217.4
24	93.3	109.6	163.3	202.2	242.1	298.1	343.4
48	124.3	146.6	221.7	277.5	336.0	420.0	489.5
72	143.2	168.7	255.9	321.6	391.5	492.9	578.1

6.5.4. Worked Example 3: Calculation of Spatial Pattern for Design Flood Estimation

Book 2, Chapter 6, Section 3 recommends that estimation of design flood events of 1% AEP and more frequent, the spatial pattern for design event should be estimated using the spatial pattern derived from the design rainfall grids across the catchment for the 1% AEP and for a duration that is anticipated to correspond to the duration of the rainfall burst that is likely to be critical at the specified location.

For the Stanley River catchment, this approach was demonstrated using the 24 hour duration IFD data. For the catchment to Woodford, point rainfall depths at each of the subcatchment centroids were divided by the weighted average of the point rainfall depths to derive the non-dimensional spatial pattern, as computed in Table 2.6.4 and mapped in the top panel of Figure 2.6.7. To model the 1% AEP 24 hour design flood event for the catchment, the non-dimensional spatial pattern was multiplied by the catchment average design rainfall depth to Woodford for this duration (468.6 mm, after application of the ARF), as computed in Table 2.6.4 and mapped in the top panel of Figure 2.6.8.

The process was repeated for the Stanley River to Somerset Dam, with the map of the non-dimensional spatial pattern shown in the bottom panel of Figure 2.6.7 and the map of the design depths for the 1% AEP, 24 hour duration event in the bottom panel of Figure 2.6.8.

Table 2.6.4. Calculation of Design Spatial Pattern for Stanley River at Woodford

Centroid Latitude (°)	Centroid Longitude (°)	1% AEP, 24 hour Design Point Rainfall Depth at Centroid (mm)	Point Rainfall Depth Divided by Weighted Average of Point Rainfall Depths (%)	Depth to be Applied to Model 1% AEP, 24 hour Design Event (mm)
-26.8467	152.8510	511.4	101.4	475.2
-26.8060	152.8320	518.5	102.8	481.8
-26.7895	152.8776	570.4	113.1	530.0
-26.8201	152.8719	530.9	105.3	493.3
-26.8631	152.8756	517.7	102.7	481.0
-26.8110	152.8000	527.0	104.5	489.7
-26.8135	152.9077	551.3	109.3	512.3

Centroid Latitude (°)	Centroid Longitude (°)	1% AEP, 24 hour Design Point Rainfall Depth at Centroid (mm)	Point Rainfall Depth Divided by Weighted Average of Point Rainfall Depths (%)	Depth to be Applied to Model 1% AEP, 24 hour Design Event (mm)
-26.8998	152.7978	467.1	92.6	434.0
-26.8828	152.8459	493.3	97.8	458.4
-26.9171	152.7620	454.2	90.1	422.0
-26.8557	152.7656	490.0	97.2	455.3
-26.9190	152.8552	502.6	99.7	467.0
-26.8568	152.8185	484.6	96.1	450.3
-26.9354	152.8096	465.2	92.2	432.3
-26.8445	152.7889	508.9	100.9	472.9
Weighted Average		504.3	100.0	468.6

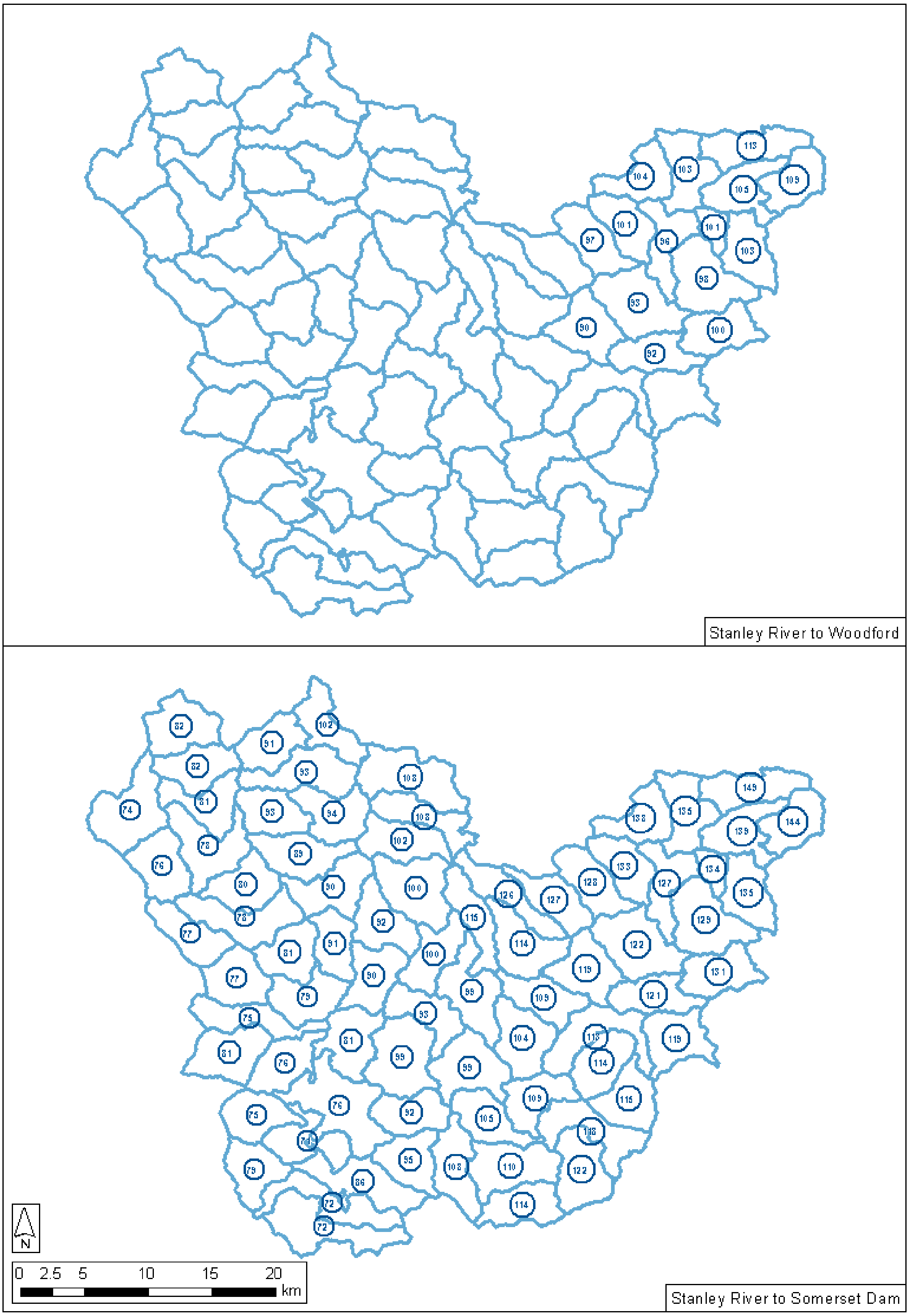


Figure 2.6.7. Non-dimensional Spatial Pattern (percentage of catchment average design rainfall depths) for Events with AEP of 1% and more Frequent for Stanley River to Woodford (top panel) and Stanley River to Somerset Dam (bottom panel)

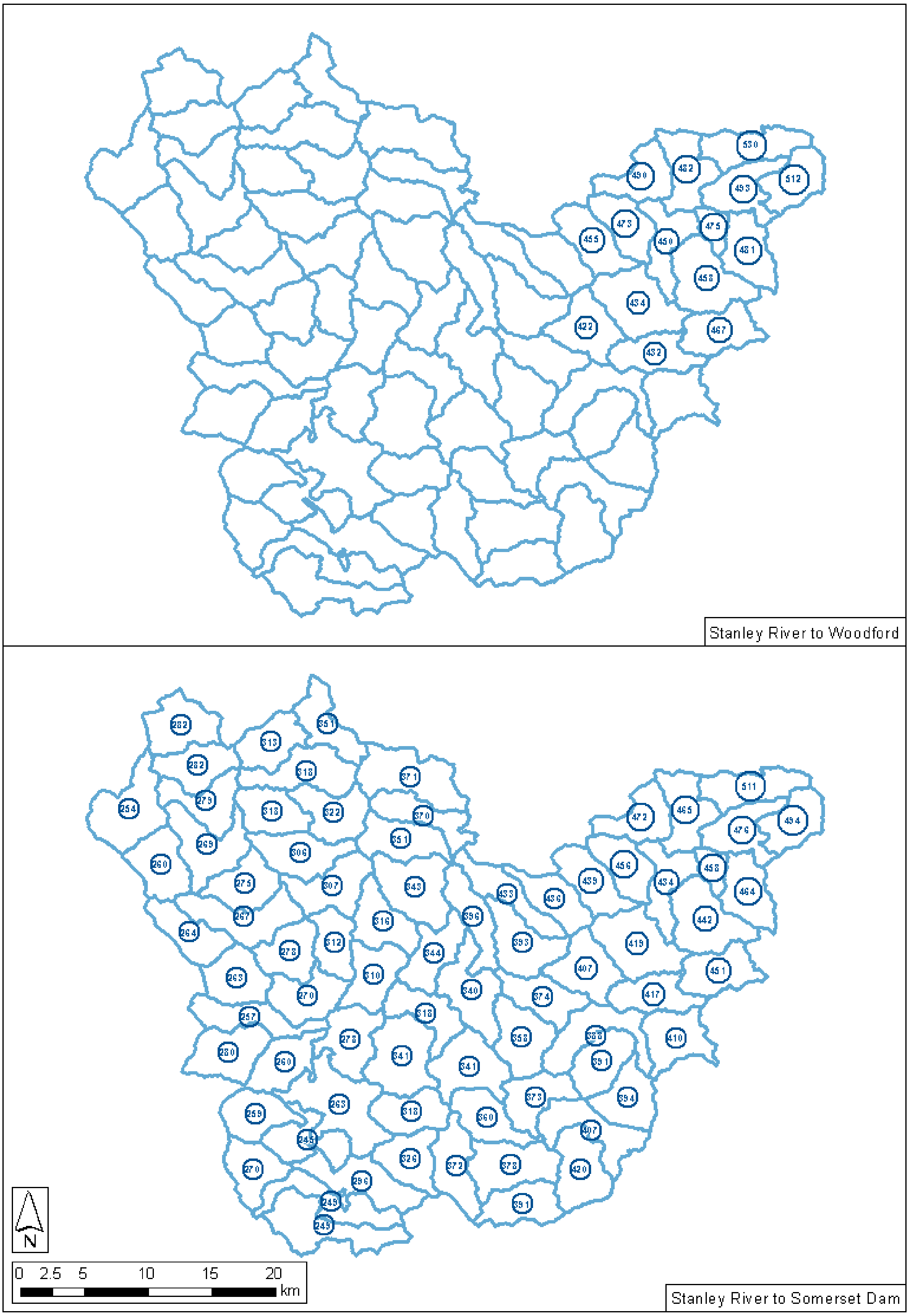


Figure 2.6.8. Design Spatial Pattern of Design Rainfall Depths 1% AEP 24 hour Event for Stanley River to Woodford (top panel) and Stanley River to Somerset Dam (bottom panel)

6.5.5. Worked Example 4: Application to Design Flood Estimation

Design flood peak estimates were produced for the Stanley River at its outlet (inflow to Somerset Dam) using a RORB runoff-routing model of the catchment. A more complete description of this case study is contained in Jordan et al. (2015).

Design peak flow estimates at Somerset Dam inflow were produced from a number of Monte Carlo simulations that were implemented within RORB. There were a number of common elements to all of these simulations:

- all adopted the same catchment average design IFD information multiplied by the areal reduction factor for the applicable duration from Jordan et al. (2013);
- all were run using the stratified Monte Carlo sampling scheme that is implemented within RORB (Laurenson et al., 2010);
- all were run for rainfall burst durations of 6, 9, 12, 18, 24, 36 and 48 hours, with the peak flow defined by the highest flow from among these durations at each AEP;
- all simulations sampled from the same non-dimensional probability distribution of initial loss values defined by Ilahee (2005), scaled by a median initial loss of 40 mm;
- all adopted a constant continuing loss rate of 1.7 mm/hour across all subcatchments;
- all adopted a RORB non-linearity parameter, *m*, value of 0.8; and
- all simulations adopted RORB delay parameter, k_c , values of 20 for the catchment upstream of Peachester, 20 for the catchment between Peachester and Woodford, 16 for the catchment upstream of Mount Kilcoy and 45 for the residual catchment to Somerset Dam inflow.

The Monte Carlo simulations differed from one another in their approach to sampling of spatial, temporal and space-time patterns across the catchment, as shown in Table 2.6.5.

Table 2.6.5. RORB Model Scenarios Run for Worked Example on Stanley River Catchment to Somerset Dam

<table>
<tr><th>Case</th><th>Spatial Pattern(s)</th><th>Temporal Pattern(s)</th></tr>
<tr><td>1</td><td>Single spatial pattern derived from IFD analysis, 1% AEP 24 hour spatial pattern</td><td>Random sampling from a set of 13 temporal patterns for each duration, derived from the bursts of the corresponding duration within the 13 selected events listed in Table 1 of Jordan et al. (2015)</td></tr>
<tr><td>2</td><td colspan="2">Random sampling from a set of 13 space-time patterns for each duration, derived from the bursts of the corresponding duration within the 13 selected events listed in Table 1 of Jordan et al. (2015)</td></tr>
</table>

Sinclair Knight Merz (2013) fitted a Generalised Extreme Value (GEV) distribution to the estimated annual maxima inflows to Somerset Dam over the period between 1955 and 2013.

The estimated inflow flood peak for the 1893 flood of 6200 m³/s was included as a censored flow in the analysis. The distribution fitted to the estimated observed inflows was used to test the performance of the RORB model simulations.

Figure 2.6.9 shows that both cases of RORB model simulations all provide an excellent match to the fitted flood frequency quantiles across the range between 5% and 0.2% AEP. Design peak inflow floods to Somerset Dam were insensitive to whether space-time patterns are randomly sampled or only temporal patterns are randomly sampled in the Monte Carlo simulation (case 1 versus case 2).

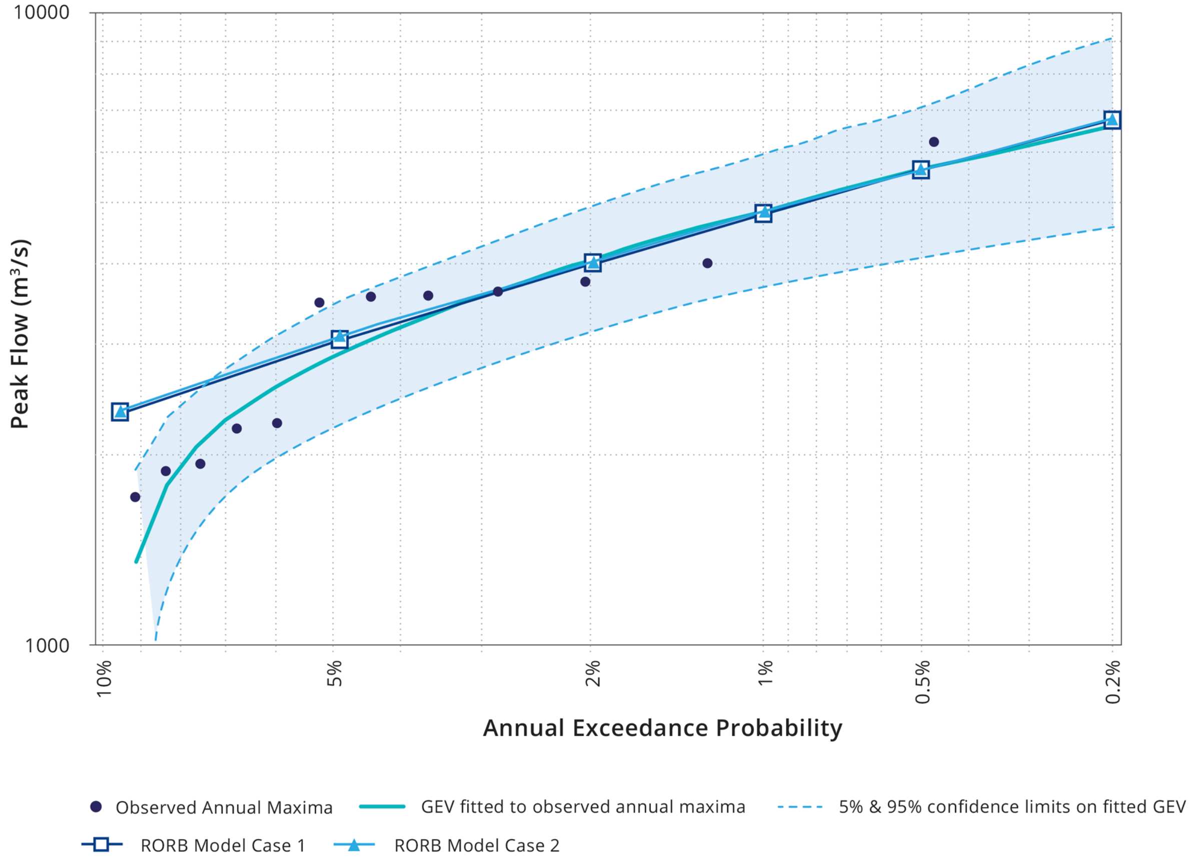


Figure 2.6.9. Flood Frequency Curves for Stanley River at Somerset Dam Inflow Derived from Analysis of Estimated Annual Maxima and from RORB Model Simulations

6.6. Recommended Further Research

6.6.1. Deriving Spatial and Space-Time Patterns of Rainfall for Events

The capacity to collect and archive remotely sensed rainfall estimates and to provide that information to practitioners is growing. It is recommended that the Bureau of Meteorology continues to invest in routinely archiving remotely sensed rainfall data, particularly from its network of ground based weather radars. It is recommended that the Bureau of Meteorology continues to expand the provision of quality controlled and bias corrected space-time rainfall estimates to practitioners, for use across the industry. It is recommended that tools should be further developed and disseminated to practitioners to facilitate the use of remotely sensed rainfall data.

It is recommended that further research is conducted into quality control of remotely sensed estimates of the space-time pattern of rainfall.

It is recommended that further research is conducted to improve methods for mean field bias correction of remotely sensed rainfall data. The recommendations on the approaches that should be adopted for mean field bias correction should be updated in these guidelines in accordance with the findings from this research.

At the time of writing, there was not an agreed optimum method for deriving space-time rainfall patterns from rainfall gauge data for Australian catchments, although Verworn and Haberlandt (2011) provide reasonable guidance. It is recommended that further research is conducted to identify a superior method (or set of potential methods) that are demonstrated to reliably produce more accurate estimation of the space-time rainfall field from gauge observations. It may be that the optimum method depends upon meteorological characteristics of the storm, density of rainfall gauges, orographic characteristics of the region or other factors. It is recommended that further research is conducted to explore these influences on the selection of optimum spatial and space-time interpolation methods for flood model calibration and design flood estimation.

6.6.2. Space-Time Patterns for Calibration of Rainfall Runoff Models to Historical Floods

As discussed in Book 2, Chapter 6, Section 2, the calibrated parameter values for a rainfall runoff model for a particular flood event may be sensitive to the method used to derive the space-time rainfall field for the event, particularly where the field is interpolated from a network of rain gauges only. It is recommended that further research is conducted into the sensitivity of rainfall-runoff routing model parameter estimation to assumptions made in the process of estimating the space-time rainfall field gauged rainfall data.

6.6.3. Spatial and Space-Time Patterns for Design Flood Estimation

It is recommended that further research is conducted into hydrometeorological drivers for space-time rainfall patterns that lead to flood events across different regions of Australia. The research should be used to inform practitioners on how they may choose between the space-time patterns of rainfall from different historical rainfall events to form the populations of space-time, spatial and temporal patterns in design flood simulation schemes. Research may investigate seasonal influences on space-time patterns of rainfall for use in design flood estimation.

There has been limited assessment on methods for selection of space-time patterns for use in Monte Carlo simulation schemes for design flood estimation. Further research should be conducted in this area, to provide more robust guidance on:

- The minimum number of space-time or temporal and spatial patterns in the sampling population(s);
- The range of AEP represented by the populations of space-time or spatial and temporal patterns to be sampled compared to the AEP of the depth of the rainfall burst; and
- The relative probabilities to be applied in the selection of patterns from the relevant populations.

It is recommended that further research is conducted into the validity of transposing space-time patterns from one location to another. The research should assist in defining valid regions over which transposition of space-time patterns is acceptable and conversely boundaries between regions over which transposition should not occur. The research should also consider other aspects of transposition, such as the validity or otherwise of rotating space-time patterns and the maximum recommended angles for rotation.

Further research should be conducted into methods for stochastic generation of space-time rainfall patterns. The research should investigate how orographic influences should be incorporated into the stochastic generation algorithms in a way that replicates the space-time variability of rainfall observed in historic rainfall events. Research should also develop more definitive guidance on appropriate statistical tests to demonstrate that the stochastically generated space-time rainfall patterns replicate the space-time statistical characteristics of historical rainfall events that are sufficiently large to have caused flood events.

6.6.4. Potential Influences of Climate Variability and Climate Change

Climatic variability at inter-decadal scales is likely to influence the relative occurrence and severity of different types of heavy rainfall events. Hydrometeorological understanding of the connection between storm types and ARFs may enable predictions of the future trend in ARFs that will occur as the climate changes over coming decades. Hydrometeorological understanding of the connection between storm types and space-time rainfall patterns may also allow for more accurate guidance to practitioners on the potential changes in space-time patterns that is predicted as a result of climate change.

6.7. References

Abbs, D. and Rafter, T. (2009), Impact of Climate Variability and Climate Change on Rainfall Extremes in Western Sydney and Surrounding Areas: Component 4 - dynamical downscaling, CSIRO.

Ball, J.E. and Luk, K.C. (1998), Modelling spatial variability of rainfall over a catchment, J. Hydrologic Engineering, 3(2), 122-130.

Barnston, A.G., (1991), An empirical method of estimating raingauge and radar rainfall measurement bias and resolution, J. Applied Meterology, 30: 282-296.

Bradley, S.G., Gray, W.R., Pigott, L.D., Seed, A.W., Stow, C.D. and Austin, G.L. (1997), Rainfall redistribution over low hills due to flow perturbation, J. Hydrology, 202, 33-47.

Collier, C.G. (1996), Applications of Weather Radar Systems: A Guide to Uses of Radar in Meteorology and Hydrology. 2d ed. John Wiley, p: 383

Ilahee, M. (2005), Modelling Losses in Flood Estimation, A thesis submitted to the School of Urban Development Queensland University of Technology in partial fulfilment of the requirements for the Degree of Doctor of Philosophy, March 2005.

Intergovernmental Panel on Climate Change, (2013), Climate Change 2013: The Physical Science Basis, Contribution of Working Group I to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change [Stocker,T.F., D. Qin, G.-K. Plattner, M. Tignor, S.K. Allen, J. Boschung, A. Nauels, Y. Xia, V. Bex and P.M. Midgley (eds.)]. Cambridge University Press, Cambridge, United Kingdom and New York, NY, USA, p: 1535.

Jones, D., Wang, W. and Fawcett, R. (2009), High-quality spatial climate data-sets for Australia, Australian Meteorological and Oceanographic Journal, 58: 233-248.

Jordan, P., Nathan, R. and Seed, A. (2015), Application of spatial and space-time patterns of design rainfall to design flood estimation. Engineers Australia, Hydrology and Water Resources Symposium 2015.

Jordan, P., Weinmann, P.E., Hill, P. and Wiesenfeld, C. (2013), Collation and Review of Areal Reduction Factors from Applications of the CRC-FORGE Method in Australia, Final Report, Australian Rainfall and Runoff Revision Project 2: Spatial Patterns of Rainfall, Engineers Australia, Barton, ACT.

Joss, J. and Waldvogel, A. (1990), Precipitation measurement and hydrology, in Radar in Meteorology, D. Atlas, Ed., American Meteorological Society, pp: 577-606.

Laurenson, E.M., Mein, R.G. and Nathan, R.J. (2010), RORB Version 6, Runoff Routing Program, User Manual, Version 6.14, Monash University and Sinclair Knight Merz, Melbourne, Victoria.

Seed, A.W. and Austin, G.L. (1990), Sampling errors for raingauge derived mean-areal daily and monthly rainfall, J. Hydrology, 118: 163-173.

Sinclair Knight Merz, (2013), Brisbane River Catchment Dams and Operational Alternatives Study, Generation of Inflow Hydrographs and Preliminary Flood Frequency Analysis, Revision 1, Brisbane, Queensland.

Umakhanthan, K. and Ball, J.E. (2005), Rainfall models for catchment simulation. Australian Journal of Water Resources, 9(1), 55-67.

Urbonas, B.R., Guo, J.C.Y. and Janesekok, M.P. (1992), Hyetograph density effects on urban runoff modelling, Proc. International Conference on Computer Applications in Water Resources, Tamkang University, Tamsui, Taiwan, pp: 32-37.

Verworn, A. and Haberlandt, U. (2011), Spatial interpolation of hourly rainfall - effect of additional information, variogram inference and storm properties, Hydrology and Earth System Sciences, 15: 569-584.

Woolhiser, D.A. (1992), Modeling daily precipitation - progress and problems, in Statistics in the environmental and earth sciences, edited by A.T. Walden and P. Guttorp, p: 306, Edward Arnold, London, U.K.

Chapter 7. Continuous Rainfall Simulation

Ashish Sharma, Ratnasingham Srikanthan, Raj Mehrotra, Seth Westra, Martin Lambert

Chapter Status	Final
Date last updated	14/5/2019

7.1. Use of Continuous Simulation for Design Flood Estimation

Design floods can be estimated based on either historical flood data using at-site Flood Frequency Analysis or Regional Flood Frequency Estimation, or derived using rainfall data and a suitable hydrologic model to simulate flows. When using a hydrologic model, two options exist. The first is the event-based approach which converts the design rainfall storm to a corresponding design flood using a hydrologic model. The second is the continuous simulation approach, which converts a continuous rainfall time series to a flow time series using a hydrologic model, followed by the application of a frequency analysis on the flows to estimate the design flood. The generation of the rainfall time series for this latter approach is the focus of this chapter.

While a clear case is often present when deciding between a Flood Frequency Analysis and an event-based approach for estimating the design flood, it is less clear when a continuous simulation approach should be used in place of an event-based approach. In general, the primary benefits of continuous simulation approaches arise when the relationship between the catchment's antecedent moisture stores and the flood-producing rainfall event are not independent of each other, or change over time (Blazkova and Beven, 2002; Boughton and Droop, 2003; Cameron et al., 2000; Lamb and Kay, 2004). Continuous simulation allows an explicit representation of the joint probability of antecedent moisture conditions and flood-producing rainfall data, which can be challenging for event-based approaches. Therefore key areas where continuous simulation approaches are likely to be useful include the following:

- Catchments with large moisture stores which have a significant relationship between antecedent rainfall and the annual maximum flood (Pathiraja et al., 2012);
- Examining the joint probability of flooding arising from the confluences of streams which are subject varying spatial rainfall distributions;
- Situations where the relationship between historical antecedent conditions and flood-producing rainfall are not representative of the design period. This is may occur as a result of climate change, but may also be relevant when calibrating over a period that is over-represented in terms of El Niño or La Niña events (e.g. Pui et al. (2011));
- Situations requiring a quantification of the uncertainty of flood quantities, where continuous simulation approaches can provide a natural method for representing the dependence between flood-producing rainfall and the antecedent conditions;
- Situations where the initial level of flood and reservoir storages are unknown and these influence the resulting downstream flood flows; and

• When using a virtual laboratory to test proposed simpler event-based approaches.

Consider the example in Figure 2.7.1 which uses data for a South Australian catchment to illustrate the workings of the three approaches used for design flood estimation. While Figure 2.7.1 uses the 90 day antecedent rainfall to illustrate its relation with the extreme rainfall, similar joint relationships could exist between antecedent rainfall for longer periods, or other more subtle rainfall characteristics that are difficult to summarise using a simple metric.

The first two panels illustrate the working of a flood frequency or event-based modelling approach for design flood estimation. The last panel illustrates a continuous simulation model that attempts to capture the strong relationship in extreme rainfall with the 90 day Antecedent rain.

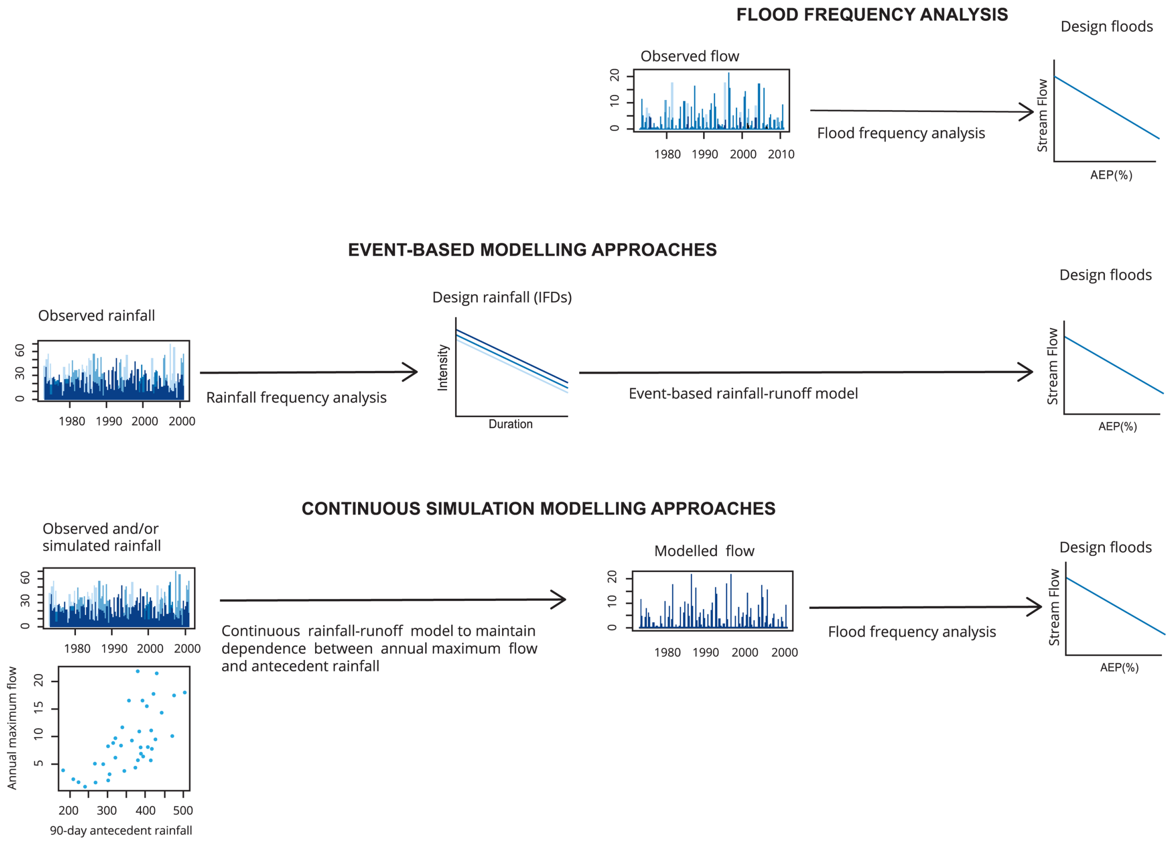


Figure 2.7.1. Flood Events for a Typical Australian Catchment - Scott Creek, South Australia

As highlighted in Figure 2.7.1, continuous simulation approaches for design flood estimation require continuous rainfall sequences as the primary data input. Although continuous rainfall data exist in some locations for periods of several decades or longer, for most locations in Australia the continuous data is either unavailable, too short or of insufficient quality to support continuous rainfall-runoff modelling. This chapter therefore presents the basis and techniques for stochastically generating continuous rainfall records in a catchment. Also discussed are:

i. Generic issues regarding the accuracy of rainfall observations and methods for identifying errors in rainfall time series;

ii. Approaches for infilling rainfall data at a point location;

iii. When to generate multi-site data as compared to lumped or single site rainfall;

iv. Approaches for generating data at locations where rainfall records are not available; and

v. Implications of non-stationarity in the rainfall record as a result of urbanisation and climate change.

Book 2, Chapter 7, Section 2 discusses the approaches used to prepare rainfall data for use in stochastic generation or other modelling studies. Book 2, Chapter 7, Section 3 discusses a conceptual framework that underlies stochastic rainfall generation at point or multiple locations. Alternatives for generation of daily rainfall are discussed Book 2, Chapter 7, Section 4, while Book 2, Chapter 7, Section 4 discusses alternatives for disaggregation of daily rainfall to sub-daily time scales. Alternatives that generate continuous rainfall sequences without reference to a daily total at point and multiple locations conclude the presentation. Worked examples illustrating the applications of some of the models presented are included to assist with practical implementations (Refer to Book 2, Chapter 7, Section 4).

7.2. Rainfall Data Preparation

7.2.1. Errors in Rainfall Measurements

Stochastic rainfall generation aims to generate continuous rainfall sequences that are representative of the underlying climate. Hence, it is important that the observed rainfall is a true representation of the underlying climate, and is not influenced by potential measurement or sampling inaccuracies that may lead to biased rainfall sequences. The first step of stochastic rainfall generation is to identify and correct for noticeable errors in the observed rainfall record.

Rainfall measurements can be susceptible to a range of errors:

- *Effect of wind, wetting, evaporation and splashing on daily rainfall measurements* – The World Meteorological Organisation (World Meteorological Organisation, 1994) states that these factors can result in the measured daily rainfall being less than the true rainfall by anywhere between three and 30%.

- *Errors in tipping bucket measurements* – Tipping bucket rainfal gauges are the preferred means of continuous rainfall measurement over the world. While reasonably accurate at low rainfall intensities, tipping bucket rainfall gauges can underestimate the rainfall when intensities are high due to the water lost as a result of the tipping motion of the rainfall gauge. Typical errors for intensities greater than 200 mm/hr can range from 10-15% of the true rainfall (La Barbera et al., 2002). A simple model for characterising gauge measurement errors was proposed by Ciach (2003), marking them inversely proportional to the measured rainfall intensity.

- *Homogeneity of rainfall measurements* - The double mass curve is a commonly used technique to identify and correct for changes in the exposure or location of the gauge, changes in the manner in which data is collected, or any other changes that result in a systematic bias in the measurements compared to the general trend in nearby locations. An example of such change is illustrated in Figure 2.7.2 for a hypothetical rainfall record, where the change appears to have occurred around 1955 with the slope of the mass curve changing from 0.95 to 0.75 from that point onwards. Changes such as those illustrated in Figure 2.7.2 should be investigated in greater detail and corrective measures (e.g.

multiplicative scaling) may need to be used. Note that similar comparative checks can also be used in the context of identifying ‘odd’ rainfall gauge locations from the regional average (slope of the double mass curve will be significantly different to 1).

- *Homogeneity of gridded rainfall* – An important source of error in gridded rainfall data is related to the use of varying number of rainfall gauges over time in the process of reconstructing records. (Hasan et al., 2014) investigated at the extent of this variability in the context of radar rainfall estimation expressed this as a function of the grid size and the density of gauges within the grid. Using daily gauge data for Sydney, the coefficient of variation of the rainfall for a 1 km x 1 km grid cell having a single gauge was estimated as 1.35, with reductions in this value as more gauges were included, and increases when extended to larger grid sizes. This error was found to be considerably larger than the measurement error discussed before (Ciach, 2003). While there is no clear way of addressing this error, its variation over time can be factored in the specification of any model that is developed using this as inputs (Chowdhury and Sharma, 2007).

- *Effect of untagged multi-day accumulations in daily rainfall data* – As nearly a third of the long-term daily rainfall records are recorded at Post Offices and other public buildings, the occurrence of multi-day readings (representing Saturday to Monday) recorded on the first working day after the weekend is frequent. An example of one such station is illustrated in Figure 2.7.3. Viney and Bates (2004) outline a hypothesis test for identifying the periods in a rainfall record that reflect significant multi-day accumulations. While there is no simple corrective procedure that can be employed, common-sense alternatives such as comparing with data at nearby locations (after ascertaining that they do not suffer from the same problem), and using the persistence structure of the non-accumulated data to disaggregate the accumulated values, should be adopted. It should be noted that while such accumulations may not affect calculations in yield or water balance studies, their implications in flood estimation studies can be significant.

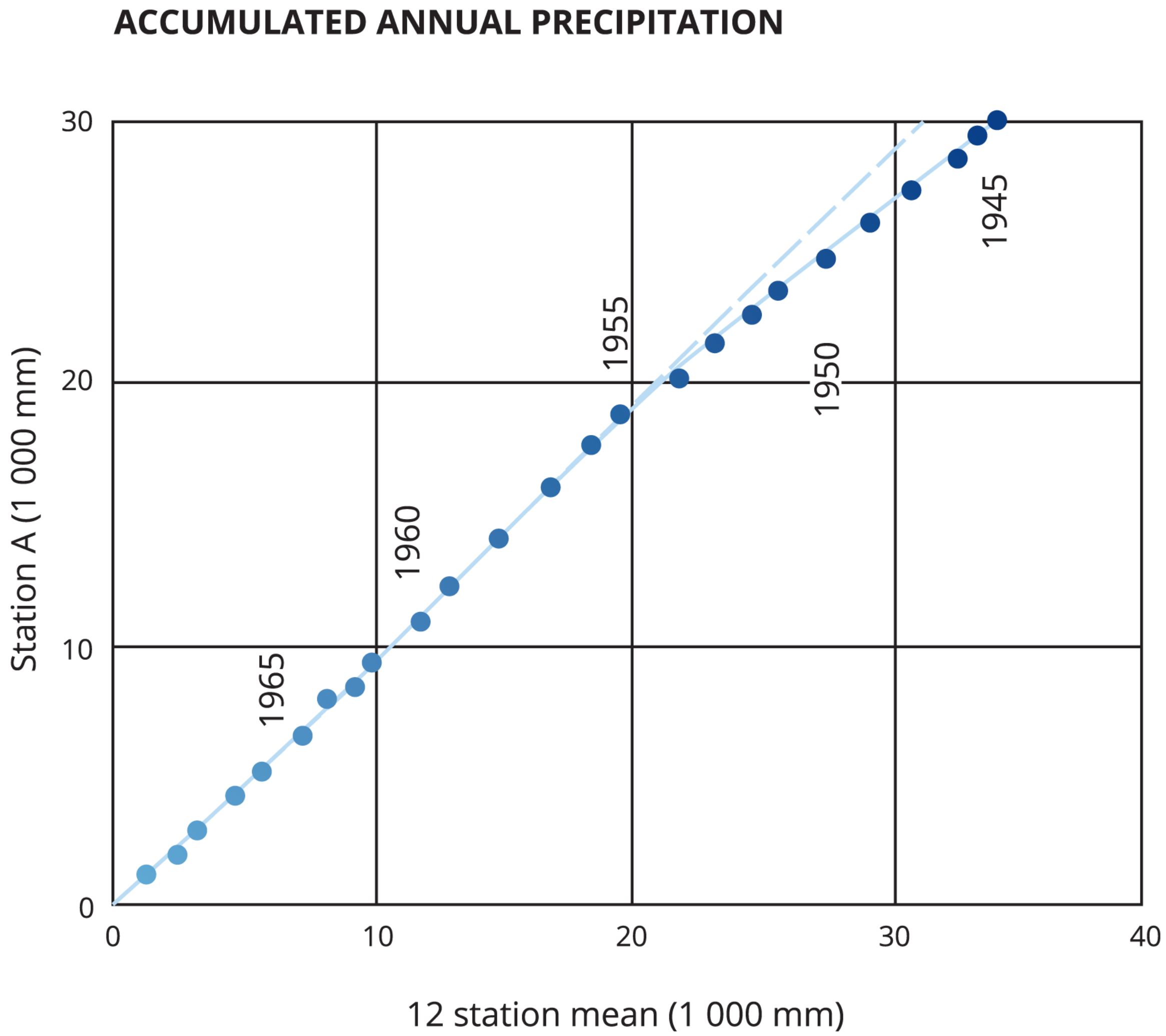


Figure 2.7.2. Double Mass Curve Analysis for Rainfall at Station A (from World Meteorological Organisation (1994))

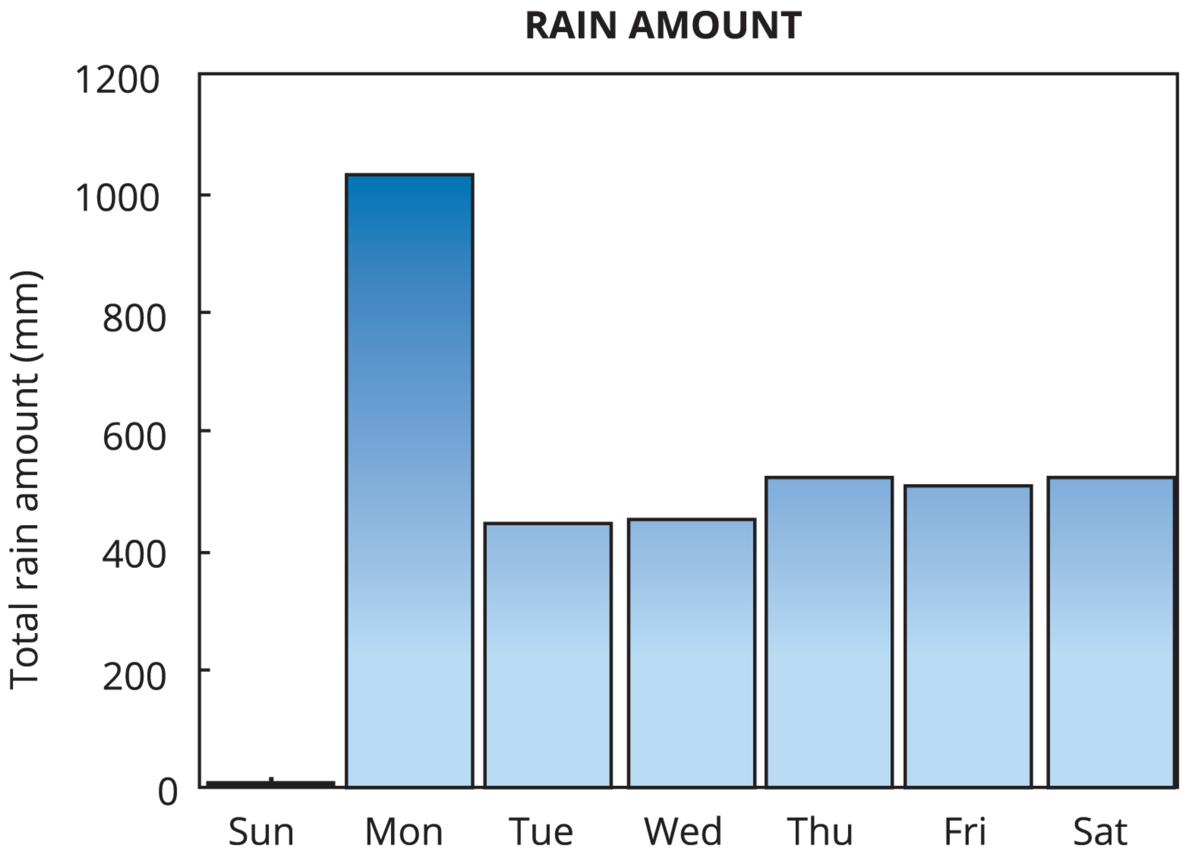


Figure 2.7.3. Total Rainfall Amounts for Rainfall Station 009557 over the Period 1956-1962 (from Viney and Bates (2004))

7.2.2. Options for Catchments with no Rainfall Records

One of the advantages of event-based approaches for design flood estimation, is the availability of design rainfall data in different parts of Australia. These data are derived through spatial interpolation of Intensity Frequency Duration (IFD) parameters, with assumptions on the changes one would expect from gauged to ungauged locations. In contrast, continuous simulation either requires observed rainfall time series at each location of interest, or a procedure to generate such series based on data from nearby locations. For situations where observed rainfall data are not available, the following alternatives can be considered:

- *Use of gridded rainfall products* -Given the need to use catchment averages of rainfall and potential evapotranspiration in a range of hydrologic studies, datasets of gridded rainfall and temperature have been produced for Australia and elsewhere. Two gridded datasets used routinely in Australia are the SILO and the Australian Water Availability Project (AWAP) daily rainfall 5 km x 5 km gridded datasets. The SILO project (Jeffrey et al., 2001) by the Queensland Centre for Climate Applications, Department of Natural Resources, aimed to develop a comprehensive archive of key meteorological variables (Maximum and Minimum Temperature, Rainfall, Class-A pan Evaporation, Solar Radiation and Vapour Pressure) through interpolation on a 0.05° grid extending from latitude 10°S to 44°S and longitude 112°E to 154°E. The project has also resulted in a patched daily rainfall series at 4600 locations extending back to 1890. In addition, the AWAP dataset was produced by the CSIRO and the Bureau of Meteorology (Jones et al., 2009) at the same resolution using a different averaging procedure. These datasets have been compared (Beesley et al., 2009) and found to be similar in many respects, while still resulting in a dampening of high extremes due to averaging, as well as a over-simulation of the number of wet days in a year. Similar biases occur in the representation of persistence attributes, possibly distorting the specification of antecedent conditions prior to large rainfall events. Care must be taken when using such datasets, especially if the intention is to simulate flow extremes for the catchment.

- *Use of radar or satellite derived rainfall measurements* - While the above mentioned gridded data are based on spatial interpolation of gauged rainfall alone, another option that has been pursued with success is to combine gauge and remotely sensed rainfall, which is known to improve accuracy especially in remote locations with limited gauge coverage. Examples of approaches that have produced and assessed such combined datasets include (Chappell et al., 2013). While they suffer from the same problems as other gridded datasets, the advantages they offer in remote locations should be taken into consideration.

- *Use of statistical interpolation techniques based on nearby daily and sub-daily gauge records* - Refer to the alternatives for continuous simulation at ungauged locations presented later in the chapter. These alternatives use separate approaches for daily and sub-daily continuous generation at ungauged locations. The daily alternative amounts to identifying nearby gauges that "mirror" key characteristics that would be expected of daily rainfall at the location of interest. These nearby gauge records are then transformed to the current location by adjusting for any difference in their annual mean. Each nearby gauge is assigned a probability depending on how "similar" it may be to the location of interest, which allows characterisation of the uncertainty associated with this procedure. In the sub-daily case, a second step is adopted. Once the daily record has been generated, it is disaggregated using data on sub-daily fragments based on a different set of characteristics that define the sub-daily climate of the location. More details on these procedures are presented later.

7.2.3. Missing Rainfall Observations

Rainfall records often contain missing observations that need to be filled using appropriate techniques. This problem is often compounded when records from multiple sites are to be used for analysis. The World Meteorological Organisation (World Meteorological Organisation, 1994) expresses caution against filling more than 10% of the rainfall records as the aggregate rainfall information may be influenced by interpretation. Some of the methods recommended for filling short gaps in the rainfall record are as follows:

- *Normal Ratio Method* – This method estimates the missing rainfall $\widehat{P_g}$ at gauge g as a weighted average of the measured rainfall at nearby rainfall gauges:

$$\widehat{P_g} = \frac{\sum_{i=1}^{G} \frac{\overline{P_g}}{\overline{P_i}} P_i}{G} \tag{2.7.1}$$

where *G* represent the total number of rainfall gauges, $\overline{P_g}$ and $\overline{P_i}$ the average annual rainfall at gauges *g* and *i* respectively, and P_i the rainfall at gauge *i* for the time period being filled. Care must be taken to ensure that the "host" rainfall gauges have similar climatic conditions as the gauge where the missing observations are being infilled.

- *Quadrant Method* – This method is related to the Normal Ratio method, but aims to account for the proximity of the rainfall gauges to the target location. The missing rainfall

$\widehat{P_g}$ at gauge *g* is estimated as a weighted average of observations at four rainfall gauges, one in each quadrant using north-south and east-west lines that intersect the location of gauge *g*. The rainfall is estimated as:

$$\widehat{P_g} = \frac{\sum_{i=1}^{4} \frac{1}{d_i^2} P_i}{\sum_{i=1}^{4} \frac{1}{d_i^2}} \quad (2.7.2)$$

where d_i is the Euclidean distance between gauges *i* and *g*.

- *Isohyetal Method* – This method involves drawing isohyets (lines of equal rainfall) for the storm duration over the network of rainfall gauges available, and inferring the rainfall at the missing rainfall gauge by interpolation. The accuracy of the Isohyetal method depends significantly on the number of rainfall gauges used and the interpolation algorithm being used to construct the isohyets.

- *Copula based interpolation* – Bárdossy and Pegram (2014) presented an alternative for interpolating existing data to infill missing values at a station of interest. They used a copula-based specification of the conditional probability distribution of the missing rainfall based on values at nearby gauges. They compared their approach with both regression and other spatial interpolation based alternatives and found it to perform better using daily rainfall data from South Africa. Another advantage of their approach is that it can include conditioning on exogenous variables which could include atmospheric fields that are common to all stations in the area of interest, thereby allowing additional information on the nature of precipitation.

The above methods are fairly intuitive and modifications of the basic logic outlined are common. For instance, in situations where data from nearby rainfall gauges are hard to find, the interpolation is often from previous years of record at the same rainfall gauge, the period being chosen to represent the same season and similar antecedent rainfall conditions.

The methods suggested above should be used with care, with consideration for the distributional changes that occur as a result of the interpolation. For instance, if the stations used for spatial averaging are at significant distances to the station where the interpolation is required, then the interpolated rainfall is likely to be ‘smoother’ than the rainfall that would have occurred at that location, potentially leading to an overestimation of wet days and an underestimation of peak rainfall. Similarly, if the interpolation is performed at each time step independently, the dependence of rainfall from one time to the next may not be accurately represented. These considerations attain importance particularly when short time steps (daily and sub-daily) are considered, and when the missing periods are a significant portion of the overall record.

Missing data within historical rainfall records can be a serious problem, the amount of which can affect the type of model structure considered. Few researchers explain adequately how this is dealt with. Cowpertwait (1991) described a replacement strategy to handle missing data but it is not apparent that this approach will be adequate with significant missing or rejected data. Katz and Parlange (1995) and Gyasi-Agyei (1999) ignore and discard months with any missing data. As a result, valuable information could be lost, particularly if there is limited data in the first place. For some months of the year Gyasi-Agyei (1999) discarded up to half of the available data. With an event-based approach, discarding storm events or inter-event times containing missing intervals should introduce no significant bias into the calibration, provided the occurrence of this corrupted data is random. Therefore, if part of a month of data is missing it does not invalidate the remaining good quality data in that month.

7.3. Stochastic Rainfall Generation Philosophy

Stochastic generation of daily or sub-daily rainfall sequences requires the specification of a probabilistic model of rainfall over time. Such a probability model should account for the following features of daily or sub-daily rainfall:

- The significant probability mass for zero values (no rain);
- The seasonality of a range of rainfall statistics, including wet/dry days, averages and extremes;
- The low-frequency variability, which causes below- or above-average rainfall to persist for multiple consecutive years;
- The short-range (day-to-day and within-day) persistence of wet and dry periods; and
- The highly skewed distribution of rainfall, with the rainfall features often of most interest in a design flood estimation context being located at the tail of the distribution.

Simulation of these aspects of rainfall requires careful formulation of the rainfall generation model, often by using conditional variables that enforce this variability at multiple timescales.

Finally, although the current chapter does not discuss the case of stochastic generation at multiple locations, this added consideration would require the specification of multivariate conditional probability distributions characterising both the temporal evolution of the process, as well its links in space.

In general, single site rainfall generation approaches fall into the following categories:

1. Daily rainfall generation;
2. Sub-daily rainfall generation; and
3. Sub-daily rainfall generation through disaggregation of daily rainfall.

Many alternative models exist for each of these categories, as do their extensions to ungauged or partly gauged locations. Readers are referred to Sharma and Mehrotra (2010) for a review on these alternatives. A subset of these alternatives is discussed in Book 2, Chapter 7, Section 4. It should be noted that some of the sub-daily models simulate daily rainfall very well when aggregated to daily (refer to Frost et al. (2004)).

7.4. Rainfall Generation Models

7.4.1. Daily Rainfall Generation

7.4.1.1. Overview of Daily Rainfall Generation Techniques

Generation of daily rainfall sequences requires the formulation of procedures for generating rainfall occurrences (wet or dry) and amounts (for the wet days). As rainfall occurs in bursts, it is important to represent the day-to-day persistence in the rainfall. This can be accomplished by assuming rainfall is a Markovian process, with the nature of persistence defined by the order of Markovian dependence. A first-order Markovian process assumes rainfall depends only on the rainfall (amount or occurrence) on the previous day.

Assuming first or low-order dependence can result in the number of wet days in a year being similar from one year to the next. This is contrary to the nature of rainfall in Australia and elsewhere, with considerable variations from one year to the next often modulated by low-frequency climatic anomalies such as the El Nino Southern Oscillation (ENSO) phenomenon. This inability of rainfall generation models to simulate observed variability at aggregated (annual or longer) scales is referred to as “over-dispersion” (Katz and Parlange, 1998).

In addition to the representation of persistence, a rainfall generation model needs to also allow for seasonal variation. This is often accomplished by allowing model parameters to be estimated on a seasonal or monthly basis. While these distributions are characterised by sample parameter estimates at locations with sufficient observational records, these can also be regionalised for use in ungauged locations.

Table 2.7.1 (adapted from Sharma and Mehrotra (2010)) summarises the approaches used for generation of daily rainfall. The higher-order Markov approaches listed are especially relevant for Australia, given the significant low-frequency variability that characterises Australian rainfall. Misrepresentation of this variability can have serious implications in the representation of pre-burst antecedent conditions, as well as the relationship between the rainfall extremes and the longer-range antecedent rainfall, given both are known to be modulated by climatic anomalies responsible for such variability in rainfall time series.

Table 2.7.1. Alternative Methods for Stochastic Generation of Daily Rainfall

Model	Description/Advantages/ Drawbacks	References
Daily Rainfall Occurrence Generation		
Low-order Markov Chain Models	Based on wet day probabilities. For some regions generates rainfall series with too few long dry spells.	(Buishand, 1977; Caskey, 1963; Feyerherm and Bark, 1965; Feyerherm and Bark, 1967; Gabriel and Neumann, 1962; Hopkins and Robillard, 1964; Racsko et al., 1991; Selvalingam and Miura, 1978; Stern and Coe, 1984; Wilks, 1998; Chapman, 1997)
Higher-order Markov Chain Models	Based on wet day probabilities of few consecutive days. The approach increases the length of the Markov model’s ‘memory’ of antecedent wet and dry days. The number of parameters (i.e., transition probabilities) required increases exponentially as the order increases, being 2k for a k^{th}-order chain. These	(Coe and Stern, 1982; Dennett et al, 1983; Gates and Tong, 1976; Jones and Thornton, 1997; Mehrotra and Sharma, 2007a; Mehrotra and Sharma, 2007b; Mehrotra et al., 2012; Pegram, 1980; Singh and Kripalani, 1986)

Model	Description/Advantages/ Drawbacks	References
	models improves the representation of observed inter- annual variance in the simulations but still fell short of observed climatic variability on average.	
'Hybrid-order' Markov Models	The Markov 'memory' extends further back in time for the dry spells only.	(Stern and Coe, 1984; Wilks, 1999a)
Alternating Renewal Process Based Models	These spell-length models operate by fitting probability distributions to observed relative frequencies of wet and dry spell lengths. The approach may not be suited in arid regions or in cases with less than 25 years of observations.	(Buishand, 1977; Racsko et al., 1991; Roldan and Woolhiser, 1982; Woolhiser, 1992; Wilks, 1999a)
	Non-parametric wet-dry spell length models.	(Lall et al., 1996; Sharma and Lall, 1999)
Daily Rainfall Generation including Amount		
Parametric Precipitation Amounts Models	Based on some distribution like a two parameter gamma distribution, exponential and mixed exponential distribution. These models assume that precipitation amounts on wet days are independent, and follow the same distribution.	(Wilks, 1999b; Coe and Stern, 1982; Richardson, 1981; Woolhiser and Pegram, 1979; Woolhiser and Roldán, 1982; Woolhiser and Roldán, 1986)
Wet Spell Based Precipitation Amount Models	These models allow different probability distributions for precipitation amounts depending on that day's position in a wet spell (separate models for start, mid and end of a wet spell).	(Chapman, 1997; Buishand, 1977; Chin and Miller, 1980; Cole and Sheriff, 1972; Wilks, 1999b)
Non-parametric Precipitation Amount Models	A non-parametric kernel density estimation based procedure is used to simulate the rainfall conditional on previous time step value of rainfall and/or other variables.	(Harrold et al., 2003a; Mehrotra and Sharma, 2006; Oriani et al., 2014)
Multi-state Markov Models	These Markov models simulate both precipitation occurrence and amounts, by	(Boughton, 1999; Gregory et al., 1993; McMahon and Srikanthan, 1983; Srikanthan

Model	Description/Advantages/ Drawbacks	References
	defining different ranges of precipitation amounts as constituting distinct states. The outcome of this approach depends on the choice of the number of states, their ranges and on the distributions used for wet day amounts in any given state. These models involve comparatively large numbers of parameters, and thus require quite long data records in order to be estimated well.	and McMahon, 1985; Haan et al., 1976)
Cluster Based Point Processes Models	Rainfall process is described using cluster of rectangular pulses. In the approach, storms arrive according to a Poisson process and are represented by clusters of rainfall cells temporally displaced from the storm centre.	(Evin and Favre, 2012; Kavvas and Delleur, 1981; Kim et al., 2014; Leblois and Creutin, 2013; Leonard et al., 2008; Onof et al., 2000; Ramirez and Bras, 1985; Rodriguez-Iturbe et al., 1984; Rodriguez-Iturbe et al., 1987; Rodriguez-Iturbe et al., 1988; Waymire and Gupta, 1981a; Waymire and Gupta, 1981b; Wheater et al., 2000)
Copula Theory Based Models	Multi-variate copulas are used to describe the spatial structure of rainfall amounts and occurrences.	(Bárdossy and Pegram, 2009; Serinaldi, 2009)
Multi-fractal Simulation Techniques	These models characterise rainfall by scale invariant (scaling) and fractal properties.	(Seed et al., 1999; Menabde et al, 1997; Jha et al., 2015)
Conditioning on Co-variates	Monthly statistics of rainfall, long-range forecasts of the monthly statistics, random numbers or a 'hidden' mixture approach to capture some inter-annual variability.	(Jones and Thornton, 1997; Katz et al., 2003; Wilks, 1999a)
Conditioning on Previous Time History of Simulated Rainfall	Rainfall occurrences and amounts are simulated conditional on the recent past rainfall behaviour.	(Harrold et al., 2003b; Harrold et al., 2003a; Mehrotra and Sharma, 2007a; Mehrotra and Sharma, 2007b; Sharma and O'Neill , 2002)

Model	Description/Advantages/ Drawbacks	References
Conditioning on Some Aspect of Large-scale Atmospheric Circulation/ Weather Patterns	Using the Lamb Weather Type weather classification, monthly Southern Oscillation Index (SOI), North Atlantic Oscillation Index (NAOI), North Atlantic sea surface temperature (SST) anomalies and other atmospheric predictors.	(Katz and Parlange, 1998; Hay et al., 1991; Charles et al., 1999; Hughes and Guttorp, 1994; Woolhiser, 1992; Wilby et al., 1998; Kim et al., 2014; Kim et al., 2012; Wallis and Griffiths, 1997; Bárdossy and Plate, 1992; Serinaldi and Kilsby, 2014; Kleiber et al., 2012; Carey-Smith et al., 2014; Heaps et al., 2015)
Model Nesting at Multiple Time Scales	Rainfall amounts are adjusted at monthly/seasonal and annual time scales to maintain the desired variability at higher time scales.	(Boughton, 1999; Srikanthan and Pegram, 2009; Wang and Nathan, 2007; Lambert et al., 2003; Thyer and Kuczera, 2003a; Thyer and Kuczera, 2003b)

The daily generation models in Table 2.7.1 are often formulated using high quality observed rainfall records, and then regionalised for use anywhere. Regionalisation of a rainfall generation model is accomplished either by interpolating model parameters for use at ungauged locations, or by sampling data from other locations as representative for the location of interest. In the discussion that follows, two methods - the regionalised Nested Transition Probability Model (N-TPM) and the Regionalised Modified Markov Model (RMMM), are summarised due to their widespread use in Australia and the availability of software to facilitate implementation within the country.

7.4.1.2. Nested Transition Probability Matrix Approach

The Transition Probability Model (TPM) offers a simple and effective characterisation of Markov order-one persistence in the daily rainfall generation process (Srikanthan et al., 2003). In the TPM, the daily rainfalls are divided into a maximum of seven states. State 1 is dry (no rainfall) and the other states are wet. The rainfall amounts in the largest state are generated using a Gamma distribution. The model operates by estimating the transition probability of sampling a state given the state of the preceding time step. Hence, if seven states are used, a 7 x 7 transition probability matrix needs to be estimated from the data. As only Markov order-one dependence is assumed, a correction is needed to ensure that simulated rainfall exhibits sufficient variability at an annual time scale. This correction occurs by rescaling of the daily rainfall amounts, thereby inflating the variability of rain on each day, while keeping the fraction of wet days in a year constant.

The TPM has been applied in a number of studies, and exists in a regionalised form for use anywhere in Australia. The computer program for the TPM can be obtained from the Stochastic Climate Library as part of the e-Water Toolkit (http://toolkit.net.au/Tools/SCL). Parameters for major city centres and recommendations for ungauged locations are provided within the software. Table 2.7.2 and Table 2.7.3 present the number of states and the rainfall amount associated with highest state used for major city centres in Australia. If the number of states is less than seven the upper limit of the last state is infinite. Figure 2.7.4

provides regional extents that are used in applying the method to other locations not included in the tables.

Table 2.7.2. Number of States used for Different Rainfall Stations in the Transition Probability Model (Srikanthan et al., 2003)

Station	Latitude °S	Longitude °E	J	F	M	A	M	J	J	A	S	O	N	D
Melbourne	37 49	144 58	6	6	6	6	6	6	6	6	6	6	6	6
Lerderderg	37 30	144 22	6	6	6	6	6	6	6	6	6	6	6	6
Monto	24 51	151 01	6	6	6	6	6	6	6	6	6	6	6	6
Cowra	33 49	148 42	6	6	6	6	6	6	6	6	6	6	6	6
Adelaide	34 56	138 35	6	6	6	6	6	6	6	6	6	6	6	6
Perth	31 57	115 51	6	6	6	6	6	6	6	6	6	6	6	6
Sydney	33 52	151 12	7	7	7	7	7	7	7	7	7	7	7	7
Brisbane	27 28	121 06	7	7	7	7	7	7	7	7	7	7	7	7
Mackay	21 06	149 06	7	7	7	7	7	7	7	7	7	7	7	7
Kalgoorlie	30 47	21 27	5	5	5	5	5	5	5	5	5	5	5	5
Alice Springs	23 49	133 53	4	4	4	4	4	4	4	4	4	4	4	4
Onslow	21 40	115 07	4	4	4	3	4	3	4	3	3	3	3	3
Bamboo Springs	22 03	119 38	6	6	6	5	5	5	2	2	2	2	2	2
Broome	17 57	122 15	7	7	7	3	3	3	3	3	3	3	3	4
Darwin	12 27	130 48	7	7	7	7	3	2	2	2	3	7	7	7

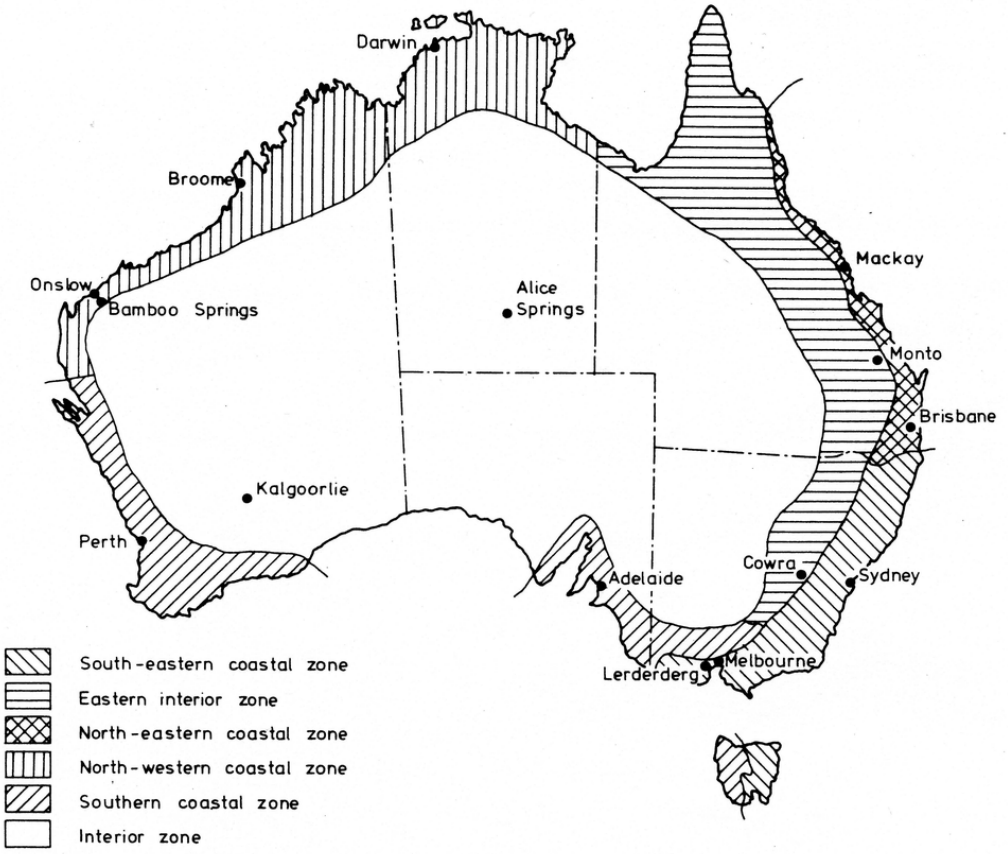


Figure 2.7.4. Rainfall Stations used in Table 2.7.1 for the Transition Probability Model (Srikanthan et al., 2003)

Table 2.7.3. State Boundaries for Rainfall Amounts in the Transition Probability Model

State Number	Upper State Boundary Limit (mm)
1	0.0
2	0.9
3	2.9
4	6.9
5	14.9
6	30.9
7	∝

As the Transition Proability Method requires a correction for the misrepresentation of low-frequency variability, several alternatives have been developed to address this limitation. The Nested Transition Probability Method (Srikanthan and Pegram, 2009) operates by aggregating the sequences of rainfall from the TPM to first a monthly and then to an annual time scale. Once aggregated, rainfall is modelled as a Markov order-one process at the aggregated time scale, accounting for the lag-one auto-correlation and variability that is manifested in the aggregated process. This offers an effective means of correcting variability in rainfall across a range of time scales, making the generated series more useable for hydrological applications. As with the TPM, the computer program for the Nested TPM can be obtained from the Stochastic Climate Library as part of the e-Water Toolkit (http://toolkit.net.au/Tools/SCL).

7.4.1.3. Regionalised Modified Markov Model

The Regionalised Modified Markov Model (RMMM) offers a non-parametric basis for daily rainfall generation at any location in Australia in a manner that ensures generated sequences mimic observed rainfall in its representation of distributional features as well as low-frequency variability. The RMMM is a regionalised version of the Modified Markov Model (MMM) (Mehrotra and Sharma, 2007a) which simulates rainfall by characterising the rainfall occurrence by a variable-order Markovian process that is designed to simulate low-frequency variability. This variable-order Markov process is defined by assuming that daily rainfall occurrence depends on the rainfall state on the previous day as well as the aggregated rainfall for the past 30 and 365 days. The use of the aggregated rainfall conditioning variables allows the generated sequences to reflect the dependence there exists in observed rainfall across different temporal scales. Furthermore, use of aggregated variables allows invoking of the Central Limit Theorem and approximating their probability distribution as a Gaussian distribution, thereby simplifying parameter estimation and implementation. As a result of using the aggregated variables, the number of wet days in a year exhibit variability that is consistent with the observed record, in contrast to the Nested TPM approach that offers similar variability with rainfall amounts alone. Once the rainfall occurrences have been generated, rainfall amounts are generated using a non-parametric kernel density estimation approach.

The algorithm for generating daily rainfall using the Modified Markov Model is presented in Algorithm for step-wise daily rainfall generation using Modified Markov Model (Mehrotra and Sharma, 2007a) .

Algorithm for step-wise daily rainfall generation using Modified Markov Model (Mehrotra and Sharma, 2007a)

1. For all calendar days of the year calculate the transition probabilities of the standard first-order Markov model using the observations falling within the moving window of 31 days centered on each day. Denote these transition probabilities as p11 for previous day being wet and p01 for previous day being dry.

2. Also estimate the means, variances and co-variances of the higher time scale predictor variables separately for occasions when current day is wet/day and previous day is wet/dry. Mehrotra and Sharma (2007a), identified 2 variables namely, previous 30 and 365 days wetness state)

3. Consider a day. Ascertain appropriate critical transition probability to the day t based on previous day's rainfall state of the generated series. If previous day is wet, assign critical probability p as p11 otherwise assign p01.

4. Calculate the values of the 30 and 365 days wetness state for the day t and the available generated sequence (Jo). To have values of wetness state in the beginning of the simulation randomly pickup a year from the historical record and calculate values of 30 and 365 days wetness states.

5. Modify the critical transition probability p of step 3 using the following equation and, conditional means, variances, co-variances and tth day value of higher time scale predictors for the generated day t. Denote the modified transition probability as $\hat{p}$.

$$\hat{p} = p_{1\mathrm{i}} \frac{\frac{e^{\left\{-\frac{1}{2}\left(X_t - \mu_{1,i}\right)V_{1,i}^{-1}\left(X_t - \mu_{1,i}\right)'\right\}}}{\sqrt{\det\left(V_{1,i}\right)}}}{\left[\frac{e^{\left\{-\frac{1}{2}\left(X_t - \mu_{1,i}\right)V_{1,i}^{-1}\left(X_t - \mu_{1,i}\right)'\right\}}}{\sqrt{\det\left(V_{1,i}\right)}} p_{1\mathrm{i}}\right] + \left[\frac{e^{\left\{-\frac{1}{2}\left(X_t - \mu_{1,i}\right)V_{1,i}^{-1}\left(X_t - \mu_{1,i}\right)'\right\}}}{\sqrt{\det\left(V_{1,i}\right)}} \left(1 - p_{1\mathrm{i}}\right)\right]}$$

   where X_t is the predictor set at time t, the $\mu_{1,i}$ parameters represent the mean $E\left(X_t \middle| J_t = 1, J_{t-1} = i\right)$ and $V_{1,i}$ is the corresponding variance-co-variance matrix. Similarly, $\mu_{0,i}$ and $V_{0,i}$ represent, respectively, the mean vector and the variance-co-variance matrix of X when $\left(J_{t-1} = i\right)$ and $\left(J_t = 0\right)$. The $p_{1\mathrm{i}}$ parameters represent the baseline transition probabilities of the first order Markov model defined by $P\left(J_t = 1 \middle| J_{t-1} = i\right)$ and $\det()$ represents the determinant operation.

6. Compare $\hat{p}$ with the uniform random variate $u_t(k)$ for station k. If $u_t(k) \leq \hat{p}$, assign rainfall occurrence, Jo_t for the day t as 1 otherwise zero.

7. Move to the next date in the generated sequence and repeat steps 2-5 until the desired length of generated sequence is obtained.

Readers are referred to Mehrotra and Sharma (2007a) for details of the Modified Markov Model rainfall generation algorithm. A R-package to generate daily rainfall at multiple locations given observed rainfall time series has been developed Mehrotra et al. (2015) and is available for download from Hydrology@UNSW Software website (http://

www.hydrology.unsw.edu.au/download/software/multisite-rainfall-simulator). The package exists as a Multi-site Rainfall Simulator (abbreviated MRS), offering the capability to generate rainfall at multiple locations of interest while maintaining their spatial dependence attributes in sequences, but simplifies to the Modified Markov Model when used to generate rainfall for a single location.

7.4.1.3.1. Regionalisation

The Modified Markov Modelrequires a representative sample of daily rainfall for generation to proceed. This restricts its application only to locations having long-length observed records. An attempt to regionalise the Modified Markov Model was presented in Mehrotra et al. (2012), using a similar approach to the regionalised sub-daily generation model Westra et al. (2012) in Book 2, Chapter 7, Section 4. Unlike the regionalised version of the Nested TPM (Book 2, Chapter 7, Section 4), here the regionalisation involved identifying rainfall records for locations deemed ‘similar’ to the target, followed by rescaling to adjust for changed climatology, and then pooling to take account of relative similarities each nearby location bears to the target location. This pooled record was then used as the basis for generating the daily rainfall sequences.

As the regionalised approach relies on using data from nearby rainfall stations, it is necessary to:

1. identify metrics to determine whether two stations are ‘similar’; and

2. predict the probability that stations within a ‘neighbourhood’ of the target site are similar by regressing against physiographic indicators such as the difference in latitude, longitude, elevation and relative distance to coast between station pairs.

The relative distance to coast is obtained by dividing the difference in distance to coast between two stations by the distance to coast of the target site. This is done to account for the fact that the relative influence of distance to coast is likely to be greater for two stations having greater proximity to the coastline.

‘Similarity’ between any two sites was assessed based on the similarity in the bivariate probability distributions of a daily-scale attribute of interest, and the annual rainfall total. Table 2.7.3 outlines the attributes used in formulation of the RMMM. Each of the attributes listed were used to define similarity between stations based on a two sample, 2 dimensional Kolgomorov-Smirnov test (Fasano and Fanceschini, 1987). The resulting classification of similarity (‘1’ for similar and ‘0’ for dissimilar) for each attribute was pooled in a logistic regression framework, using the difference in latitude for the two stations, difference in longitude, and difference in the relative distance to coast as covariates.

Table x presents Daily scale attributes used to define similarity between locations. Each of these variables were estimated for each location and each year of record, and then paired to assess the best basis for defining ‘similarity’ between stations. Using 2708 separate rain gauge stations with at least 25 years of data, this resulted in a total of 3,665,278 station pairs.

Table 2.7.4. Daily Scale Attributes used to Define Similarity between Locations

Daily Maxima	Daily Maximum rainfall for DJF, MAM, JJA, SON
7 Day Maxima	Maximum 7-day total rainfall for each season

Wet/Dry Spell Lengths	Maximum Wet/Dry spell length for each season
Rainfall Amount per Spell	Total Rainfall in maximum wet spell for each season
Daily Averages	• Average rainfall amount per wet day for year • Average rainfall amount per day (wet or dry) for each season
Number of Wet Days	• Total number of wet days each year • Total number of wet days each season

This logistic regression framework can then be used to determine the similarity between any two stations for the attribute of interest. Therefore, for a given target location where rainfall is to be generated, one can now rank stations with data from most similar to least similar for each attribute. The approach adopted in RMMM is to form an average rank using all attributes for all nearby stations, and use the lowest *S* ranks to identify the stations to use as the basis of rainfall generation. To account for the relative similarity across these *S* stations, each station is selected with a probability equal to:

$$w_i = \frac{1/r_i}{\sum_{k=1}^{S} 1/r_k} \tag{2.7.3}$$

where w_i represents the weight associated with the i^{th} station and r_i the rank associated with that station, used as the basis for probabilistically selecting nearby stations in the Modified Markov Model. Lower ranked stations, which, by definition have rainfall attributes which are most statistically similar to the target site, attain higher weight and therefore a higher probability of being used in MMM. This rationale is summarised in Figure 2.7.5.

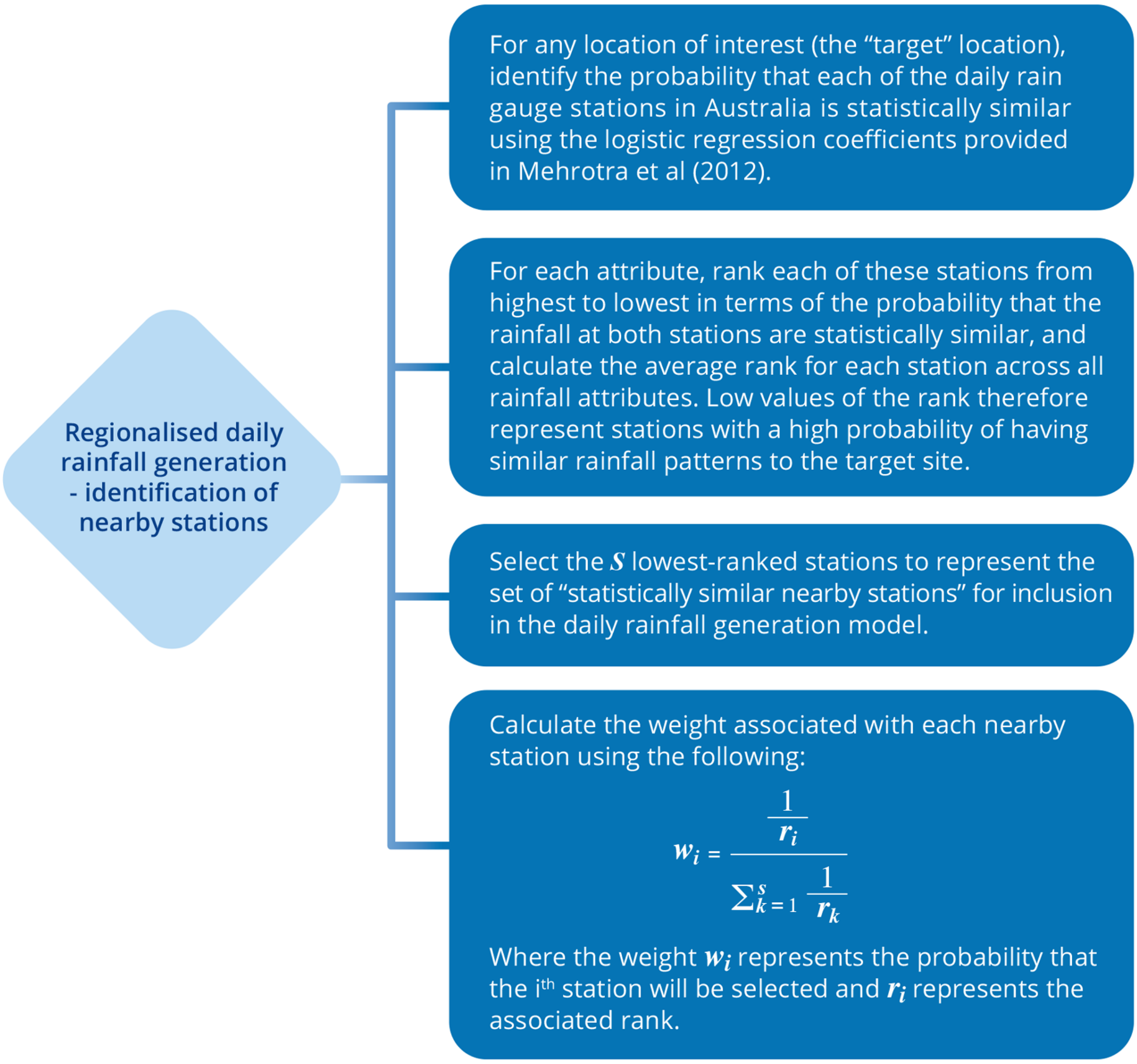


Figure 2.7.5. Identification of "Similar" Locations for Daily Rainfall Generation using RMMM

Once the 'similar' *S* stations have been identified, the generation of rainfall sequences at the target location proceeds as per the generation algorithm for MMM in Algorithm for step-wise daily rainfall generation using Modified Markov Model (Mehrotra and Sharma, 2007a), with the inclusion of two additional steps. The first of these steps involves a rescaling of the "similar" locations identified as described in Figure 2.7.5. The second of these steps is a probabilistic selection of the "similar" locations, based on the weights associated with each location. These steps are summarised in Figure 2.7.6.

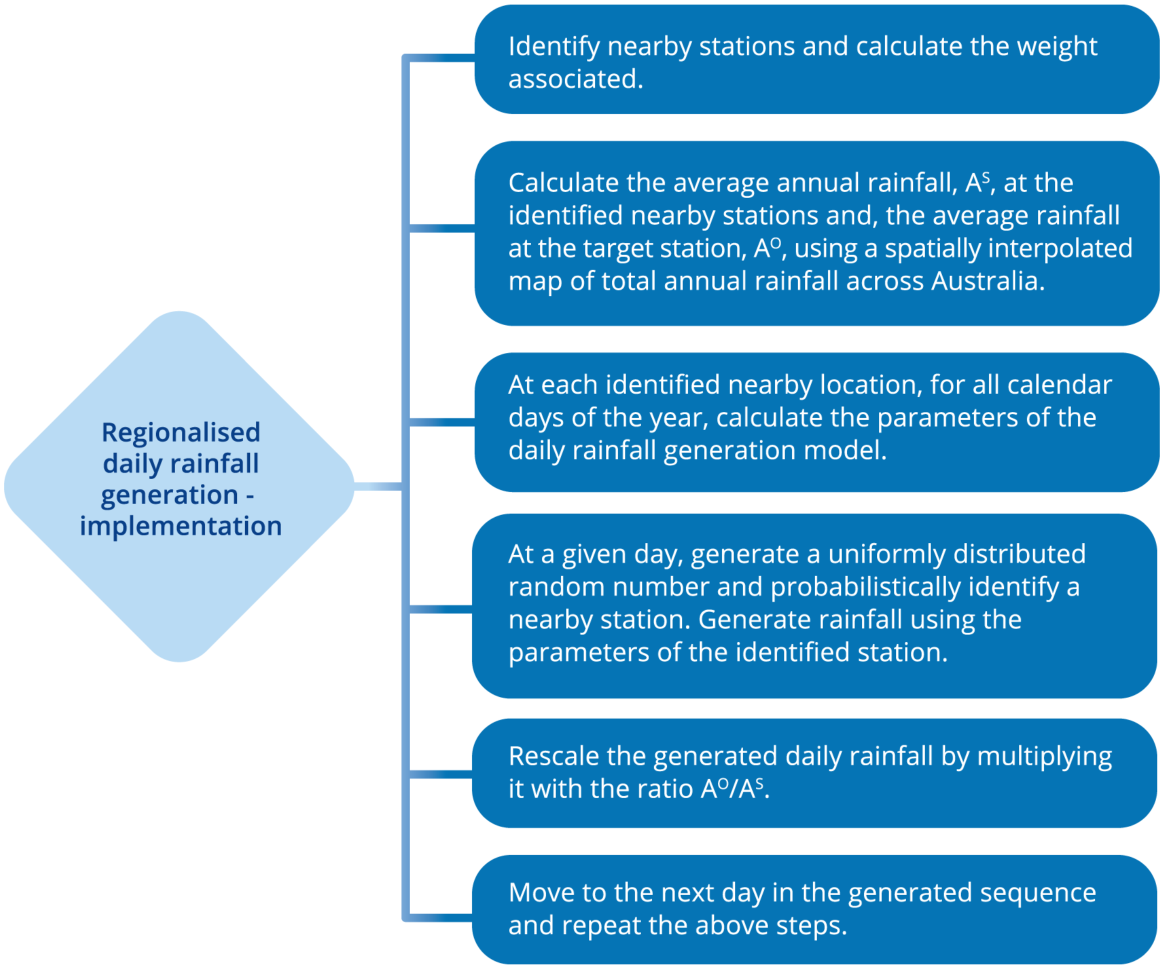


Figure 2.7.6. Generation of Daily Rainfall Sequences using the Regionalised Modified Markov Model Approach

It should be noted that the low frequency variable states (30 and 365 day wetness states in MMM) are ascertained based on the generated sequence, and hence represent the probabilistic average from the collation of the locations that have been selected as “similar” for the generation procedure. The software first identified “similar" locations to the target location of interest, and then estimates the parameters of the MMM for these locations. As the criterion for selecting “similar” locations is defined as a function of differences in latitude, longitude, elevation and rescaled distances from the coast, a new location with daily observations can be included for the procedure to work. The parameters of the logistic regression model have been ascertained using high quality daily rainfall observations, and will be updated with significant updates in the daily rainfall datasets available in Australia.

It should also be noted that use of actual rainfall data from similar locations is followed by a rescaling approach to account for changed climatology results in maximal use of observed rainfall. The use of MMM has been shown to produce generated rainfall with low frequency variability and extremes that are consistent with observations. Given not one but multiple similar locations are used, the likelihood of over sampling rainfall attributes from a misclassified similar location is reduced. An assessment by Mehrotra et al. (2012) indicates that the method is able to capture the important attributes that define daily rainfall in both gauged and ungauged locations in Australia.

7.4.2. Sub-daily Rainfall Generation

While considerable research has been done on the generation of daily rainfall sequences for design flood estimation, information is often required at a sub-daily time scale. Sub-daily rainfall sequences are generated using two approaches. In the first case, rainfall is generated assuming a model formulated based on sub-daily rainfall observations. In the second case, sub-daily rainfall is generated conditional to daily rainfall through a disaggregation algorithm, the aim being to utilise the value of the much longer daily rainfall data and adopt a sensible approach to convert it to finer time steps. Table 2.7.5 summarises many of the sub-daily rainfall generation approaches available in the literature.

Table 2.7.5. Commonly used Sub-daily Rainfall Generation Models

Model	Description/Advantages/ Drawbacks	References
Poisson Cluster Process Based Models	Represents rainfall events as clusters of rain cells where each cell is considered a pulse with a random duration and random intensity. A rainfall generation model, however, can also be used for rainfall disaggregation.	(Cowpertwait, 2010; Cowpertwait et al., 2002; Leonard et al., 2008; Koutsoyiannis et al., 2003; Onof and Townend, 2004; Wheater et al., 2000; Gyasi-Agyei, 2013; Rodriguez-Iturbe et al., 1987; Rodriguez-Iturbe et al., 1988; Eagleson, 1978; Heneker et al., 2001; Koutsoyiannis and Pachakis, 1996; Menabde and Sivapalan, 2000)
Scale Invariance Theory Based Models	Utilises the moment scaling function and an appropriate probability distribution for the weights.	(Waymire and Gupta, 1981a; Menabde et al, 1997; Seed et al., 1999; de Lima and Grasman, 1999; Deidda et al., 1999; Gupta and Waymire, 1993; Schertzer and Lovejoy, 1987; Sivakumar et al., 2001; Lovejoy and Schertzer, 1990; Molnar and Burlando, 2005; Olsson and Berndtsson, 1998; Over and Gupta, 1996)
Parametric and non-parametric stochastic disaggregation models	Based on disaggregation of daily rainfall based on distribution of sub-daily rainfall statistics/ rainfall values.	(Arnold and Williams, 1989; Connolly et al., 1998; Cowpertwait et al., 1996; Econopouly et al, 1990; Hershenhorn and Woolhiser, 1987; Sharma and Srikanthan, 2006; Westra et al., 2012; Koutsoyiannis, 2001; Koutsoyiannis and Onof, 2000)

Book 2, Chapter 7, Section 4 and Book 2, Chapter 7, Section 4. discuss two approaches recommended for use in Australia. Both approaches exist in a regionalised form and can be adopted at any location within the country. The Disaggregated Rainfall Intensity Pulse approach represents a sub-daily rainfall generator that is calibrated using sub-daily data and parameters regionalised for use anywhere, while the Regionalised Method of Fragments is a daily to sub-daily disaggregation approach that relies on either the observed daily rainfall or a generated daily rainfall sequence to convert to a sub-daily scale.

7.4.2.1. Disaggregated Rainfall Intensity Pulse

Also known as 'alternating renewal' or 'profile-based' models, event-based models break the rainfall process into a series of events characterised by inter-arrival time, storm duration and mean storm intensity. Early work on such models by Eagleson (1978) involved simulating rainfall arrivals using a Poisson distribution, the time between events and the event duration using an exponential distribution, and the storm event depth using a gamma distribution. Since this time these models have undergone significant development, including the elucidation of the self-similarity concept, in which storms are found to exhibit similar internal structure despite differing durations and storm depths, thus providing a basis for the disaggregation of storm events into within-storm temporal patterns (Koutsoyiannis, 2001; Garcia-Guzman and Aranda-Oliver, 1993), and the development of a generalised exponential distribution for representing inter-storm and storm durations (Lambert and Kuczera, 1998).

The Disaggregated Rectangular Intensity Pulse (DRIP) model was developed by (Heneker et al., 2001) with the view to addressing several perceived deficiencies in existing event-based models, particularly with regard to the simulation of extreme rainfall and aggregation statistics. The DRIP modelling process is divided into two stages. The generation stage (Figure 2.7.7) is represented by three random variables: dry spell or inter-event time t_a, the wet spell or storm duration t_d, and the average intensity i, with t_a and $t_d$both described by a generalised exponential distribution and the intensity (i) described by a Generalised Pareto distribution. In the second stage, the individual events are disaggregated through a constrained random walk (Figure 2.7.7b) to represent the rainfall temporal pattern for each event.

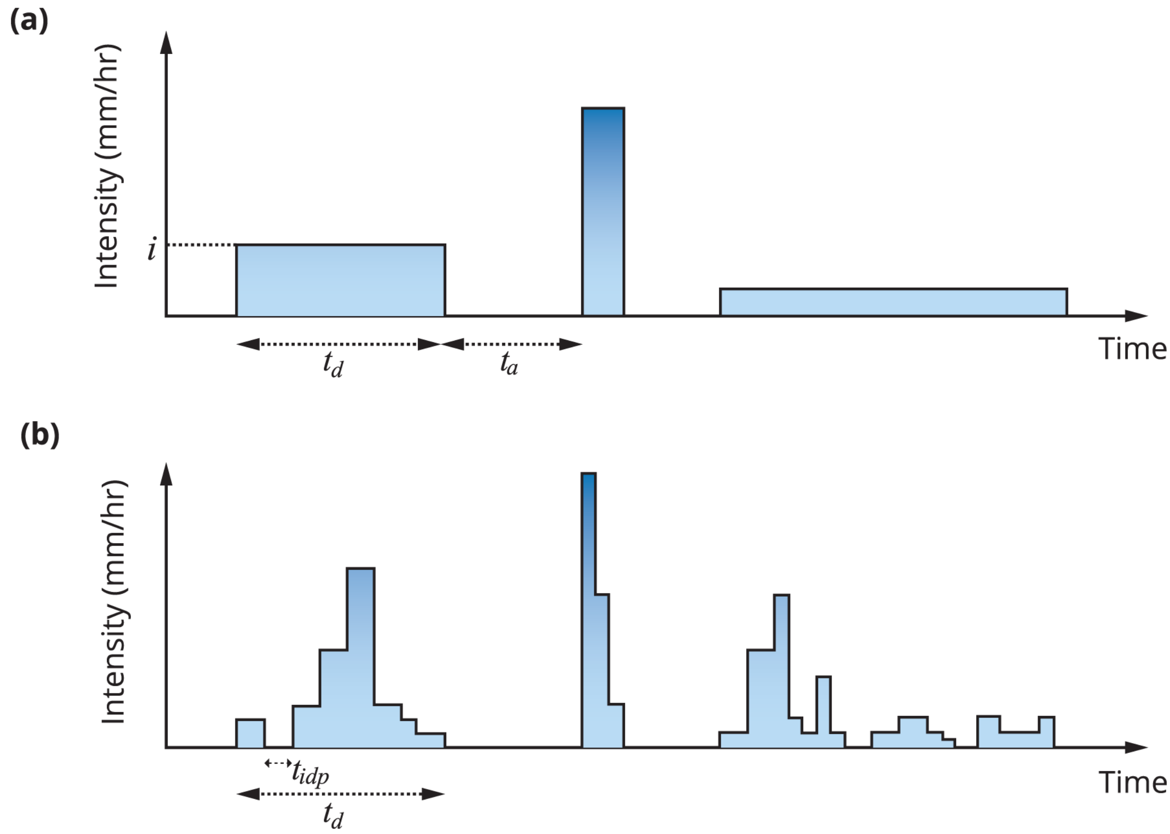


Figure 2.7.7. Disaggregated Rectangular Intensity Pulse Model (extracted from Heneker et al. (2001))

The random walk through a non-dimensional time-depth space is illustrated in Figure 2.7.2. This is then used to disaggregate the rectangular pulse to time steps of the order of six or fewer minutes. Time during the storm is non-dimensionalised by $\tau = {}^{t}\!/\!{}_{t_d}$ where t is the time since the start of the storm and depth is non-dimensionalised by $\delta = {}^{d(t)}\!/\!{}_{it_d}$ where $d(t)$ is the cumulative rainfall up to time t. The random walk progresses in discrete time intervals $\Delta\tau$ from coordinate (0,0) to (1,1) in Figure 2.7.8, always with a non-negative slope. There are two possibilities for a jump from τ to $\tau + \Delta\tau$:

1. An internal dry spell (represented by a horizontal segment in Figure Figure 2.7.8) whose probability of occurrence is defined by a probability distribution; or

2. A rainfall burst (represented by a sloping segment in Figure Figure 2.7.8) whose non-dimemsional depth $\Delta\delta$ is sampled from a probability distribution.

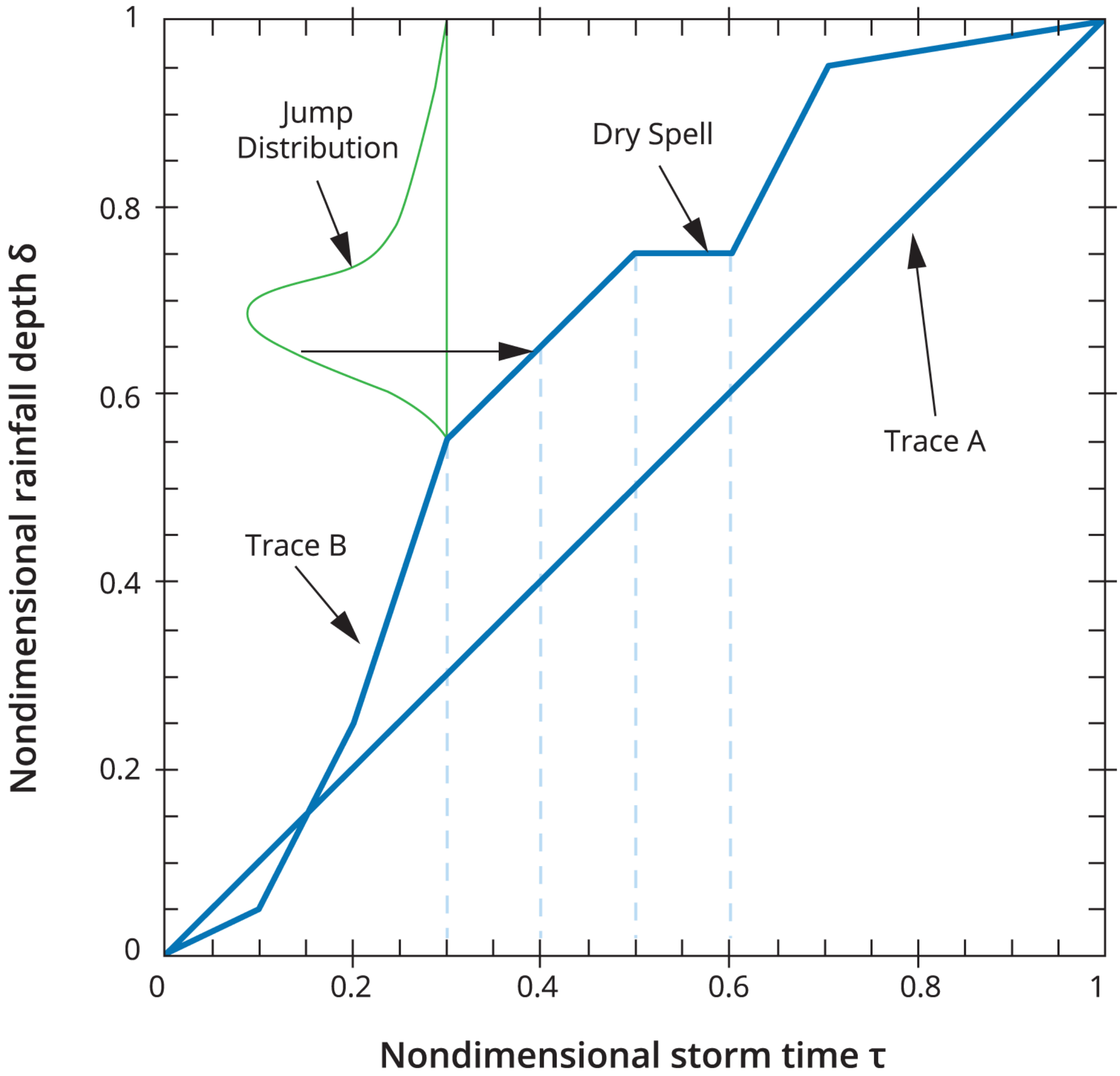


Figure 2.7.8. Schematic of Non-dimensional Random Walk used in DRIP disaggregate pulses

To fit a probability distribution to the observed inter-event time t_a and storm duration $t_d$populations, a procedure was employed to extract independent events from the continuous historical record. After analysis of correlation results, Heneker et al. (2001) adopted a minimum inter-event time of 2 hours to distinguish independent storms and inter-storm periods. This value provides a balance between ensuring consecutive events are sufficiently independent and the need to have as much calibration storm data as possible within a fixed length historical record. While different minimum inter-event times have been reported (e.g. (Grace and Eagleson, 1966; Sariahmed and Kisiel, 1968; Koutsoyiannis and Xanthopoulos, 1990; Heneker et al., 2001)), Heneker et al. (2001) showed that 2 hours was shown to assure independence of storm events across numerous Australian sites.

The generalised exponential distribution developed by (Lambert and Kuczera, 1998) was used to model the distributions of inter-event time and storm duration. The generalised exponential distribution takes the form:

$$F(x|\theta_t) = P(X \leq x|\theta_t) = 1 - e^{[-g_x(x,\theta_t)]}, x > 0 \quad (2.7.4)$$

where X is an independently distributed random variable and θ_t is a parameter vector which may be dependent on$t$ defined as the time at the start of the storm or inter-event time, and $g_x(x|\theta_t)$ is a kernel function. The kernel chosen by Heneker et al. (2001) to best fit the data was a combination of functions based on the Generalised Pareto Distribution (Rosjberg et al., 1992) and the power law:

$$\log_e[1 - F(x|\theta_t)] = -g(x,\theta_t) = \frac{1}{\theta_1}\log_e\left(1 - \theta_1\frac{x}{\theta_2}\right) - \theta_3 x^{\theta_4}, \theta_1 < 0, \theta_2, \theta_3, \theta_4 > 0 \quad (2.7.5)$$

The parameter vector θ_t is estimated using maximum likelihood techniques. The DRIP parameters are usually calibrated for each month of the year to capture seasonal variability in the rainfall process. Figure 2.7.9 and Figure 2.7.10 illustrate observed and fitted probability distributions for inter-event and storm durations for Melbourne for select months and demonstrate the good fit typically achieved by the generalized exponential distribution. Noting that exponentially distributed data would plot as a straight line in Figure 2.7.9 and Figure 2.7.10, the use of an exponential distribution for inter-event and storm durations would be clearly inappropriate. A detailed comparison of the DRIP model with other point process models is given in Frost et al. (2004).

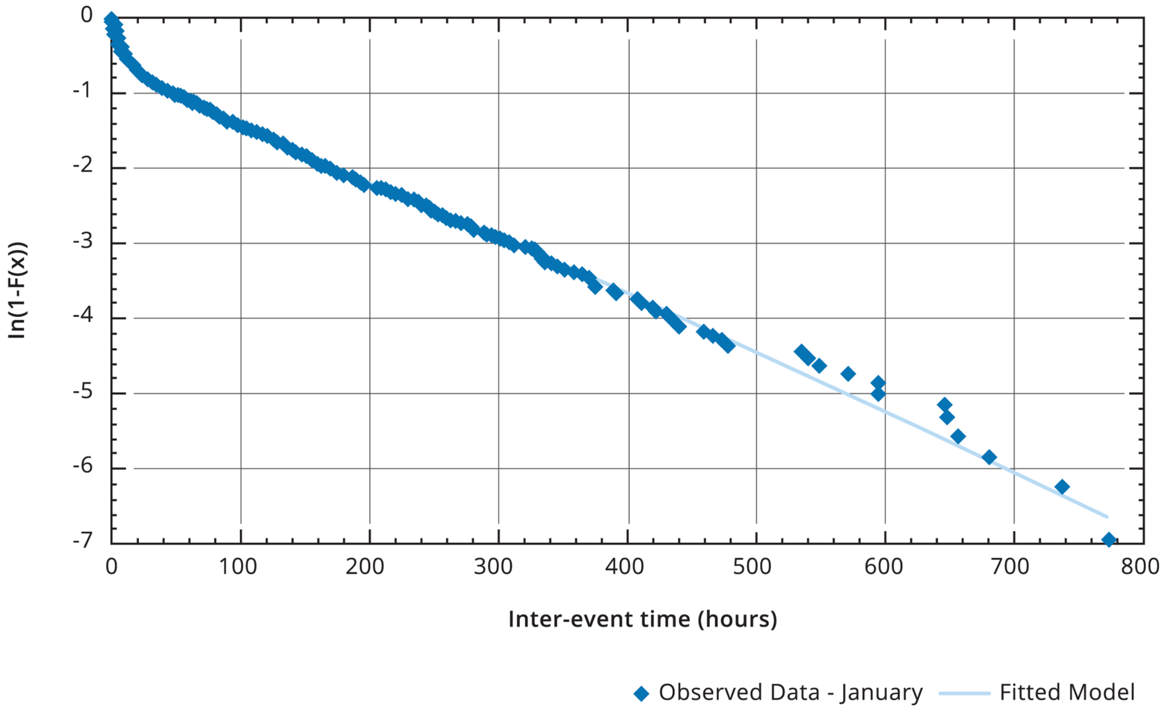


Figure 2.7.9. Heneker et al. (2001) Model Fitted to Monthly Inter-event Time Data for Melbourne in January

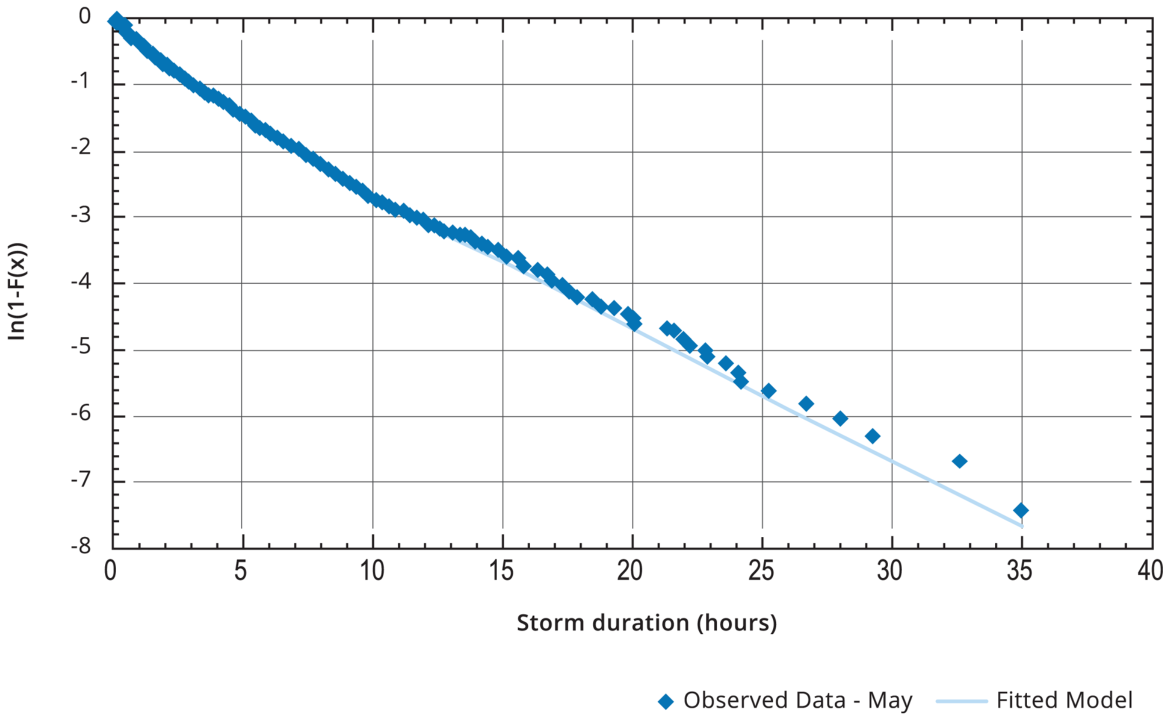


Figure 2.7.10. Heneker et al. (2001) Model Fitted to Monthly Storm Duration Data for Melbourne in May

Recently, DRIP has been extended to any location where sufficient daily data is available, thus greatly augmenting the domain of the approach. The basis of this regionalisation is a 'master-target' scaling relationship in which model calibration is undertaken at a 'master' site with a long pluviograph record which is then updated and scaled to the 'target' site of interest using the information from either a short pluviograph or daily rainfall record (Jennings, 2007), with testing providing encouraging results for separations of up to 190 km between the master and the target.

The software for DRIP is available via the Stochastic Climate Library as part of the e-Water Toolkit (http://toolkit.net.au/Tools/DRIP).

7.4.2.2. Regionalised Method of Fragments

The regionalised method of fragments offers a mechanism to disaggregate observed or generated daily rainfall sequences to a sub-daily time scale. The disaggregation rationale for the method is patterned after the Method of Fragments (Boughton, 1999) that resamples the observed near-continuous fractions (or fragments) of daily accumulated rainfall for use with any daily total that is closest in magnitude. This approach assumes that the sub-daily rainfall structure depends solely on the daily rainfall, an assumption that can lead to discontinuities in the generated sub-daily sequences between two adjacent days. Taking this on board, Westra et al. (2012) modified the basic Method of Fragments approach in two ways. The first modification was to the traditional fragments approach to work at ungauged locations. The second modification was the use of a "state-based" conditioning approach (Sharma and Srikanthan, 2006) that makes use of information about the state of rainfall on the preceding and the next day, in an attempt to reduce the disconnect in sub-daily rainfall attributes across daily boundaries.

Figure 2.7.11 illustrates the rationale behind the regionalised version of the state-based Method of Fragments procedure used in Westra et al. (2012). A sub-daily time-step of 6-minutes is used in Figure 2.7.11, although no change in the procedure is needed if an alternate sub-daily time-step is to be adopted. Here, $I(R_t)$ represents the state (wet or dry) of the rainfall on day t. Conditioning the selection of a “similar” day in the historical record involves selecting from a subset of days that (a) fall within a calendar window representative of the season (chosen equal to +/-15 days in Westra et al. (2012)), and (b) represent the same state $\left(I(R_{t-1}), I(R_{t=1}), I(R_{t+1})\right)$. Once these sub-sets of days are identified, they are ranked based on their similarity with the rainfall amount that is sought to be disaggregated. This forms the sample of days the fragments can be resampled from. Resampling proceeds probabilistically using the k-nn resampling approach of Lall and Sharma (1996). Once the fragments have been resampled, they are scaled back to rainfall amounts by multiplication with the daily rainfall total for the day being disaggregated.

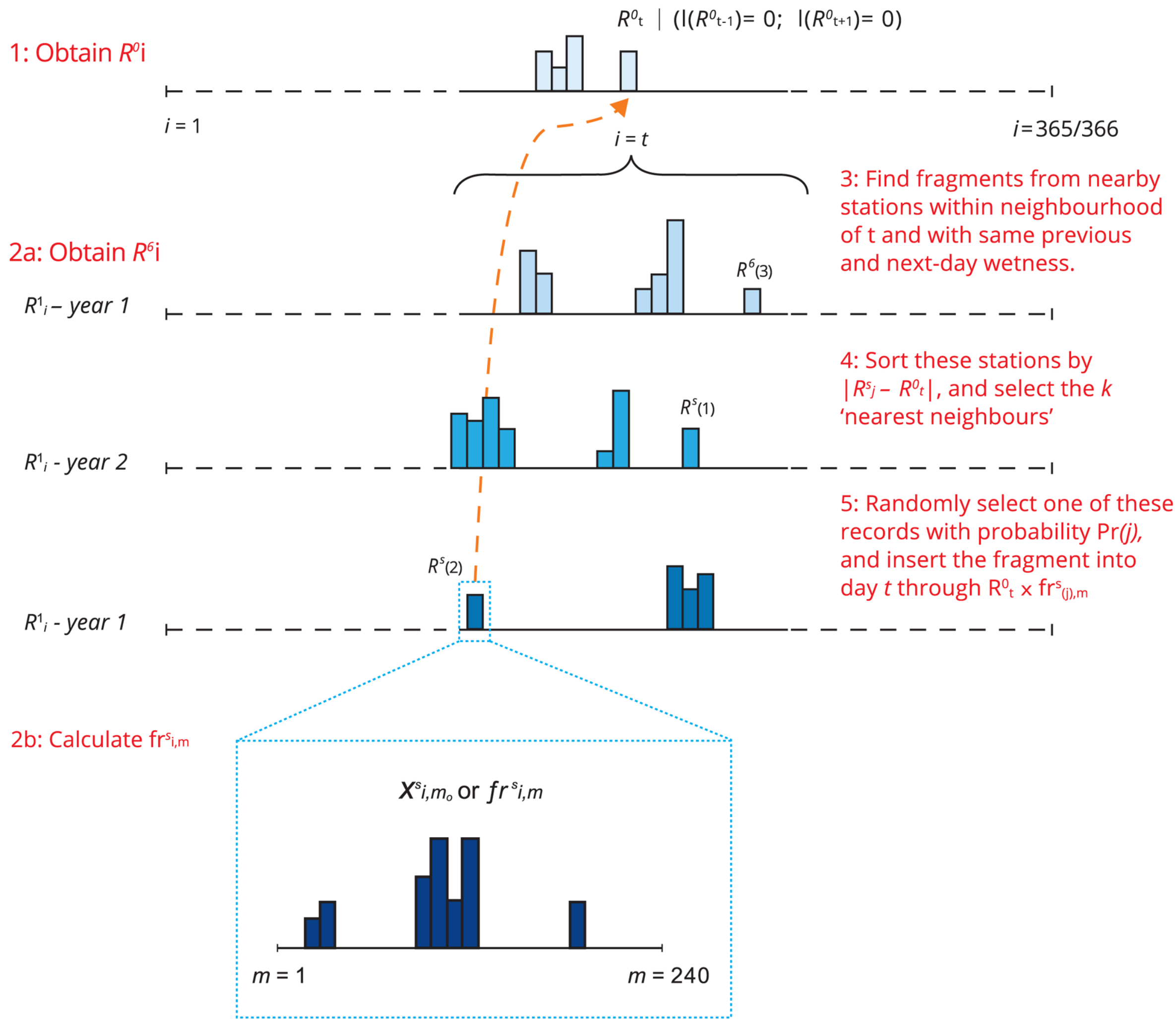


Figure 2.7.11. State-based Method of Fragments Algorithm used in the Regionalised Method of Fragments Sub-daily Rainfall Generation Procedure

The logic used to regionalise the method is similar to that adopted in case of the Regionalised Modified Markov Model (RMMM) (Book 2, Chapter 7, Section 4). Here, the importance of regionalisation is all the more given the paucity of sub-daily rainfall records in most parts of Australia (and the world). However, here the aim of the regionalisation is not to identify locations having similar rainfall attributes as the target, but a similar daily to sub-daily disaggregation relationships. As with the daily rainfall generation, a range of criteria were used to characterise this relationship. These are listed in Table 2.7.6. Each of these variables were estimated for each location, and then paired to assess the best basis for defining ‘similarity’ between stations.

Table 2.7.6. Sub-daily Attributes used to Define Similarity between Locations

Maximum Sub-daily Intensity	Maximum Intensity Fraction for each day for 6, 12, 30, 60, 120, 180 and 360 minute durations.
Fraction of Zeroes	Fraction of zero rainfall time-steps within each day at a 6 minute time scale.
Timing of Maximum Intensity	The timing associated with the maximum intensity fraction for the day for 6, 12, 30, 60, 120, 180 and 360 minute time steps.

Using 232 separate rain gauge stations with at least 30 years of data, a total of 26 796 station pairs were formulated for each attribute. The similarity in each attribute across each pair was then assessed using a two sample two dimensional Kolmogorov Smirnov test. Using a significance level of 5%, this allowed the identification of pairs where the attributes were similar. This then allowed the identification of covariates that could be used to distinguish “similar” locations to allow the regionalisation to proceed. Use of attributes pertaining to the maximum sub-daily fractions at multiple durations, as well as the timing of the maximum, allowed similarity to be defined taking both diurnal pattern characterisation and rainfall magnitudes into account. The use of fraction of zeroes allowed distinction between locations having dominantly convective extremes from those that were spread over the day.

The results of the significance testing described above were used as the basis for formulating a logistic regression relationship for each attribute, with regression coefficients being allowed to vary with season. The predictor variables found to be significant in defining the relationship were the differences in latitude, longitude, elevation and the relative distance to the coast. Based on this relationship, given any location in Australia, the user can identify a subset of sub-daily locations having attributes that are most similar to the target location sequences are needed at. This information is expressed as a probability, which is then used to identify a defined number of sub-daily locations for use in the RMOF procedure.

The logistic regression of the binomial (0 for insignificant and 1 for a significant test outcome) response for each sub-daily attribute can be expressed as:

$$\Pr(u = 1) = \mathrm{logit}(z) = \frac{e^z}{e^z + 1} \tag{2.7.6}$$

The logit function transforms the continuous predictor variables in Table 2.7.7 to the range [0,1] as required when modelling a binomial response. In this equation, **z** is defined as:

$$z = \beta_0 + v_1\beta_1 + \dots + v_5\beta_5 \tag{2.7.7}$$

with β representing the regression coefficients in Table 2.7.7 for the five predictor variables used.

Table 2.7.7. Logistic Regression Coefficients for the Regionalised Method of Fragments Sub-daily Generation Model[a]

		Logistic Regression Coefficients					
Season	Sub-daily Rainfall Attribute	Intercept	Latitude	Longitude	Latitudex Longitude	Distance Coast	Elevation
DJF	6 min intensity	0.426	-0.345	-0.0377	0.0064	-0.186	-0.00089
DJF	1 hr intensity	0.823	-0.333	-0.0425	0.0093	-0.231	-0.00075
DJF	Fraction of zeros	-0.375	-0.253	-0.0318	0.0075	-0.242	-0.00065
DJF	6 min time	0.0979	-0.137	-0.0099	0.0022	-0.453	-0.00141
MAM	6 min intensity	-0.067	-0.192	-0.0065	NS	-0.218	-0.00130
MAM	1 hr intensity	0.308	-0.178	-0.0074	NS	-0.107	-0.00098
MAM	Fraction of zeros	-0.806	-0.157	-0.0105	0.0025	-0.165	-0.00060
MAM	6 min time	1.256	-0.140	-0.0226	-0.0034	-0.227	-0.00092
JJA	6 min intensity	-0.197	-0.097	-0.0110	0.0034	-0.096	-0.00198
JJA	1 hr intensity	0.471	-0.0102	-0.0204	0.0033	NS	-0.00335
JJA	Fraction of zeros	-0.365	-0.073	-0.0171	0.0031	-0.101	-0.00116
JJA	6 min time	2.078	-0.098	-0.0321	0.0037	-0.156	-0.00069
SON	6 min intensity	0.474	-0.387	-0.0722	0.0129	NS	-0.00146
SON	1 hr intensity	0.824	-0.325	-0.0835	0.0135	NS	-0.00132
SON	Fraction of zeros	-0.382	-0.239	-0.0623	0.0104	-0.087	-0.00095
SON	6 min time	1.028	-0.162	-0.0287	0.0042	-0.317	NS

[a]All predictors were found to be statistically significant (usually with a p-value <0.001 level), with the exception of several predictors labelled as NS (Not Significant). Seasons include December-January-February (DJF), March-April-May (MAM), June-July-August (JJA) and September-October- November (SON).

This allows the identification of the most to least similar sub-daily locations for each attribute of interest, which forms the basis for identification of a subset of locations used to sample the fragments. As multiple sub-daily attributes are considered in this choice, this subset is selected based on a common rank averaged across all the attributes for each season. The number of locations the fragments are pooled from depend on their respective data lengths as a total of 500 years of data (including zeroes) is needed for the approach to work.

7.4.3. Identifying ‘Nearby’ Stations - Application to Sydney Airport

Book 2, Chapter 7, Section 4 provides a demonstration of a single application of the approach at one location: Sydney Airport (Gauge number 066037). This location represents a relatively long-record pluviograph station, and therefore provides a useful record for verification of the method.

The approach to identifying ‘nearby’ stations is as follows:

- For all the 1396 pluviograph stations in Australia (excluding the Sydney Airport gauge), calculate each of the regression predictors; namely, difference in latitude, longitude, latitude*longitude, elevation and normalised distance to coast, relative to the Sydney Airport station;
- Having developed the 1396 x 5 predictor matrix, apply the regression model presented in Equation (2.7.6) and Equation (2.7.7) using the regression coefficients shown in Table 2.7.2 for each season and attribute to calculate the probability Pr(u =1);
- Separately for each season and attribute, rank the probabilities from highest to lowest;
- For each season calculate the average rank for each station across all attributes;
- Select the S lowest ranked stations for inclusion in the disaggregation model.

This algorithm yields different choices of stations for each season, as physiographic influences may vary depending on the dominant synoptic systems occurring and different times of the year. It is noted that the selection of the size of S represents a somewhat subjective decision, as larger values of S increase the probability of selecting stations which are statistically different to the target station, whereas smaller values of S will result in small sample sizes. For this case we a total of 500 years of data (including zero rainfalls) distributed over the 13 stations (S=13).

These lowest ranked 13 stations for the summer season are shown in Figure 2.7.12. As expected, the lowest ranked stations (i.e. those with the greatest chance of being ‘similar’ to Sydney Airport, brown dots) are those which are most proximate to this station, generally within a small distance to coast, and all are at low coastal elevations. In this case, therefore, the stations appear to be selected over a wide range of latitudes, which is probably due to the strong increases in elevation and relative distance to coast with changing longitude.

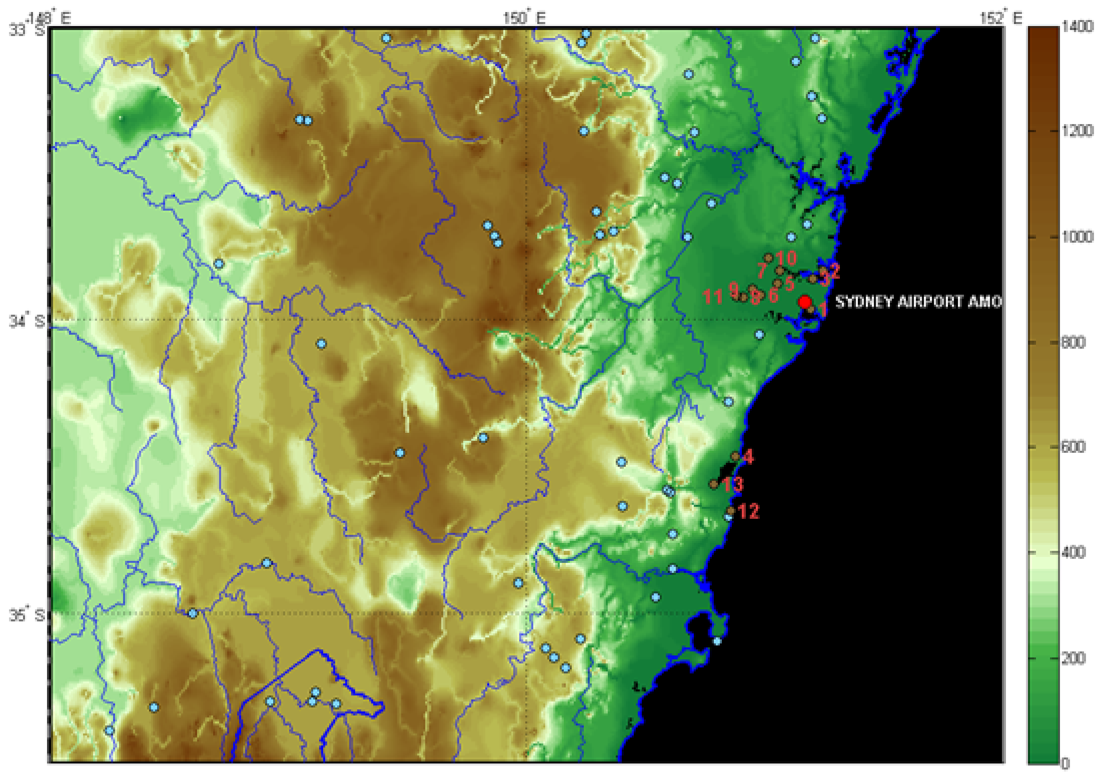


Figure 2.7.12. Sydney Airport and nearby pluviograph stations

It should be noted that the RMOF approach can be expanded to use more sub-daily data than the 1306 stations used in the example for Sydney Airport presented above. New data can be included without the need to update the coefficients of the logistic regression model unless these inclusions are substantial enough to change the distributional characteristics of the data being used. This allows improvements in the representativeness of the continuous simulations as more data over time.

It should also be noted that the RMOF can be used at completely locations having no sub-daily or daily rainfall observations, or to disaggregate daily rainfall records at locations where sub-daily data is not available. In the first case, the use of a daily generation approach is recommended such as the RMMM to generate daily sequences that should then be disaggregated using RMOF. In the latter case, the observed daily sequence can be used directly as the basis for disaggregation.

The software for the RMOF approach is available on request from the authors at this stage, and will be uploaded to the Hydrology@UNSW Software website after a formal review process (http://www.hydrology.unsw.edu.au/download/software/). This document will be updated to reflect the full location once the download of the software is completed.

7.4.4. Modification of Generated Design Rainfall Attributes

The stochastic nature of the algorithms described in previous sections mean that the stochastic sequences will also produce stochastic estimates of design rainfall. In many cases this is a desirable outcome of the approach, as it enables the representation of uncertainty associated with design rainfall. However, in some situations, it may be desirable to post process the design rainfall characteristics obtained through stochastic generation in order to reflect published Intensity Frequency Duration curves. This is likely to be particularly useful when conducting comparisons between the outputs of continuous versus event-based models, or when seeking to understand the role of a catchment's antecedent moisture content conditional to pre-specified design rainfall features.

For cases where it is necessary to have consistency between the Bureau of Meteorology IFDs and the IFDs derived from continuous simulation, a modification in the generation algorithms for RMMM and RMOF was proposed. The main steps involved as illustrated in Figure 2.7.13. First, annual extreme rainfall is corrected at multiple durations so that the IFD based on the generated rainfall matches up with the observed IFD (henceforth referred as 'target IFD'). Second, the non-extreme rainfall (i.e. rainfall that is not part of the annual extreme series) is corrected in such a way that the cumulative rainfall before and after correction is maintained. The dry periods are kept the same before and after bias correction, hence no correction is required for dry periods. As the majority of the data is in the non-extreme category, the corrections are markedly smaller for the non-extreme case.

Due to the inter-dependence of the extreme rainfall across various durations, it is necessary to apply the above corrections in a recursive manner, with each recursion repeating the above steps using a new set of durations exhibiting the maximum difference between the generated and target intensities. This recursion is applied until the following objective function reaches a minimum:

$$\mathrm{RMAE}_{\mathrm{AEP}} = \frac{\left|\mathrm{IFD}_{\mathrm{AEP}}^{T} - \mathrm{IFD}_{\mathrm{AEP}}^{G}\right|}{\mathrm{IFD}_{\mathrm{AEP}}^{T}} \tag{2.7.8}$$

where the objective to be minimised is a dimensionless standardised error measure referred as Relative Mean Absolute Error (RMAE) for consistent comparison across various durations and exceedance probabilities. The RMAE at each of the Annual Exceedance Probabilities (AEP) is estimated through the mean of the absolute difference between the target IFD ($\mathrm{IFD}_{\mathrm{AEP}}^{T}$) and generated IFD ($\mathrm{IFD}_{\mathrm{AEP}}^{G}$) scaled by the target IFD.

Minimisation of the RMAE in Equation (2.7.8) requires the specification of the set of target durations to be used in its adjustment. The choice of durations is governed by the dependence that the extremes for one duration have with the extremes for another. For instance, it is more likely for 6 minute extremes to be a subset of 30 minute extremes than 6 hour extremes (say). In such a case, the durations should be selected keeping an interval that maximises the independence between the extremes being evaluated. In practice, the procedure uses two recursions with separate durations. For both recursions, three target durations, i.e. *D* = 6 min, 1 hr and 3 hrs are considered, which keeps the distance between the durations far enough to reduce the dependence between them. Options exist to use a broader set of durations in the second iteration (6 min, 30 min, 1 hr, 3 hr, 6 hr, 12 hr) although assessment with data for selected city centres in Australia indicated the benefits from this were not significant.

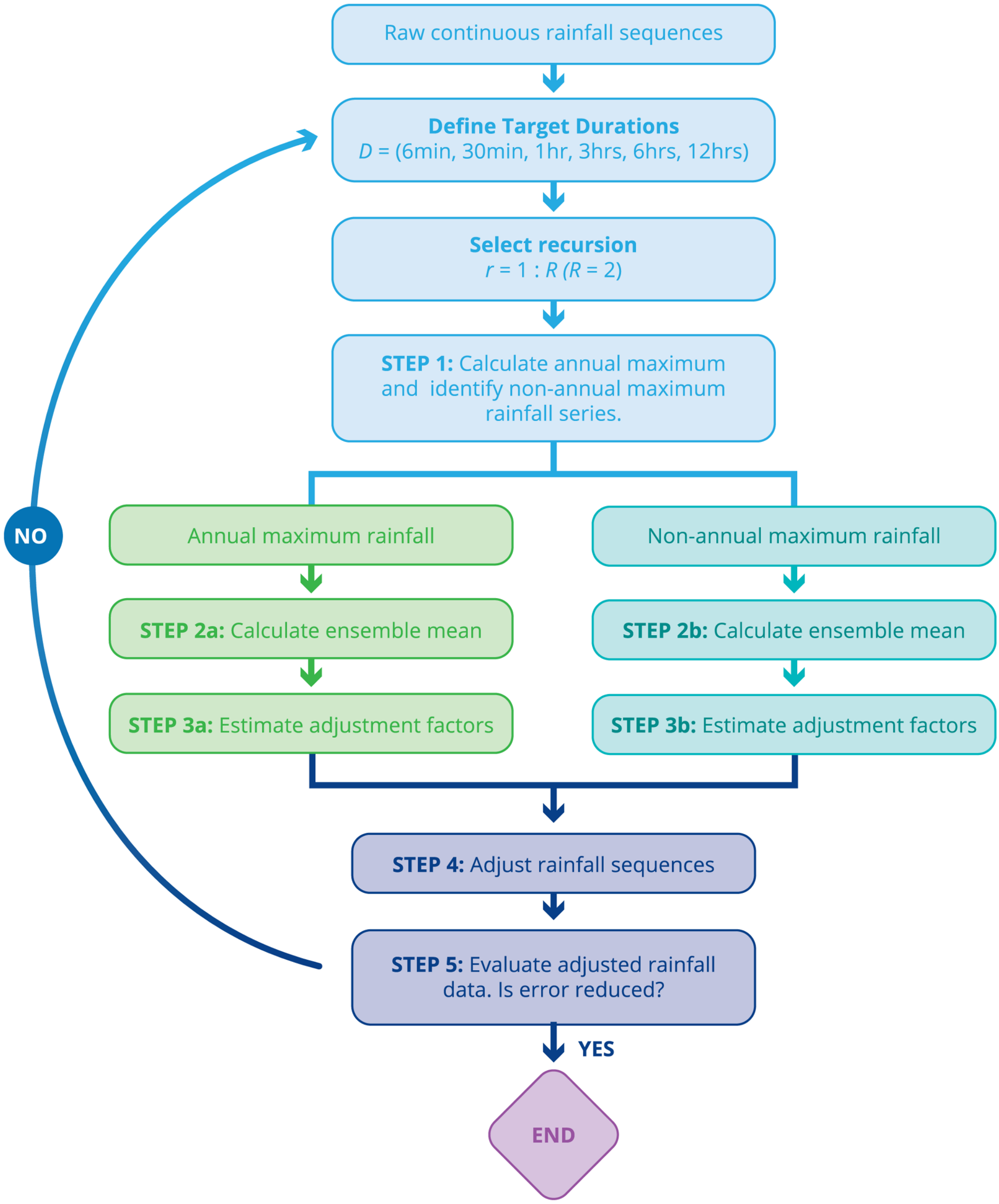


Figure 2.7.13. Main Steps Involved in the Adjustment of Raw Continuous Rainfall Sequences to Preserve the Intensity Frequency Duration relationships

The software for the post-processing approach described above is available on request from the authors at this stage, and will be uploaded to the Hydrology@UNSW Software website after a formal review process (http://www.hydrology.unsw.edu.au/download/software/). This document will be updated to reflect the full location once the download of the software is completed.

7.4.5. Example of Daily and Sub-Daily Rainfall Generation

This example presents the generation of daily, sub-daily and corrected sub-daily rainfall for Alice Springs, including the case where it is assumed that data for the location is not available. Hence, the results here represent a typical example practitioners may face when generating rainfall sequences for any ungauged location in Australia.

Alice Springs is an arid region with average annual rainfall of 280 mm. The observed record at Alice Springs Airport exists for 67 years (1942-2008) and the sub-daily record for 57 years (1951-2007, with missing periods). Each of the statistics presented are based on 100 realisations of length 67 years.

7.4.5.1. Daily Rainfall Generation

For daily rainfall generation, two options are considered;

a. observed rainfall record at the location is available (at-site generation); and

b. no daily rainfall record is available i.e. location is ungauged (regionalised generation).

Table 2.7.8. Statistical Assessment of Daily Rainfall from RMMM for Alice Springs using 100 Replicates 67 years Long

Attribute	Observed	Simulated	
		At-site	Regionalised
Average Annual Wet Days (Nos)	41	40	31
Average Annual Rainfall (mm)	279	297	306
Average Standard Deviation of Annual Wet Days (Nos)	13	12	15
Average Standard Deviation of Annual Rainfall (mm)	152	160	189

Figure 2.7.14 presents annual rainfall simulations for Alice Springs using 100 replicates. The probability distribution of annual rainfall is well represented even in the case of the regionalised simulation where at-site data was not used. This indicates a reasonable representation of the inter-annual variability that characterises Australian rainfall.

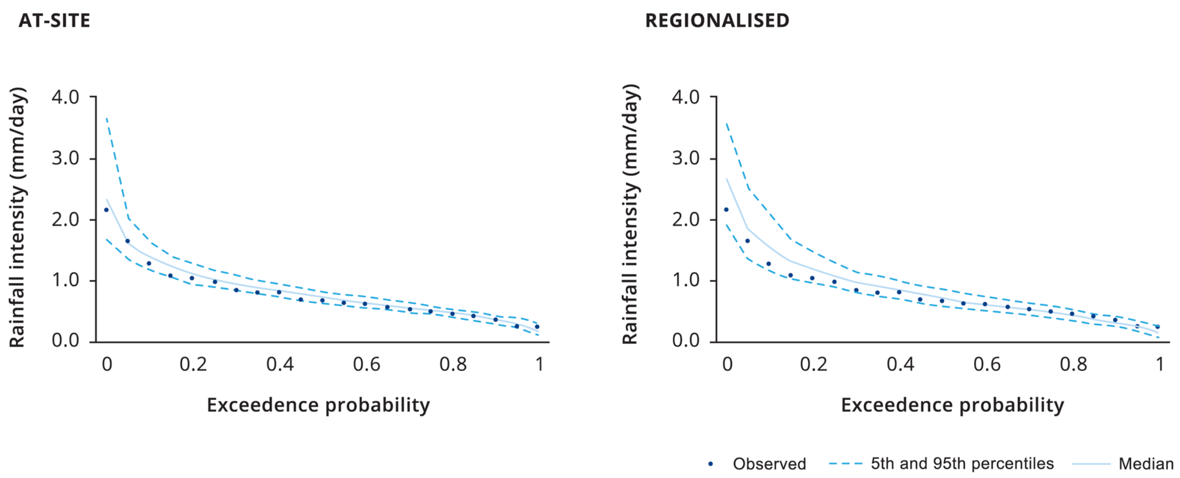


Figure 2.7.14. Annual Rainfall Simulations for Alice Springs using 100 Replicates

As with the annual rainfall in Figure 2.7.13, the extremes are reasonably well simulated even for the regionalised case, except for the most extreme event on record, which the model under-simulates in the regionalised setting (Figure 2.7.15).

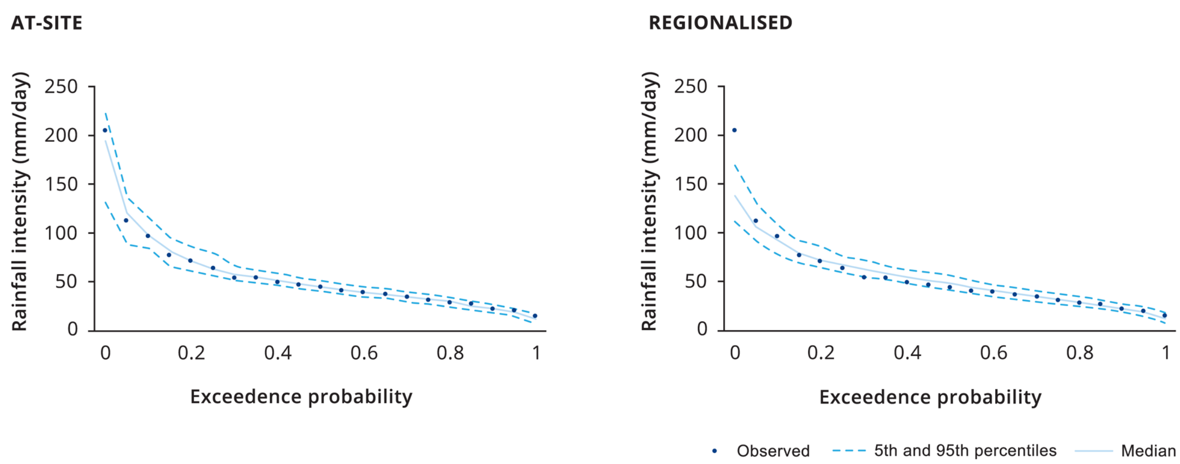


Figure 2.7.15. Intensity Frequency relationship for 24 hour Duration.

It should be noted that the results of the RMMM approach use 2708 daily rainfall stations with long records, instead of the complete daily rainfall observation dataset for Australia. One can expect better representation of underlying rainfall attributes as better and longer datasets are used.

7.4.5.2. Sub-Daily Rainfall Generation

Sub-daily rainfall generation is based on the Regionalised Method of Fragments approach (RMOF) in Book 2, Chapter 7, Section 4. Keeping in mind data availability scenarios for sub-daily rainfall generation, the following generation options are possible:

A. *Daily and sub-daily rainfall record at the location of interest is available*- Daily time series are disaggregated using available at-site sub-daily time series. To obtain multiple simulations, the same daily rainfall time series is used (at-site daily and at-site sub-daily).

B. *Only a daily rainfall record at the location of interest is available* - Daily time series is disaggregated using sub-daily time series from nearby locations (regionalised sub-daily). To obtain multiple simulations, the same daily rainfall time series is used (at-site daily and regionalised sub-daily).

C. *No daily or sub-daily rainfall record at the location of interest is available* - First multiple realisations of daily time series are obtained using regionalised daily model. In the second step, each daily time series is disaggregated using sub-daily time series from nearby locations (regionalised sub-daily) (regionalised daily and regionalised sub-daily).

Selected results from this assessment are presented in Table 2.7.9 and Figure 2.7.15. The deterioration in the representation of extremes in the shorter duration case, is observed, when regionalised options are considered, especially for the smallest duration (6 minute).

Table 2.7.9. Performance of extremes and representation of zeroes (for 6 minute time-steps) from the sub-daily rainfall generation using RMOF for at-site generation using observed sub-daily data (option 1), at-site disaggregation using observed daily data (option 2), and the purely regionalised case (option 3).

	Average Annual Maximum Rainfall (mm) in Spell of			
Duration	**Observed**	**Option A**	**Option B**	**Option C**
6 min	5.5	6.75	6.77	8.02
30 min	16.71	18.07	18.23	20.97
1 hr	22.14	24.19	24.17	26.56
3 hr	32.58	34.77	33.56	34.94
6 hr	39.61	41.73	39.79	40.74
12 hr	48.18	47.65	46.5	46.78
Percentage of zeros	98.54	98.62	98.78	98.68

Top panel presents results for option A, at-site daily rainfall and fragments, middle panel presents results for option B, regionalised daily rainfall at-site fragments, while the bottom panel presents results for option C, regionalised results using ‘nearby’ daily as well as sub-daily records. Dark Blue dots represent observed data, the solid line represents the median of 100 simulations, and dashed lines represent the 5 and 95 percentile simulated values

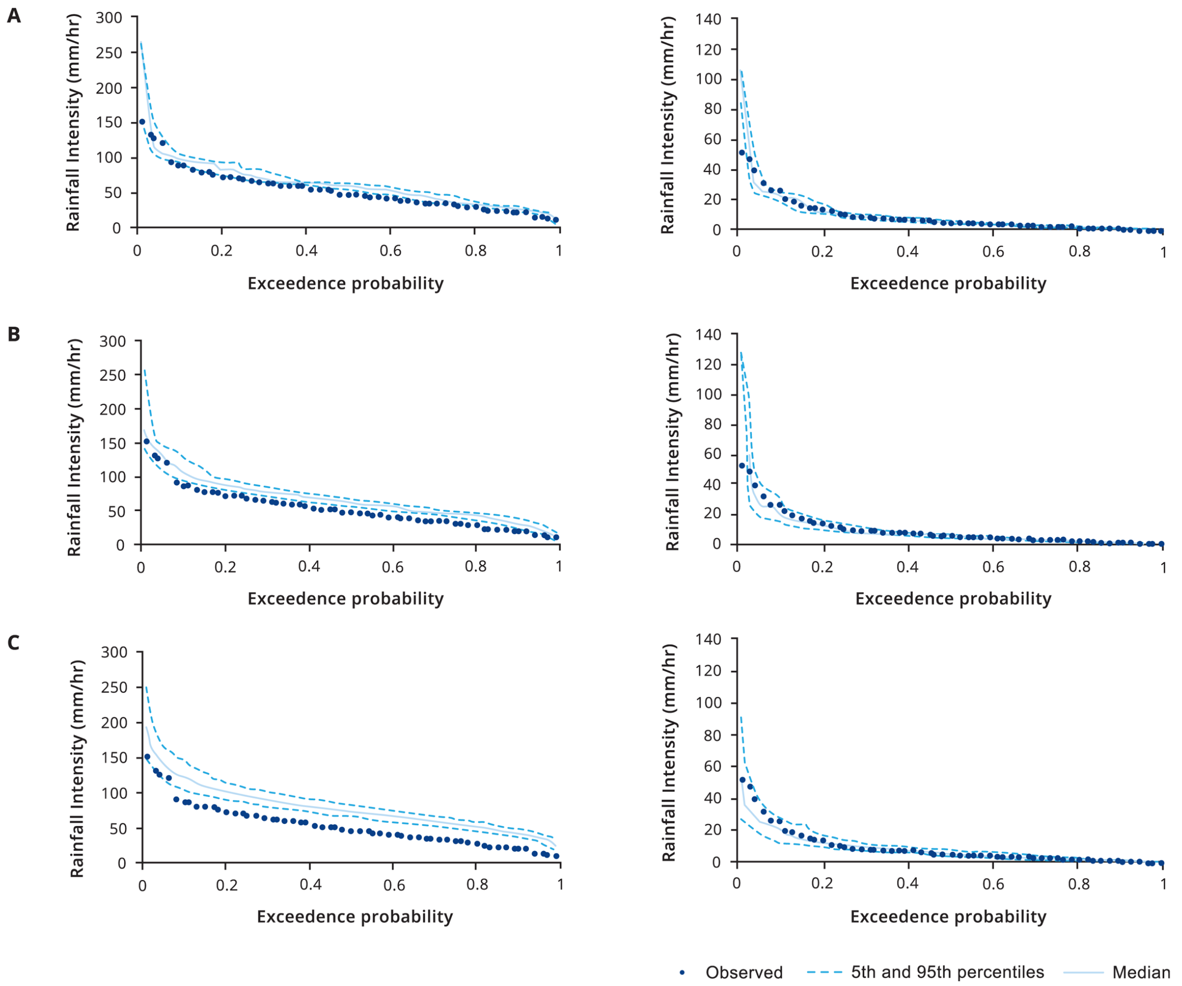


Figure 2.7.16. 6 minute (left column) and 6 hour (right column) Annual Maximum Rainfall against Exceedance Probability for Alice Springs.

7.4.5.3. Post-Processing of Continuous Rainfall to Correct for Intensity Frequency Duration Biases

As illustrated in Figure 2.7.16, in cases where observed rainfall datasets used for continuous simulation are of poor quality or are pooled from locations that are dis-similar to the target, the RMMM and RMOF approaches will simulate sequences with different IFD attributes compared to those published by the Bureau of Meteorology Intenisty Frequency Duration data. This is addressed using a post-processing step that involves scaling of the continuous sequences to alter extremes while attempting to maintain the average annual rainfall for the location of interest.

Results from this post-processing step for the continuous rainfall sequences from RMOF for Alice Springs are presented in Figure 2.7.17. The broken lines (blue and green) indicate the 5 and 95 percentiles for raw and bias corrected data, respectively. The continuous series that was generated has not used rainfall data from Alice Springs for the purpose of generation. In addition to representing low frequency variability characteristics through the proper simulation of daily rainfalls, these continuous sequences are able to mimic actual IFDs and annual rainfall totals, thus making them suitable for continuous flow simulation.

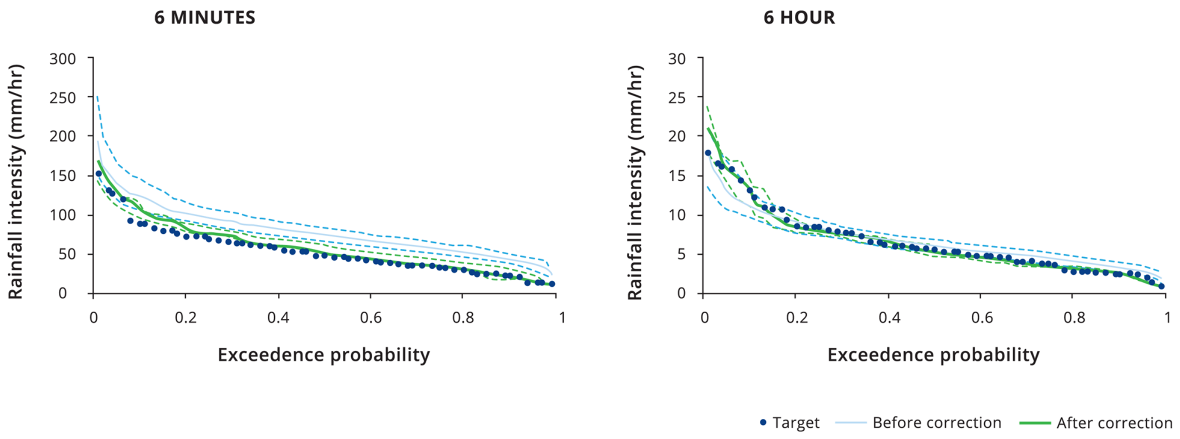


Figure 2.7.17. Intensity Duration Frequency Relationships for Target and Simulated Rainfall before and after Bias Correction at Alice Springs

7.5. Implications of Climate Change

The implications of climate change on design rainfall have been discussed in Book 1, Chapter 6 and are not repeated here. The focus of this section is to discuss how the procedures for continuous simulation described here may be altered to account for climate change. This may be particularly important if there are changes in extreme rainfall, in antecedent rainfall, or in the dependence between the two, all of which will have significant impacts on the resulting design flood.

Both the daily and the sub-daily continuous simulation alternatives discussed here will be affected by climate change. Practitioners may need to use daily rainfall sequences that are representative of future warmer climates, and are referred to the statistical downscaling extensions of the RMMM daily generation approach discussed in Mehrotra and Sharma (2010) for an alternative for generating daily sequences for any location of interest. This generation requires selection of appropriate Global Climate Models (GCMs) and atmospheric predictors, followed by sensible correction of GCM simulations to remove known biases. Practitioners are referred to (Sharma et al., 2013) for a review of the approaches used commonly for these purposes.

Generation of sub-daily sequences will require modification of the RMOF to alternatives that take into account changes to extremes at sub-daily timescales (Westra et al., 2014) as well as changes to associated temporal patterns (Wasko and Sharma, 2015). An alternative that can be used to accommodate these changes is presented in (Westra et al., 2013). In general, approaches for stochastically generating continuous (sub-daily) rainfall sequences under a future climate are a rapidly evolving area of research, and detailed advice on theory and approaches for continuous simulation under a future climate are outside of the scope of this document.

7.6. References

Arnold, J.G. and Williams, J.R. (1989), Stochastic Generation of Internal Storm Structure at a Point, Transactions of the ASAE, 32(1), 161-0167.

Beesley, C.A., Frost, A.J. and Zajaczkowski, J. (2009), A comparison of the BAWAP and SILO spatially interpolated daily rainfall datasets, paper presented at 18th World IMACS / MODSIM Congress, MSSANZ, Cairns, Australia.

Blazkova, S. and Beven, K. (2002), Flood frequency estimation by continuous simulation for a catchment treated as ungauged (with uncertainty), Water Resources Research, Volume 38(8).

Boughton, W.C. (1999), A daily rainfall generating model for water yield and flood studiesRep. Report 99/9, 21pp pp, CRC for Catchment Hydrology, Monash University, Melbourne.

Boughton, W. and Droop, O. (2003), Continuous simulation for design flood estimation - a review, Environmental Modelling & Software, 18(4), 309-318.

Buishand, T.A. (1977), Stochastic modeling of daily rainfall sequences, Meded. Landbouwhogesch. Wageningen, 77(3), 211.

Bárdossy, A. and Pegram, G. (2014), Infilling missing precipitation records - A comparison of a new copula-based method with other techniques, Journal of Hydrology, 519, Part A, pp: 1162-1170.

Bárdossy, A. and Pegram, G.G.S. (2009), Copula based multisite model for daily precipitation simulation, Hydrol. Earth Syst. Sci., 13(12), 2299-2314.

Bárdossy, A. and Plate, E.J. (1992), Space-time model for daily rainfall using atmospheric circulation patterns, Water Resources Research, 28(5), 1247-1259.

Cameron, D., Beven, K., Tawn, J. and Naden, P. (2000), Flood frequency estimation by continuous simulation (with likelihood based uncertainty estimation), Hydrology and Earth System Sciences, 4(1), 23-34.

Carey-Smith, T., Sansom, J. and Thomson, P. (2014), A hidden seasonal switching model for multisite daily rainfall, Water Resources Research, 50(1), 257-272.

Caskey, J.E. (1963), A MARKOV CHAIN MODEL FOR THE PROBABILITY OF PRECIPITATION OCCURRENCE IN INTERVALS OF VARIOUS LENGTH, Mon. Wea. Rev., 91(6), 298-301.

Chapman, T.G. (1997), Stochastic models for daily rainfall in the Western Pacific, Mathematics and Computers in Simulation, 43: 351-358.

Chappell, A., Renzullo, L.J., Raupach, T.H. and Haylock, M. (2013), Evaluating geostatistical methods of blending satellite and gauge data to estimate near real-time daily rainfall for Australia, Journal of Hydrology, 493: 105-114.

Charles, S.P., Bates, B.C. and Hughes, J.P. (1999), A spatiotemporal model for downscaling precipitation occurrence and amounts, Journal of Geophysical Research-Atmospheres, 104(D24), 31657-31669.

Chin, E.H. and Miller, J.F. (1980), On the Conditional Distribution of Daily Precipitation Amounts, Mon. Wea. Rev., 108(9), 1462-1464.

Chowdhury, S. and Sharma, A. (2007), Mitigating Parameter Bias in Hydrological Modelling due to Uncertainty in Covariates, Journal of Hydrology, 340(3-4), 197-204.

Ciach, G.J. (2003), Local random errors in tipping-bucket rain gauge measurements, Journal of Atmospheric and Oceanic Technology, 20(5), 752-759.

Coe, R. and Stern, R.D. (1982), Fitting Models to Daily Rainfall Data, Journal of Applied Meteorology, 21(7), 1024-1031.

Cole, J.A. and Sherriff, J.D.F. (1972), Some single- and multi-site models of rainfall within discrete time increments, Journal of Hydrology, 17(1-2), 97-113.

Connolly, R.D., Schirmer, J. and Dunn, P.K. (1998), A daily rainfall disaggregation model, Agricultural and Forest Meteorology, 92(2), 105-117.

Cowpertwait, P.S.P. (1991), Further developments of the Neyman-Scott clustered point process for modeling rainfall. Water Resources Research, 27: 1431-1438.

Cowpertwait, P.S.P. (2010), A spatial-temporal point process model with a continuous distribution of storm types, Water Resources Research, 46(12).

Cowpertwait, P.S.P., Kilsby, C.G. and O'Connell, P.E. (2002), A space-time Neyman-Scott model of rainfall: Empirical analysis of extremes - art. no. 1131, Water Resources Research, 38(8), 1131-1131.

Cowpertwait, P.S.P., O'Connell, P.E., Metcalfe, A.V. and Mawdsley, J.A. (1996), Stochastic point process modelling of rainfall. II. Regionalisation and disaggregation, Journal of Hydrology, 175(1-4), 47-65.

Das, D., Kodra, E., Obradovic, Z. and Ganguly, A.R. (2012), Mining extremes: Severe rainfall and climate change, in Frontiers in Artificial Intelligence and Applications, edited, pp: 899-900.

Deidda, R., Benzi, R. and Siccardi, F. (1999), Multifractal modeling of anomalous scaling laws in rainfall, Water Resources Research, 35(6), 1853-1867.

Dennett, M.D., Rodgers, J.A. and Stern, R.D. (1983), Independence of Rainfalls through the Rainy Season and the Implications for the Estimation of Rainfall Probabilities, Journal of Climatology, Volume 3(4), pp.375-384.

Eagleson, P.S. (1978), Climate, Soil, and Vegetation 2. The Distribution of Annual Precipitation Derived From Observed Storm Sequences, Water Resources Research, 14(5), 713-721.

Econopouly, T.W., Davis, D.R. and Woolhiser, D.A. (1990), Parameter transferability for a daily rainfall disaggregation model, Journal of Hydrology, 118(1-4), 209-228.

Evin, G., and Favre, A.C. (2012), Further developments of a transient Poisson-cluster model for rainfall, Stochastic Environmental Research and Risk Assessment, 27(4), 831-847.

Fasano, G. and Franceschini, A. (1987), A Multidimensional Version of the Kolmogorov-Smirnov Test, Mon Not R Astron Soc, 225(1), 155-170.

Feyerherm, A.M. and Bark, L.D. (1965), Statistical Methods for Persistent Precipitation Patterns, Journal of Applied Meteorology, 4(3), 320-328.

Feyerherm, A.M. and Bark, L.D. (1967), Goodness of Fit of a Markov Chain Model for Sequences of Wet and Dry Days, Journal of Applied Meteorology, 6(5), 770-773.

Frost, A., R. Srikanthan, and P. Cowpertwait (2004), Stochastic generation of point rainfall data at subdaily timescales: a comparison of DRIP and NSRP, Technical report 04/09, Cooperative Research Centre - Catchment Hydrology, Melbourne, Australia.

Gabriel, K.R. and Neumann, J. (1962), A Markov chain model for daily rainfall occurrence at Tel Aviv, Q.J Royal Met. Soc., 88(375), 90-95.

Garcia-Guzman, A., and Aranda-Oliver, E. (1993), A stochastic model of dimensionless hyetograph, Water Resources Research, 29(7), 2363-2370.

Gates, P. and Tong, H. (1976), On Markov Chain Modeling to Some Weather Data, Journal of Applied Meteorology, 15(11), 1145-1151.

Grace, R.A. and Eagleson, P.S. (1966). The Synthesis of Short-Time-Increment Rainfall Sequences, Hydrodynamics Lab. Report 91, Dept. of Civil Engineering, Massachusetts Institute of Technology, Cambridge, MA.

Gregory, J.M., Wigley, T.M.L. and Jones, P.D. (1993), Application of Markov models to area-average daily precipitation series and interannual variability in seasonal totals, Clim. Dyn., 8: 299-310.

Gupta, V.K. and Waymire, E.C. (1993), A Statistical Analysis of Mesoscale Rainfall as a Random Cascade, Journal of Applied Meteorology, 32(2), 251-267.

Gyasi-Agyei, Y. (1999), Identification of regional parameters of a stochastic model for rainfall disaggregation. Journal of Hydrology. 223: 148-163.

Gyasi-Agyei, Y. (2013), Evaluation of the effects of temperature changes on fine timescale rainfall, Water Resources Research, 49(7), 4379-4398.

Haan, C.T., Allen, D.M. and Street, J.D. (1976), A Markov chain model of daily rainfall, Water Resour. Res., 12: 443-449.

Harrold, T.I., Sharma, A. and Sheather, S.J. (2003a), A nonparametric model for stochastic generation of daily rainfall amounts - art. no. 1343, Water Resources Research, 39(12), 1343-1343.

Harrold, T.I., Sharma, A. and Sheather, S.J. (2003b), A nonparametric model for stochastic generation of daily rainfall occurrence - art. no. 1300, Water Resources Research, 39(10), 1300-1300.

Hasan, M.M., Sharma, A., Johnson, F., Mariethoz, G., Seed, A. (2014), Correcting bias in radar Z-R relationships due to uncertainty in point raingauge networks. Journal of Hydrology 519(B), 1668-1676.

Hay L.E., McCabe G.J., Wolock D.M. and Ayers M.A. (1991), Simulation of precipitation by weather type anaylsis. Water Resources Research, 27: 493-501.

Heaps, S.E., Boys, R.J. and Farrow, M. (2015), Bayesian modelling of rainfall data by using non-homogeneous hidden Markov models and latent Gaussian variables, Journal of the Royal Statistical Society: Series C (Applied Statistics), 64(3), 543-568.

Heneker, T.M., Lambert, M.F. and Kuczera, G. (2001), A point rainfall model for risk-based design, Journal of Hydrology, 247(1-2), 54-71.

Hershenhorn, J. and Woolhiser, D.A. (1987), Disaggregation of daily rainfall, Journal of Hydrology, 95(3-4), 299-322.

Hopkins, J.W. and Robillard, P. (1964), Some Statistics of Daily Rainfall Occurrence for the Canadian Prairie Provinces, Journal of Applied Meteorology, 3(5), 600-602.

Hughes, J.P. and Guttorp, P. (1994), Incorporating Spatial Dependence and Atmospheric Data in a Model of Precipitation, Journal of Applied Meteorology, 33(12), 1503-1515.

Jeffrey, S.J., Carter, J.O., Moodie, K.B. and Beswick, A.R. (2001), Using spatial interpolation to construct a comprehensive archive of Australian climate data, Environmental Modelling & Software, 16(6), 309-330.

Jennings S. (2007). A High Resolution Point Rainfall Model Calibrated to Short Pluviograph or Daily Rainfall Data, PhD Thesis, The University of Adelaide, Australia.

Jha, S.K., Mariethoz, G., Evans, J., McCabe, M.F. and Sharma, A. (2015), A space and time scale dependent non-linear geostatistical approach for downscaling daily precipitation and temperature, Water Resources Research, available at: http://onlinelibrary.wiley.com/doi/10.1002/2014WR016729/full accesses [20 July 2015].

Jones, P.G. and Thornton, P.K. (1997), Spatial and temporal variability of rainfall related to a third-order Markov model, Agricultural and Forest Meteorology, 86(1-2), 127-138.

Jones, D., Wang, W.and Fawcett, R. (2009), High-quality spatial climate data-sets for Australia, Australian Meteorological and Oceanographic Journal, 58: 233-248.

Katz, R. and Parlange, M.B. (1995), Generalizations of chain-dependent processes: Application to hourly precipitation. Water Resour. Res., 31: 1331-1341.

Katz, R.W. and Parlange, M.B. (1998), Overdispersion phenomenon in stochastic modeling of precipitation, Journal of Climate, 11(4), 591-601.

Katz, R.W., Parlange, M.B. and Tebaldi, C. (2003), Stochastic modeling of the effects of large-scale circulation on daily weather in the southeastern US, Climatic Change, 60(1-2), 189-216.

Kavvas, M.L. and Delleur, J.W. (1981), A stochastic cluster model of daily rainfall sequences, Water Resources Research, 17(4), 1151-1160.

Kim, D., Kim, J. and Cho, Y.-S. (2014), A Poisson Cluster Stochastic Rainfall Generator That Accounts for the Interannual Variability of Rainfall Statistics: Validation at Various Geographic Locations across the United States, Journal of Applied Mathematics, 2014, pp: 1-14.

Kim, Y., Katz, R.W. Rajagopalan, B., Podestá, G.P. and Furrer, E.M. (2012), Reducing overdispersion in stochastic weather generators using a generalized linear modeling approach, Climate Research, 53(1), 13-24.

Kleiber, W., Katz, R.W. and Rajagopalan, B. (2012), Daily spatiotemporal precipitation simulation using latent and transformed Gaussian processes, Water Resources Research, 48(1).

Koutsoyiannis, D. (2001), Coupling stochastic models of different timescales, Water Resources Research, 37(2), 379-391.

Koutsoyiannis, D. and Onof, C. (2000), A computer program for temporal rainfall disaggregation using adjusting procedures, paper presented at 25th General Assembly of the European Geophysical Society, EGS, Nice.

Koutsoyiannis, D. and Pachakis, D. (1996), Deterministic chaos versus stochasticity in analysis and modeling of point rainfall series, Journal of Geophysical Research, 101(D21), 26441-26451.

Koutsoyiannis, D. and Xanthopoulos, T. (1990). A dynamic model for short-scale rainfall disaggregation, Hydrological Sciences Journal, 35(3), 303-322.

Koutsoyiannis, D., Onof, C. and Wheater, H.S. (2003), Multivariate rainfall disaggregation at a fine timescale - art. no. 1173, Water Resources Research, 39(7), 1173-1173.

La Barbera, P., Lanza, L.G. and Stagi, L. (2002), Tipping bucket mechanical errors and their influence on rainfall statistics and extremes, Water Science and Technology, 45(2), 1-10.

Lall, U. and Sharma, A. (1996), A nearest neighbor bootstrap for time series resampling, Water Resources Research, 32(3), 679-693.

Lall, U., Rajagopalan, B. and Tarboton. D.G. (1996), A nonparametric wet/dry spell model for resampling daily precipitation, Water Resources Research, 32(9), 2803-2823.

Lamb, R., and Kay, A.L. (2004), Confidence intervals for a spatially generalized, continuous simulation flood frequency model for Great Britain, Water Resources Research, 40(7).

Lambert, M., and Kuczera, g. (1998), Seasonal generalized exponential probability models with application to interstorm and storm durations, Water Resources Research, 34(1), 143-148.

Lambert M.F., Whiting J.P. and Metcalfe, A.V. (2003). A non-parametric hidden Markov model for climate state identification. Hydrology and Earth Systems Sciences, 7(5), pp. 652-667.

Leblois, E., and Creutin J.-E. (2013), Space-time simulation of intermittent rainfall with prescribed advection field: Adaptation of the turning band method, Water Resources Research, 49(6), 3375-3387.

Leonard, M., Lambert, M.F. Metcalfe, A.V. and Cowpertwait, P.S.P. (2008), A space-time Neyman-Scott rainfall model with defined storm extent, Water Resources Research, 44(9).

Lovejoy, S. and Schertzer, D. (1990), Multifractals, universality classes and satellite and radar measurements of cloud and rain fields, J. Geophys. Res., 95(D3), 2021-2034.

McMahon, T.A. and Srikanthan, R. (1983), Statistical characteristics of Australian Rainfall and EvaporationRep.

Mehrotra, R., and Sharma, A. (2006), A nonparametric stochastic downscaling framework for daily rainfall at multiple locations, Journal of Geophysical Research-Atmospheres, 111(D15101), doi:10.1029/2005JD006637.

Mehrotra, R., and Sharma, A. (2010), Development and Application of a Multisite Rainfall Stochastic Downscaling Framework for Climate Change Impact Assessment, Water Resources Research, 46.

Mehrotra, R., and Sharma, A. (2007a), Preserving low-frequency variability in generated daily rainfall sequences, Journal of Hydrology, 345(1-2), 102-120.

Mehrotra, R. and Sharma, A. (2007b), A semi-parametric model for stochastic generation of multi-site daily rainfall exhibiting low-frequency variability, Journal of Hydrology, 335(1-2), 180-193.

Mehrotra, R., Westra, S., Sharma, A. and Srikanthan, R. (2012), Continuous rainfall simulation: 2 - A regionalised daily rainfall generation approach, Water Resources Research, 48(1), W01536

Mehrotra, R., Li, J., Westra, S. and Sharma, A. (2015), A programming tool to generate multi-site daily rainfall using a two-stage semi parametric model Environmental Modelling & Software, 63: 230-239.

Menabde, M. and Sivapalan, M. (2000), Modeling of rainfall time series and extremes using bounded random cascades and Levy-stable distribution, Water Resources Research, 36(11), 3293-3300.

Menabde, M., Seed, D., Harris and Austin, G. (1997), Self-similar random fields and rainfall simulation, J. Geophys. Res., 102(D12), 13509-13515.

Micevski, T. and Kuczera, G. (2009), Combining Site and Regional Flood Information Using a Bayesian Monte Carlo Approach, Water Resources Research(doi: 10.1029/2008WR007173), available at: online on http://www.agu.org/journals/wr/papersinpress.shtml.

Molnar, P. and Burlando, P. (2005), Preservation of rainfall properties in stochastic disaggregation by a simple random cascade model, Atmospheric Research, 77(1-4), 137-151.

Olsson, J. and Berndtsson, R. (1998), Temporal rainfall disaggregration based on scaling properties, Water Science and Technology, 37(11), 73-79.

Onof, C. and Townend, J. (2004), Modelling 5-minute rainfall extremes, paper presented at British Hydrological Society International Conference, London.

Onof, C., Chandler, R.E. Kakou, A., Northrop, P., Wheater, H.S. and Isham, V. (2000), Rainfall modelling using Poisson-cluster processes: a review of developments, Stochastic Environmental Research and Risk Assessment, 14(6), 384-411.

Oriani, F., Straubhaar, J., Renard, P. and Mariethoz, G. (2014), Simulation of rainfall time series from different climatic regions using the direct sampling technique, Hydrology and Earth System Sciences, 18(8), 3015-3031.

Over, T. M. and Gupta, V.K. (1996), A space-time theory of mesoscale rainfall using random cascades, J. Geophys. Res., 101(D21), 26319.

Pathiraja, S., Westra, S. and Sharma, A. (2012), Why Continuous Simulation? The Role of Antecedent Moisture in Design Flood Estimation, Water Resources Research, 48(W06534).

Pegram, G.G.S. (1980), An Autoregressive Model for Multilag Markov Chains, Journal of Applied Probability, 17(2), 350.

Pui, A., Lal, A. and Sharma, A. (2011), How does the Interdecadal Pacific Oscillation affect design floods in Australia?, Water Resources Research, 47(5).

Pui, A., Sharma, A., Santoso, A. and Westra, S. (2012), Impact of the El Niño Southern Oscillation, Indian Ocean Dipole, and Southern Annular Mode on daily to sub-daily rainfall characteristics in East Australia, Monthly Weather Review, 140: 1665-1681.

Racsko, P., Szeidl, L. and Semenov, M. (1991), A serial approach to local stochastic weather models, Ecological Modelling, 57(1-2), 27-41.

Ramirez, J. A. and Bras, R.L. (1985), Conditional Distributions of Neyman-Scott Models for Storm Arrivals and Their Use in Irrigation Scheduling, Water Resources Research, 21(3), 317-330.

Richardson, C.W. (1981), Stochastic Simulation of Daily Precipitation, Temperature, and Solar-Radiation, Water Resources Research, 17(1), 182-190.

Rodriguez-Iturbe, I., Gupta, V.K. and Waymire, E. (1984), Scale considerations in the modeling of temporal rainfall, Water Resources Research, 20(11), 1611-1619.

Rodriguez-Iturbe, I., Cox, D.R. and Isham, V. (1987), Some Models for Rainfall Based on Stochastic Point Processes, Proceedings of the Royal Society A: Mathematical, Physical and Engineering Sciences, 410(1839), 269-288.

Rodriguez-Iturbe, I., Cox, D.R. and Isham, V. (1988), A Point Process Model for Rainfall: Further Developments, Proceedings of the Royal Society A: Mathematical, Physical and Engineering Sciences, 417(1853), 283-298.

Roldan, J., and Woolhiser, D.A. (1982), Stochastic daily precipitation models 1. A comparison of occurrence models, Water Resources Research, 18(5), 1451-1459.

Rosjberg, D., Madsen, H. and Rasmussen, P. F. (1992). Prediction in Partial Duration Series with Generalized Pareto-Distirbuted Exceedences, Water Resources Research, 28(11), 3001-3010.

Sariahmed, A. and Kisiel, C.C. (1968). Synthesis of sequences of summer thunderstorm volumes for the Atterbury watershed in the Tucson area. Proceedings International Association Hydrologic Sciences Symposium on Use of Analog and Digital Computers in Hydrology, 2: 439-447.

Schertzer, D. and Lovejoy, S. (1987), Physical modeling and analysis of rain and clouds by anisotropic scaling multiplicative processes, J. Geophys. Res., 92(D8), 9693.

Seed, A.W., Srikanthan, R. and Menabde, M. (1999), A space and time model for design storm rainfall, Journal of Geophysical Research-Atmospheres, 104(D24), 31623-31630.

Selvalingam, S. and Miura, M. (1978), STOCHASTIC MODELING OF MONTHLY AND DAILY RAINFALL SEQUENCES, J Am Water Resour As, 14(5), 1105-1120.

Serinaldi, F. (2009), A multisite daily rainfall generator driven by bivariate copula-based mixed distributions, J. Geophys. Res., 114(D10).

Serinaldi, F. and Kilsby, C.G. (2014), Simulating daily rainfall fields over large areas for collective risk estimation, Journal of Hydrology, 512: 285-302.

Sharma, A. and Lall, U. (1999), A nonparametric approach for daily rainfall simulation, Mathematics and Computers in Simulation, 48(4-6), 361-371.

Sharma, A. and Mehrotra, R. (2010), Rainfall Generation, in Rainfall: State of the Science, edited by F.Y. Testik and M. Gebremichael, p: 287, American Geophysical Union, San Francisco.

Sharma, A. and O'Neill, R. (2002), A nonparametric approach for representing interannual dependence in monthly streamflow sequences - art. No.1100, Water Resources Research, 38(7), 1100-1100.

Sharma, A. and Srikanthan, R. (2006), Continuous Rainfall Simulation: A Nonparametric Alternative, paper presented at 30th Hydrology and Water Resources Symposium, Institution of Engineers (Australia), Launceston.

Sharma, A., Mehrotra, R. and Johnson, F. (2013), A New Framework for Modeling Future Hydrologic Extremes: Nested Bias Correction as a Precursor to Stochastic Rainfall Downscaling (invited), in Climate Change Modeling, Mitigation, and Adaptation, edited by R.Y. Surampalli, T.C. Zhang, C.S.P. Ojha, B.R. Gurjar, R. D. Tyagi and C.M. Kao, p: 698, American Society of Civil Engineers, Reston, Virginia.

Singh, S.V. and Kripalani, R.H. (1986), Analysis of persistence in daily monsoon rainfall over India, Journal of Climatology, 6(6): 625-639.

Sivakumar, B., Berndtsson, R., Olsson, J. and Jinno, K. (2001), Evidence of chaos in the rainfall-runoff process, Hydrological Sciences Journal, 46(1), 131-145.

Srikanthan, R. and McMahon, T.A. (1985), Stochastic generation of rainfall and evaporation data, Tech. Paper No. 84, Aust. Water Resour. Council, Canberra, 1985, Australian Water Resources Council, Technical Paper No.84.

Srikanthan, R. and Pegram, G. (2009), A Nested Multisite Daily Rainfall Stochastic Generation Model, Journal of Hydrology, Available at: http://www.science-direct.com/ until publication.

Srikanthan, R., McMahon, T., Harrold, T. and Sharma, A. (2003), Stochastic Generation of Daily Rainfall DataRep., CRC for Catchment Hydrology Technical Report 03, Melbourne, Australia.

Steinschneider, S., and Lall, U. (2015), A hierarchical Bayesian regional model for nonstationary precipitation extremes in Northern California conditioned on tropical moisture exports, Water Resources Research, 51(3), 1472-1492.

Stern, R.D. and Coe, R. (1984), A Model Fitting Analysis of Daily Rainfall Data, Journal of the Royal Statistical Society. Series A (General), 147(1), 1-34.

Thyer, M. and Kuczera, G. (2003a), A hidden Markov model for modelling long-term persistence in multi-site rainfall time series 1. Model calibration using a Bayesian approach Journal of Hydrology, 275(1-2), 12-26.

Thyer, M. and Kuczera, G. (2003b), A hidden Markov model for modelling long-term persistence in multi-site rainfall time series. 2. Real data analysis Journal of Hydrology, 275(1-2), 27-48.

Viney, N.R. and Bates, B.C. (2004), It never rains on Sunday: The prevalence and implications of untagged multi-day rainfall accumulations in the Australian high quality data set, International Journal of Climatology, 24: 1171-1192.

Wallis, T.W.R. and Griffiths, J.F. (1997), Simulated meteorological input for agricultural models, Agricultural and Forest Meteorology, 88(1-4), 241-258.

Wang, Q. J. and Nathan, R.J. (2007), A method for coupling daily and monthly time scales in stochastic generation of rainfall series, Journal of Hydrology, 346(3-4), 122-130.

Wasko, C. and Sharma, A. (2015), Steeper temporal distribution of rain intensity at higher temperatures within Australian storms, Nature Geoscience, 8(7), 527-529.

Waymire, E. and Gupta, V.K. (1981a), The mathematical structure of rainfall representations: 1. A review of the stochastic rainfall models, Water Resources Research, 17(5), 1261-1272.

Waymire, E., and Gupta, V.K. (1981b), The mathematical structure of rainfall representations: 2. A review of the theory of point processes, Water Resources Research, 17(5), 1273-1285.

Westra, S., Mehrotra, R., Sharma, A. and Srikanthan, R. (2012), Continuous rainfall simulation: 1 - A regionalised sub-daily disaggregation approach, Water Resources Research, 48(1), W01535.

Westra, S., Fowler, H.J., Evans, J.P., Alexander, L.V., Berg, P., Johnson, F., Kendon, E.J., Lenderink, g. and Roberts, N.M. (2014), Future changes to the intensity and frequency of short-duration extreme rainfall, Rev Geophys, 52.

Westra, S., Evans, J.P., Mehrotra, R. and Sharma, A. (2013), A conditional disaggregation algorithm for generating fine time-scale rainfall data in a warmer climate, Journal of Hydrology, 479: 86-99.

Wheater, H. S., Isham, V.S., Cox, D.R., Chandler, R.E., Kakou, A., Northrop, P.J., Oh, L., Onof, C. and Rodriguez-Iturbe, I. (2000), Spatial-temporal rainfall fields: modelling and statistical aspects, Hydrology and Earth System Sciences, 4(4), 581-601.

Wilby, R. L., Wigley, T.M.L., Conway, D., Jones, P.D., Hewitson, B.C., Main, J. and Wilks, D.S. (1998), Statistical Downscaling of General Circulation Model Output - a Comparison of Methods, Water Resources Research, 34(11), 2995-3008.

Wilks, D.S. (1998), Multisite generalization of a daily stochastic precipitation generation model, Journal of Hydrology, 210: 178-191.

Wilks, D.S. (1999a), Multisite downscaling of daily precipitation with a stochastic weather generator, Climate Research, 11(2), 125-136.

Wilks, D.S. (1999b), Simultaneous stochastic simulation of daily precipitation, temperature and solar radiation at multiple sites in complex terrain, Agricultural and Forest Meteorology, 96(1-3), 85-101.

Woolhiser, D.A. (1992), Modeling daily precipitation - progress and problems, in Statistics in the environmental and earth sciences, edited by A.T. Walden and P. Guttorp, p: 306, Edward Arnold, London, U.K.

Woolhiser, D.A. and Pegram, G.G.S (1979), Maximum Likelihood Estimation of Fourier Coefficients to Describe Seasonal Variations of Parameters in Stochastic Daily Precipitation Models, Journal of Applied Meteorology, 18(1), 34-42.

Woolhiser, D.A. and Roldán J. (1982), Stochastic daily precipitation models: 2. A comparison of distributions of amounts, Water Resources Research, 18(5), 1461-1468.

Woolhiser, D.A. and Roldán, J. (1986), Seasonal and Regional Variability of Parameters for Stochastic Daily Precipitation Models: South Dakota, U.S.A, Water Resources Research, 22(6), 965-978.

World Meteorological Organisation (WMO) (1994), WMO Guide to Hydrological Practices, World Meteorological Organisation, Geneva, p: 770.

de Lima, M.I.P. and Grasman, J. (1999), Multifractal analysis of 15-min and daily rainfall from a semi-arid region in Portugal, Journal of Hydrology, 220(1-2), 1-11.

BOOK 3

Peak Flow Estimation

Peak Flow Estimation

Table of Contents

1. Introduction 1
   1.1. Why Estimate Peak Discharges? 1
   1.2. Book Contents 1
   1.3. Selection of Method 2
   1.4. Scope 2
   1.5. Terminology 3
   1.6. References 3
2. At-Site Flood Frequency Analysis 5
   2.1. Introduction 5
   2.2. Conceptual Framework 5
      2.2.1. Definition of Flood Probability Model 5
      2.2.2. Annual Maximum and Peak-Over-Threshold Perspectives 6
      2.2.3. Advantages and Disadvantages of Flood Frequency Analysis 11
      2.2.4. Range of Application 12
   2.3. Selection and Preparation of Data 13
      2.3.1. Requirements of Data for Valid Analysis 13
      2.3.2. Types of Flood Data 14
      2.3.3. Annual Maximum Flood Gauged Series 15
      2.3.4. Peak-Over-Threshold Gauged Series 16
      2.3.5. Monthly and Seasonal Gauged Series 17
      2.3.6. Extension of Gauged Records 18
      2.3.7. Rating Curve Error in Gauged Discharges 19
      2.3.8. Historical and Paleo Flood Information 21
      2.3.9. Data Characterising Long-Term Climate Persistence 22
      2.3.10. Regional Flood Information 24
      2.3.11. Missing Records 24
   2.4. Choice of Flood Probability Model 25
      2.4.1. General 25
      2.4.2. Choice of Distribution Family for Annual Maximum Series 26
      2.4.3. Choice of Distribution for Peak-Over-Threshold Series 31
   2.5. Choice of Flood Quantile Estimator 31
      2.5.1. Expected Parameter Quantiles 32
      2.5.2. Expected AEP Quantiles 33
      2.5.3. Selection of Flood Quantile Estimator 34
   2.6. Fitting Flood Probability Models to Annual Maxima Series 34
      2.6.1. Overview of Methods 34
      2.6.2. Probability Plots 35
      2.6.3. Bayesian Calibration 37
      2.6.4. L-moments Approach 45
      2.6.5. Method of Moments Approach 51
   2.7. Fitting Flood Probability Models to Peak Over Threshold Series 51
      2.7.1. Probability Plots 51
      2.7.2. Fitting Peak-Over-Thresholds Models 52
   2.8. Supplementary Information 52
      2.8.1. Example 1: Extrapolation and Process Understanding 52
      2.8.2. Example 2: Accuracy of Daily Gauged Discharges 54
      2.8.3. Example 3: Fitting a probability model to gauged data 55
      2.8.4. Example 4: Use of binomial censored historical data 69
      2.8.5. Example 5: Use of regional information 73
      2.8.6. Example 6: Censoring PILFs using multiple Grubbs-Beck test 78

2.8.7. Example 7: Improving poor fits using censoring of low flow data 85
2.8.8. Example 8: A Non-Homogeneous Flood Probability Model 88
2.8.9. Example 9: L-moments fit to gauged data 92
2.8.10. Example 10: Improving poor fits using LH-moments 94
2.8.11. Example 11: Fitting a probability model to POT data 98
2.9. References 99
3. Regional Flood Methods 105
3.1. Introduction 105
3.2. Conceptual Framework 106
3.2.1. Definition of Regional Flood Frequency Estimation 106
3.2.2. Formation of Regions 106
3.2.3. Development of Regional Flood Frequency Estimation Technique 107
3.2.4. Data Required to Develop Regional Flood Frequency Estimation Technique 108
3.2.5. Accuracy Considerations 109
3.3. Statistical Framework 110
3.3.1. Region Of Influence (ROI) Approach 110
3.3.2. Parameter Regression Technique 110
3.3.3. Generalised Least Squares Regression 111
3.3.4. Development of Confidence Limits for the Estimated Flood Quantiles 112
3.4. RFFE Techniques for Humid Coastal Areas 112
3.4.1. Data Used to Develop RFFE Technique 112
3.4.2. Adopted RFFE Regions 115
3.4.3. Adopted Estimation Equations 116
3.5. RFFE Techniques for Arid/Semi-Arid Areas 116
3.5.1. Data Used to Develop RFFE Technique 116
3.5.2. Adopted Regions 117
3.5.3. Adopted Estimation Equations 118
3.6. Fringe Zones 118
3.7. Relative Accuracy of the RFFE Technique 119
3.8. Bias Correction 121
3.9. Combining Regional and At-Site Quantile Estimates 122
3.10. Impact of Climate Change 122
3.11. Progressive Improvement of RFFE Model 2015 v1 122
3.12. RFFE Implementation and Limitations 123
3.12.1. Overview of RFFE Accuracy 123
3.12.2. RFFE Implementation 124
3.13. Practical Considerations for Application of the RFFE Technique 125
3.13.1. Urban Catchments 125
3.13.2. Catchments Containing Dams and Other Artificial Storage 126
3.13.3. Catchments Affected by Mining 126
3.13.4. Catchments with Intensive Agricultural Activity 126
3.13.5. Catchment Size 127
3.13.6. Catchment Shape 127
3.13.7. Atypical Catchment Types 127
3.13.8. Catchment Location 128
3.13.9. Arid and Semi-arid Areas 128
3.13.10. Baseflow 129
3.14. RFFE Model 2015 129
3.15. Worked Examples 133

3.15.1. Application of RFFE Model 2015 to the Wollomombi River at Coinside, NSW (Region 1) (A Catchment Having No Major Regulation, No Major Natural or Artificial Storage and No Major Land Use Change) 133
3.15.2. Application for the Four Mile Brook at Netic Rd, SW Western Australia (Region 5) (a Catchment Having No Major Regulation, No Major Natural or Artificial Storage and no Major Land Use Change) 135
3.15.3. Application for the Morass Creek at Uplands, VIC (Region 1) (a Catchment Having Significant Natural Floodplain Storage Where RFFE Model 2015 Output is Not Directly Applicable) 138
3.16. Further Information on the Development and Testing of RFFE Technique 2015 141
3.17. References 143

List of Figures

3.2.1. Peak Over Threshold series ... 8

3.2.2. Annual Exceedance Probability (AEP) - Exceedances per Year (EY) Relationship .. 11

3.2.3. Hydrograph for a 1000 km^2 Catchment Illustrating Difficulty of Assessing Independence of Floods ... 13

3.2.4. Depiction of Censored and Gauged Flow Time Series Data ... 15

3.2.5. Rating Curve Extension by Fitting to an Indirect Discharge Estimate ... 20

3.2.6. Rating Curve Extension by Slope-Conveyance Method ... 21

3.2.7. Annual Average Interdecadal Pacific Oscillation Time Series ... 23

3.2.8. NSW Regional Flood Index Frequency Curves for Positive and Negative Interdecadal Pacific Oscillation epochs (Kiem et al., 2003) ... 24

3.2.9. Rating error multiplier space diagram for rating curve ... 39

3.2.10. Bilinear channel storage-discharge relationship ... 53

3.2.11. Simulated rainfall and flood frequency curves with major floodplain storage activated at a threshold discharge of 3500 m^3/s ... 54

3.2.12. Comparison between true peak flow and 9:00 am flow for Goulburn River at Coggan 55

3.2.13. Comparison between true peak flow and 9:00 am flow for Hunter River at Singleton ... 55

3.2.14. TUFLOW Flike Splash Screen ... 56

3.2.15. Create New .fld file ... 57

3.2.16. Flike Editor Screen ... 58

3.2.17. Observed Values Screen ... 59

3.2.18. Import Gauged Values Screen ... 60

3.2.19. View Gauged Values in Text Editor ... 61

3.2.20. Observed Values screen with Imported Data ... 62

3.2.21. Rank Data Screen ... 63

3.2.22. General Screen – After Data Import ... 64

3.2.23. Blank TUFLOW-Flike Screen ... 65

3.2.24. Probability Plot ... 66

3.2.25. Results File ... 67

3.2.26. Probability Plot using Gumbel Scale ... 68

3.2.27. Censoring observed values tab ... 70

3.2.28. Configured Flike Editor ... 71

3.2.29. Probability plot of the Singleton data with historic information ... 72

3.2.30. Probability plot of the Singleton data with historic information ... 73

3.2.31. Gaussian prior distributions ... 75

3.2.32. Prior for Log-Pearson III window ... 75

3.2.33. Probability plot of with prior regional information 76

3.2.34. Comparison between the results from Example 3 and Example 5 77

3.2.35. TUFLOW Flike editor window with Wimmera data 79

3.2.36. Initial probability plot for Wimmera data with GEV 80

3.2.37. Results of the multiple Grubbs-Beck test 81

3.2.38. Excluded gauged values 82

3.2.39. Censoring of observed values 83

3.2.40. GEV fit - 56 years AM of gauged discharge - Using multiple Grubbs-Beck test 84

3.2.41. Bayesian fit to all gauged data Gumbel probability plot 85

3.2.42. Bayesian fit with 5 low outliers censored after application of multiple Grubbs-Beck test 86

3.2.43. Bayesian fit with floods below 250 m^3/s threshold treated as censored observations 88

3.2.44. Histogram of IPO- and IPO+ flood ratios 89

3.2.45. Log-Normal fit to 43 years of IPO+ data for the Clarence river at Lilydale (units ML/day). 90

3.2.46. Log-Normal fit to 33 years of IPO- data for the Clarence river at Lilydale (units ML/day). 90

3.2.47. Log-Normal fit to 76 years of data for the Clarence river at Lilydale (units ML/day). 91

3.2.48. Marginal, IPO+ and IPO+ log-Normal distributions for the Clarence River at Lilydale 92

3.2.49. Flike Editor configured for L-moments 93

3.2.50. Configured Flike Editor 95

3.2.51. L-moment fit - Albert River at Broomfleet 96

3.2.52. LH-moment fit with shift H=4 97

3.2.53. Plot of the fitted POT exponential model against the observed POT series 99

3.3.1. Geographical Distribution of the Adopted 798 Catchments from Humid Coastal Areas of Australia and 55 Catchments from Arid/Semi-arid Areas 113

3.3.2. Adopted Regions for RFFE Technique in Australia 115

3.3.3. Standardised Residuals vs. Normal Scores for Region 1 Based on Leave One Out Validation for AEPs of 10% and 5% 120

3.3.4. Screen Shot of RFFE Model 2015 (Landing Page) 130

3.3.5. RFFE Model 2015 Screen Shot for Data Input for the Wollomombi River at Coinside, NSW 131

3.3.6. RFFE Model 2015 Screen Shot for Model Output for the Wollomombi River at Coinside, NSW (Region 1) 132

3.3.7. RFFE Model 2015 vs. At-site FFA Flood Estimates for the Wollomombi River at Coinside, NSW (Region 1) 134

3.3.8. RFFE Model 2015 Screen Shot for Data Input for Four Mile Brook at Netic Rd, SW WA (Region 5) 136
3.3.9. RFFE Model 2015 Screen Shot for Model Output for Four Mile Brook at Netic Rd, SW WA (Region 5) 137
3.3.10. RFFE Model 2015 vs. At-site FFA Flood Estimates for Four Mile Brook at Netic Rd, SW WA (Region 5) 138
3.3.11. RFFE Model 2015 Screen Shot for Data Input for the Morass Creek at Uplands, VIC (Region 1) 139
3.3.12. RFFE Model 2015 Screen Shot for Model Output for the Morass Creek at Uplands, VIC (Region 1) 140
3.3.13. RFFE Model 2015 vs. At-site FFA Flood Estimates (for the Morass Creek at Uplands, VIC) (Region 1) 141
3.3.14. Standardised Residuals vs. Normal Scores for Region 1 Based on Leave-one-out Validation for AEPs of 50%, 20%, 2% and 1% 142
3.3.15. Standardised Residuals vs. Normal Scores for Region 1 Based on Leave-one-out Validation for AEPs of 2% and 1% 143

List of Tables

3.2.1. Selected Homogeneous Probability Models Families for use in Flood Frequency Analysis ... 27

3.2.2. Frequency factors for standard normal distribution. ... 29

3.2.3. L-moments for several distributions (from Stedinger et al. (1993)) ... 46

3.2.4. LH-moments for GEV and Gumbel distributions (from Wang (1997)) ... 48

3.2.5. Polynomial coefficients for use with Equation (3.2.73) ... 49

3.2.6. Selected Results ... 68

3.2.7. Gauged flows on the Hunter River at Singleton ... 69

3.2.8. Posterior Mean, Standard Deviation and Correlation for the LP III ... 72

3.2.9. Comparison of Selected Quantiles with 90% Confidence Limits ... 72

3.2.10. Comparison of LP III Parameters with and without prior information ... 77

3.2.11. Selected Results ... 77

3.2.12. Selected Results ... 84

3.2.13. L-moment and GEV Parameter Estimates ... 93

3.2.14. Comparison of Quantiles using a Bayesian and LH-moments Inference Methods . 98

3.3.1. Summary of Adopted Catchments from Humid Coastal Areas of Australia ... 114

3.3.2. Distribution of Shape Factors for the Selected Catchments ... 115

3.3.3. Details of RFFE Technique for Humid Coastal Areas of Australia ... 115

3.3.4. Summary of adopted stations from arid/semi-arid areas of Australia ... 117

3.3.5. Details of RFFE technique for arid/semi-arid regions ... 118

3.3.6. Upper bound on (absolute) median relative error (RE) from leave-one-out validation of the RFFE technique (without considering bias correction) ... 121

3.3.7. Region Names for Application of the RFFE Model 2015 (see Figure 3.3.2 for the Extent of the Regions) ... 129

3.3.8. Application Data for the Wollomombi River at Coinside, NSW (Region 1) (Basic Input Data) ... 134

3.3.9. Fifteen Gauged Catchments (Used in the Development of RFFE Model 2015) Located Closest to Wollomombi River at Coinside, NSW ... 135

3.3.10. Application Data for Four Mile Brook at Netic Rd, SW Western Australia (Region 5) (Basic Input Data) ... 136

3.3.11. Application Data for the Morass Creek at Uplands, VIC (Region 1) (Basic Input Data) ... 138

3.3.12. Further Information on the Development and Testing of RFFE Technique 2015 .. 141

Chapter 1. Introduction

James Ball, Erwin Weinmann, George Kuczera

Chapter Status	Final
Date last updated	14/5/2019

1.1. Why Estimate Peak Discharges?

As outlined in earlier books, there are many alternative forms of design flood problems and hence there are many alternative flood characteristics requiring estimation of a design flood quantile. For many fluvial design flood problems (i.e. problems associated with estimating design flood quantiles at a riverine location), estimation of the quantile of the peak discharge is the critical flood characteristic. This estimation is required as part of the design process for many structures in rural and urban environments (for example culverts and small to medium bridges) and particularly so for small and medium sized catchments. In many of these discharge dominated design problems, an estimation of the full hydrograph and other flood characteristics is not necessary and hence only the peak characteristics of the flood hydrograph require estimation. Where estimation of the full flood hydrograph is required, techniques outlined in other sections of ARR are required in preference to the approaches presented in this book.

Following the concepts outlined in Book 1 for estimation of design flood parameters, where adequate data of sufficient quality are available, it is recommended that an at-site Flood Frequency Analysis (FFA) be used for estimation of the design peak flood discharges quantiles. Details of suitable approaches are outlined in Book 3, Chapter 2.

For many other situations no observed data of a suitable quality for at-site Flood Frequency Analysis are available for estimation of the desired flood quantiles. It is recommended that in these situations, Regional Flood Frequency Estimation (RFFE) techniques be applied. Details of suitable approaches are outlined in Book 3, Chapter 3.

While a consistent methodology for Regional Flood Frequency Estimation for any region in Australia is outlined in Book 3, Chapter 3, designers are reminded of the guidance provided in Book 1, Chapter 1, Section 1; namely, where circumstances warrant, flood engineers have a duty to use other procedures and data that are more appropriate for their design flood problem than those recommended in this Edition of Australian Rainfall and Runoff. This guidance is particularly relevant where approaches have been developed for limited regions of the country without the aim of these approaches being suitable for application across the whole country or being subject to same development testing as the RFFE model proposed herein. An example of this situation is the Pilbara Region of Western Australia where independent studies by Davies and Yip (2014) and Flavell (2012) have developed Regional Flood Frequency Estimation techniques for this region.

1.2. Book Contents

This book contains three chapters with the final two chapters dealing with alternative approaches to the estimation of the peak flood discharge for design purposes. Provided in the this chapter is a general introduction to the contents of this book. Following this introduction, at-site Flood Frequency Analysis is presented in Book 3, Chapter 2. While these analysis techniques are applicable only to catchments where gauged information is available, the philosophy of Flood Frequency Analysis and its application underpin many of the approaches presented in Book 3, Chapter 3 for rural ungauged catchments. It is considered, therefore, to be a fundamental component of the estimation of peak flood quantiles.

Presented in Book 3, Chapter 3 is a range of regional flood methods for estimation of peak flood discharge quantiles in ungauged catchments. These techniques use the results of at-site flood frequency analyses at gauged sites to derive peak discharge estimation procedures for ungauged locations in the same hydrologic region. As the flood characteristics vary considerably between different regions, a range of methods (similar philosophical development but differing in parameter values) have been developed to suit the specific conditions and requirements in different regions.

Different to previous versions of Australian Rainfall and Runoff, in the development of this edition of Australian Rainfall and Runoff there has been an assumption that users will have computing resources available. The techniques presented in the following sections therefore require computing resources for their implementation. Therefore, the discussion in the following sections focusses on both the theoretical basis of the techniques and their implementation.

1.3. Selection of Method

Following the discussion in Book 1, the primary criterion for the selection of the methods recommended in ARR is that the methods should be based on observed flood data in the region of interest and have been peer reviewed by the profession.

In early editions of Australian Rainfall and Runoff, application of this criterion was not always possible because of the paucity of observed flood data technology limitations and the limited analysis of the available data. Hence it was necessary to recommend many arbitrary methods based purely on engineering judgement. The previous approaches towards estimation of the Rational Method runoff coefficient (“C”) for urban catchments is an example of this necessity. As discussed by Hicks et al. (2009), the approach for estimation of the urban runoff coefficient presented by O'Loughlin and Robinson (1987) did not have a scientific foundation but was included to provide the necessary guidance in the application of this method.

For significant portions of Australia, this is no longer the case, and data are available for the development of techniques that have undergone review by the profession from both a scientific and a practical perspective. In these regions, the continued use of arbitrary design methods and information cannot be justified.

It is worthwhile noting that the continued collection of data is necessary to enable ongoing and continued improvements in the design methods, particularly in the robustness of predictions and the detection of inappropriate flood quantile estimates.

1.4. Scope

This book has been prepared as a guide or manual, rather than a mandatory code of practice. Rules and methods appropriate to various situations are presented, together with relevant background information. Since catchments and the problems involved are diverse, and the related technology is changing, recommendations herein should not be taken as binding. They should be considered together with other information and local experience when being implemented.

The contents of this book within Australian Rainfall and Runoff are intended for a wide readership including engineers, students, technicians, surveyors and planners. Readers should be familiar with the basic concepts of catchment hydrology and hence have a basic knowledge of hydrology and hydraulics.

1.5. Terminology

Many terms associated with design flow estimation have been used in a loose manner, and sometimes quite incorrectly and in a misleading fashion. As outlined in Book 1, Chapter 2, the National Committee on Water Engineering of Engineers Australia had three major concerns:

- Clarity of meaning;
- Technical correctness; and
- Practicality and acceptability.

In view of the loose and frequently incorrect manner in which many terms often are used, it was considered that Australian Rainfall and Runoff should adopt terminology that is technically correct, as far as this is possible and in harmony with other objectives. Even if this terminology is not entirely popular with all users, it was considered that Engineers Australia has a responsibility to encourage and educate engineers regarding correct and consistent terminology. It was recognised also by the National Committee on Water Engineering that as well as being correct technically, the terms adopted should be relatively simple and suitable for use in practical design as this would facilitate acceptance by the profession.

The issue of terminology is particularly relevant to the usage of the term model. There are many and varied usages of this term within the field of design flood estimation. For example, the software used for implementation of a particular approach commonly is called a model by users while others refer to the model as the encapsulation of the design flood estimation approach, the calculations necessary for implementation of the approach (usually in software but could be hand calculations) and the data necessary for implementation of the approach. In the following definitions of the terms “model”, “technique” and “approach”, the explanations used are suitable for the guidance contained within this book.

While the major terminology is discussed in Book 1 of Australian Rainfall and Runoff, those terms pertinent only to the contents of this book are presented herein.

1.6. References

Davies, J.R. and Yip, E. (2014), Pilbara Regional Flood Frequency Analysis, Proc. Hydrology and Water Resources Symposium, Perth, February 2014, Engineers Australia, pp: 182-189.

Flavell, D. (2012), Design Flood Estimation in Western Australia, Australian Journal of Water Resources, 16(1), 1-20.

Hicks, B., Gray, S. and Ball, J.E. (2009), A Critical Review of the Urban Rational Method, Proceedings of H2009, 32nd Hydrology and Water Resources Symposium, Engineers Australia, ISBN 978-08258259461.

O'Loughlin, G.G. and Robinson, D.K., (1987), Urban stormwater drainage, Chapter 14 in Australian Rainfall and Runoff - A Guide to Flood Estimation, DH Pilgrim editor, I.E.Aust, Barton, ACT.

Chapter 2. At-Site Flood Frequency Analysis

George Kuczera, Stewart Franks

Chapter Status	Final
Date last updated	14/5/2019

2.1. Introduction

Flood Frequency Analysis (FFA) refers to procedures that use recorded and related flood data to identify underlying probability model of flood peaks, at a particular location in the catchment, which can then be used to perform risk-based design and flood risk assessment, while providing input to regional flood estimation methods.

The primary purpose of this chapter is to present guidelines on performing Flood Frequency Analyses[1].Often judgment will need to be exercised when applying these techniques. To inform such judgments, this chapter describes the key conceptual foundations that underpin Flood Frequency Analysis – the practitioner will need an understanding of elementary probability theory and statistics to get maximum benefit. In addition, a number of worked examples are provided to aid deeper insight with the implied caveat that the examples are not exhaustive in their scope. While it is expected that most practitioners will use software written by others to implement the methods described in this chapter, sufficient information is provided to enable practitioners to develop their own software applications.

2.2. Conceptual Framework

2.2.1. Definition of Flood Probability Model

In Flood Frequency Analysis flood peaks are considered to be random variables. Following convention the random variable denoting the flood peak is denoted by an upper-case symbol (e.g. Q) whereas a specific realisation (or sample) is denoted by the lower-case symbol (e.g. q) – where there is no ambiguity, lower-case symbols will be used.

It is assumed that each realization q is statistically independent of other realisations. This is the standard assumption in Flood Frequency Analysis and is believed to be widely applicable (e.g. Stedinger et al. (1993)).

In its most general form, the flood probability model can be described by its Probability Density Function (pdf) $p(s|\theta(x))$ where $\theta(x)$ is the vector (or list) of parameters dependent on x, a vector of exogenous or external variables such as climate indexes. The symbol ‘|’ is interpreted as follows: the variable to the left of ‘|’ is a random variable, while the variables to the right of ‘|’ are known values.

[1]The chapter represents a substantial revision of Chapter 10 of the 3rd Edition of Australian Rainfall and Runoff (Pilgrim and Doran, 1987). Where appropriate, original contribution by Pilgrim and Doran has been retained. Major changes include introduction of non-homogeneous probability models, replacement of product log-moments with more efficient estimation methods, use of Bayesian methods to make better use of available flood information (such as censored flow data, rating curve error and regional information), reduced prescription about the choice of flood probability model, improved identification of potentially influential low flows and guidance on fitting frequency curves to “difficult” data sets.

The distribution function of Q is defined as the non-exceedance probability $P(Q \leq q)$ and is related to the pdf by:

$$P(Q \leq q|\theta(x)) = \int_0^q p(s|\theta(x))ds \tag{3.2.1}$$

Empirically, the pdf of q is the limiting form of the histogram of q, as the number of samples approaches infinity. Importantly, Equation (3.2.1) shows that the area under the pdf is interpreted as probability.

Homogeneous flood probability model:

The simplest form of flood probability model arises when the parameters θ do not depend on an exogenous vector x. In such a case, each flood peak is considered to be a random realisation from the same probability model $p(q|\theta)$. Under this assumption, flood peaks form a homogeneous time series.

Non-homogeneous flood probability model:

A more complicated situation arises when flood peaks do not form a homogeneous time series. This may arise for a number of reasons including the following:

- Rainfall and flood mechanisms may be changing over time. For example, long-term climate change due to global warming, land use change and river regulation may render the flood record non-homogeneous.

- Climate may experience pseudo-periodic shifts that persist over periods lasting from several years to several decades. There is growing evidence that parts of Australia are subject to such forcing and that this significantly affects flood risk, for example, (Franks and Kuczera, 2002; Franks, 2002a; Franks, 2002b; Kiem et al., 2003; Micevski et al., 2003).

The practioner needs to assess the significance of such factors and identify appropriate exogenous variables x to condition the flood probability model. Although this chapter will provide some guidance it is stressed that this is an area of continuing research – practitioners are therefore advised to keep abreast of new developments.

2.2.2. Annual Maximum and Peak-Over-Threshold Perspectives

Flood Frequency Analysis deals with the probability distribution of significant peak discharges[2]. Throughout the year, there are typically many flood peaks associated with individual storm events. This is demonstrated in Figure 3.2.1 which illustrates a time series record of continuous streamflow discharge. Two types of flood data can be extracted from such a record. In turn, two measures of flood risk can be estimated:

- Annual Maximum (AM) Series

[2]Flood stage is typically not used in Flood Frequency Analysis for a number of reasons. Flood stage is dependent on the geometric properties of the cross-section. As a result, the probability models described in this chapter may not adequately fit peak stage data. Furthermore, the task of regionalizing flood frequency becomes more difficult because of the confounding influence of cross-sectional geometry. Finally, if the cross-section at which stage is measured changes over time, the stage time series will not be consistent over time precluding the use of frequency analysis.

- Peak-Over-Threshold Series

2.2.2.1. Annual Maximum (AM) Series

The AM series is formed by extracting maximum discharge in each year. This yields the series $\{w_1,..,w_n\}$ where w_i is the maximum discharge in the i^{th} year of the n-year record.

The data in the AM series can be used to estimate the probability that maximum flood discharge in a year exceeds a particular magnitude w. In ARR, this probability is called the Annual Exceedance Probability AEP(w) and is formally defined as:

$$AEP(w) = P(W \leq w|\theta(x)) = \int_{q}^{\infty} p(s|\theta(x))ds \tag{3.2.2}$$

where w is the maximum flood discharge in a year. Often it is convenient to express the AEP as a percentage X% or alternatively for rare events. as a ratio 1 in Y. For example, the 1% AEP is equivalent to an AEP of 1 in 100 or 0.01.

2.2.2.2. Peak-Over-Threshold Series

The POT series is formed by extracting every statistically independent peak discharge (that exceeds a threshold discharge), from the record. This yields the series $\{q_1,..,q_m\}$ where q_i is the peak discharge associated with the i^{th} statistically independent flood event in the n-year record. Typically the threshold discharge is selected so that m is about 2 to 3 times greater than n.

The data in the POT series can be used to estimate the probability distribution of the time to the next peak discharge that exceeds a particular magnitude:

$$\mathrm{P}(\text{Time to next peak exceeding } q \leq t) = 1 - e^{-EY(q)t} \tag{3.2.3}$$

where t is time expressed in years and EY(q), the number of exceedances per year, is the expected number of times in a year that the peak discharge exceeds q.

Theory of Peak-Over-Threshold and Annual Maximum Series

Annual Exceedance Probability AEP

The objective is to derive the distribution of the maximum flood peak within a specified interval of time. Referring to the continuous streamflow times series Figure 3.2.1, let the random variable q be a local peak discharge defined as a discharge that has lower discharge on either side of the peak. This presents an immediate problem as any bump on the hydrograph would produce a local peak. To circumvent this problem, we focus on peaks greater than some threshold defined as q_o. The threshold is selected so that the peaks above the threshold are sufficiently separated in time to be statistically independent of each other.

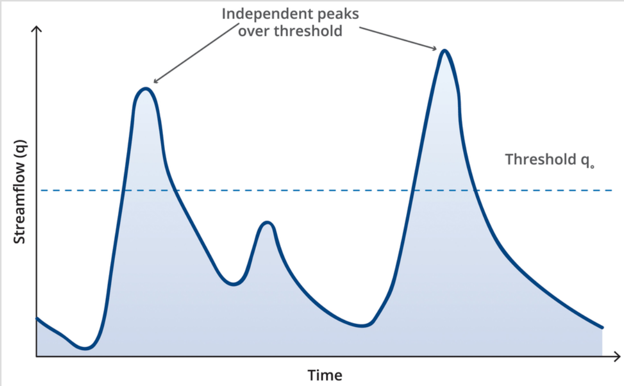


Figure 3.2.1. Peak Over Threshold series

It is assumed that all peaks above the threshold q_o are sampled from the same distribution denoted by the pdf $p(q|q > q_0)$.

Suppose, over a time interval of length t years, there are k peaks over the threshold q_o. This defines the POT time series $\{q_1,\ldots,q_n\}$, which consists of k independent realisations sampled from the pdf $p(q|q > q_0)$.

Let w be the maximum value in the POT time series; that is,

$$w = \max\{q_1, \ldots, q_k\} \tag{3.2.4}$$

For w to be the maximum value, each peak within the POT series must be less than or equal to w. In probability theory this condition is expressed by the compound event consisting of the intersection of the following k events:

$$\{(q_1 \leq w) \cap (q_2 \leq w) \cap \ldots \cap (q_k \leq w)\} \tag{3.2.5}$$

Because the peaks are assumed to be statistically independent, the probability of the compound event is the product of the probabilities of the individual events. Therefore the probability that the random variable W≤ w in a POT series with n events occurring over the interval t simplifies to:

$$\begin{aligned} P(W \leq w, t) &= P[(q_1 \leq w) \cap \ldots \cap (q_k \leq w)] \\ &= P(q_1 \leq w)P(q_2 \leq w)\ldots P(q_k \leq w) \\ &= P(q \leq w)^k \end{aligned} \tag{3.2.6}$$

The number of POT events k occurring over an interval t is random. Suppose that the random variable k follows a Poisson distribution with ν being the average number of POT events per year; that is:

$$P(W \leq \nu, t) = \frac{(\nu t)^n e^{-\nu t}}{n!}, n = 0, 1, 2, \ldots \tag{3.2.7}$$

Application of the total probability theorem yields the distribution of the largest peak magnitude over the time interval with length t:

$$\begin{aligned} P(W \leq w, t) &= \sum_{n=0}^{\infty} P(W \leq n, t) P(W \leq \nu, t) \\ &= e^{[-(\nu t)P(q > w)]} \end{aligned} \tag{3.2.8}$$

where $P(W \leq w, t)$ is the probability that the largest peak over time interval t is less than or equal to w. When the time interval t is set to one year, Equation (3.2.8) defines the distribution of the AM series.

ARR defines Annual Exceedance Probability as:

$$AEP(w) = 1 - P(W \leq w | t = 1) \tag{3.2.9}$$

where AEP(w) is the probability of the largest peak in a year exceeding magnitude w.

Exceedances per Year (EY)

We now derive the probability distribution of the time to the next flood peak which has a magnitude in excess of w. With regard to Equation (3.2.8), if the largest peak during the interval t is less than or equal to w, then the time to the next peak with magnitude in excess of w must be greater than t. It therefore follows that the distribution of the time to the next peak with magnitude exceeding w is

$$\begin{aligned} &P(\text{Time to next peak exceeding } w \leq t) = 1 - e^{[-\nu P(q > w)t]} \\ &= 1 - e^{[-EY(w)t]} \end{aligned} \tag{3.2.10}$$

This is recognised as an exponential distribution with parameter $\nu P(q > w)$ which is the expected number of peaks exceeding w per year.

ARR defines this parameter as EY(w) which stands for Exceedances per Year, but more strictly, is the expected number of peaks that exceed w in a year.

Linking AEP and EY

If we select a particular peak magnitude w, combining Equation (3.2.8), Equation (3.2.9) and Equation (3.2.10) yields the following relationship between EY(w) and AEP(w):

$$\begin{aligned} AEP(w) &= 1 - P(W \leq w | t = 1) \\ &= 1 - e^{[-\nu P(q > w)]} \\ &= 1 - e^{[-EY(w)]} \end{aligned} \quad (3.2.11)$$

If we express AEP(w) as 1/Y(w) then Equation (3.2.11) can be rewritten as:

$$\begin{aligned} EY(w) &= -\log_e[1 - AEP(w)] \\ &= -\log_e\left[1 - \frac{1}{Y(w)}\right] \end{aligned} \quad (3.2.12)$$

This relationship assumes peaks in the POT series are statistically independent and that there is no seasonality in the sense that the probability density of the POT peak above a threshold $p(q|q > q_0)$ does not change over the year. While the no-seasonality assumption appears questionable on first inspection, in practice the threshold q_0 is selected so that the expected number of peaks exceeding the threshold q_0 in any year is of the order of 1. This is done to ensure the POT peaks are genuine floods and statistically independent. As a consequence of the high threshold selected in practice, the impact of seasonality is diminished.

2.2.2.3. When to use Annual Maximum and Peak-Over-Threshold Series

The risk measures AEP and Exceedances per Year (EY) are intimately connected. The analysis presented in Theory of Peak-Over-Threshold and Annual Maximum Series shows that:

$$\begin{aligned} EY(w) &= -\log_e[1 - AEP(w)] \\ &= -\log_e\left[1 - \frac{1}{Y(w)}\right] \end{aligned} \quad (3.2.13)$$

where AEP(w) is expressed as the ratio 1 in Y(w). This relationship is plotted in Figure 3.2.2. For AEPs less than 10% (0.1 or 1 in 10 i.e. events rarer than 10% AEP), EY and AEP are numerically same, from a practical perspective. However, as the AEP increases beyond 10% (i.e. for events more frequent than 10% AEP), EY increases more rapidly than AEP. This occurs because in years with a large annual maximum peak, the smaller peaks of that year may exceed the annual maximum peak in other years.

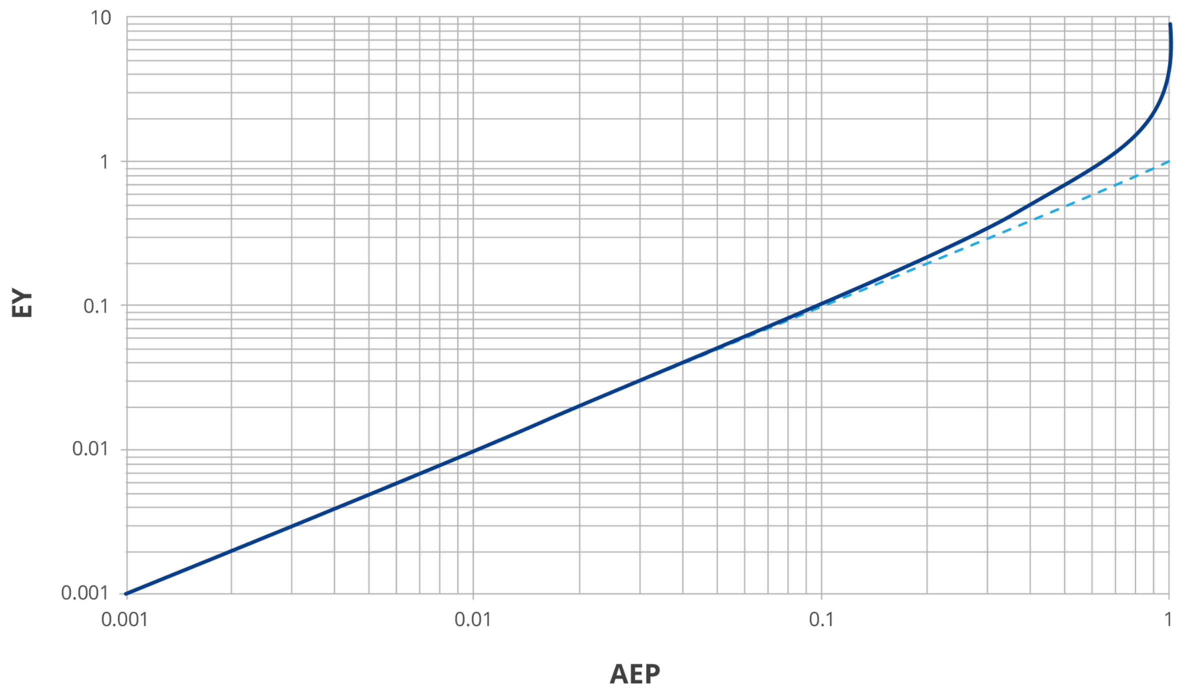


Figure 3.2.2. Annual Exceedance Probability (AEP) - Exceedances per Year (EY) Relationship

The question arises when should one use AM or POT approaches. Consistent with the guidelines provided in Book 1, the following guideline is offered:

i. AEP of interest < 10% (i.e. events rarer than 10% AEP)

   AEPs, in this range, are generally required for estimation of a design flood for a structure or works at a particular site. Use of AM series is preferred as it yields virtually identical answers to POT series in most cases, provides a more robust[3] estimate of low AEP floods and is easier to extract and define.

ii. EY of interest > 0.2 events per year (i.e. events more frequent than 0.2 EY)

   Use of a POT series is generally preferred because all floods are of interest in this range, whether they are the highest in the particular year of record or not. The AM series may omit many floods of interest. The POT series is appropriate for estimating design floods with a relatively high EY in urban stormwater contexts and for diversion works, coffer dams and other temporary structures. However, in practice, flow records are not often available at sites where minor works with a design EY greater than 0.1 events per year is required.

2.2.3. Advantages and Disadvantages of Flood Frequency Analysis

Practitioners need to be aware of the advantages and disadvantages of Flood Frequency Analysis.

[3]In a POT series, there are typically 2 to 3 times more peaks than in the corresponding AM series. The additional peaks are largely smaller peaks. In the case when the data is not well fitted by the chosen probability model, the fit to the upper part of the distribution may be compromised in order to obtain a good fit to the smaller peaks where the bulk of the data lies.

Flood peaks are the product of a complex joint probability process involving the interaction of many random variables associated with the rainfall event, antecedent conditions and the rainfall-runoff transformation. Peak flood records represent the integrated response of the storm event with the catchment. They provide a direct measure of flood exceedance probabilities. As a result, Flood Frequency Analysis is not subject to the potential for bias, possibly large, that can affect alternative methods based on design rainfall (Kuczera et al., 2006).

Other advantages of Flood Frequency Analysis include its comparative simplicity and capacity to quantify uncertainty arising from limited information.

Offsetting these significant advantages are several disadvantages:

- The true probability distribution family is unknown. Unfortunately, different models can fit the flood data with similar capability, yet can diverge in the right hand tail when extrapolated beyond the data.
- Short records may compromise the utility of flood estimates. Confidence limits inform the practitioner about the credibility of the estimate.
- It may be difficult or impossible to adjust the data if the catchment conditions under which the flood data were obtained have changed during the period of record, or are different to those applying to the future economic life of a structure or works being designed.

Considerable extrapolation of rating curves is necessary to convert recorded stage to discharge for the largest flood peaks at most Australian gauging stations. In addition, the probability of malfunction of recording instruments is increased during major floods. Suspect floods and the years in which they occurred may be omitted in analysis of Annual Maximum series, but this reduces the sample size and may introduce bias if the suspect floods are all major events. These problems are inherent to the calibration of all methods employing major flood peaks. At this stage it is not clear whether Flood Frequency Analysis is more sensitive to such problems than other methods.

2.2.4. Range of Application

As noted the true flood probability family, M, is unknown. In practice, the choice of model is guided by goodness of fit to data. Therefore, use of the fitted frequency curve for AEPs reflected in the data is regarded as an interpolation exercise deemed reliable in the sense that confidence limits capture the uncertainty. However, when the frequency curve is extrapolated well beyond the observed data, confidence limits which quantify the effect of sampling variability on parameter uncertainty may underestimate the true uncertainty - model bias may be significant and even dominant. Book 3, Chapter 2, Section 8 demonstrates the need to understand the processes affecting flood peaks beyond the observed record and illustrates the pitfall of blind extrapolation.

Large extrapolation of a flood frequency curve is not recommended. It is acknowledged that prescribing strict limits on the minimum AEP does not have a strong conceptual foundation. The limits to extrapolation should be guided by consideration of confidence limits, which are affected by the information content of the data and choice of flood model, and by judgments about model bias which cannot be quantified. In situations where the analyst is prepared to make the judgment that the processes operating in the range of the observed record continue to operate for larger floods, model bias may be deemed to be manageable – of course the effects of sampling uncertainty may be so amplified under significant extrapolation to render the frequency estimate of little value.

2.3. Selection and Preparation of Data

2.3.1. Requirements of Data for Valid Analysis

For a valid frequency analysis, the data used should constitute a random sample of independent values, ideally from a homogeneous population. Streamflow data are collected as a continuous record, and discrete values must be extracted from this record as the events to be analysed. The problem of assessing independence of events, and of selecting all independent events, is illustrated by the streamflow record for a 1000 km^2 catchment in Figure 3.2.3. It is clear that peaks A and B are not independent of each other but are serially correlated, while peak D is independent of A and B. However, the independence of peak C in regards to A and B is open to question, as it is difficult to determine the independent peaks in the record - B and D, or B, C and D. Methods for selecting the peaks included in the analysis are described in the following subsections.

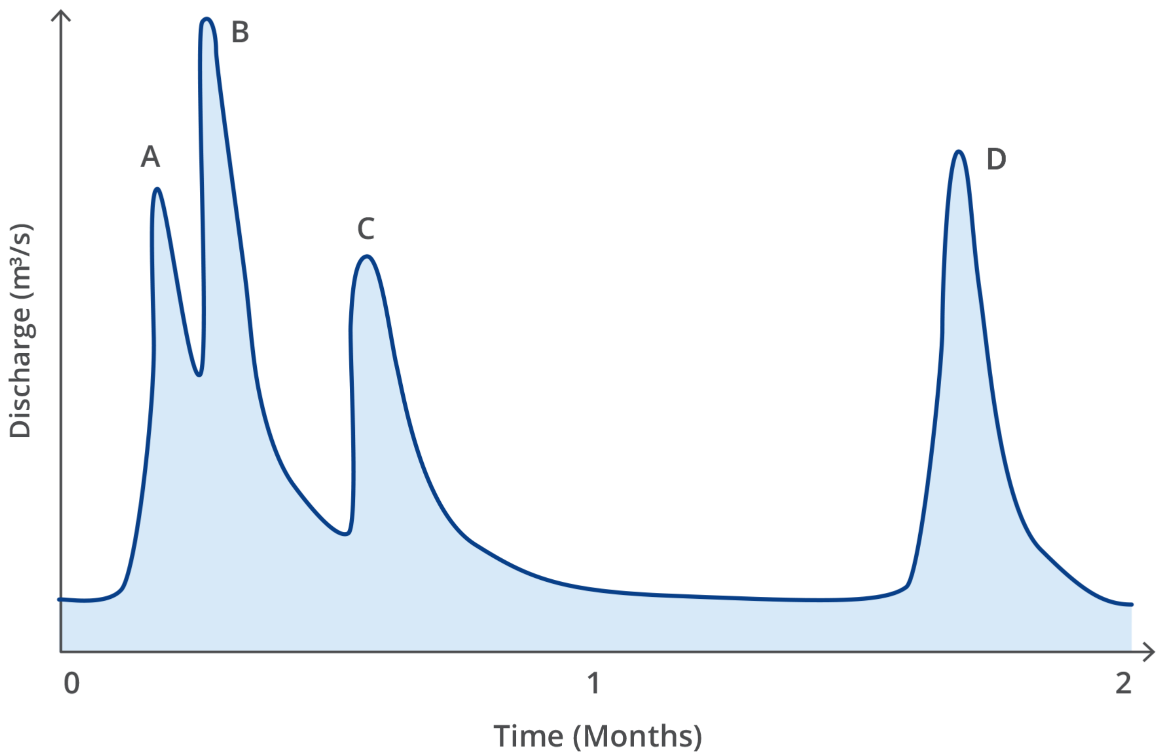


Figure 3.2.3. Hydrograph for a 1000 km^2 Catchment Illustrating Difficulty of Assessing Independence of Floods

Lack of homogeneity of the population of floods is another practical problem, especially if the data sample from the past is used to derive flood estimates applicable to the design life of the structure or works in future. Examples of changes in collection of data or in the nature of the catchment that lead to lack of homogeneity are:

1. Inability to allow for change of station rating curve, for example, resulting from insufficient high-stage gauging;

2. Change of gauging station site;

3. Construction of large storages, levees and channel improvements;

4. Growth in the number of farm dams on the catchment; and
5. Changes in land use such as clearing, different farming practices, soil conservation works, re-forestation, and urbanisation.

The record should be carefully examined for these and other causes of lack of homogeneity. In some cases, recorded values can be adjusted by means such as routing pre-dam floods through the storage to adjust them to equivalent present values, correcting rating errors wherever possible, or making some adjustment for urbanisation. Such decisions must be made largely by judgement. As with all methods of flood estimation, it is important that likely conditions during the design life are considered, instead of existing conditions at the time of design. Some arbitrary adjustment of derived values for likely changes in the catchment may be possible, but the recorded data must generally be accepted for analysis and design. Fortunately, the available evidence indicates that unless changes to the catchment involve large proportions of the total area or large changes in the storage on the catchment, the effects on flood magnitudes are likely to be low. In addition, the effects are likely to be larger for frequent floods than for the rare floods that are of primary interest in design.

2.3.2. Types of Flood Data

In the most general sense, flood peak data can be classified as either being gauged or censored.

2.3.2.1. Gauged Data

Gauged data consists of a time series of flood discharge estimates. Such estimates are based on observed peak (or instantaneous) stages (or water levels). A rating curve is used to transform stage observations to discharge estimates. When extrapolated, the rating curve can introduce large systematic errors into discharge estimates.

It is important to check how the peak discharges were obtained from the gauged record. Peak discharges may be derived from daily readings, possibly with some intermediate readings during some floods, for part of the record, and continuous readings from the remainder of the record. If part of the record consists of daily readings, it is necessary to assess whether daily readings adequately approximate the instantaneous peak discharge (refer to Book 3, Chapter 2, Section 8 for instances of adequate and inadequate approximations). If the daily reading is deemed as an unreliable estimate of the peak discharge during that day, the reading need not be discarded but treated as a censored discharge.

2.3.2.2. Censored Data

Censored data consists of a time series of indicator values defined as:

$$I_t(q) = \begin{Bmatrix} 1 \text{ if } \mathrm{t}^{th} \text{ flood peak} > \text{threshold } q \\ -1 \text{ if } \mathrm{t}^{th} \text{ flood peak } \leq \text{ threshold } q \end{Bmatrix} \tag{3.2.14}$$

They arise in a number of ways. For example, prior to gauging, water level records may be kept only for rare floods above some perception threshold. Therefore, all we may know is that there were n_a flood peaks above the threshold and n_b peaks below the threshold. Sometimes, frequent floods below a certain threshold may be deliberately excluded, since the overall fit gets unduly influenced by small floods.

Figure 3.2.4 presents a graphical depiction of gauged and censored time series data. In the first part of the record, all the peaks are below a threshold, while in the second part, daily readings define a lower threshold for the peak. Finally, in the third part, continuous gauging yields instantaneous peaks.

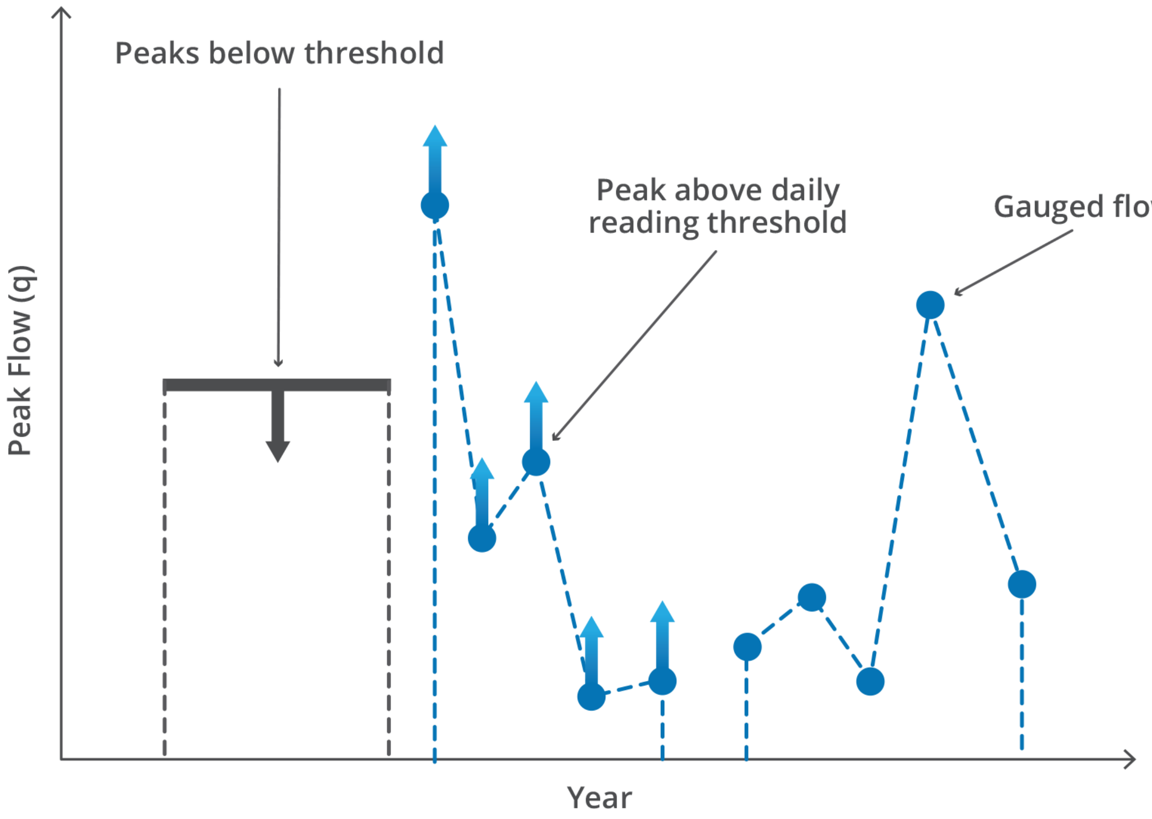


Figure 3.2.4. Depiction of Censored and Gauged Flow Time Series Data

2.3.3. Annual Maximum Flood Gauged Series

This is the most common method of selecting the floods to be analysed. The series comprised of the highest instantaneous rate of discharge in each year of record. The year may either be a calendar year or a water year, the latter usually commencing at the end of the period of lowest average flow during the year. Where flows are highly seasonal, especially with a wet summer, use of the water year is preferable. The highest flow in each year is selected, whether it is a major flood or not, and all other floods are neglected, even though some will be much larger than the maximum discharges selected from some other years. For *n* years of data, the annual flood series will consist of *n* values.

The Annual Maximum series has at least two advantages:

1. As the individual annual maximum discharges are likely to be separated by considerable intervals of time, it is probable that the values will be independent. Checking the dates of the annual maxima to ensure that they are likely to be independent, is a simple procedure that should be followed. If the highest annual value occurred at the start of a year and was judged to be dependent on the annual maximum at the end of the previous year, the lower of these two values should be discarded and the second highest discharge in that year substituted.

2. The series is easily and unambiguously extracted. Most data collection agencies have annual maxima on computer file and/or hard copy.

2.3.4. Peak-Over-Threshold Gauged Series

A POT flood series consists of all floods with peak discharges above a selected base value, regardless of the number of such floods occurring each year. The POT series is also referred to as the partial duration series or basic stage series. The number of floods *m* generally will be different to the number of years of record *n*, and will depend on the selected base discharge. ASCE (1949) recommended that the base discharge should be selected so that *m* is greater than *n*, but that there should not be more than 3 or 4 floods above the base in any one year. These two requirements can be incompatible. The U.S. Geological Survey (Dalrymple, 1960) recommended that *m* should equal 3*n*. If a probability distribution is to be fitted to the POT series, the desirable base discharge and average number of floods per year selected depend on the type of distribution. These distributions are discussed further in Book 3, Chapter 2, Section 4. For the compound model using a Poisson distribution of occurrences and an exponential distribution of magnitudes, Tavares and Da Silva (1983), and Jayasuriya and Mein (1985) found that *m* should equal 2*n* or greater, and the U.K Flood Studies Report (Natural Environment Research Council, 1975) recommended that m should equal 3*n* to 5*n*. For fitting the Log Pearson III (LP III) distribution, the values of the moments depend on the number of floods selected and the base discharge. McDermott and Pilgrim (1982) and Jayasuriya and Mein (1985) found that best results were obtained in this case when *m* equalled *n*.

An important advantage of the POT series is that when the selected base value is sufficiently high, small events that are not really floods are excluded. With the AM series, non-floods in dry years may have an undue influence on shape of the distribution. This is particularly important for Australia, where both the range of flows and the non-occurrence of floods are greater than in many other countries such as the United States and the United Kingdom. For this reason it would also be expected that the desirable ratio of *m* to *n* would be lower in Australia than in these countries (refer to Book 3, Chapter 2, Section 3).

A criterion for independence of successive peaks must also be applied in selecting events. As discussed by Laurenson (1987), statistical independence requires physical independence of the causative factors of the flood, mainly rainfall and antecedent wetness. This type of independence is necessary if the POT series is used to estimate the distribution of annual floods. On the other hand, selection of POT series floods for design flood studies should consider the consequences of the flood peaks in assessing independence of events where damages or financial penalties are the most important design variables. Factors to be considered might include duration of inundation and the time required to repair flood damage. In both cases, the size or response time of the catchment will have some effect.

The decision regarding a criterion for independence, therefore requires subjective judgement by the practitioner, designer or analyst in each case. There is often conflict that some flood effects are short-lived, perhaps only as long as inundation, while others, such as the destruction of an annual crop, may last as long as a year. It is thus not possible to recommend a simple and clear-cut criterion for independence. The circumstances and objectives of each study, and the characteristics of the catchment and flood data, should be considered in each case before a criterion is adopted. It is inevitable that the adopted criterion will be arbitrary to some extent.

While no specific criterion can be recommended, it may be helpful to consider some criteria that were used in past studies:

- Bulletin 17B of the Interagency Advisory Committee on Water Data (1982) states that no general criterion can be recommended and the decision should be based on the intended use in each case, as discussed above. However, in Appendix 14 of that document, a study by Beard (1974) is summarised where the criterion is that it should use independent flood peaks should be separated by five days plus the natural logarithm of the square miles of drainage area, with the additional requirement that intermediate discharges must drop to below 75% of the lower of the two separate flood peaks. This may only be suitable for catchments larger than 1000 km^2. Jayasuriya and Mein (1985) used this criterion.

- The UK Flood Studies Report (Natural Environment Research Council, 1975) used a criterion that flood peaks should be separated by three times the time to peak and that the flow should decrease between peaks to two-thirds of the first peak.

- McIllwraith (1953), in developing design rainfall data for flood estimation, used the following criteria, based on the rainfall causing the floods:

  - For rainfalls of short duration up to two hours, only the one highest flood within a period of 24 hours.

  - For longer rainfalls, a period of 24 hours in which no more than 5 mm of rain could occur between rain causing separate flood events.

- In a study of small catchments, Potter and Pilgrim (1971) used a criterion of three calendar days between separate flood events but lesser events could occur in the intervening period. This was the most satisfactory of five criteria tested on data from seven small catchments located throughout eastern New South Wales. It also gave the closest approximation to the above criteria used by McIllwraith (1953).

- Pilgrim and Doran (1987) and McDermott and Pilgrim (1983) adopted monthly maximum peak flows to give an effective criterion of independence in developing a design procedure for small to medium sized catchments. This was based primarily on the assumption that little additional damage would be caused by floods occurring within a month, and thus closer floods would not be independent in terms of their effects. This criterion was also used by Adams and McMahon (1985) and Adams (1987).

The criteria cited above represent a wide range and illustrate the difficult and subjective nature of the choice. It is stressed that these criteria have been described for illustrative purposes only. In each particular application the practitioner, designer or analyst should choose a criterion suitable to the analysis and relevant to all of the circumstances and objectives.

2.3.5. Monthly and Seasonal Gauged Series

In some circumstances, series other than the AM or POT series may be used. The monthly and seasonal series are the most useful.

Maximum monthly flows are an approximation to the POT series in most parts of Australia, as the probability of two large independent floods occurring in the same month is low. Tropical northern Australia, the west coast of Tasmania and the south-west of Western Australia may be exceptions. It should be noted that not every monthly maximum flood will be selected, but only those large enough to exceed a selected base discharge, as is the case for the POT series. The monthly series has two important advantages over the POT series, which it approximates:

1. It is more easily extracted, as most gauging authorities have monthly maximum discharges on file.

2. It can be argued that a flood occurring within a month of a previous large flood is of little concern in design, as repairs will not have been undertaken and little additional damage will result.

With the monthly series, care is required to check any floods selected in successive months for independence. Where the dates are close, the lower value should be discarded. The second highest flood in that month could then be checked from the records, but this would generally not be worthwhile. An example of use of the monthly series is described by Pilgrim and McDermott (1982).

Seasonal flood frequencies are sometimes required. For these cases, the data are selected for the particular month or season as for the annual series, and the flood frequency analysis is carried out in a similar fashion to that for the annual series.

2.3.6. Extension of Gauged Records

It may sometimes be possible to extend the recorded data by values estimated from longer records on adjacent catchments, by use of a catchment rainfall-runoff model, or by use of historical data from before the commencement of records. If this can be done validly, the effective sample size of the data will be increased and the reliability of the analysis will be greater. However, care is necessary to ensure that the extended data is valid and real information has been added. Several procedures can be used and are outlined in the following sections:

- Regression Relationship with Data from an Adjacent Catchment
- Use of a Catchment Rainfall-Runoff Model
- Station-Year Method

2.3.6.1. Regression Relationship with Data from an Adjacent Catchment

If a regression of flood peaks for the study catchment on peaks for an adjacent catchment can be established for the period of concurrent record, the relation can be used to estimate values for the study catchment for a longer period, when records are only available on the adjacent catchment. The data should first be plotted on linear and log-log scales. A regression equation can then be fitted to the values or alternatively, the graphical relation can be used directly with a smooth curve fitted by eye.

The principal shortcoming of the regression approach is that uncertainty in the transfer process is ignored resulting in an overstatement of information content. To guard against this, an approximate criterion for deciding whether the regression should be used is that the correlation coefficient of the relation should exceed 0.85 (Fiering, 1963; Matalas and Jacobs, 1964). More rigorous criteria are discussed in ARR 1987 Book 3 Section 2.6.5.

Care is needed when annual floods are used. The dates of the corresponding annual floods on the adjacent catchments should be compared. Not infrequently, the dates are different, resulting in a lack of physical basis for the relation. Although relationships of this type seem to have been used in some regional flood frequency procedures, it is recommended that regressions should only be used when the corresponding floods result from the same storm. This problem is discussed further by Potter and Pilgrim (1971).

When floods resulting from the same storm on adjacent catchments are plotted against each other, there is often a large scatter. Frequently, a large flood occurs on one catchment but only a small flood occurs on the other. The scatter is generally greater than for the physically unrealistic relation using floods which are the maximum annual values on the two catchments but which may have occurred on different dates. The resulting relation using floods that occurred in the same storm is often so weak that it should not be used to extend records.

Wang (2001) describes a Bayesian approach that rigorously makes allowance for the noise in the transfer process. This approach is considered superior to the traditional regression transfer.

2.3.6.2. Use of a Catchment Rainfall-Runoff Model

A catchment rainfall-runoff model can range from a simple rainfall-runoff regression to a catchment modelling system that simulates either continuous runoff hydrographs or single event hydrograph from rainfall data. This discussion relates primarily to the latter type of model. The calibration of such a model for a period with concurrent rainfall and runoff records and its subsequent use to extend streamflow records for the period when rainfall data are available, while an attractive approach, should only be used with great caution. Appreciable differences often occur between observed and modelled runoff, especially in periods not used in calibration and in periods with runoff not represented in the calibration. Estimation of model parameters involves considerable uncertainty. Greatest accuracy in modelling can be expected in calculating discharges around the mean value, and larger errors are likely in extreme values such as the large flood peaks required for frequency analysis. Overall, the use of catchment models to extend flood records should be adopted with caution.

2.3.6.3. Station-Year Method

This method is included only to warn against its shortcomings. In this procedure, records from several adjacent catchments are joined "end-to-end" to give a single record equal in length to the sum of the lengths of the constituent records. As discussed by Clarke-Hafstead (1942) for rainfall data, spatial correlation between the records of the adjacent stations invalidates the procedure.

2.3.7. Rating Curve Error in Gauged Discharges

Though it is widely accepted that discharge estimates for large floods can be in considerable error, there is limited published information on these errors and how they can be allowed for in a Flood Frequency Analysis. Rating error can arise from a number of mechanisms:

1. For large floods the rating curve typically is extrapolated or fitted to indirect discharge estimates. This can introduce a systematic but unknown bias.

2. If the gauging station is located at a site with an unstable cross-section the rating curve may shift causing a systematic but unknown bias.

The conceptual model of rating error presented in this section is based on Kuczera (1999) and is considered to be rudimentary and subject to refinement. It is assumed the cross-section is stable with the primary source of rating error arising from extension of the rating curve to large floods.

Potter and Walker (1981) and Potter and Walker (1985) observe that flood discharge is inferred from a rating curve which is subject to discontinuous measurement error. Consider Figure 3.2.5 which depicts a rating curve with two regions having different error characteristics. The interpolation zone consists of that part of the rating curve well defined by discharge-stage measurements; typically the error Coefficient of Variation (CV) would be small, say 1 to 5%. In the extension zone the rating curve is extended by methods such as slope-conveyance, log-log extrapolation or fitting to indirect discharge estimates. Typically such extensions are smooth and, therefore, can induce systematic under- or over-estimation of the true discharge over a range of stages. The extension error CV is not well known but (Potter and Walker, 1981; Potter and Walker, 1985) suggest it may be as high as 30%.

Figure 3.2.5 and Figure 3.2.6 illustrate two cases of smooth rating curve extension wherein systematic error is introduced. In Figure 3.2.5, the estimate was below the true discharge. In the absence of any other information the rating curve is extended to pass smoothly through this point thereby introducing a systematic underestimate of large flood discharges. Even if more than one indirect discharge estimate were available, it is likely the errors will be correlated because the same biases in estimating Manning's n, conveyance and friction slope would be present

In Figure 3.2.6 the rating curve is extended using the slope-conveyance method. The method relies on extrapolating gauged estimates of the friction slope so that the friction slope asymptotes to a constant value. Depending on how well the approach to asymptotic conditions is defined by the data considerable systematic error in extrapolation may occur. Perhaps of greater concern is the assumption that Manning's n and conveyance can be reliably estimated in the overbank flow regime particularly when there are strong contrasts in roughness along the wetted perimeter.

Though Figure 3.2.5 represents an idealisation of actual rating curve extension two points of practical significance are noted:

1. The error is systematic in the sense that the extended rating curve is likely to diverge from the true rating curve as discharge increases. The error, therefore, is likely to be highly correlated- in fact, it is perfectly correlated in the idealisation of Figure 3.2.5.

2. The interpolation zone anchors the error in the extension zone. Therefore, the error in the extension zone depends on the distance from the anchor point and not from the origin. This error is termed incremental because it originates from the anchor point rather than the origin of the rating curve.

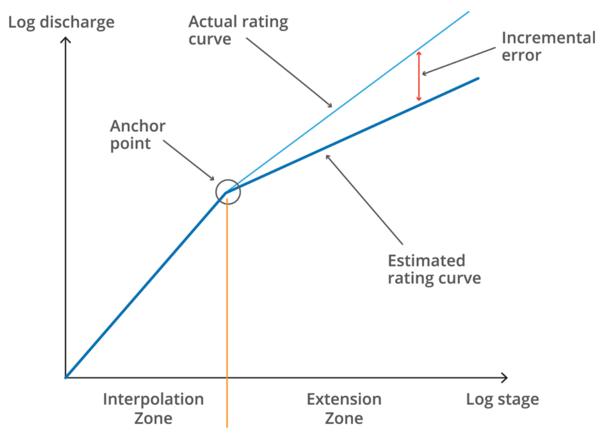


Figure 3.2.5. Rating Curve Extension by Fitting to an Indirect Discharge Estimate

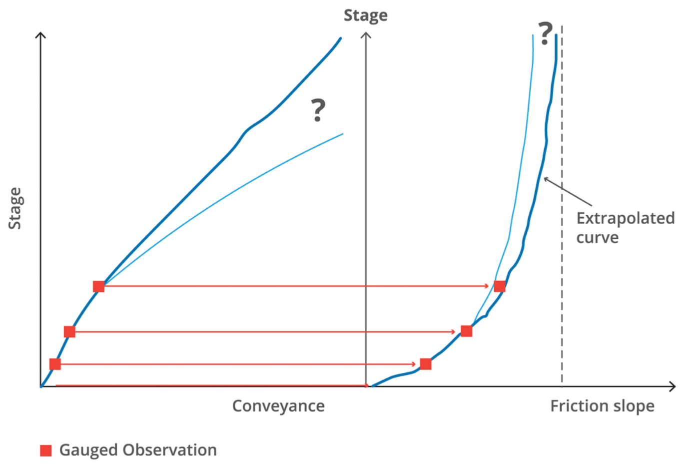


Figure 3.2.6. Rating Curve Extension by Slope-Conveyance Method

2.3.8. Historical and Paleo Flood Information

A flood may have occurred before the period of gauged record and known to be the largest flood, or flood of other known rank, over a period longer than that of the gauged record. Such floods can provide valuable information and should be included in the analysis if possible.

Care is needed in assessing historical floods. Only stages are usually available, and these may be determined by flood marks recorded on buildings or structures, by old newspaper reports, or from verbal evidence. Newspaper or other photographs can provide valuable information. Verbal evidence is often untrustworthy, and structures may have been moved. A further problem is that the channel morphology, and hence the stage-discharge relation of the stream, may have changed from those applying during the period of gauged record.

It is desirable to carry out Flood Frequency Analyses both by including and excluding the historical data. The analysis including the historical data should be used unless in the comparison of the two analyses, the magnitudes of the observed peaks, uncertainty regarding the accuracy of the historical peaks, or other factors, suggest that the historical peaks are not indicative of the extended period or are not accurate. All decisions made should be thoroughly documented.

Considerable work has been carried out in the United States on the assessment of paleofloods. These are major floods that have occurred outside the historical record, but which are evidenced by geological, geomorphological or botanical information. Techniques of paleohydrology have been described by Costa (1978), Costa (1983), Costa (1986) and Kochel et al. (1982) and more recently by O'Connell et al. (2002), and a succinct summary is given by Stedinger and Cohn (1986). Although high accuracy is not possible with these estimates, they may only be marginally less accurate than other estimates requiring extrapolation of rating curves, and they have the potential for greatly extending the database and providing valuable information on the tail of the underlying flood distribution. A procedure for assessing the value of paleoflood estimates of Flood Frequency Analysis is given by Hosking and Wallis (1986). Only a little work on this topic has been carried out in Australia, but its potential has been indicated by its use to identify the five largest floods in the last 700 years in the Finke River Gorge in central Australia (Baker et al., 1983; Baker, 1984), and for more frequent floods, by identification of the six largest floods that occurred since a major flood in 1897 on the Katherine River in the Northern Territory (Baker, 1984). While the use of paleoflood data should be considered, it needs to be recognized that there are not many sites where paleofloods can be estimated and that climate changes may have affected the homogeneity of long-term flood data.

2.3.9. Data Characterising Long-Term Climate Persistence

There is growing evidence that flood peaks are not identically distributed from year to year in some parts of Australia and that flood risk is dependent on long-term climate variability. The idea of alternating flood and drought dominated regimes that exist on decadal and longer timescales was first proposed by Erskine and Warner (1988). More recently, analyses of changes in climate state affecting flood risk have been published (refer to Franks and Kuczera (2002), Franks (2002a), and Franks (2002b)). The climate-dependence of flood risk is an important consideration when assessing flood risk. Most flood frequency applications will require assessment of long-term flood risk; that is, flood risk that is independent of a particular current climate state. If a flood record is sufficiently long to sample all climate states affecting flood risk, a traditional analysis assuming homogeneity will yield the long-term flood risk. Unfortunately many flood records are relatively short and may be dominated by one climate state. Blind use of such data can result in substantial bias in long-term flood risk estimates. For this reason it may be necessary to obtain climate index data which characterizes long-term persistence in climate and to investigate the homogeneity of the flood distribution.

A number of known climate phenomena impact on Australian climate variability. Most well known is the inter-annual El Nino/Southern Oscillation (ENSO). The cold ENSO phase, La Nina, results in a marked increase in flood risk across Eastern Australia, whereas El Nino years are typically without large floods (Kiem et al., 2003).

There is also mounting evidence that longer-term climate processes also have a major impact on flood risk. The Interdecadal Pacific Oscillation (IPO) is a low frequency climate process related to the variable epochs of warming and cooling in the Pacific Ocean and is described by an index derived from low pass filtering of Sea Surface Temperature (SST) anomalies in the Pacific Ocean (Power et al., 1998; Power et al., 1999; Allan, 2000). The IPO is similar to the Pacific Decadal Oscillation (PDO) of Mantua et al (1997), which is defined as the leading principal component of North Pacific monthly sea surface temperature variability.

The IPO time series from 1870 is displayed in Figure 3.2.7. It reveals extended periods where the index either lies below or above zero. Power et al. (1999) have shown that the association between ENSO and Australian climate is modulated by the IPO- a strong association was found between the magnitude of ENSO impacts during negative IPO phases, whilst positive IPO phases showed a weaker, less predictable relationship. Additionally, Kiem et al. (2003) and Kiem and Franks (2004) analysed New South Wales flood and drought data and demonstrated that the IPO negative state magnified the impact of La Nina events. Moreover, they demonstrated that the IPO negative phase, related to mid-latitude Pacific Ocean cooling, appears to result in an increased frequency of cold La Nina events. The net effect of the dual modulation of ENSO by IPO is the occurrence of multi-decadal periods of elevated and reduced flood risk. To place this in context, Figure 3.2.8 shows regional flood index curves based on about 40 NSW sites for the different IPO states (Kiem et al., 2003) – the 1% AEP flood during years with a positive IPO index corresponds to the 1 in 6 AEP flood during years with a negative IPO index. Micevski et al. (2003) investigating a range of sites in NSW found that floods occurring during IPO negative periods were, on average, about 1.8 times bigger than floods with the same frequency during IPO positive periods.

A key area of current research is the spatial variability of ENSO and IPO impacts. The associations between ENSO, IPO and eastern Australian climate have been investigated from a mechanistic approach. Folland et al. (2002) showed that ENSO and IPO both affect the location of the South Pacific Convergence Zone (SPCZ) providing a mechanistic justification for the role of La Nina and IPO negative periods in enhancing flood risk in eastern Australia.

Whilst the work to date has primarily focused on eastern Australia, a substantial step change in climate also occurred in Western Australia around the mid-1970's, in line with the IPO and PDO indices (Franks, 2002b), however the role of ENSO is less clear and is likely to be additionally complicated by the role of the Indian Ocean.

The finding that flood risk in parts of Australia is modulated by low frequency climate variability is recent. Practitioners are reminded that this is an area of active research and therefore should keep abreast of future developments.

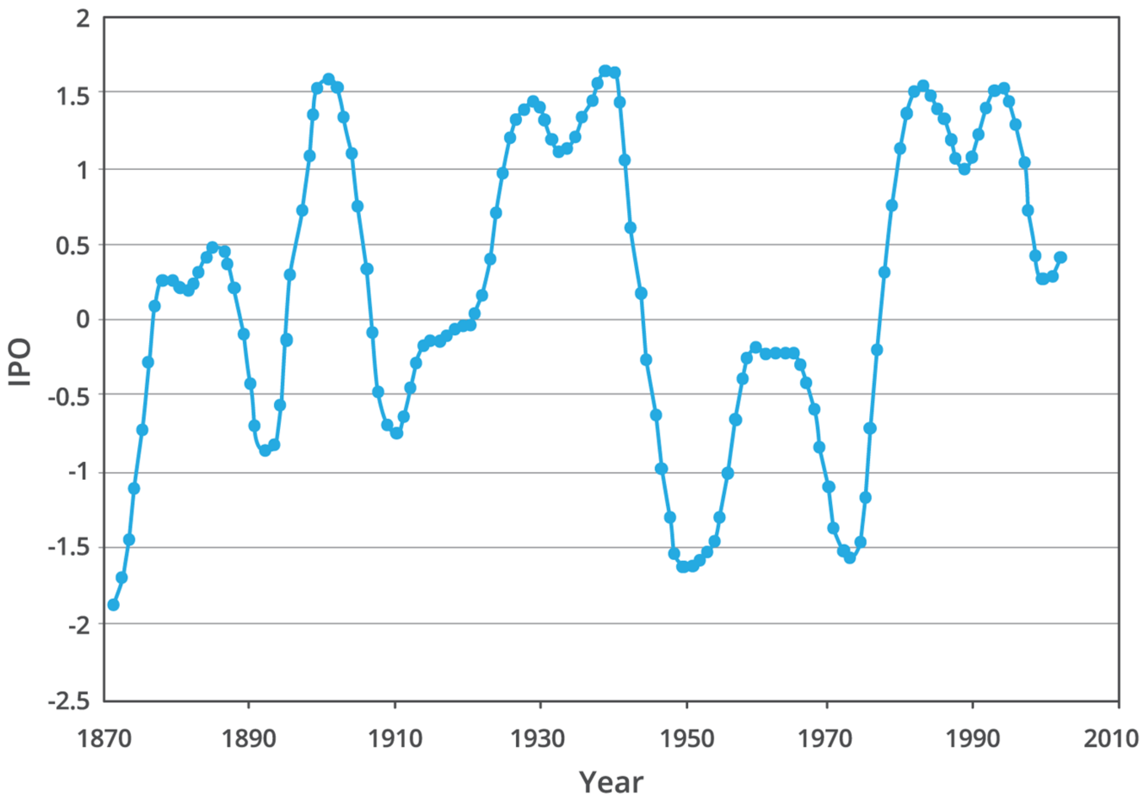


Figure 3.2.7. Annual Average Interdecadal Pacific Oscillation Time Series

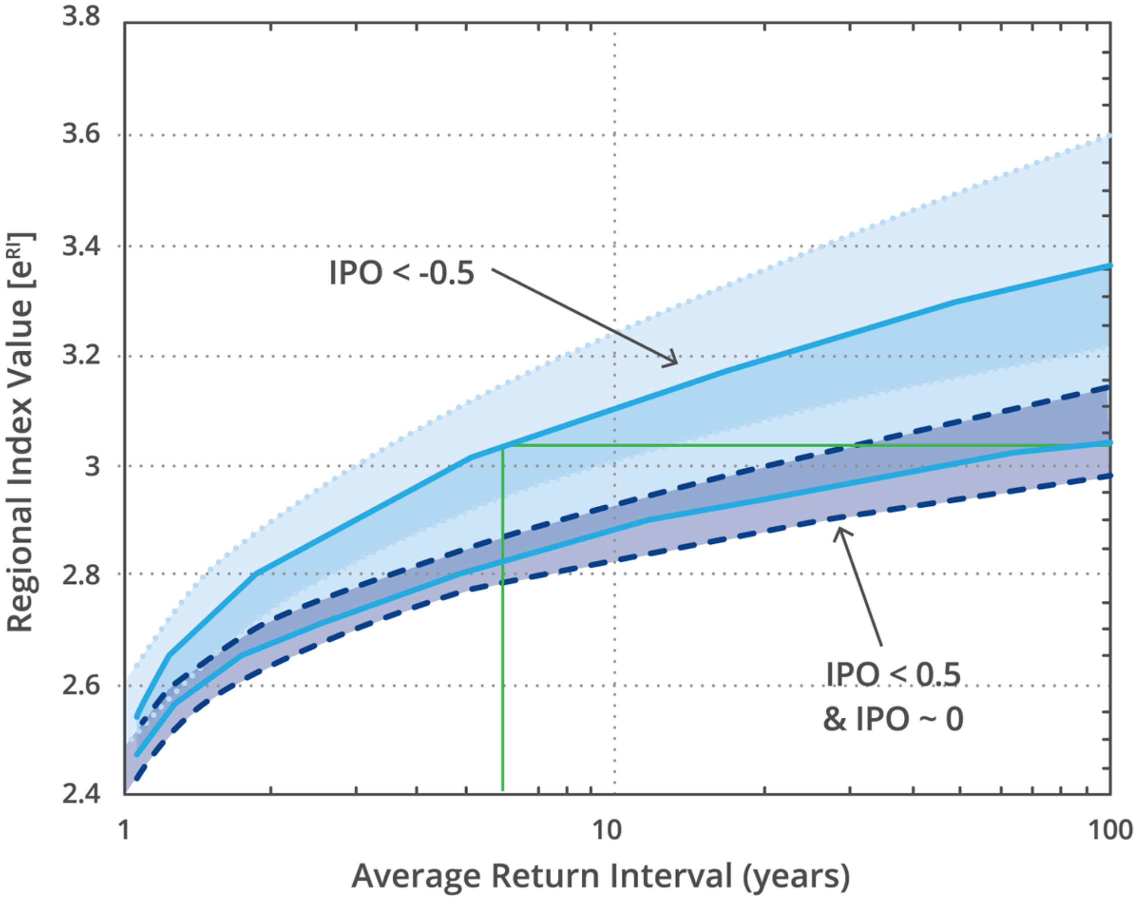


Figure 3.2.8. NSW Regional Flood Index Frequency Curves for Positive and Negative Interdecadal Pacific Oscillation epochs (Kiem et al., 2003)

2.3.10. Regional Flood Information

Whereas the primary focus of this chapter is Flood Frequency Analysis using at-site information, the accuracy of the frequency analysis can be improved, substantially in some cases, by augmenting at-site information with regional information. Subsequent chapters in this Book describe methods for estimating flood frequency at ungauged sites. Provided such methods also provide estimates of uncertainty, the regional information can be pooled with the at-site information to yield more accurate results. Book 3, Chapter 2, Section 6 shows how regional information on flood probability model parameters can pooled with at-site information. When pooling at-site and regional information it is important to establish that both sources of information are consistent – that is, they yield statistically consistent results.

2.3.11. Missing Records

Streamflow data frequently contain gaps for a variety of reasons including the malfunction of recording equipment. Rainfall records on the catchment and streamflow data from nearby catchments may indicate the likelihood of a large flood having occurred during the gap. A regression may be able to be derived to enable a missing flood to be estimated, but as discussed in Book 3, Chapter 2, Section 3, the degree of correlation is often insufficient for a quantitative estimate.

For AM series the missing record period is of no consequence and can be included in the period of record, if it can be determined that the largest discharge for the year occurred outside the gap, or that no large rainfall occurred during the gap. However the rainfall records and streamflow on nearby catchments might indicate that a large flood could have occurred during the period of missing record. If a regression with good correlation can be derived from concurrent records, the missing flood can be estimated and used as the annual flood for the year. If the flood cannot be estimated with reasonable certainty, the whole year should be excluded from the analysis.

For POT series data, treatment of missing records is less clear. McDermott and Pilgrim (1982) tested seven methods, leading to the following recommendations based on the assumption that the periods of missing data are random occurrences and are independent of the occurrence of flood peaks.

1. Where a nearby station record exists covering the missing record period, and a good relation between the flood peaks on the two catchments can be obtained, then use this relation and the nearby station record to fill in the missing events of interest.

2. Where a nearby station record exists covering the missing record period, and the relation between the flood peaks on the two catchments is such that only the occurrence of an event can be predicted but not its magnitude, then:

   - For record lengths less than 20 years, ignore the missing data and include the missing period in the overall period of record;

   - For record lengths greater than 20 years, subtract an amount from each year with missing data proportional to the ratio of the number of peaks missed to the total number of ranked peaks in the year.

3. Where no nearby station record exists covering the missing record period, or where no relation between flood peaks on the catchment exists, then ignore the missing data and include the missing record period in the overall period of record.

2.4. Choice of Flood Probability Model

2.4.1. General

As noted in Book 3, Chapter 2, Section 2, it is assumed that each flood peak in an AM or POT series is statistically independent of other flood peaks in the series. In addition, the flood probability model, described by its probability density function (pdf) $p(q|\theta)$, must be specified.

There is no universally accepted flood probability model. Many types of probability distributions have been applied to Flood Frequency Analysis. Unfortunately, it is not possible to determine the true form of distribution (for example, Cunnane (1985)), and there is no rigorous analytical proof that any particular probability distribution for floods is the correct theoretical distribution. The appropriateness of these distributions can be tested by examining the fit of each distribution to observed flood data. Various empirical tests of different distributions have been carried out with recorded data from many catchments, however, conclusive evidence is not possible largely because gauged records are of insufficient length to eliminate the confounding effect of sampling variability. Examples of sampling experiments illustrating this problem are given by Alexander (1957) and Dalrymple (1960). The choice of flood probability model is further exacerbated by recent evidence that in certain parts of Australia, the flood record is not homogeneous due to variations in long-term climate controls.

Given these considerations, it is inappropriate to be prescriptive with regard to choice of flood probability model. As a general rule, the selected probability distribution family should be consistent with available data. It is recognised that more than one probability distribution family may be consistent with the data. One approach to deal with this problem is to select the distribution family on the basis of best overall fit to a range of catchments within a region or landscape space – L-moment diagrams offer a useful tool for judging overall goodness of fit (refer to Stedinger et al. (1993), Section 18.3.3 for more details).

2.4.2. Choice of Distribution Family for Annual Maximum Series

Two distribution families are suggested as reasonable initial choices for AM series, namely the Generalized Extreme Value (GEV) and Log Pearson III (LP III) families. These families fit most AM flood data adequately. Nonetheless, the practitioner is reminded that there is no rigorous justification for these families, which is particularly important when extrapolating – Book 3, Chapter 2, Section 8 demonstrates the importance of understanding the mechanisms controlling flood response. The following sections describe the GEV and LP III distributions and some other distributions that may be more appropriate in certain circumstances.

2.4.2.1. Generalized Extreme Value (GEV) Distribution

Table 3.2.1 lists the pdf $p(q|\theta)$ distribution function $P(Q \leq q|\theta)$ and product moments for the GEV distribution. It has three parameters: τ, the location parameter, α the scale parameter and κ the shape parameter. When the shape parameter κ equals 0, the GEV simplifies to the Gumbel distribution, whose details are also presented in Table 3.2.1. For positive values of κ there exists an upper bound, while for negative κ, there exists a lower bound. The 1 in Y AEP quantile q_Y is given by:

$$q_Y = \begin{cases} \tau + \dfrac{\alpha}{\kappa}\left[1 - \left(-\log_e\left(1 - \dfrac{1}{Y}\right)\right)^{\kappa}\right], \kappa \neq 0 \\ \tau - \alpha\log_e\left(-\log_e\left(1 - \dfrac{1}{Y}\right)\right), \kappa = 0 \end{cases} \tag{3.2.15}$$

Table 3.2.1. Selected Homogeneous Probability Models Families for use in Flood Frequency Analysis

Family	Distribution	Moments
Generalized Extreme Value (GEV)	$p(q\|\theta) = \frac{1}{\alpha} e^{\left\{-\left[1-\frac{\kappa(q-\tau)}{\alpha}\right]^{\frac{1}{\kappa}}\right\}} \left[1-\frac{\kappa(q-\tau)}{\alpha}\right]^{\frac{1}{\kappa}-1}$ $P(Q \leq q\|\theta) = e^{\left\{-\left[1-\frac{\kappa(q-\tau)}{\alpha}\right]^{\frac{1}{\kappa}}\right\}}$ $when \quad \kappa > 0, q < \tau + \frac{\alpha}{\kappa};$ $when \quad \kappa < 0, q > \tau + \frac{\alpha}{\kappa}$	$Mean(q) = \tau + \frac{\alpha}{\kappa}[1-\Gamma(1+\kappa)]$ $for\ \kappa > -1$ $Variance(q) = \frac{\alpha^2}{\kappa^2}\left[\Gamma(1+2\kappa) - [\Gamma(1+\kappa)]^2\right]$ $for\ \kappa > -\frac{1}{2}$ $where\ \Gamma(\ )$ is the gamma function
Gumbel	$p(q\|\theta) = \frac{1}{\alpha} e^{-\frac{(q-\tau)}{\alpha}} e^{-e^{-\frac{(q-\tau)}{\alpha}}}$ $P(Q \leq q\|\theta) = e^{-e^{-\frac{(q-\tau)}{\alpha}}}$	$\mathrm{Mean}(q) = \tau + 0.5772\alpha$ $\mathrm{Variance}(q) = \frac{\pi^2\alpha^2}{6}$ $\mathrm{Skew}(q) = 1.1396$
Log Pearson III (LP III)	$p(q\|\theta) = \frac{\|\beta\|}{q\Gamma(\alpha)} [\beta(\log_e q - \tau)]^{\alpha-1} \mathrm{e}^{-\beta(\log_e q - \tau)}$ $\alpha > 0$ when $\beta > 0, \log_e q > \tau;$ when $\beta < 0, \log_e q < \tau$	$\mathrm{Mean}(\log_e q) = m = \tau + \frac{\alpha}{\beta}$ $\mathrm{Variance}(\log_e q) = s^2 = \frac{\alpha}{\beta^2}$ $\mathrm{Skew}(\log_e q) = g = \begin{cases} \frac{2}{\sqrt{\alpha}} \text{ if } \beta > 0 \\ -\frac{2}{\sqrt{\alpha}} \text{ if } \beta < 0 \end{cases}$
log-Normal	$p(q\|\theta) = \frac{1}{q\sqrt{2\pi s^2}} e^{\left[-\frac{1}{2s^2}(\log_e q - m)^2\right]}$ $q > 0, s > 0$	$\mathrm{Mean}(\log_e q) = \mathrm{m}$ $\mathrm{Variance}(\log_e q) = s^2$
Generalized Pareto	$p(q\|\theta) = \frac{1}{\beta}\left(1-\frac{\kappa(q-q_*)}{\beta}\right)^{\frac{1}{\kappa}-1}$ $P(Q \leq q\|\theta) = 1-\left(1-\frac{\kappa(q-q_*)}{\beta}\right)^{\frac{1}{\kappa}}$ $when \quad \kappa < 0, q_* \leq q < \infty;$ $when \quad \kappa > 0, q_* \leq q \leq \frac{\beta}{\kappa}$	$Mean(q) = q_* + \frac{\beta}{1+\kappa}$ $Variance(q) = \frac{\beta^2}{(1+\kappa)^2(1+2\kappa)}$ $Skew(q) = \frac{2(1-\kappa)(1+2\kappa)^{\frac{1}{2}}}{(1+3\kappa)} . \kappa > -1/3$
Exponential	$p(q\|\theta) = \frac{1}{\beta} e^{\left(-\frac{q-q_*}{\beta}\right)}$ $P(Q \leq q\|\theta) = 1 - e^{\left(-\frac{q-q_*}{\beta}\right)}, q \geq q_*$	$\mathrm{Mean}(q) = q_* + \beta$ $\mathrm{Variance}(q) = \beta^2$ $\mathrm{Skew}(q) = 2$

Of the widely used distribution families, the GEV distribution has the strongest theoretical appeal as it is the asymptotic distribution of extreme values for a wide range of underlying parent distributions. In the context of flood frequency, suppose there are N flood peaks in a year. Provided N is large and the flood peaks are identically and independently distributed, the distribution of the largest peak discharge in the year approaches the GEV under quite general conditions. However, it is questionable whether these assumptions are satisfied in practice.

The number of independent flood peaks in any year may not be sufficient to ensure asymptotic behaviour particularly for catchments that tend to aridity. Moreover, in strongly seasonal climates, it is unlikely that the within-year independent flood peaks are random realisations from the same probability distribution.

The GEV has gained widespread acceptance (for example, Natural Environment Research Council (1975), Wallis and Wood (1985), and (Stedinger et al., 1993)).

2.4.2.2. Log Pearson III Distribution

Table 3.2.1 lists the pdf $p(q|\theta)$ and product moments for Log Pearson III (LP III) distribution. It has three parameters: τ, the location parameter, α the scale parameter and β the shape parameter. When the skew of log q is zero, the distribution simplifies to the log-Normal, whose details are provided in Table 3.2.1.

The LP III distribution is widely accepted in practice as it consistently fits flood data as well, if not better than other probability families. It has performed best of those that have been tested on data for Australian catchments (Conway, 1970; Kopittke et al., 1976; McMahon, 1979; Mcmahon and Srikanthan, 1981). It is the recommended distribution for the United States in Bulletin 17B of the Interagency Advisory Committee on Water Data (1982).

The distribution, however, is not well-behaved from an inference perspective. Direct inference of the parameters α, β and τ can cause numerical problems. For example, when the skew of log q is close to zero, the shape parameter α tends to infinity. Experience indicates it is preferable to fit the first three moments of log q rather than α, β and τ. Note that τ is a lower bound for positive skew and an upper bound for negative skew.

A problem arises when the absolute value of the skew of log q exceeds 2; that is, when $\alpha \geq 1$. When $\alpha < 1$, the LP III has a gamma-shaped density. However, when $\alpha \geq 1$, the density changes to a J-shaped function. Indeed when $\alpha = 1$, the pdf degenerates to that of an exponential distribution with scale parameter β and location parameter τ. For $\alpha \geq 1$, the J-shaped density seems to be over-parameterised with three parameters. In such circumstances, it is pointless to use the LP III. It is suggested that either the GEV or the Generalized Pareto (GP) Distributions should be used as a substitute.

An analytical form of the distribution function is not available for the LP III and log-Normal distributions. To compute the quantile q_Y (that is, the discharge with a 1 in Y AEP) the following equation may be used:

$$\log(q_Y) = m + K_Y(g)s \tag{3.2.16}$$

where m, s and g are the mean, standard deviation and skewness of the log discharge and K_Y is a frequency factor well-approximated by the Wilson-Hilferty transformation:

$$K_Y(g) = \begin{cases} \frac{2}{g}\left[\left\{\frac{g}{6}\left(Z_Y - \frac{g}{6}\right) + 1\right\}^3 - 1\right] \text{ if } |g| > 0 \\ 0 \text{ if } g = 0 \end{cases} \tag{3.2.17}$$

for |g| < 2 and AEPs ranging from 99% to 1% AEP. The term Z_Y is the frequency factor for the standard normal distribution which has a mean of zero and standard deviation of 1; Z_Y is the value of the standard normal deviate with exceedance probability $^1/_Y$. Table 3.2.2 lists Z_Y for selected exceedance probabilities. Comprehensive tables of K_Y can be found in (Pilgrim, 1987).

Table 3.2.2. Frequency factors for standard normal distribution.

AEP (%)	Y in AEP of 1 in Y	Z_Y
50	2	0.0000
20	5	0.8416
10	10	1.2816
5	20	1.6449
2	50	2.0537
1	100	2.3263
0.5	200	2.5758
0.2	500	2.8782
0.1	1000	3.0902

2.4.2.3. Generalized Pareto Distribution

In POT modelling, the GP distribution is often found to satisfactorily fit the data. Table 3.2.1 lists the pdf $p(q|\theta)$, distribution function $P(Q \leq q|\theta)$ and product moments for the GP distribution. It has three parameters: q_*, the location parameter, β, the scale parameter and κ, the shape parameter. When κ equals zero, the distribution simplifies to the exponential distribution, also described in Table 3.2.1.

$$q_Y = \begin{cases} q_* + \frac{\beta}{\kappa}\left[1 - \left(\frac{1}{Y}\right)^{\kappa}\right], \kappa \neq 0 \\ q_* - \beta \log_e\left(\frac{1}{Y}\right), \kappa = 0 \end{cases} \tag{3.2.18}$$

The GP distribution has an intimate relationship with the GEV. If the GP describes the distribution of peaks over a threshold, then for Poisson arrivals of the POT peaks with ν being the average number of arrivals per year, it can be shown that the distribution of Annual Maximum peaks is GEV with shape parameter κ, scale parameter and location parameter:

$$\alpha = \beta \nu^{-\kappa} \tag{3.2.19}$$

and location parameter

$$\tau = q_* + \frac{\beta}{\kappa} \tag{3.2.20}$$

2.4.2.4. Zero-threshold mixture model

In certain parts of Australia, the AM flood series may contain one or more years of zero or virtually zero flow. In such cases, the flood probability model is best described by a two-component mixture model. In any year, there are two possibilities:

1. There is a (fixed) probability P_0 that the peak flow equals the zero-threshold flow q_0, which may be zero or a near-zero flow; and

2. There is a probability 1- P_0 that the peak flow exceeds the threshold. In that case the distribution of the peak flow follows a standard probability model.

A formal definition of this model can be found in Formal Definition of Zero-Threshold Mixture Distribution.

Formal Definition of Zero-Threshold Mixture Distribution

The zero-threshold mixture model has a distribution function:

$$P(Q \leq q|\theta) = \begin{cases} P_0 \text{ if } q = q_0 \\ P_0 + (1 - P_0)\dfrac{P(Q \leq q|\theta) - P(Q \leq q_0|\theta)}{P(Q > q_0|\theta)} \text{ if } q > q_0 \end{cases} \tag{3.2.21}$$

where q_0 is the zero-threshold flow, P_0 is the probability of the AM peak equaling q_0 and $P(Q \leq q|\theta)$ is a probability model such as described in Table 3.2.1.

The pdf of the mixture model can be expressed using the generalized probability density which allows the random variable to take discrete values as well as continuous values:

$$p(q|\theta) = \begin{cases} P_0 \text{ if } q = q_0 \\ \dfrac{1 - P_0}{P(Q > q_0|\theta)} p(q|\theta) \text{ if } q > q_0 \end{cases} \tag{3.2.22}$$

2.4.2.5. Multi-Component or Mixture Models

In some areas, flooding may be affected by different types of meteorological events (for example, intense tropical cyclones and storms characterised by more usual synoptic conditions) or by changing hydraulic controls (e.g., see Book 3, Chapter 2, Section 8), causing abnormal slope changes in the frequency curve. In such cases, the GEV or LP III families may not adequately fit the data. This type of problem may be of particular importance in northern Australia, and an example is described by Ashkanasy and Weeks (1975). It may be desirable to separate the data by cause and analyse each set separately. The above reference describes a procedure using a combination of two log-Normal distributions. A two-component extreme value distribution has been described by Rossi et al. (1984) and Fiorentino et al. (1985), and developed and applied to U.K. data by Beran et al. (1986). Alternatively, the four or five parameter Wakeby distribution may be used to fit such data (Houghton, 1978).

2.4.2.6. Non-Homogeneous Models

If the evidence suggests that flood risk is affected by multi-decadal climate persistence, the use of non-homogeneous probability models may need to be investigated. The concern is that ignoring the non-homogeneity of the flood record may lead to biased estimates of long-term flood risk.

If a non-homogeneous probability model with pdf $p(q|\theta(x))$ is identified, this cannot be used to estimate long-term or marginal flood risk. This is because flood risk is dependent on the exogeneous variables x. To estimate long-term flood risk, the dependence on x must be removed using the total probability rule to give:

$$P(Q \leq q) = \int_{x} \left(\int_{0}^{q} p(z \mid \theta(x)) dz \right) p(x) dx \tag{3.2.23}$$

where p(x) is the pdf of the exogenous variables.

If the gauged record adequately samples the distribution of x, it is not necessary to identify a non-homogeneous model. It suffices to fit a probability model to all the record to estimate the long-term flood risk.

However, if the gauged record does not adequately sample x, significant bias in flood risk may result if only at-site data are used. In such instances, it will be necessary to employ regional frequency methods that take the non-homogeneity into account. This is an area of current research. Practitioners are advised to keep abreast of new developments.

Book 3, Chapter 2, Section 8 illustrates the impact on flood risk arising from multi-decadal persistence in climate state as represented by the IPO index. It illustrates how the exogenous variable x can be constructed, demonstrates the serious bias in flood risk that can arise if short records do not adequately sample different climate states and illustrates the use of Equation (3.2.23).

2.4.3. Choice of Distribution for Peak-Over-Threshold Series

In some cases, it may be desirable to analytically fit a probability distribution to POT data. Distributions that have been used to describe the flood peak above a threshold include the exponential, GP and LP III.

2.5. Choice of Flood Quantile Estimator

This section considers the following question: Given data D, what is the best estimate of the 1 in Y AEP flood discharge.

If the true value of θ were known, then the pdf $p(q|\theta)$ can be used to compute the flood quantile $q_Y(\theta)$. For AM series the 1 in Y AEP quantile is defined as:

$$P(Q \leq q_Y \left| \theta \right) = \frac{1}{Y} = \int_{q_Y}^{\infty} p(q \left| \theta \right) dq \tag{3.2.24}$$

However, in practice the true value of θ (as well as the distribution family) is unknown. All that is known about θ, given the data D, is summarized by a probability distribution with pdf $p(\theta|D)$. Book 3, Chapter 2, Section 6 describes how this distribution may be obtained – this distribution is the posterior distribution if performing a Bayesian analysis or the sampling distribution if performing a bootstrap analysis.

If the true value of θ is not known, it follows that the true value of the quantile $q_Y(\theta)$ is not known. The uncertainty about θ described by the pdf $p(\theta|D)$, translates into uncertainty about the quantile, described by the quantile predictive pdf $p(q_Y|D)$. The question then arises, which value from the quantile predictive pdf $p(q_Y|D)$should be adopted as the flood quantile estimate? This section presents two approaches for determining the best estimate of a flood discharge with a 1 in Y AEP knowing the pdf $p(\theta|D)$.

2.5.1. Expected Parameter Quantiles

In general the estimation of a design quantile should be guided by the consequences of under or over-design (Slack et al., 1975). This section considers the case where the consequence of over- and under-design is expressed by some measure of the difference between the true and estimated 1 in Y AEP quantiles – this difference is called the quantile error.

The loss function $L[q_Y(D), q_Y(\theta)]$ describes the loss or consequence when the true quantile $q_Y(\theta)$, which depends on θ, is incorrectly estimated by $q_Y(D)$, which depends on the data D. Because the true value of θ is uncertain, the best estimator $q_Y(D)_{opt}$ is the one that minimises the expected loss:

$$q_Y(D)_{opt} \leftarrow \min \int_\theta L[q_Y(D), q_Y(\theta)] p(\theta|D) d\theta \tag{3.2.25}$$

The optimal quantile estimator depends on the choice of loss function.

2.5.1.1. Quadratic loss

The quadratic loss function may be appropriate when the consequences of under or over-design are judged to be same and loss is proportional to the square of the quantile error. The loss function is expressed as:

$$L[q_Y(D), q_Y(\theta)] \propto [q_Y(D) - q_Y(\theta)]^2 \tag{3.2.26}$$

The expected value of this loss function is referred to as the Mean Squared Error (MSE), which shows that the optimal quantile estimator is the expected value of $q_Y(D)$ (DeGroot, 1970):

$$E[q_Y|D] = \int_\theta q_Y(\theta) p(\theta|D) d\theta \tag{3.2.27}$$

Stedinger (1983) observes that this integral may not exist for some of the probability distributions used in Flood Frequency Analysis. To avoid this problem the following first-order approximation may be used:

$$E[q_Y|D] = q_Y[E(\theta|D)] \tag{3.2.28}$$

where $E(\theta\,|\,D)$ is the expected parameter given the data D:

$$E[\theta\,|\,D] = \int \theta p(\theta\,|\,D) d\theta \tag{3.2.29}$$

The term $q_Y\,[\mathrm{E}(\theta|\mathrm{D})]$ is referred to as the expected parameter 1 in Y AEP quantile. When this term is used it is understood that the quantile is computed with θ assigned $[E(\theta\,|\,D)]$.

2.5.1.2. Linear asymmetric loss

The linear asymmetric loss function may be appropriate when the consequences of under and over-design are judged to be different. The loss function is expressed as:

$$L[q_Y(D),\mathrm{q}_Y(\theta)] \;= \begin{cases} \alpha(q_Y(\theta) - q_Y(D)) & \text{if } q_Y(\theta) > q_Y(D), \text{ underdesign} \\ \beta(q_Y(D) - q_Y(\theta)) & \text{if } q_Y(\theta) \le q_Y(D), \text{ overdesign} \end{cases} \tag{3.2.30}$$

where α and β are the loss coefficients for under and over-design respectively. It can be shown that the quantile estimator that minimises the expected asymmetric linear loss must satisfy the following (DeGroot, 1970):

$$P\left(Q_Y \le q_Y(D)_{opt}\,\middle|\,D\right) = \frac{\alpha}{\alpha + \beta} \tag{3.2.31}$$

When α equals $\beta, q_Y(D)_{opt}$, it is the median of the quantile predictive distribution. However, when α equals 4β, it implies that the consequences of under-design are four times more severe than over-design, which is the 80-percentile of the predictive distribution, a far more conservative estimate than the median.

2.5.2. Expected AEP Quantiles

This section considers a different perspective on selecting a quantile estimator. In the previous section the uncertainty in the quantile for a given AEP was considered. In this section the uncertainty in AEP for a given q is considered.

Stedinger (1983) showed that the dependence of the flood peak pdf on uncertain parameters can be removed using total probability to yield the design flood distribution:

$$p(q|D) = \int_\theta p(q|\theta) p(\theta|D) d\theta \tag{3.2.32}$$

This distribution only depends on the data D (and the assumed probability family).

The design flood quantile q_Y with X% AEP can be derived by solving:

$$p(q > q_Y|D) = \int_\theta \left(\int_{q_Y}^{\infty} p(q|\theta) dq \right) p(\theta|D) d\theta = \frac{1}{Y} \tag{3.2.33}$$

This quantile is greater than the expected parameter X% AEP quantile $q_Y\,[\mathrm{E}(\theta|\mathrm{D})]$. To gain further insight suppose we make q_Y equal to the expected parameter X% AEP quantile and compute its design flood AEP using Equation (3.2.33). Noting the inner integral is the probability of the flood peak exceeding q_Y given a particular value of θ , Equation (3.2.33) can be rewritten as:

$$p(q > q_Y|D) = \int_{\theta} \left(\int_{q_Y}^{\infty} p(q|\theta)dq \right) p(\theta|D)d\theta$$
$$= \int_{\theta} p(q > q_Y|D)p(\theta|D)d\theta \quad (3.2.34)$$
$$= \frac{1}{Y}$$

It thus follows that the expected parameter 1 in Y AEP quantile $q_Y[E(\theta \mid D)]$. Moreover, it follows that the expected parameter 1 in Y AEP quantile $q_Y[E(\theta \mid D)]$ has an expected AEP given by $P(q > q_Y \mid D)$ which exceeds $1/_Y$.

2.5.3. Selection of Flood Quantile Estimator

The choice of flood quantile estimator depends on whether the design is being carried out for many sites or a single site, on whether risk, actual discharge or average annual damage is of primary interest in design and on whether the consequences of under and over-design are different. The choice is somewhat subjective and may be a matter of policy. As a general guide, a non-exhaustive list of recommendations is given below:

1. In situations where there is indifference to the consequences of under or over-design, either the expected parameter or expected AEP quantile can be chosen.

   Expected parameter quantiles may be applicable where:

   i. Sizing of a structure for a given AEP is the primary consideration and minimizing quadratic loss, as expressed by Equation (3.2.26), is a reasonable criterion.

   ii. Unbiased estimates of annual flood damages are required (Doran and Irish, 1980).

   Expected AEP quantiles may be applicable where:

   i. Design is required for many sites in a region, and the objective is to attain a desired AEP over all sites.

   ii. Probability of exceedance is of primary importance for design at a single site (such as in floodplain management).

2. If there are significant differences between the consequences of under and over-design, consideration should be given to using Equation (3.2.31) as the flood quantile estimator.

2.6. Fitting Flood Probability Models to Annual Maxima Series

2.6.1. Overview of Methods

Fitting a flood probability model involves three steps:

1. Calibrating the model to the available data D to determine the parameter values consistent with the data D.

2. Estimation of flood quantiles and their confidence limits.

3. Evaluation of goodness of fit and consistency of model with data.

Two calibration approaches are described involving Bayesian and L-moment techniques. For each approach, the algorithms are documented and illustrated with worked examples. Implementation of the algorithms in software requires specialist skill; therefore, a typical practitioner is advised to make use of the available software. The use of the method of product-moments applied to log flows is not recommended. The choice of calibration methods depends on the type of data available (gauged and censored), the extent of measurement error associated with the rating curve and the availability of regional information about parameters.

2.6.2. Probability Plots

An essential part of a Flood Frequency Analysis is the construction of an empirical distribution function, better known as a probability plot. In such a plot, an estimate of AEP is plotted against the observed discharge. This enables one to draw a smooth curve as an empirical probability distribution or to visually check the adequacy of a fitted distribution.

The following steps describe the production of a probability plot for gauged Annual Maximum floods:

- Rank the gauged discharges in descending order (that is, from largest to smallest) yielding the series $\{q_{(1)}, q_{(2)},\ldots,q_{(n)}\}$ where $q_{(i)}$ is the rank i or the ith largest flood;
- Estimate the AEP for each $q_{(i)}$ using a suitable plotting position; and
- Using suitable scales plot the estimated AEP against $q_{(i)}$.

For analysis of the AM series, a general formula (Blom, 1958) for estimating the AEP of an observed flood is:

$$P_{(i)} = \frac{\mathrm{i} - \alpha}{n + 1 - 2\alpha} \tag{3.2.35}$$

where i is the rank of the gauged flood, n is the number of years of gauged floods and α is a constant whose value is selected to preserve desirable statistical properties.

There are several choices for α:

- $\alpha = 0$ yields the Weibull plotting position that produces unbiased estimates of the AEP of $q_{(i)}$;
- $\alpha = 0.375$ yields the Blom's plotting position that produces unbiased quantile estimates for the normal distribution; and
- $\alpha = 0.4$ yields the Cunnane (1978) plotting position that produces nearly unbiased quantile estimates for a range of probability families.

While there are arguments in favour of plotting positions that yield unbiased AEPs, usage has favoured plotting positions that yield unbiased quantiles. To maintain consistency, it is recommended that the Cunnane plotting position is used, namely:

$$P_{(i)} = \frac{i - 0.4}{n + 0.2} \tag{3.2.36}$$

A more complete discussion on plotting positions can be found in Stedinger et al. (1993).

It is stressed that plotting positions should not be used as an estimate of the actual AEP or EY of an observed flood discharge. Such estimates should be obtained from the fitted distribution.

Judicious choice of scale for the probability plot can assist the evaluation of goodness of fit. The basic idea is to select a scale so that the data plot as a straight line if the data is consistent with the assumed probability model.

This is best illustrated by an example. Suppose that floods follow an exponential distribution, then from Table 3.2.1, the distribution function is:

$$P(Q \le q) = 1 - e^{-\frac{q - q_*}{\beta}} \tag{3.2.37}$$

Replacing q by $q_{(i)}$ and $1 - P(Q \le q)$ by the plotting position of $q_{(i)}$ gives:

$$1 - P_{(i)} = 1 - e^{-\frac{q_{(i)} - q_*}{\beta}} \tag{3.2.38}$$

Making $q_{(i)}$ the subject of the equation yields

$$q_{(i)} = q_* - \beta \log_e P_{(i)} \tag{3.2.39}$$

If $q_{(i)}$ is plotted against $\log_e P_{(i)}$, the data will plot approximately as a straight line if they are consistent with the exponential distribution.

Examples for other distributions include:

- For the Gumbel distribution plot $q_{(i)}$ against $-\log\left[-\log\left(1 - P_{(i)}\right)\right]$. Data following a GEV distribution will plot as a curved line.
- For the log normal distribution plot log $q_{(i)}$ against the standard normal deviate with exceedance probability $P_{(i)}$. Data following a LP III distribution will plot as a curved line.

When visually evaluating the goodness of fit care needs to be exercised in judging the significance of departures from the assumed distribution. Plotting positions are correlated. As a result, they do not scatter about the fitted distribution independently of each other. The correlation can induce “waves” or regions of systematic departure from the fitted distribution. To guard against this it is suggested that statistical tests be used to assist goodness-of-fit assessment. Stedinger et al. (1993) discuss the use of the Kolmogorov-Smirnov test, the Filiben probability plot correlation test and L-moment diagrams and ratio tests.

The estimation of plotting positions for censored and historic data is more involved and in some cases can be inaccurate refer to Stedinger et al. (1993) for more details.

2.6.3. Bayesian Calibration

2.6.3.1. Overview

The Bayesian approach is a very general approach for calibrating and identifying models. The Handbook of Hydrology Stedinger et al. (1993) observes that “the Bayesian approach... allows the explicit modeling of uncertainty in parameters and provides a theoretically consistent framework for integrating systematic flow records with regional and other hydrologic information”. However, it is only with the advent of new computational methods that Bayesian methods can be routinely applied to flood frequency applications.

The core of the Bayesian approach is described below – refer to Lee (1989) and Gelman et al. (1995) for general expositions. The data D is hypothesised to be a random realisation from a probability model with pdf $p(D|\theta)$ where θ is a vector of unknown parameters. The pdf $p(D|\theta)$ is given two labels depending on the context. When $p(D|\theta)$ is used to describe the probability model generating the sample data D for a given θ, it is called the sampling distribution. However, when inference about the parameter θ is sought, $p(D|\theta)$ is called the likelihood function to emphasise that the data D is known and the parameter θ is the object of attention. The same notation for the sampling distribution and likelihood function is used to emphasise its oneness.

In Bayesian inference, the parameter vector θ is considered to be a random vector whose probability distribution describes what is known about the true value of θ. Prior to analysing the data D, knowledge about θ, given the probability model, is summarised by the pdf $p(\theta)$. This density, referred to as the prior density, can incorporate subjective belief about θ.

Bayes theorem is then used to process the information contained in the data D by updating what is known about the true value of θ as follows:

$$p(D|\theta) = \frac{p(D|\theta)p(\theta)}{p(D|\theta)} \propto p(D|\theta)p(\theta) \qquad (3.2.40)$$

The posterior density $p(\theta|D)$ describes what is known about the true value of θ given the data D, prior information and the probability model. The denominator p(D) is the marginal likelihood defined as:

$$p(D) = \int p(D|\theta)p(\theta)d\theta \qquad (3.2.41)$$

Usually the marginal likelihood is not computed as it does not depend on θ and serves as a normalizing constant.

2.6.3.2. Likelihood Function

The key to a Bayesian analysis is the formulation of the likelihood function. In the context of flood frequency, analysis two formulations are considered. The first assumes there is no error in the flood data. The focus is on the contribution to the likelihood function made by gauged and censored data. The second case generalises the likelihood function to allow for error-in-discharge estimates.

2.6.3.3. Likelihood function: No-error-discharge Case

Suppose the following data are available:

1. A gauged record of n true flood peaks $\{q_1,...,q_n\}$; and
2. m censored records in which a_i annual flood peaks in $(a_i + b_i)$ ungauged years exceeded a threshold with true discharge s_i, i=1,..,m.

This data is denoted by D = $\{q_i$, i=1,..,n; (a_i, b_i, s_i), i=1,..,m$\}$. It is shown in Likelihood function: No-error-discharge case that the likelihood function is:

$$p(D\,|\,\theta) \propto \prod_{i=1}^{n} p(q_i\,|\,\theta) \prod_{i=1}^{m} \left[1 - P(Q \le s_i\,|\,\theta)\right]^{a_i} P(Q \le s_i\,|\,\theta)^{b_i} \quad (3.2.42)$$

Likelihood function: No-error-discharge case

The likelihood function is, by definition, the joint pdf of the observed given the parameter vector θ.

The likelihood function for the gauged data is the joint pdf of the n gauged floods. Given the AM flood peaks are statistically independent, the likelihood can be simplified to (Stedinger and Cohn, 1986):

$$p(q_1, ..., q_n|\theta) = \prod_{i=1}^{n} p(q_i|\theta) \quad (3.2.43)$$

The likelihood of the binomial censored data relies on the fact that the probability of observing exactly x exceedances in n years is given by the binomial distribution

$$P(x|n,\pi) = {}^{n}C_{x}(1-\pi)^{n-x}\pi^{x} \quad (3.2.44)$$

where π is the probability of an exceedance.

Provided each censoring threshold does not overlap over time with any other censoring threshold, the likelihood of the censored data becomes:

$$p(\text{censored data}|\theta) = \prod_{i=1}^{m} \left[1 - P(Q \le s_i|\theta)\right]^{a_i} P(Q \le s_i|\theta)^{b_i} = \prod_{i=1}^{m} P(a_i, b_i|s_i, \theta) \quad (3.2.45)$$

where $P(a_i, b_i|s_i, \theta)$ is the binomial probability of observing exactly a_i exceedances above the threshold discharge s_i in (a_i+b_i).

2.6.3.4. The Likelihood Function: Error-in-discharge Case

The incorporation of rating errors into the likelihood function complicates matters. Likelihood function: Error-in-discharge case outlines the derivation of the likelihood function using the simple rating error model presented in Book 3, Chapter 2, Section 3. There has been limited research into the use of this likelihood and on the characterisation of rating errors. Kuczera (1996) shows that, as rating error grows for floods well in excess of the largest gauged discharge, less weight is given to high floods in the calibration. This loss of information about the right tail of the flood distribution is sensitive to the magnitude of rating errors when extrapolating beyond gauged discharges. Unfortunately, little is known about these errors and hence caution is recommended if using this advanced likelihood.

Likelihood function: Error-in-discharge case

Figure 3.2.9 presents a rating error space diagram. In zone 1 (Figure 3.2.9), the interpolation zone it is assumed the rating error multiplier e_1 equals 1 – that is, errors within the rated part of the rating curve are deemed negligible. As a result the estimated discharge w equals the true discharge q. However, in zone 2, the extension zone, the rating error multiplier e_2 is assumed to be a random variable with mean of 1. The anchor point (q_1,w_1) separates the interpolation and extension zones. The rating error model can represented mathematically as:

$$w = \begin{cases} q \text{ if } q \leq q_1 \\ w_1 + e_2(q - q_1) \text{ if } q > q_1 \end{cases} \quad (3.2.46)$$

The rating error multiplier e_2 is sampled only once at the time of extending the rating curve. Therefore, all flood discharge estimates exceeding the anchor value of q_1 (which equals w_1) are corrupted by the same rating error multiplier. It must be stressed that the error e_2 is not known – at best, only its probability distribution can be estimated. For practical applications one can assume e_2 is distributed as either a log-Normal or normal distribution with mean 1 and standard deviation σ_2.

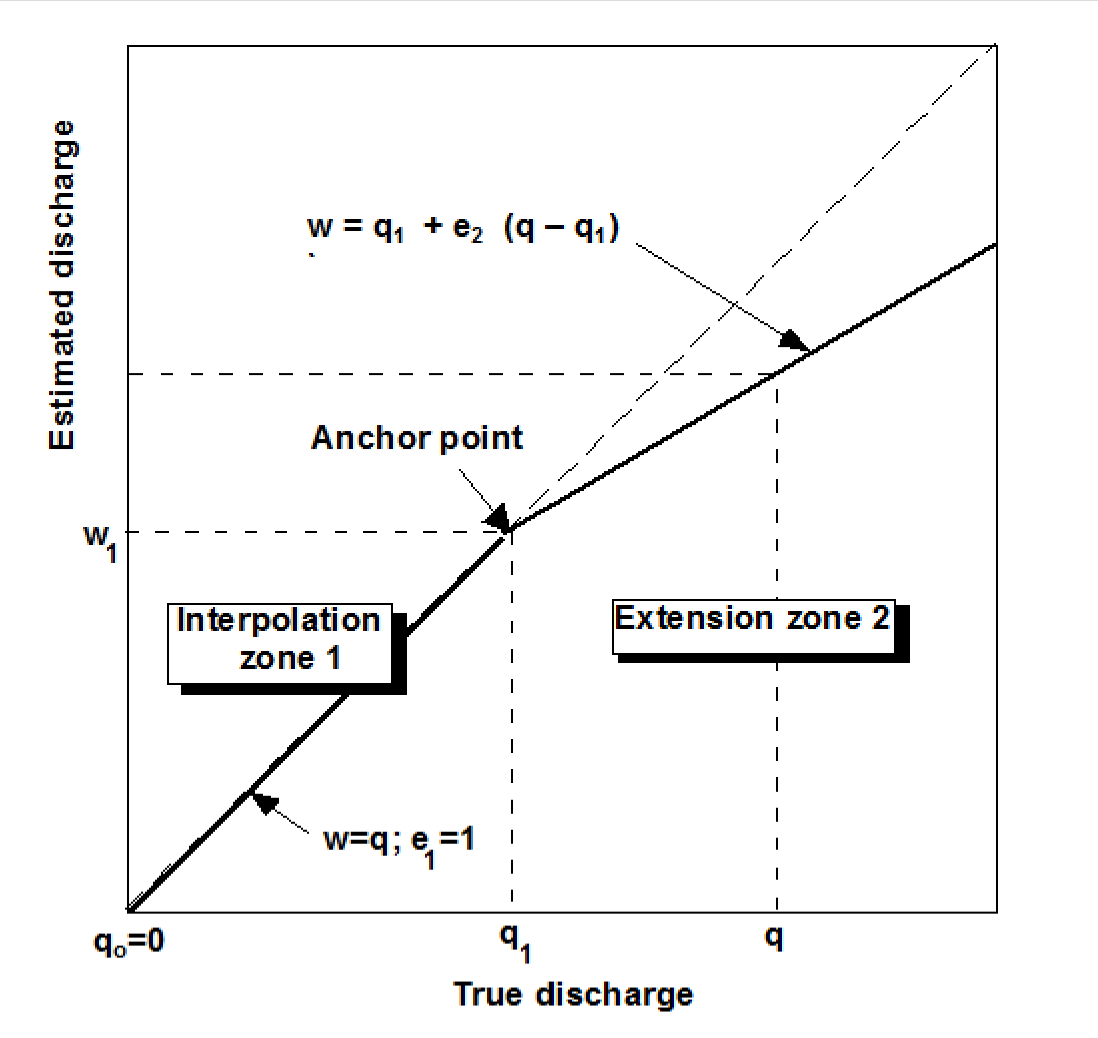


Figure 3.2.9. Rating error multiplier space diagram for rating curve

Data are assigned to each of the two zones, i=1,2, in the rating error space diagram. The rating error multiplier standard deviation for the extension zone σ_2 is assigned a value with $\sigma_1 = 0$. There are n_i annual flood peak estimates w_{ji} satisfying the zone constraint $w_{i-1} \leq w_{ji} < w_i$, j=1,..,$n_i$ where w_0=0 and w_2= ∞. In addition, there are m_i threshold discharge estimates w_{ji} for which there are a_{ji} exceedances in (a_{ji}+b_{ji}) years, j=1,..,m_i. Collectively this data is represented as:

$$\begin{aligned} D &= \{D_i,\ i = 1, 2\} \\ &= \left\{\left[w_{ji}, j = 1, \dots n_i; w_{ji}, a_{ji}, b_{ji}, j = 1, \dots, m_i\right], i = 1, 2\right\} \end{aligned} \quad (3.2.47)$$

Kuczera (1999) it can be shown for the two-zone rating error model of Figure 3.2.9 the likelihood reduces to:

$$p\left(D_1, D_2 | \theta_1, \sigma_2\right) = p\left(D_1, e_1 = 1 \,|\, \theta\right)\left[\int_0^\infty p\left(D_1, e_2 | \theta\right) g\left(e_2 | \sigma_2\right) de_2\right] \quad (3.2.48)$$

where

$$\begin{aligned} p\left(D_i, e_i | \theta_1\right) &= \prod_{j=1}^{n_i} \frac{1}{e_i} p\left(q_{i-1} + \frac{w_{ji} - w_{i-1}}{e_i} | \theta\right) \\ &= \prod_{j=1}^{m_i} P\left(a_{ji}, b_{ji} \middle| q_{i-1} + \frac{w_{ji} - w_{i-1}}{e_i}, \theta\right) \end{aligned} \quad (3.2.49)$$

$|g\left(e_i | \sigma_i\right)$ is the rating error multiplier pdf with mean 1 and standard deviation σ_i and $P(a, b | s, \theta)$) is the binomial probability of observing exactly a exceedances above the threshold discharge s in (a+b). This is a complex expression which can only be evaluated numerically. However, it makes the fullest use of information on annual flood peaks and binomial-censored data in the presence of rating curve error. Book 3, Chapter 2, Section 3 offers limited guidance on the choice of σ_2.

2.6.3.5. Prior Distributions

The prior pdf $p(\theta)$ reflects the worth of prior information on θ obtained preferably from a regional analysis. A convenient distribution is the multivariate normal with mean μ_p and covariance Σ_p; that is,

$$\theta \sim N\left(\mu_p, \Sigma_p\right) \quad (3.2.50)$$

In the absence of prior information, the covariance matrix can be made non-informative as illustrated in the following equation for a three-parameter model:

$$\Sigma_p \xrightarrow[v \to \infty]{} \begin{pmatrix} v & 0 & 0 \\ 0 & v & 0 \\ 0 & 0 & v \end{pmatrix} \quad (3.2.51)$$

Informative prior information can be obtained from a regional analysis of flood data.

The use of an informative prior based on regional analysis is strongly recommended in all Flood Frequency Analyses involving at-site data. Even with long at-site records, the shape parameter in the LP III and GEV distribution is subject to considerable uncertainty. Regional priors can substantially reduce the uncertainty in the shape (and even scale) parameter.

The regional procedures in Book 3, Chapter 3 are designed to express the prior information in the form of Equation (3.2.50) for the Log Pearson III probability model. They should be used in any Flood Frequency Analysis involving the Log Pearson III distribution unless there is evidence that the regional prior is not applicable to the catchment of interest.

2.6.3.6. Monte Carlo Sampling from the Posterior Distribution

The posterior pdf $P(\theta|D)$ fully defines the parameter uncertainty. However, interpreting this distribution is difficult using analytical methods. Modern Monte Carlo methods for sampling from the posterior have overcome this limitation – for example, see Gelman et al. (1995). Importance sampling from the posterior distribution describes a particular sampling procedure called importance sampling.

Importance sampling from the posterior distribution

Importance sampling is a widely used method Gelman et al. (1995) for sampling parameters from a target probability model for which there is no algorithm to draw random samples. The basic idea is to sample from a probability model for which a sampling algorithm exists – the probability model is called the importance distribution and the samples are called particles. The particles are then weighted so that they represent samples from the target distribution. The closer the importance distribution approximates the target, the more efficient the sampling.

Three steps are involved:

Step 1: Find most probable parameters of the target distribution

Any robust search method can be used to locate the value of θ, which maximises the logarithm of the posterior probability density; that is,

$$\hat{\theta} \leftarrow \max_{\theta} \log p(\theta|D) \tag{3.2.52}$$

where $\hat{\theta}$ is the most probable value of θ. The shuffled complex evolution algorithm of Duan et al. (1992) is a recommended search method.

Step 2: Obtain the importance distribution using a multi-normal approximation to the target distribution

Almost always, the log of posterior pdf $p(\theta|D)$ can be approximated by a second-order Taylor series expansion about the most probable parameter to yield the multivariate normal approximation

$$\theta|D \sim N\left(\hat{\theta}, \Sigma\right) \tag{3.2.53}$$

where θ is interpreted as the mean and the posterior covariance Σ is defined as the inverse of the Hessian

$$\Sigma = \left(-\frac{\partial^2 \log_e p(\theta \mid D)}{\partial^2 \theta}\right)^{-1} \tag{3.2.54}$$

An adaptive difference scheme should be used to evaluate the Hessian. Particular care needs to be exercised when selecting finite difference perturbations for the GEV and LP III distributions when upper or lower bounds are close to the observed data.

Step 3: Importance sampling of target distribution

The importance sampling algorithm proceeds as follows:

1. Sample N particles according to $\theta_i \leftarrow p_N(\theta_i), i = 1, \ldots N$ where $p_N(\theta)$ is the pdf of the multi-normal approximation obtained in Step 2.
2. Calculate particle probability weights according to $P(\theta_i) = \frac{p(\theta_i|D)}{p_N(\theta_i)}$,i=1,...,N
3. Scale the particle weights so they sum to 1.

2.6.3.7. Quantile Confidence Limits and Expected Probability

The posterior distribution of any function dependent on θ can be readily approximated using Monte Carlo samples. Confidence limits describe the uncertainty about quantiles arising from uncertainty in the fitted parameters. They are used in conjunction with the probability plot to evaluate goodness-of-fit. $100(1-\alpha)\%$ quantile confidence limits, or more correctly probability limits. Confidence limits can be derived as follows:

1. Draw N samples from the posterior distribution $\{\theta_i, w_i, i = 1, ..., N\}$ where w_i is the normalized weight assigned to the sample θ_i.
2. Rank in ascending order the N quantiles $\{q_Y(\theta_i), i = 1, ..., N\}$.
3. For each ranked quantile evaluate the non-exceedance probability $\sum_{j=1}^{i} w_{(j)}$ where $W_{(j)}$ is the weight for the jth ranked quantile $q_Y(\theta_j)$.
4. The lower and upper confidence limits are approximated by the quantiles whose non-exceedance probabilities are nearest to $\frac{\alpha}{2}$ and $1-\frac{\alpha}{2}$ respectively.

The expected posterior parameters can be estimated:

$$E[\theta|D] = \sum_{i=1}^{N} w_i q_Y(\theta_i) \tag{3.2.55}$$

These parameters can then be used to compute the expected parameter 1 in Y AEP quantiles described in Book 3, Chapter 2, Section 5.

Finally, the expected AEP probability for a flood of magnitude q_Y can be estimated:

$$E[P(q > q_Y|D)] = \sum_{i=1}^{N} w_i P(q > q_Y|\theta_i) \tag{3.2.56}$$

2.6.3.8. Treatment of Poor Fits

The standard probability models described in Book 3, Chapter 2, Section 4 may sometimes poorly fit flood data. Typically the goodness-of-fit is assessed by comparing observed data against the fitted probability model and its confidence limits. A poor fit may be characterised by:

- Presence of outliers in the upper or lower tail of the distribution. Outliers in Flood Frequency Analysis represent observations that are inconsistent with the trend of the remaining data and typically would lie well outside confidence limits; and
- Systematic discrepancies between observed and fitted distributions. Caution is required in interpreting systematic departures because plotting positions are correlated. Confidence limits can help guide the interpretation.

Poor fits to the standard probability models may arise for a variety of reasons including the following:

1. Small AM peaks may not be significant floods and thus may be unrepresentative of significant flood peaks;

2. By chance, one or more observed floods may be unusually rare for the length of gauged record. Recourse to the historical flood record may be useful in resolving this issue;

3. Rating curve extensions are biased resulting in a systematic under or over-estimate of large floods (see discussion in Book 3, Chapter 2, Section 3);

4. A change in hydraulic control with discharge may affect the shape of the frequency curve as illustrated in Book 3, Chapter 2, Section 8;

5. Storm events responsible for significant flooding may be caused by different meteorological mechanisms which lead to a mixed population not amenable to three-parameter distributions. This may arise when the majority of flood-producing storms are generated by one meteorological mechanism and the minority by an atypical mechanism such as a tropical cyclone; and

6. Nonhomogeneity of the flood record.

The potential causes of a poor fit need careful investigation.

If it is decided the poor fit is due to inadequacy of the probability model, three strategies are available to deal with the problem:

1. Observations can be censored;

2. The data responsible for the unsatisfactory fit may be given less weight; and

3. A more flexible probability model can used to fit the data.

Data in the upper part of the distribution is typically of more interest and therefore a strong case needs to be made to justify reduction in weight of such data.

2.6.3.9. Censoring of Potentially Influential Low Discharges

AM series contain many Annual Maximum discharges, which are less than bank full discharge. In arid zones these low peaks may be zero or very low values, not associated with any significant storm event. These low peaks may not be representative of the physical processes driving large floods (Cohn et al., 2013; Pedruco et al., 2014). The inclusion of such data in a Flood Frequency Analysis runs the risk of low peaks unrepresentative of large floods influencing the fit to the right-hand tail of the frequency distribution, which is of most interest to the hydrologist. Therefore the identification and removal of Potentially Influential Low Flows (PILFs) is considered an important step in a Flood Frequency Analysis.

Cohn et al. (2013) developed a generalisation of the Grubbs-Beck test that was recommended in Bulletin 17B (Interagency Advisory Committee on Water Data, 1982) to identify PILFs. The multiple Grubbs-Beck test, checks if the k^{th} smallest flow is unusually low and, if it is, uses this discharge to define a threshold for censoring discharges below the threshold. The test involves two steps:

1. The outward sweep starts at the median discharge and moves towards the smallest discharge. Each flow is tested at the 0.5% significance level. If the k^{th} smallest flow is identified as a low outlier, the outward sweep stops; and

2. The inward sweep starts at the smallest discharge and moves towards the median. Each discharge is tested at the 10% significance level. If the m^{th} flow is identified as a low outlier, the inward sweep stops.

The total number of low outliers is then the maximum of k and m - 1. The flows identified as low outliers are treated as censored flows.

The multiple Grubbs-Beck test is recommended for general use but must be conducted in unison with a visual assessment of the fitted frequency curve.

2.6.3.10. Software

The Bayesian approach to calibrating flood probability models is numerically complex and is best implemented in a high level programming language. The web-based software called TUFLOW FLIKE supporting the Bayesian methods described in this chapter. The reader is advised that this does not preclude use of other software if it is fit for purpose.

2.6.3.11. Worked Examples

Book 3, Chapter 2, Section 8 illustrates fitting a LP III distribution to a 31-year gauged record. Although the fit is judged satisfactory, considerable uncertainty in the 1% AEP quantile is noted.

Book 3, Chapter 2, Section 8 is a continuation of Example 3 and illustrates the benefit of incorporating censored historic flood information. In the 118 years prior to gauging only one flood exceed the largest gauged flood. This information is shown to substantially reduce quantile uncertainty.

Book 3, Chapter 2, Section 8 is a continuation of Example 3. It illustrates the value of regional information in reducing uncertainty in parameters and quantiles. It is recommended that regional information be always used unless there is contrary evidence.

Book 3, Chapter 2, Section 8 illustrates the identification of PILFs using the multiple Grubbs-Beck test and fitting a LP III distribution with PILFs treated as censored discharges. It is recommended that multiple Grubbs-Beck test be performed in all Flood Frequency Analyses.

Book 3, Chapter 2, Section 8 illustrates how three–parameter distributions such as the GEV and LP III can be made to fit data exhibiting sigmoidal behaviour. Because interest is in fitting the higher discharges the low discharges are de-emphasized by treating them as censored observations. In this example the multiple Grubbs-Beck test improved the fit but a more severe manual censoring produced an even better fit to the right-hand tail.

Book 3, Chapter 2, Section 8 illustrates application of a non-homogeneous flood probability model conditioned on the IPO index. It shows how the long-term flood risk may be estimated.

2.6.4. L-moments Approach

2.6.4.1. Overview

L-moments were developed by Hosking (1990) to overcome the bias and sensitivity of the method of product-moments approach to fitting distributions. L-moment estimators are unbiased and are less sensitive to outliers than product-moment estimators. They have been used extensively by researchers to analyse extremes and are recommended for use in Flood Frequency Analysis in the Handbook of Hydrology (Stedinger et al., 1993).

The L-moment approach is simpler than the Bayesian approach but limited in capability. It is restricted to applications involving gauged discharge data where there is no useful regional information and rating curve errors do not require special attention.

2.6.4.2. L-moments for summarising distributions

Hosking (1990) developed the L-moment theory based on order statistics. The first four L-moments are defined as:

$$\lambda_1 = \mathrm{E}[\mathrm{X}]_{1:1} \tag{3.2.57}$$

$$\lambda_2 = \frac{1}{2}\,\mathrm{E}\left[X_{2:2} - X_{1:2}\right] \tag{3.2.58}$$

$$\lambda_3 = \frac{1}{3}\,\mathrm{E}\left[X_{3:3} - 2\mathrm{X}_{2:3} + X_{1:3}\right] \tag{3.2.59}$$

$$\lambda_4 = \frac{1}{4}\,\mathrm{E}\left[X_{4:4} - 3\mathrm{X}_{3:4} + 3\mathrm{X}_{2:4} - X_{1:4}\right] \tag{3.2.60}$$

where $X_{j:m}$ is the j^{th} smallest variable in a sample of size m and E stands for expectation.

Wang (1996) justifies L-moments as follows: "When there is only one value in a sample, it gives a feel of the magnitude of the random variable. When there are two values in a sample, their difference gives a sense of how varied the random variable is. When there are three values in a sample, they give some indication on how asymmetric the distribution is. When there are four values in a sample, they give some clue on how peaky, roughly speaking, the distribution is."

When many such samples are considered, the expectations λ_1 and λ_2 give measures of location and scale. Moreover, the L-moment ratios:

$$\tau_3 = \frac{\lambda_3}{\lambda_2} \tag{3.2.61}$$

$$\tau_4 = \frac{\lambda_4}{\lambda_2} \tag{3.2.62}$$

give measures of skewness and kurtosis respectively. Hosking termed τ_3 L-skewness and τ_4 L-kurtosis. Hosking also defined the L-coefficient of variation as:

$$\tau_2 = \frac{\lambda_2}{\lambda_1} \tag{3.2.63}$$

Table 3.2.3 Summarizes L-moments for a range of distributions.

Table 3.2.3. L-moments for several distributions (from Stedinger et al. (1993))

Family	L-moments
Generalized Extreme Value (GEV)	$\lambda_1 = \tau + \frac{\alpha}{\kappa}[1 - \Gamma(1+\kappa)]$ $\lambda_2 = \frac{\alpha}{\kappa}\Gamma(1+\kappa)[1 - 2^{-\kappa}]$ $\tau_3 = \frac{2(1-3^{-\kappa})}{1-2^{-\kappa}} - 3$ $\tau_4 = \frac{1 - 5(4^{-\kappa}) + 10(3^{-\kappa}) - 6(2^{-\kappa})}{1-2^{-\kappa}}, \kappa \neq 0$

Family	L-moments
Gumbel	$\lambda_1 = \tau + 0.5772\alpha$ $\lambda_2 = \alpha \ln 2$ $\tau_3 = 0.1699$ $\tau_4 = 0.1504$
Generalized Pareto	$\lambda_1 = q_* + \frac{\beta}{1+\kappa}$ $\lambda_2 = \frac{\beta}{(1+\kappa)(2+\kappa)}$ $\tau_3 = \frac{1-\kappa}{3+\kappa}$ $\tau_4 = \frac{(1-\kappa)(2-\kappa)}{(3+\kappa)(4+\kappa)}$

2.6.4.3. L-moments estimates for gauged data sample data

The traditional L-moment estimator is based on probability weighted moments. However, Wang (1996) derived the following sample estimators directly from the definition of the first four L-moments:

$$\widehat{\lambda_1} = \frac{1}{{}^{n}C_1} \sum_{i=1}^{n} q_{(i)} \tag{3.2.64}$$

$$\widehat{\lambda_2} = \frac{1}{2} \frac{1}{{}^{n}C_2} \sum_{i=1}^{n} \left({}^{i-1}C_1 - {}^{n-i}C_1 \right) q_{(i)} \tag{3.2.65}$$

$$\widehat{\lambda_3} = \frac{1}{3} \frac{1}{{}^{n}C_3^{n}} \sum_{i=1}^{n} \left({}^{i-1}C_2 - 2\,{}^{i-1}C_1\,{}^{n-i}C_1 + {}^{n-i}C_2 \right) q_{(i)} \tag{3.2.66}$$

$$\widehat{\lambda_4} = \frac{1}{4} \frac{1}{{}^{n}C_4} \sum_{i=1}^{n} \left({}^{i-1}C_3 - 3\,{}^{i-1}C_2\,{}^{n-i}C_1 + 3\,{}^{i-1}C_1\,{}^{n-i}C_2 - {}^{n-i}C_3 \right) q_{(i)} \tag{3.2.67}$$

where $q_{(i)}, i = 1, 2, \ldots, n$ are gauged peak discharges ranked in ascending order and:

$${}^{m}C_k = \begin{Bmatrix} \frac{m!}{k!(m-k)!} if\ k \le m \\ 0\ if\ k > m \end{Bmatrix} \tag{3.2.68}$$

is the number of combinations of any k items from m items.

2.6.4.4. Parameter and quantile estimation

The method of L-moments involves matching theoretical and sample L-moments to estimate parameters. The L-moments in Table 3.2.3 are replaced by their sample estimates given by Equation (3.2.67). The resulting equations are then solved to obtain estimates of the parameters. These parameters are used to calculate the X% AEP quantiles.

2.6.4.5. LH-moments for fitting the GEV distribution

When the selected probability model does not adequately fit all the gauged data, the lower discharges may exert undue influence on the fit and give insufficient weight to the higher discharges which are the principal object of interest. To deal with this situation, Wang (1997) introduced a generalisation of L-moments called LH-moments. A more detailed exposition can be found in LH-moments for fitting the GEV Distribution.

LH-moments for fitting the GEV Distribution

LH-moments are based on linear combinations of higher order-statistics. A shift parameter $\eta = 0,1,2,3...$ is introduced to give more emphasis on higher ranked flows. LH-moments are defined as:

$$\lambda_1^{\eta} = \mathrm{E}[X_{(\eta+1):(\eta+1)}] \tag{3.2.69}$$

$$\lambda_2^{\eta} = \frac{1}{2}\,\mathrm{E}\left[X_{(\eta+2):(\eta+2)} - X_{(\eta+1):(\eta+3)}\right] \tag{3.2.70}$$

$$\lambda_3^{\eta} = \frac{1}{3}\,\mathrm{E}\left[X_{(\eta+3):(\eta+3)} - 2\mathrm{X}_{(\eta+2):(\eta+3)} + X_{(\eta+1):(\eta+3)}\right] \tag{3.2.71}$$

$$\lambda_4^{\eta} = \frac{1}{4}\,\mathrm{E}\left[X_{(\eta+4):(\eta+4)} - 3\mathrm{X}_{(\eta+3):(\eta+4)} + 3\mathrm{X}_{(\eta+2):(\eta+4)} - X_{(\eta+1):(\eta+4)}\right] \tag{3.2.72}$$

Table 3.2.4 presents the relationship between the first four LH-moments and the parameters of the GEV and Gumbel distributions.

Table 3.2.4. LH-moments for GEV and Gumbel distributions (from Wang (1997))

Family	LH-moments
Generalised extreme value (GEV)	$\lambda_1^{n}=\tau+\frac{\alpha}{\kappa}[1-\Gamma(1+\kappa)(\eta+1)^{-\kappa}]\lambda_2^{n}=\frac{(\eta+2)\alpha\Gamma(1+\kappa)}{2!\kappa}[-(\eta+2)^{-\kappa}+(\eta+1)^{-\kappa}]$ $\lambda_3^{n}=\frac{(\eta+3)\alpha\Gamma(1+\kappa)}{3!\kappa}[-(\eta+4)(\eta+3)^{-\kappa}+2(\eta+3)(\eta+2)^{-\kappa}-(\eta+2)(\eta+1)^{-\kappa}]$ $\lambda_4^{n}=\frac{(\eta+4)\alpha\Gamma(1+\kappa)}{4!\kappa}[-(\eta+6)(\eta+5)(\eta+4)^{-\kappa}+3(\eta+5)(\eta+4)(\eta+3)^{-\kappa}$ $-3(\eta+4)(\eta+3)(\eta+2)^{-\kappa}+(\eta+3)(\eta+2)(\eta+1)^{-\kappa}]$ where $\kappa\neq0$

Family	LH-moments
Gumbel	$\lambda_1^n = \tau + \alpha[0.5772 + \ln(\eta+1)] \lambda_2^n = \frac{(\eta+2)\alpha}{2!}[\ln(\eta+2) - \ln(\eta+1)$ $\lambda_3^n = \frac{(\eta+3)\alpha}{3!}[(\eta+4)\ln(\eta+3) - 2(\eta+3)\ln(\eta+2) + (\eta+2)\ln(\eta+1)]$ $\lambda_4^n = \frac{(\eta+4)\alpha}{4!\kappa}[(\eta+6)(\eta+5)\ln(\eta+4) - 3(\eta+5)(\eta+4)\ln(\eta+3)$ $+3(\eta+4)(\eta+3)\ln(\eta+2) - (\eta+3)(\eta+2)\ln(\eta+1)]$

For ease of computation Wang (1997) derived the following approximation for the shape parameter κ:

$$\kappa = a_0 + a_1[\tau_3^\eta] + a_2[\tau_3^\eta]^2 + a_3[\tau_3^\eta]^3 \tag{3.2.73}$$

where the polynomial coefficients vary with η according to Table 3.2.5.

Table 3.2.5. Polynomial coefficients for use with Equation (3.2.73)

η	a_0	a_1	a_2	a_3
0	0.2849	-1.8213	0.8140	-0.2835
1	0.4823	-2.1494	0.7269	-0.2103
2	0.5914	-2.3351	0.6442	-0.1616
3	0.6618	-2.4548	0.5733	-0.1273

η	a_0	a_1	a_2	a_3
4	0.7113	-2.5383	0.5142	-0.1027

Wang (1997) derived the following estimators for LH-moments with shift parameter η:

$$\lambda_1^\eta = \frac{1}{{}^{n}C_{\eta+1}} \sum_{i=1}^{n} {}^{i-1}C_{\eta}\, x_{(i)} \tag{3.2.74}$$

$$\hat{\lambda}_2^\eta = \frac{1}{2} \frac{1}{{}^{n}C_{\eta+2}} \sum_{i=1}^{n} \left({}^{i-1}C_{\eta+1} - {}^{i-1}C_{\eta}\, {}^{n-i}C_{1} \right) q_{(i)} \tag{3.2.75}$$

$$\hat{\lambda}_3^\eta = \frac{1}{3} \frac{1}{{}^{n}C_{\eta+3}} \sum_{i=1}^{n} \left({}^{i-1}C_{\eta+2} - 2\, {}^{i-1}C_{\eta+1}\, {}^{n-i}C_{1} + {}^{i-1}C_{\eta}\, {}^{n-i}C_{2} \right) q_{(i)} \tag{3.2.76}$$

$$\begin{aligned} \hat{\lambda}_4^\eta = \frac{1}{4} \frac{1}{{}^{n}C_{\eta+4}} \sum_{i=1}^{n} (& {}^{i-1}C_{\eta+3} - 3\, {}^{i-1}C_{\eta+2}\, {}^{n-i}C_{1} \\ & + 3\, {}^{i-1}C_{\eta+1}\, {}^{n-i}C_{2} - {}^{i-1}C_{\eta}\, {}^{n-i}C_{3})\, q_{(i)} \end{aligned} \tag{3.2.77}$$

The selection of the best shift parameter requires some form of goodness-of-fit test. Wang (1998) argued that the first three LH-moments are used to fit the GEV model leaving the fourth LH-moment available for testing the adequacy of the fit. Wang (1998) proposed the following approximate test statistic:

$$z = \frac{\hat{\tau}_4^\eta - \tau_4^\eta}{\sigma(\hat{\tau}_4^\eta | \hat{\tau}_3^\eta = \tau_3^\eta)} \tag{3.2.78}$$

where $\hat{\tau}_4^\eta$ is the sample estimate of the LH-kurtosis, τ_4^η is the LH-kurtosis derived from the GEV parameters fitted to the first three LH-moments, and $\sigma(\hat{\tau}_4^\eta | \hat{\tau}_3^\eta = \tau_3^\eta)$ is the standard deviation of $\hat{\tau}_4^\eta$ assuming the sample LH-skewness equals the LH-skewness derived from the GEV parameters fitted to the first three LH-moments. Under the hypothesis that the underlying distribution is GEV, the test statistic z is approximately normal distributed with mean 0 and variance 1. Wang (1998) describes a simple relationship to estimate $\sigma(\hat{\tau}_4^\eta | \hat{\tau}_3^\eta = \tau_3^\eta)$.

2.6.4.6. Parameter Uncertainty and Quantile Confidence Limits

The sampling distribution $p(\theta|D)$ can be approximated using the Monte Carlo method known as the parametric bootstrap described in Parametric bootstrap. This procedure yields N equi-weighted samples that approximate the sampling distribution $p(\theta|D)$. As a result, they can be used to quantify parameter uncertainty and estimate quantile confidence limits. However, because the parametric bootstrap assumes θ is the true parameter, it underestimates the uncertainty and therefore should not be used to estimate expected probabilities.

Parametric bootstrap

The sampling distribution of an estimator can be approximated using the Monte Carlo method known as the parametric bootstrap:

1. Fit the probability model to n years of gauged discharges using L or LH-moments to yield the parameter estimate θ.
2. Set i=1
3. Randomly sample n flows from the fitted distribution; that is, $q_{ji} \leftarrow p\left(q\middle|\hat{\theta}\right), j = 1, \ldots, n)$
4. Fit the model to the sampled flows $\left\{q_{ji}, j = 1, \ldots, n\right\}$ using L or LH-moments to yield the parameter estimate θ_i
5. Increment i. Go to step 3 if i does not exceed N.

This procedure yields N equi-weighted samples that approximate the sampling distribution $p(\theta|D)$. As a result, they can be used to quantify parameter uncertainty and estimate quantile confidence limits. However, because the parametric bootstrap assumes $\hat{\theta}$ is the true parameter, it underestimates the uncertainty and therefore should not be used to estimate expected probabilities.

2.6.4.7. Software

Implementation of L and LH-moments requires extensive computation. The TUFLOW FLIKE software, supports the L and LH-moment estimation described in this section.

2.6.4.8. Worked Examples

Book 3, Chapter 2, Section 8 illustrates fitting the GEV distribution using L-moments to a 47-year gauged record. Book 3, Chapter 2, Section 8 revisits Book 3, Chapter 2, Section 8 demonstrating the search procedure for finding the optimal shift in LH-moments fitting.

2.6.5. Method of Moments Approach

The method of moments used in conjunction with the LP III distribution was the recommended method in Australian Rainfall and Runoff (Pilgrim, 1987). The method was simple to implement but, unlike US practice, did not use regionalised skew. Its use is no longer recommended in Australia. Both, the Bayesian and L-moment procedures, make better use of the available information.

2.7. Fitting Flood Probability Models to Peak Over Threshold Series

2.7.1. Probability Plots

As with the analysis of Annual Maximum series, it is recommended that a probability plot of the POT data series be prepared. The plot involves plotting an estimate of the observed ARI against the discharge. The ARI of a gauged flood can be estimated using:

$$T_{(\mathrm{i})} = \frac{n + 0.2}{i - 0.4} \qquad (3.2.79)$$

where i is the rank of the gauged flood (in descending order) and n is the number of years of record.

2.7.2. Fitting Peak-Over-Thresholds Models

In some cases, it may be desirable to analytically fit a probability distribution to POT data. Two general approaches have been used.

In the first approach, the annual flow series distribution has been estimated from the POT series on the basis that the latter considers all of the relevant data and should thus provide a better estimate. The basis for the procedure is Equation (3.2.14), which links Poisson arrivals of flood peaks with a distribution of flood magnitudes above some threshold. Jayasuriya and Mein (1985), Ashkar and Rousselle (1983) and Tavares and Da Silva (1983) explored this approach using the exponential distribution. The approach has been fairly successful, but some results have diverged from the distribution derived directly from the annual series. Given the concern that the fit to the right tail of the annual maximum series may be compromised by the leverage exerted by the low flows, this approach cannot be recommended as a replacement for the analysis of annual series data. If possible, the base discharge for this approach should be selected so that the number of floods in the POT series m is at least 2 to 3 times the number of years of record n. However, it may be necessary to use a much lower value of m in regions with low rainfall where the number of recorded events that could be considered as floods is low.

The second approach uses a probability distribution as an arbitrary means of providing a consistent and objective fit to POT series data. For example, McDermott and Pilgrim (1982), Adams and McMahon (1985) and Jayasuriya and Mein (1985) used the LP III distribution – they found that selecting a threshold discharge such that m equalled n was best. Book 3, Chapter 2, Section 8 illustrates this approach using L-moments to fit an exponential distribution to a POT series.

2.8. Supplementary Information

2.8.1. Example 1: Extrapolation and Process Understanding

The importance of process understanding when extrapolating beyond the observed record is illustrated by a simple Monte Carlo experiment. A Poisson rectangular pulse rainfall model is used to generate a long record of high resolution rainfall. This is routed through a rainfall-runoff model to generate runoff into the stream system. The storage-discharge relationship for the stream is depicted by the bilinear relationship shown in Figure 3.2.10. A feature of this relationship is the activation of significant flood terrace storage once a threshold discharge is exceeded.

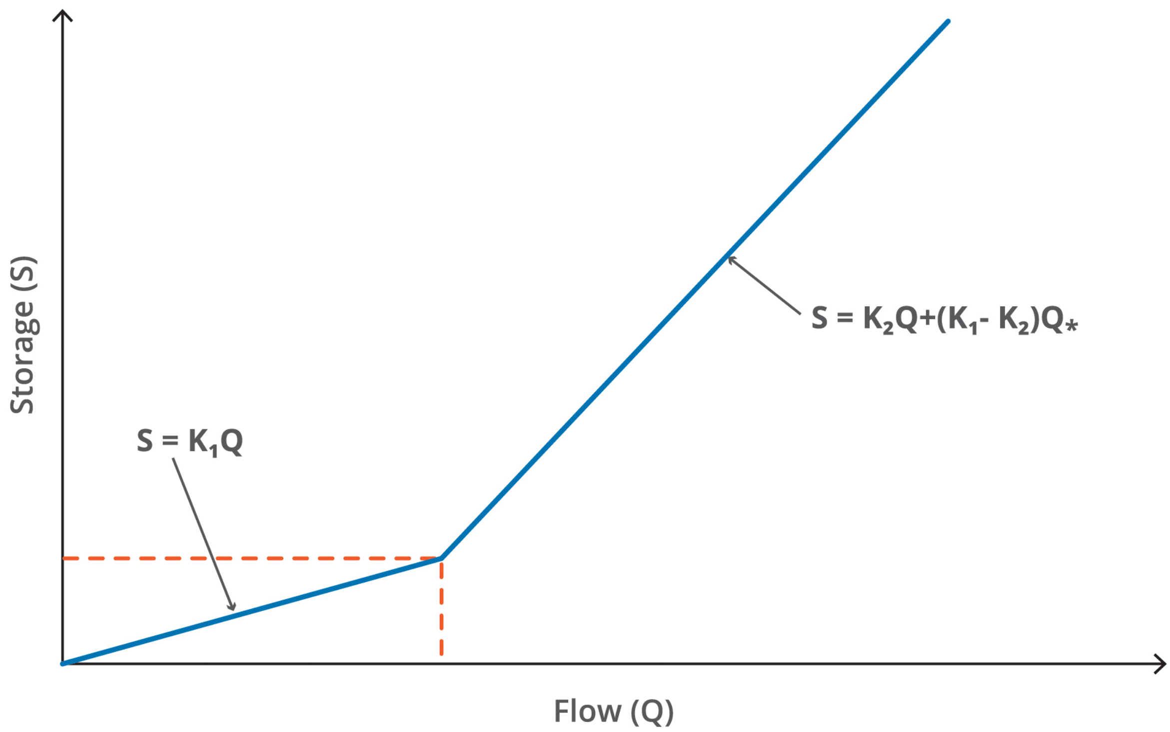


Figure 3.2.10. Bilinear channel storage-discharge relationship

The routing model parameters were selected so that major flood terrace storage is activated by floods of less than 1 in 100 AEP. This situation was chosen to represent a river with multiple flood terraces with the lowest terraces accommodating the majority of floods and the highest terrace only inundated by extreme floods.

Figure 3.2.11 presents the flood frequency curve based on 30 000 simulated years – it shows a clear break in slope around the 1 in 100 AEP corresponding to the activation of major flood terrace storage. Indeed the flood frequency curve displays downward curvature despite that the fact the rainfall frequency curve displays upward curvature in the 1 in 100 to 1 in 1000 AEP range. In contrast the flood frequency curve based on 100 years of “data” shows no evidence of downward curvature. This is because in a 100 year record there is little chance of the major flood terrace storage being activated. Indeed without knowledge of the underlying hydraulics one would be tempted to extrapolate the 100 year flood record using a straight line extrapolation. Such an extrapolation would rapidly diverge from the “true” frequency curve.

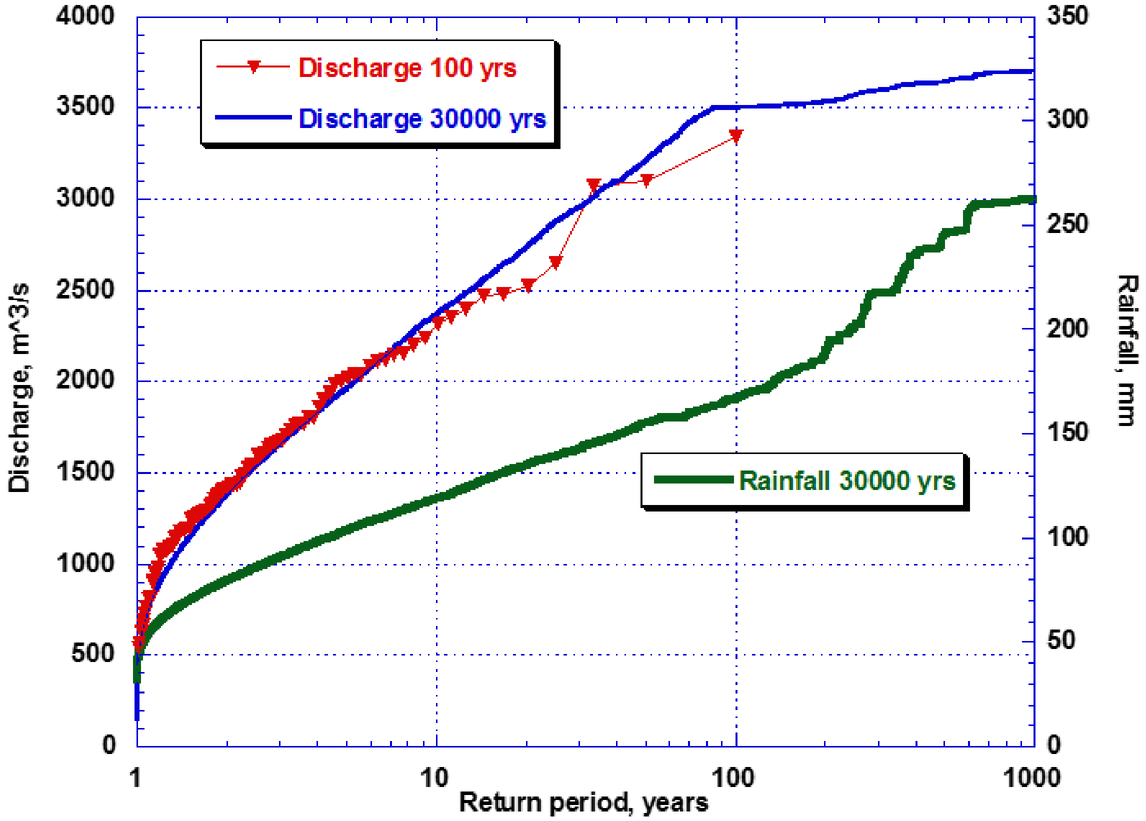


Figure 3.2.11. Simulated rainfall and flood frequency curves with major floodplain storage activated at a threshold discharge of 3500 m^3/s

Although the example idealises the dominant rainfall-runoff dynamics it delivers a very strong message. Extrapolation of flood frequency curves fitted to gauged discharges records requires the exercise of hydrologic judgment backed up by appropriate modelling. The problem of extrapolation is much more general. For example, in this case, if a rainfall-runoff approach were used with the rainfall-runoff model calibrated to small events the simulated flood frequency curve is likely to be compromised in a similar way.

2.8.2. Example 2: Accuracy of Daily Gauged Discharges

The use of daily discharge readings in Flood Frequency Analysis is most problematic for smaller catchments, which can be "flashy" in the sense that the hydrograph can rise and subside within a twenty four hour period. This effect can be quite significant, even for reasonably large catchments.

Figure 3.2.12 and Figure 3.2.13, taken from Micevski et al. (2003), compare instantaneous annual maximum discharge against the discharge recorded at 9am on the same day for two gauging stations in the Hunter Valley: Goulburn River at Coggan with area 3340 km^2 and Hunter River at Singleton with area 16 400 km^2. The dashed line represents equality. Figure 3.2.12 demonstrates that the true peak flow can be up to 10 times the 9:00 am flow. In contrast the estimation error is much smaller for the larger catchment shown in Figure 3.2.13.

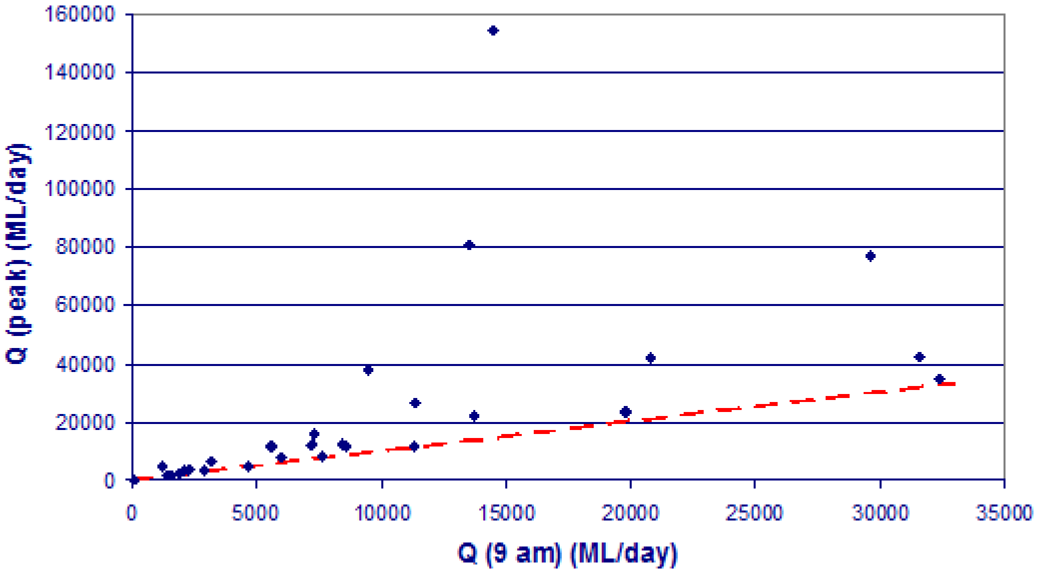


Figure 3.2.12. Comparison between true peak flow and 9:00 am flow for Goulburn River at Coggan

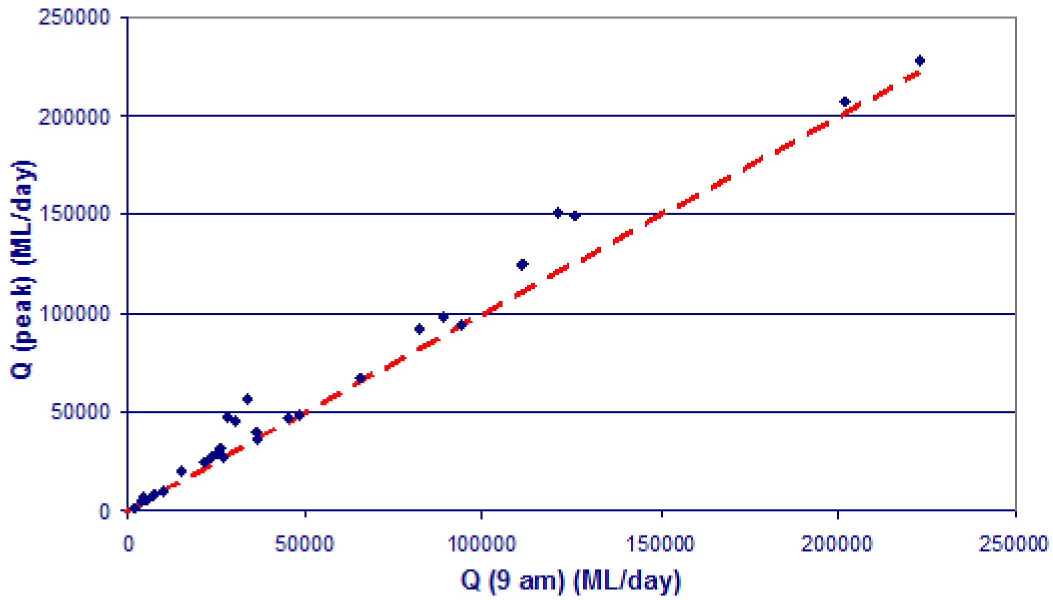


Figure 3.2.13. Comparison between true peak flow and 9:00 am flow for Hunter River at Singleton

The example demonstrates the need to check the representativeness of daily readings by comparing instantaneous peak flows against daily readings.

2.8.3. Example 3: Fitting a probability model to gauged data

2.8.3.1. Launch TUFLOW Flike

This example demonstrates undertaking a flood frequency analysis using the procedures described in this book. Specifically, this example covers the fitting of a Log Pearson Type III distribution to an annual maximum series for the Hunter River at Singleton. The analysis will be undertaken using TUFLOW Flike which has been developed to undertake flood frequency analysis as described in this book, that is, it has the ability to fit a range of statistical distributions using a Bayesian Inference method.

Once TUFLOW Flike has been obtained and installed, launch **TUFLOW Flike** and the screen in Figure 3.2.14 will appear.

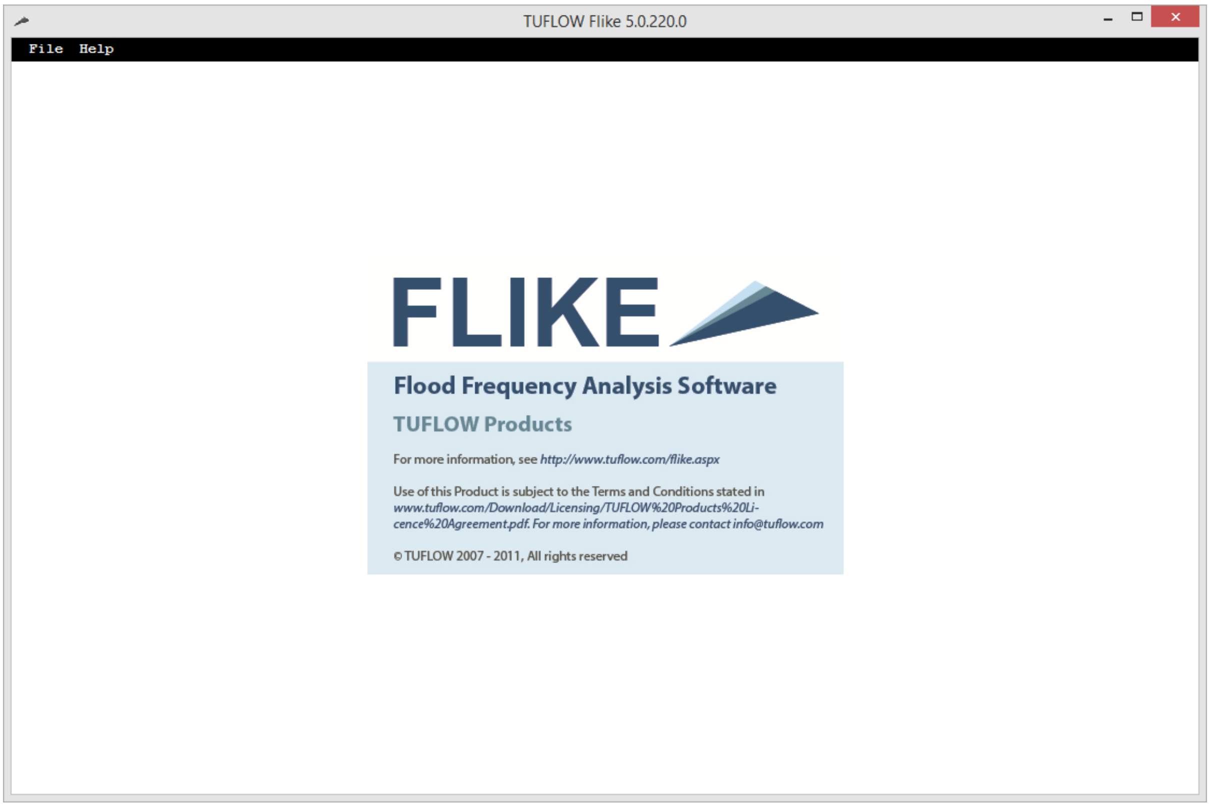


Figure 3.2.14. TUFLOW Flike Splash Screen

2.8.3.2. Create the .fld file

The first step will be to create the **.fld** file which contains information about the project. To create a new **.fld** file, select **New** from the **File** dropdown menu. This will open a new window called **Open** as shown in Figure 3.2.15.

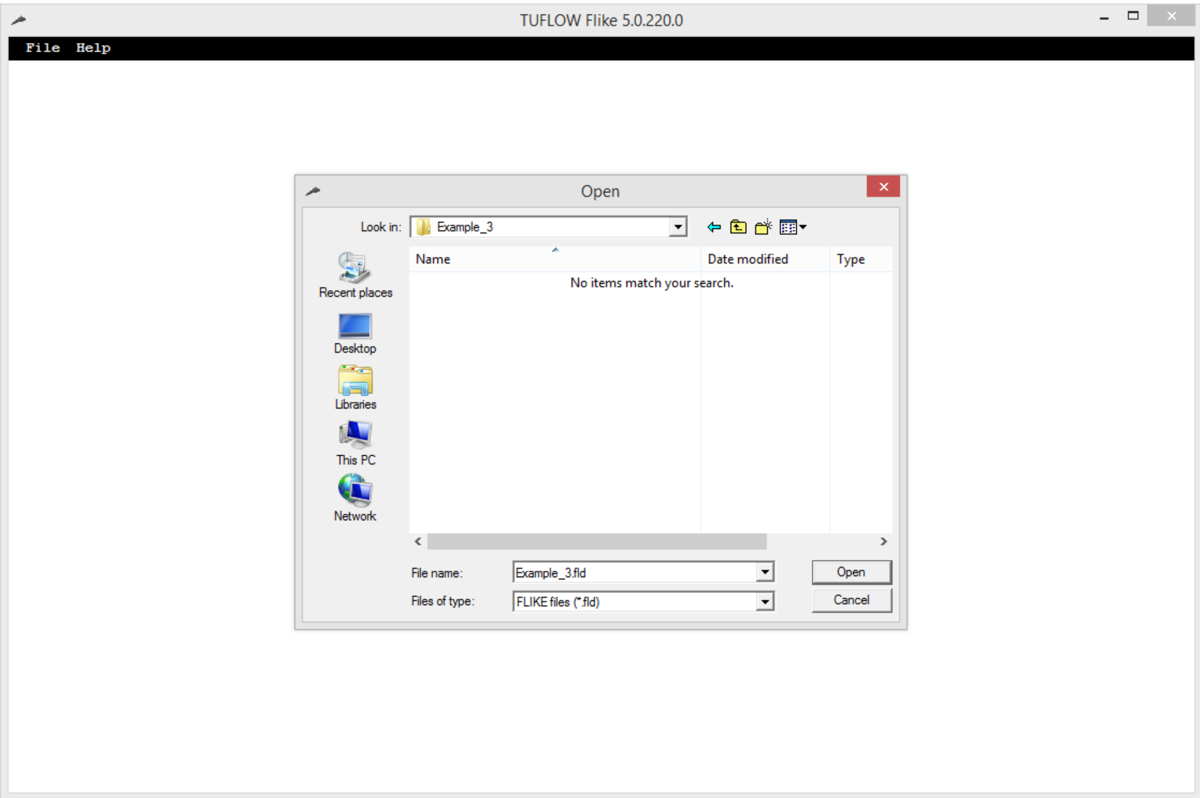


Figure 3.2.15. Create New .fld file

Create and save a new .fld file in an appropriate location, such as in a folder under the job directory, and give it a logical name, in this case **Example_3.fld**. A message will appear asking if you want to create the file, select **Yes**. Note that the window is titled **Open**, but it works for creating new files as well. Once the **.fld** file has been saved, the **Flike Editor** window will open which will be used in the next step.

2.8.3.3. Configure the Project Details

The **.fld** file is used to store the project data and configuration. Once the **.fld** has been created the **Flike Editor** window will open automatically (see Figure 3.2.16) and the project will be configured here. The first bit of information to be completed is the project a name which is filled in the **Title** text box. The project title can go over two lines.

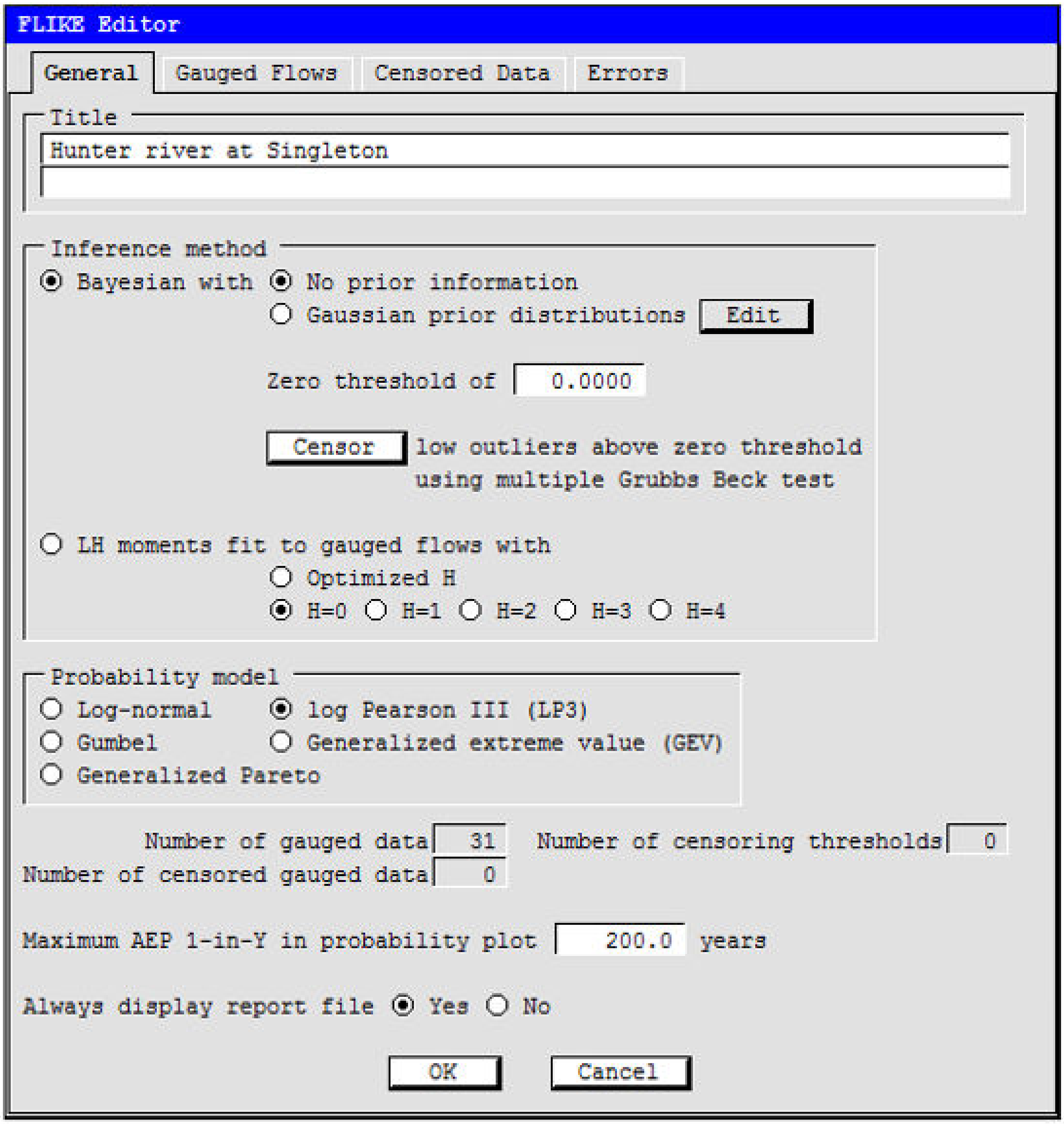


Figure 3.2.16. Flike Editor Screen

2.8.3.4. Import the Data

The next step is to import the flood series to analyse. To do this select the **Observed values** tab in the **Flike Editor** as shown in Figure 3.2.17. In this tab the flood series to be investigated will be imported.

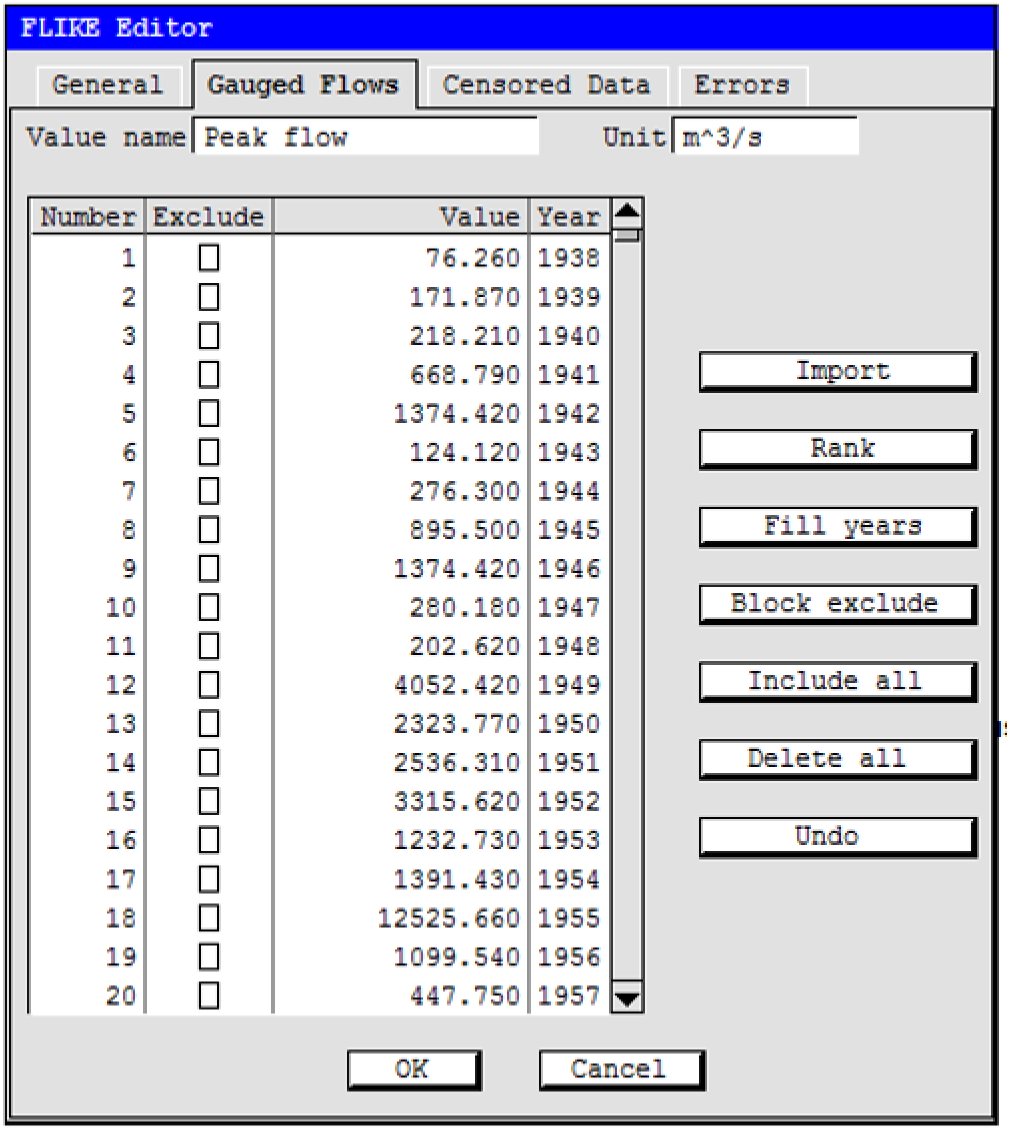


Figure 3.2.17. Observed Values Screen

To import the flood series select the **Import** button and the **Import gauged values** window opens as shown in Figure 3.2.18. Now select the **Browse** button and navigate to the Singleton flood series. This example data are included in the TUFLOW Flike download, a copy of which was installed in the **data** folder in the install location of TUFLOW-Flike. By default, this location is *C:\TUFLOW Flike\data\singletonGaugedFlows.csv*. This data also appears at the end of this example.

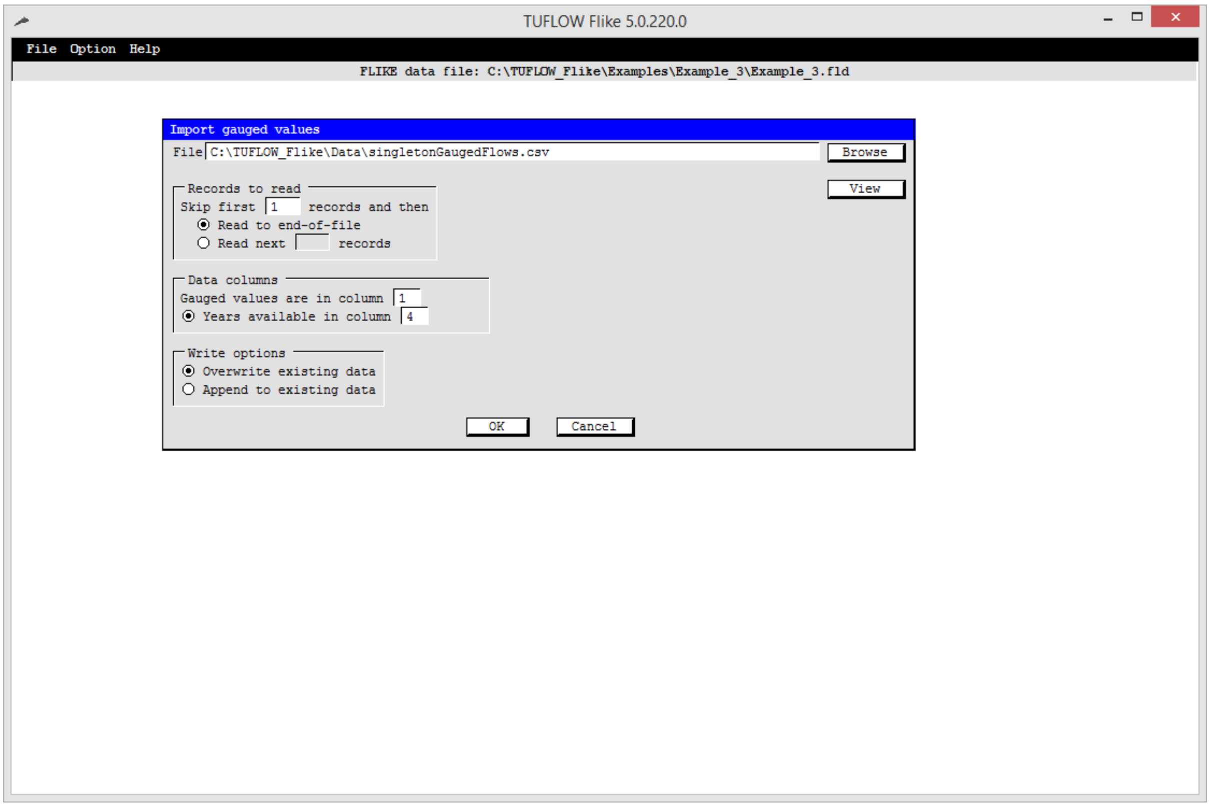
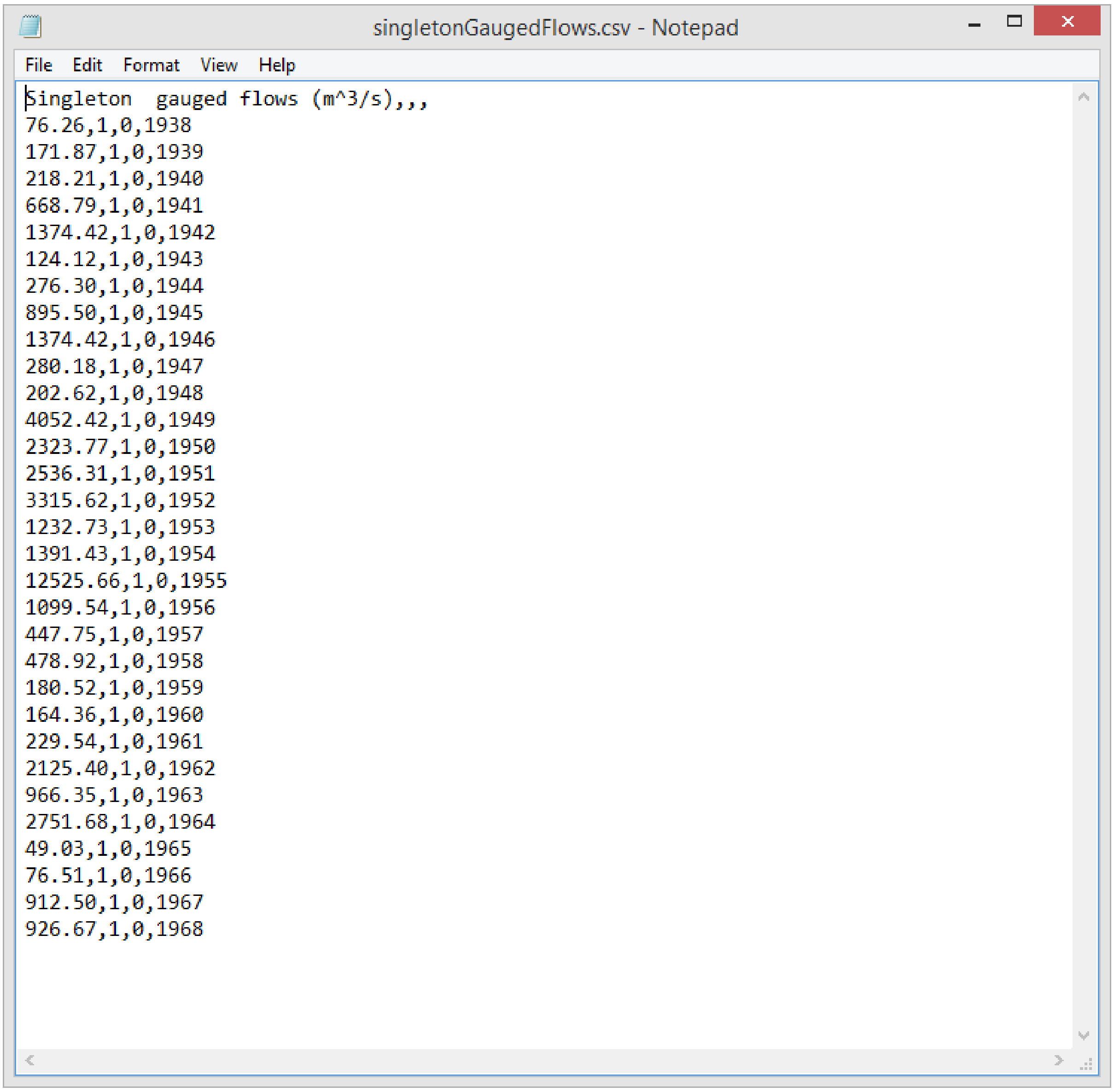


Figure 3.2.18. Import Gauged Values Screen

Once the data file has been selected, the program will return to the **Import gauged values** window. As the input data format is flexible TUFLOW Flike needs to be told how to interpret the data file. To view the format of the data, select the **View** button and the data will be open in your default text editor (see Figure 3.2.19). In the example data the first line contains a header line and the data follows this. The flow values are in the first column and the year in the fourth column. Having taken note of the data structure close the text editor and return to the **Import gauged values** window. It's a good habit to check the data in the text editor to ensure that the format of the data is known and the file has not been corrupted or includes a large number of trailing comma or whitespace. This last issue commonly occurs when deleting information from excel files, but it is easy to fix. Simply delete any trailing comma or white space in a text editor.




```
Singleton  gauged flows (m^3/s),,,
76.26,1,0,1938
171.87,1,0,1939
218.21,1,0,1940
668.79,1,0,1941
1374.42,1,0,1942
124.12,1,0,1943
276.30,1,0,1944
895.50,1,0,1945
1374.42,1,0,1946
280.18,1,0,1947
202.62,1,0,1948
4052.42,1,0,1949
2323.77,1,0,1950
2536.31,1,0,1951
3315.62,1,0,1952
1232.73,1,0,1953
1391.43,1,0,1954
12525.66,1,0,1955
1099.54,1,0,1956
447.75,1,0,1957
478.92,1,0,1958
180.52,1,0,1959
164.36,1,0,1960
229.54,1,0,1961
2125.40,1,0,1962
966.35,1,0,1963
2751.68,1,0,1964
49.03,1,0,1965
76.51,1,0,1966
912.50,1,0,1967
926.67,1,0,1968
```

Figure 3.2.19. View Gauged Values in Text Editor

The next step is to configure the import of the data. As the example data has a header, the first line needs to be skipped. Enter **1** into the **Skip first __ records and then text field**. This will skip the first line. Ensure that the **Read to the end-of-file** option is selected (this is the default). Occasionally, there may be a need to specify how many records to read, in which case this can be achieved by selecting the **Read next __ records** option and entering the desired number of records to read. Next, specify which column the flood data are in, by filling the **gauged values are in column __** text box, in this example data this is column 1. Next, select the **Years available in column __** text box and specify the column that this data is in (column 4). Finally, select **OK** to import the data. The **Import gauged values** window should look similar to Figure 3.2.18.

The **Value** and **Year** columns in the **Observed values** tab will now be filled with the data in the order that they were in the data file as shown in Figure 3.2.20. The data can be sorted by value and year using the **Rank** button. Selecting this button will open a new window (Figure 3.2.21) where there are five choices to rank by, these are:

- **Descending flow:** Ranks the data in order of values from largest to smallest
- **Ascending flow:** Ranks the data in order of values from smallest to largest
- **Descending year:** Ranks the data in order of year from largest to highest
- **Ascending year:** Ranks the data in order of year from highest to largest
- **Leave unchanged** : Leaves both the values and years unchanged

It is always a good idea to initially rank your data in descending order so you can check the largest flows. For this data series the value is 12 525.66 m^3/s. Leave the data ranked in descending order for this example.

Note that the value name and units can be specified by entering values in the **Valuename** and **Unit** text boxes. These titles do not affect the computations in any way, they do, however, assist in reviewing the results, particularly when presenting results to external audiences.

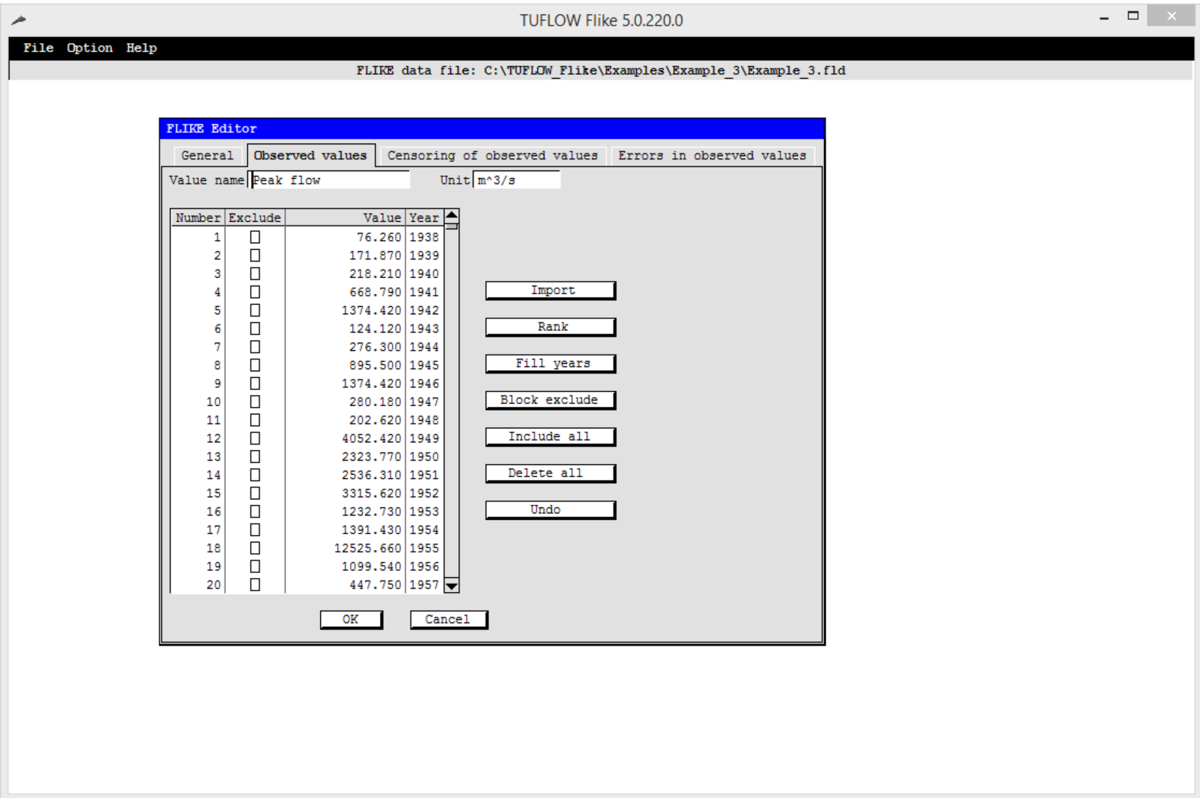


Figure 3.2.20. Observed Values screen with Imported Data

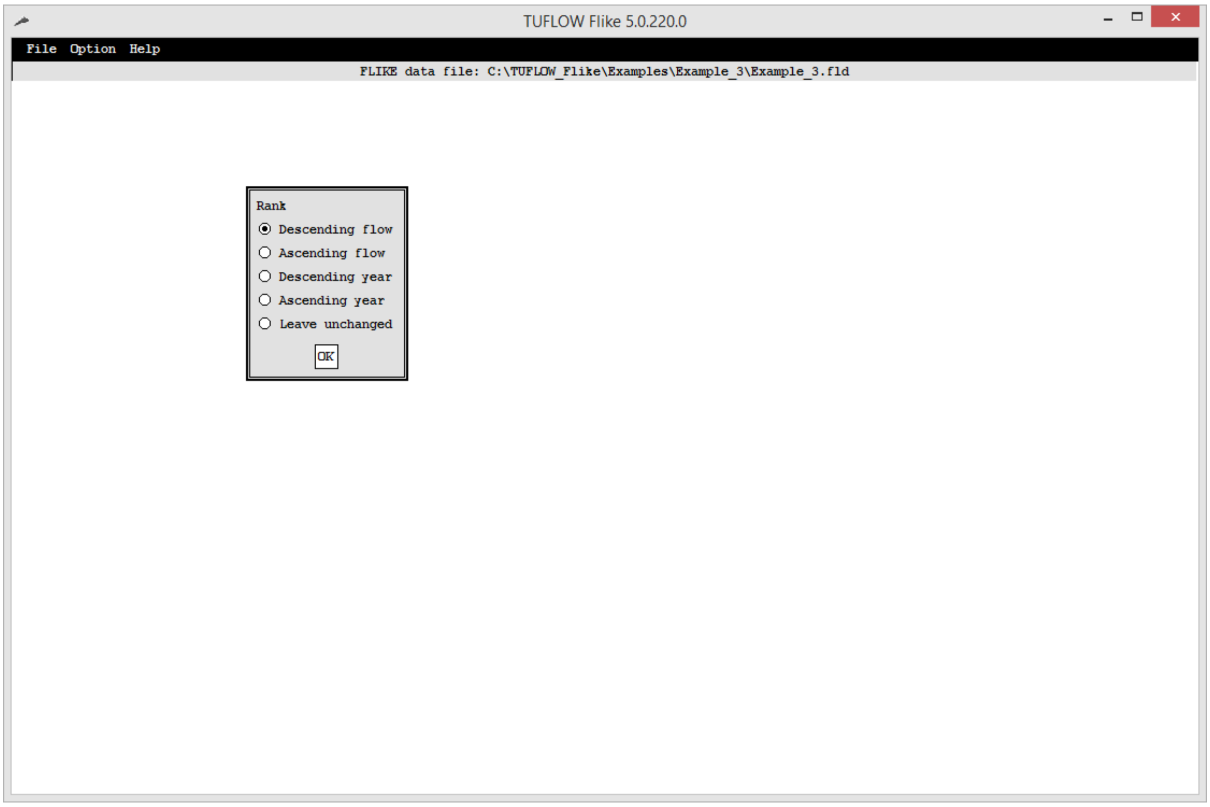


Figure 3.2.21. Rank Data Screen

2.8.3.5. Configure the distribution and fit method

Now that the data has been imported the statistical distribution can be fitted to the data. To do this, select the **General** tab. As noted above, for this example the Log Pearson Type III distribution will be fitted using the Bayesian Inference method.

Before configuring the model it is worthwhile checking that TUFLOW Flike has interpreted the data correctly. The number of observed data is reported in the **Number of observed data** text box. In this case the number of observations or length of the data series is 31 as shown in Figure 3.2.22. Before continuing, check that this is the case.

Next, select the probability model; the Log Pearson Type III. To do this ensure that the radio button next to the text **Log Pearson Type III (LP3)** is selected (this is the default) as in Figure 3.2.22.

The final task is to choose the fitting method. In this example the Bayesian Inference method will be used. To do this, ensure that the radio button next to **Bayesian with** is selected and the radio button next to **No prior information** is selected as shown in Figure 3.2.22. Again, both of these are the defaults.

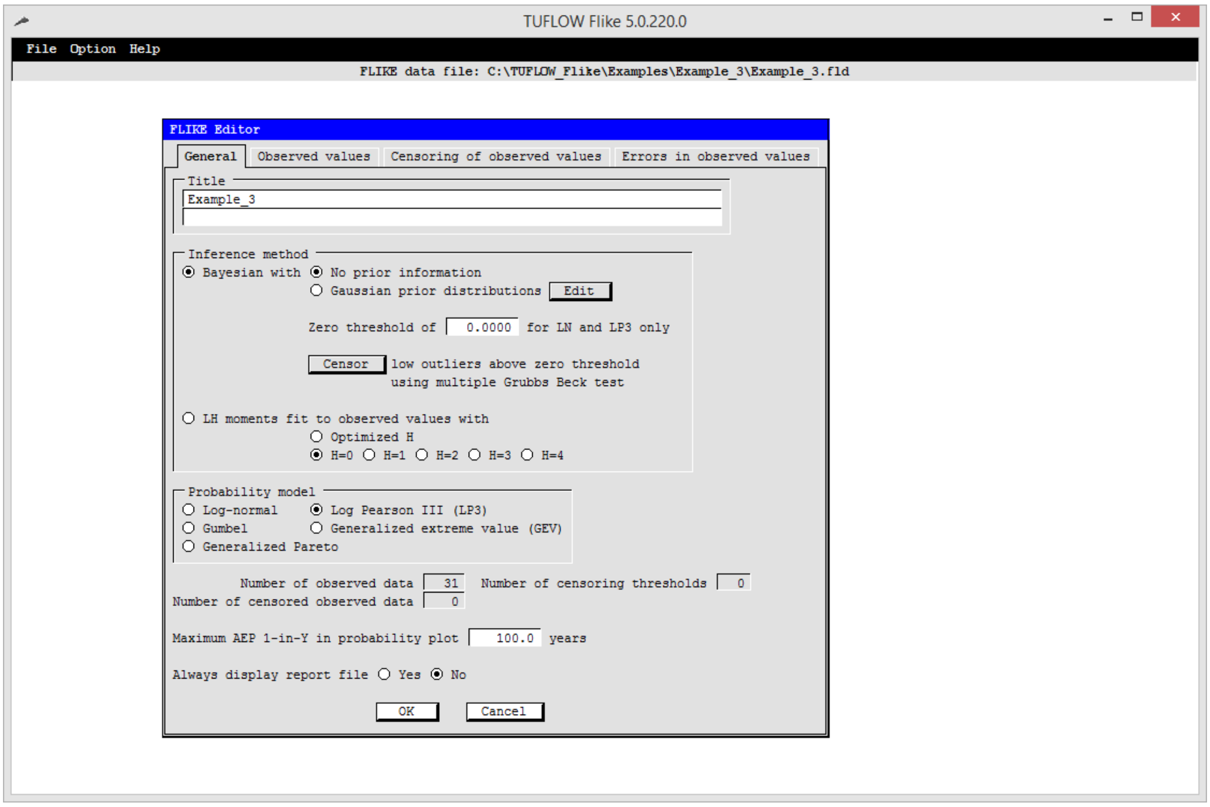


Figure 3.2.22. General Screen – After Data Import

2.8.3.6. Running TUFLOW Flike and accessing Results

TUFLOW Flike presents the results in two ways:

- As a visual plot; and
- In a text based report file.

Both of these will be explored in this example and both should be consulted when undertaking a Flood Frequency Analysis. Before we proceed with this example the length of the x-axis in the plot needs to be specified; that is, the lowest probability (rarest event) to be displayed. It is recommended to always enter a value greater than the 1 in Y AEP event that you are interested in. This is specified in the **Maximum AEP 1 in Y in probability plot ___ years** text box. In this example, enter the 1 in 200 year AEP event as shown in Figure 9 [http://localhost:8889/Fit.model.png]. By default the plot window automatically launches when a distribution is fitted.

In addition to the plot window a report file can also be automatically launched in a text editor. This can be quite helpful when you are developing a model, as it allows you to more readily compare the results. To do this select the appropriate radio button next to **Always display report file** as shown in Figure 3.2.22.

2.8.3.7. Run TUFLOW Flike

Now that the data has been imported, the distribution selected, the fit method configured and the output configured TUFLOW Flike is ready to run. To fit the model select **OK** on the **General** tab and this will return you to the **TUFLOW Flike** window, which will look quite empty as in Figure 3.2.23. In this window, select the **Option** dropdown menu and choose

Fitmodel. This will run TUFLOW-Flike and present you with a **Probability Plot** as well as opening the **Report File** in a text editor.

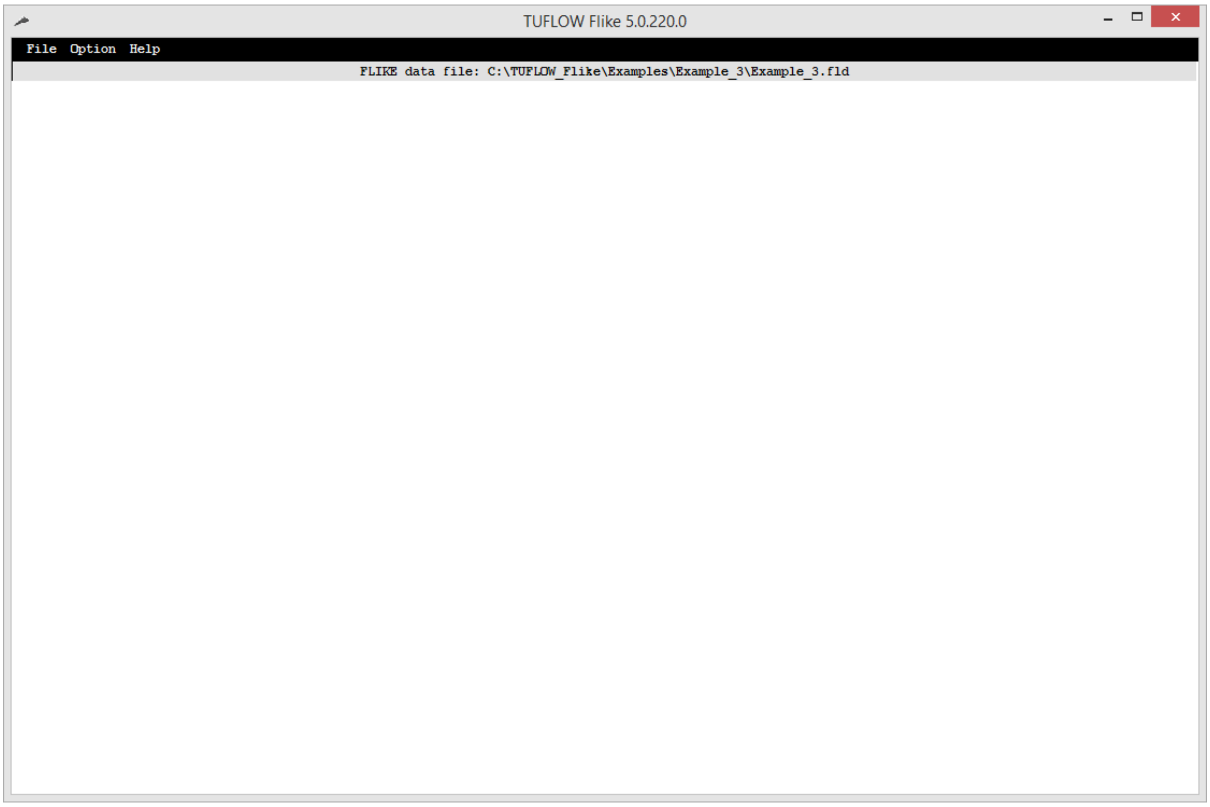


Figure 3.2.23. Blank TUFLOW-Flike Screen

2.8.3.8. Reviewing the results

When TUFLOW-Flike has finished fitting the distribution to the input data, a plot screen will appear similar to Figure 3.2.24 and the results file will be shown in the default text editor as in Figure 3.2.25.

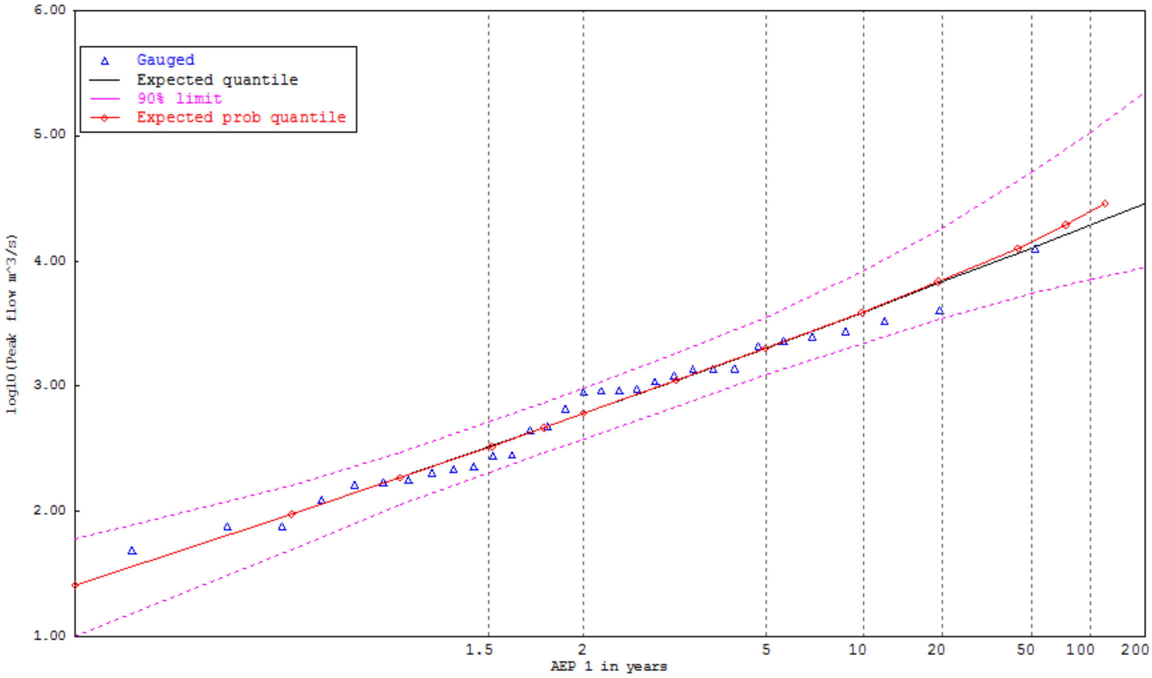
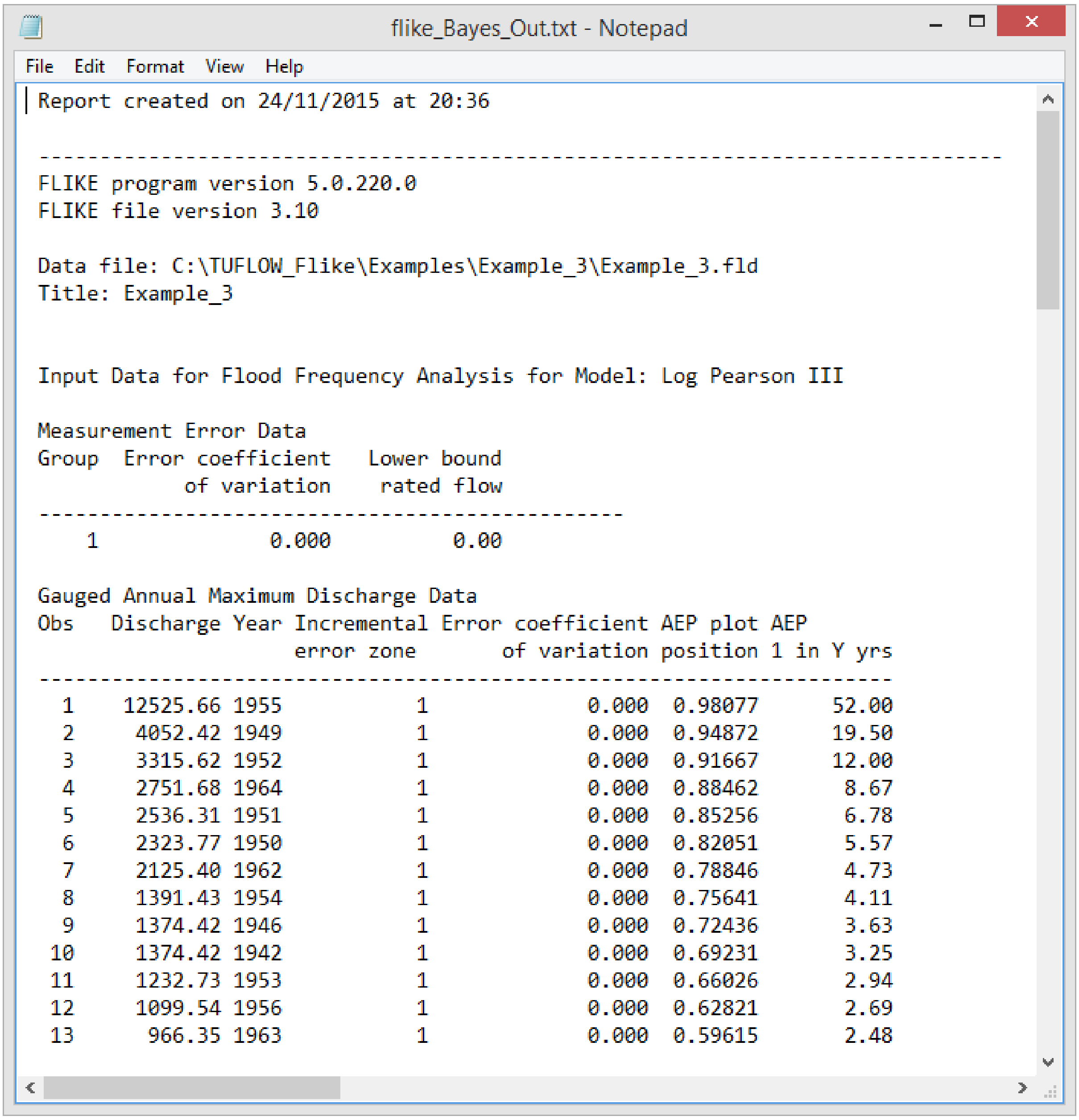


Figure 3.2.24. Probability Plot



flike_Bayes_Out.txt - Notepad

File Edit Format View Help

```
Report created on 24/11/2015 at 20:36

--------------------------------------------------------------------------------
FLIKE program version 5.0.220.0
FLIKE file version 3.10

Data file: C:\TUFLOW_Flike\Examples\Example_3\Example_3.fld
Title: Example_3


Input Data for Flood Frequency Analysis for Model: Log Pearson III

Measurement Error Data
Group   Error coefficient   Lower bound
           of variation     rated flow
------------------------------------------------
    1             0.000          0.00

Gauged Annual Maximum Discharge Data
Obs   Discharge Year Incremental Error coefficient AEP plot AEP
                     error zone       of variation position 1 in Y yrs
----------------------------------------------------------------------
  1    12525.66 1955           1           0.000  0.98077      52.00
  2     4052.42 1949           1           0.000  0.94872      19.50
  3     3315.62 1952           1           0.000  0.91667      12.00
  4     2751.68 1964           1           0.000  0.88462       8.67
  5     2536.31 1951           1           0.000  0.85256       6.78
  6     2323.77 1950           1           0.000  0.82051       5.57
  7     2125.40 1962           1           0.000  0.78846       4.73
  8     1391.43 1954           1           0.000  0.75641       4.11
  9     1374.42 1946           1           0.000  0.72436       3.63
 10     1374.42 1942           1           0.000  0.69231       3.25
 11     1232.73 1953           1           0.000  0.66026       2.94
 12     1099.54 1956           1           0.000  0.62821       2.69
 13      966.35 1963           1           0.000  0.59615       2.48
```

Figure 3.2.25. Results File

When fitting a flood series to a probability distribution it is essential that the results are viewed and reviewed. This is most easily achieved by first viewing the results in the **Probability Plot**. If the **Probability Plot** window has been closed, it can be reopened by selecting the **Option** dropdown menu and then **Viewplot**. The plot contains information about the fit as well as the quantile values and confidence limits. Within the plot window the y-axis contains information on discharge (or log discharge depending on the Plot scale selected) and x-axis displays the Annual Exceedance Probability (AEP) in terms of 1 in Y years. The plot displays the:

- Log-Normal probability plot of the gauged flows with plotting position determined using the Cunnane plotting position, shown as blue triangles;

- X% AEP quantile curve (derived using the posterior mean parameters), shown as a black line;

- 90% quantile confidence limits shown as dashed pink lines; and
- The expected probability quantile, shown as a red line.

For the data contained in this example the resulting plot displays a good fit to the gauged data and appears to have tight confidence limits with all gauged data points falling within the 90% confidence limits; by default the figure plots the logarithm of the flood peaks. The plot can be rescaled to remove the log from the flow values. Select the **Plotscale** button and choose one of the non-log options, that is, either Gumble or Exponential and the uncertainty changes as in Figure 3.2.26. This will present a more sobering perspective on the model fit with the confidence limit appearing much larger for rarer flood quantiles. This can be confirmed by reviewing the results in the **Result file**. Table 3.2.6 presents a subset of the results found in the **Result file** of selected X% AEP quantiles qY and their 90% confidence limits. For example, for the 1% AEP flood, the 5% and 95% confidence limits are respectively 37% and 546% of the quantile qY! The 0.2% AEP confidence limits are so wide as to render estimation meaningless. Note the expected AEP for the quantile qY consistently exceeds the nominal X% AEP. For example, the 1% (1 in 100) AEP quantile of 19 572 m^3/s has an expected AEP of 1.35% (1 in 74).

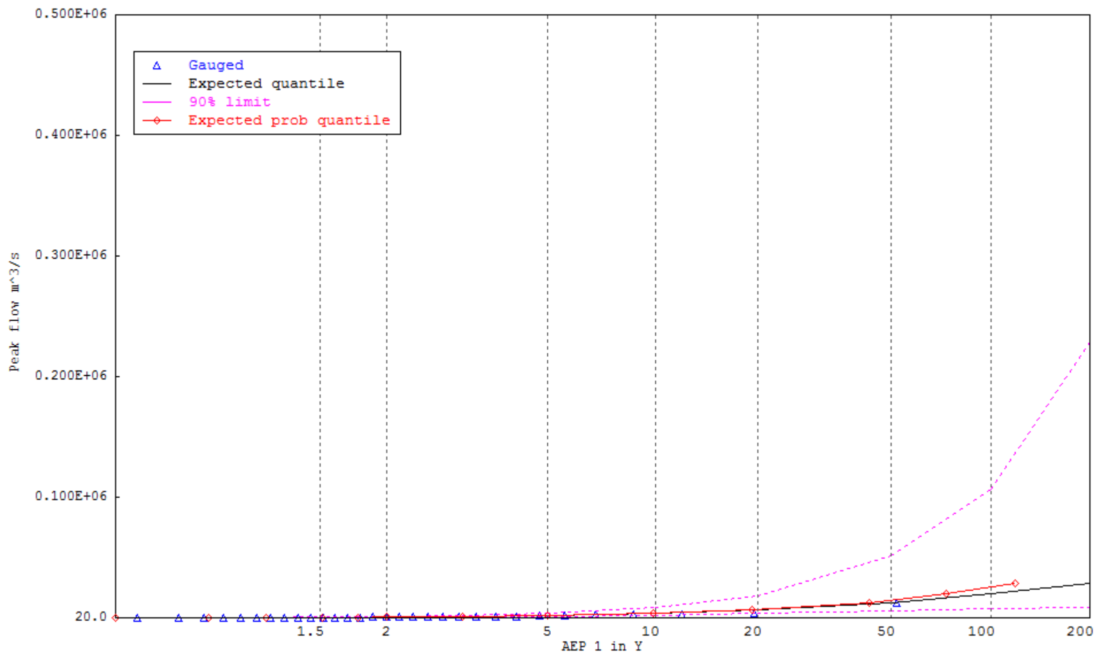


Figure 3.2.26. Probability Plot using Gumbel Scale

Table 3.2.6. Selected Results

1 in Y AEP	Quantile Estimate q_Y	Quantile confidence limits 5% limit	Quantile confidence limits 95% limit	Expected 1 in Y AEP for q_Y
10	3929	2229	8408	10.1%
50	12 786	5502	51 010	2.32%
100	19 572	7188	107 122	1.36%

1 in Y AEP	Quantile Estimate q_Y	Quantile confidence limits 5% limit	Quantile confidence limits 95% limit	Expected 1 in Y AEP for q_Y
500	47 034	11 507	570 635	0.48%

Table 3.2.7. Gauged flows on the Hunter River at Singleton

Year	Flow (m^3/s)	Year	Flow (m^3/s)	Year	Flow (m^3/s)	Year	Flow (m^3/s)
1938	76.26	1946	1374.42	1954	1391.43	1962	2125.4
1939	171.87	1947	280.18	1955	12525.66	1963	966.35
1940	218.21	1948	202.62	1956	1099.54	1964	2751.68
1941	668.79	1949	4052.42	1957	447.75	1965	49.03
1942	1374.42	1950	2323.77	1958	478.92	1966	76.51
1943	124.12	1951	2536.31	1959	180.52	1967	912.5
1944	276.3	1952	3315.62	1960	164.36	1968	926.67
1945	895.5	1953	1232.73	1961	229.54		

2.8.4. Example 4: Use of binomial censored historical data

This example is a continuation of **Example 3** and it examines the benefit of using historical flood information. In the previous example the gauged record spanned the period 1938 to 1968. The biggest flood in that record occurred in 1955 with a discharge of 12 526 m^3/s. An examination of historic records indicates that during the ungauged period 1820 to 1937 there was only one flood that exceeded the 1955 flood and that this flood occurred in 1820. The information for the 1820 flood is not from a stream gauge; rather it is from a variety of sources including newspaper articles. This information is valuable, perhaps the most valuable, even though the magnitude of the 1820 flood is not reliably known. This information can be incorporated into a Bayesian approach. The way that this is done in TUFLOW Flike is through censoring data.

From the information about the flood history at Singleton we can make the following conclusions:

- Over the ungauged period 1820 to 1937 there was:
  - One flood above the 1955 flood; and
  - 117 floods below the 1955 flood.

Note that the ungauged record length is 118 years, that is, all years from 1820 to 1937 are included as it is assumed each year has an event. Also, note that the ungauged period cannot overlap with the gauged period.

2.8.4.1. Launch TUFLOW-Flike

As in **Example 3** launch TUFLOW Flike; however, this time open the **.fld** file previously created: **Example_3.fld**. This file will be used as it contains the data that are needed for this example. To do this select the File dropdown menu and then select **Open**. Navigate to the **Example_3.fld** in the next dialogue box and open the file. The **Flike Editor** window will then appear containing all the information from Example 3.

2.8.4.2. Save Example_4.fld

The next step is to save the **Example_4.fld** file as a new file. It is best to do this immediately to ensure that no data is overwritten. To do this, select **OK** from the **Flike Editor** window which will return to the main **TUFLOW Flike** window. Select **File** again and then **Saveas**. Save the file as **Example_4.fld** in a new folder called **Example 4.**

2.8.4.3. Enter Historical Flood Information

In this step the historical flood information is entered. To edit the **Example_4.fld** data from the **TUFLOW-Flike** window select **Options** and then **Edit data**. This reopens the Flike Editor window. Now select the **Censoring of observed values** tab and this will open a window similar to Figure 3.2.27 with no data.

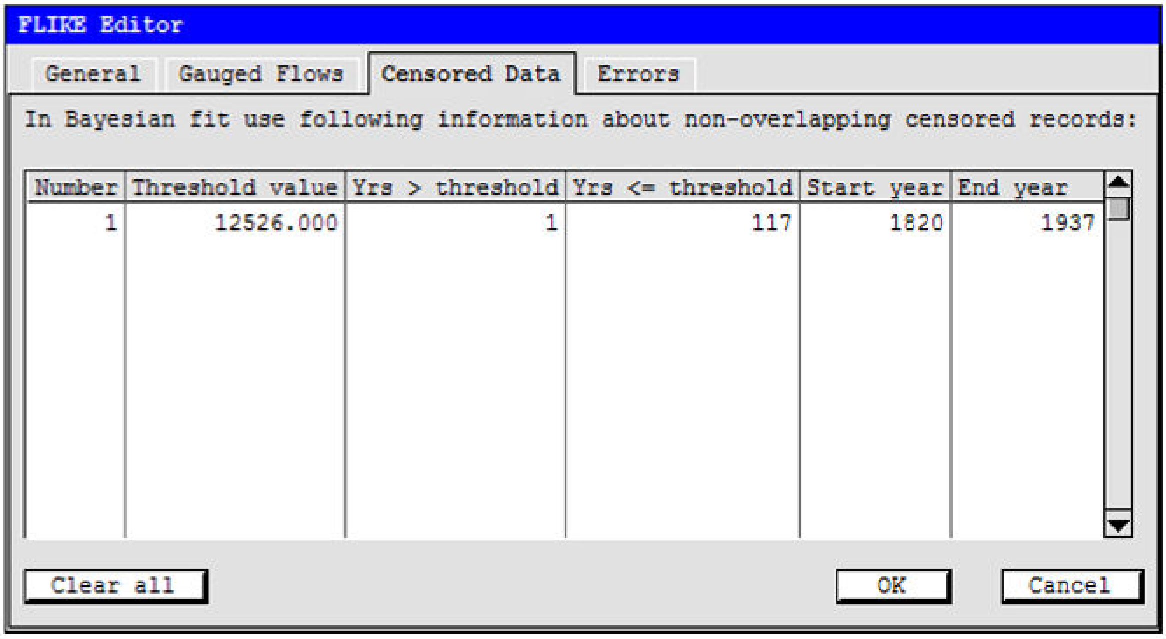


Figure 3.2.27. Censoring observed values tab

The historical data needs to be entered into the **Censoring of observed values** tab, that is, we need to let TUFLOW-Flike know that there has been one flood greater than the 1955 flood between 1820 and 1937. So:

- The **Threshold value** is 12 526m^3/s - the size of the 1955 flood.
- The Years greater than the threshold (**Yrs > threshold**) is one (1) – the 1820 flood.
- The Years less than or equal to the threshold (**Yrs <= threshold**) is 117 – there were 117 years between 1820 and 1937 with flood less than the 1820 flood.
- The **Start Year** is 1820; and

  The **End Year** is 1937.

Once the data has been entered, select OK which will return the main **TUFLOW-Flike** window. TUFLOW-Flike preforms some checks of the data to ensure that it has been entered correctly. However, these are only checks and it is up to the user to ensure they have correctly configured the historic censoring.

Return to the **General** tab by selecting **Options** and then **Edit data** and it should appear as in Figure 3.2.28. **Note the Number of censoring thresholds** text field has been populated with the number 1, so TUFLOW-Flike has recognised that there censoring has been configured.

As with the previous example, check that the **Always display report file** radio button has been selected.

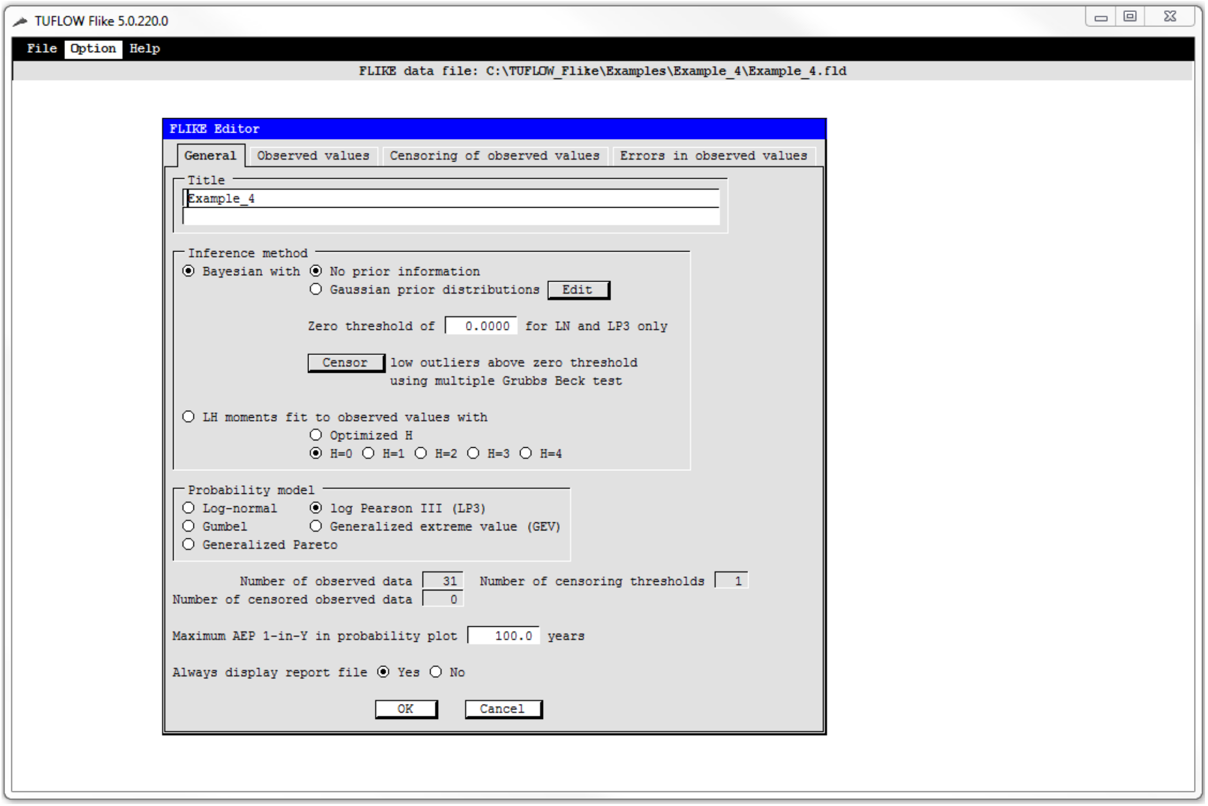


Figure 3.2.28. Configured Flike Editor

2.8.4.4. Run TUFLOW-Flike with Historic Censoring Data

On the general tab select **OK** and return to the **TUFLOW-Flike** window. As in the previous exercise select **Option** and then **Fit model**. This will run TUFLOW-Flike and when the engine has finished the **Probability Plot** will open together with the **Report File**.

2.8.4.5. Results

Table 3.2.8 presents the posterior mean, standard deviation and correlation for the Log Pearson Type III parameters: *m*, *loges* and *g* which are respectively the mean, standard deviation and skewness of loge(q) taken from the **Report File**. Comparison with Example 3 reveals the censored data have reduced by almost 17% the uncertainty in the skewness (*g*) parameter. This parameter controls the shape of the distribution, particularly in the tail region where the floods of interest are.

Table 3.2.8. Posterior Mean, Standard Deviation and Correlation for the LP III

LP III Parameter	Mean	Std. Deviation	Correlation		
m	6.365	0.237	1.000		
$\log_e s$	0.303	0.120	-0.236	1.000	
g	-0.004	0.405	-0.227	-0.409	1.000

The resulting **Probability plot** is shown in Figure 3.2.30. This figure displays on a log normal probability plot the gauged flows, the X% AEP quantile curve (derived using the posterior mean parameters), the 90% quantile confidence limits and the expected probability curve. Compared with Example 3 the tightening of the confidence limits is noticeable.

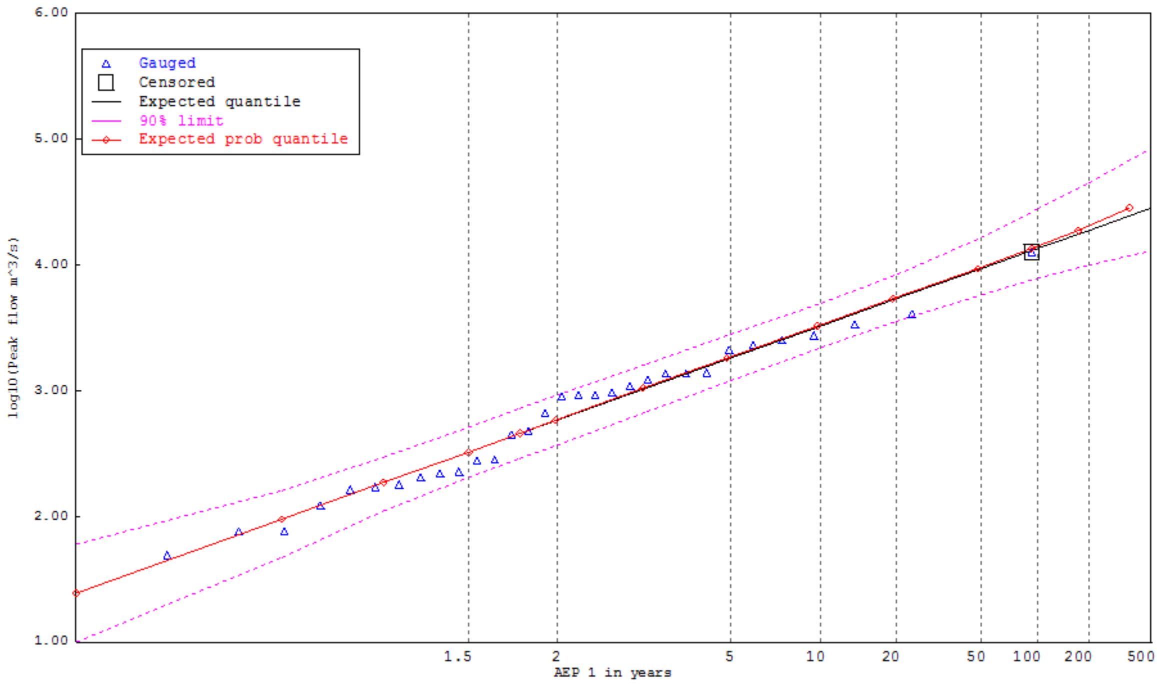


Figure 3.2.29. Probability plot of the Singleton data with historic information

The following table (Table 3.2.9) of selected 1 in Y AEP quantiles qY and their 90% confidence limits illustrates the benefit of the information contained in the historic data. For example, for the 1% AEP flood the 5% and 95% confidence limits are respectively 58% and 205% of the quantile qY! This represents a major reduction in quantile uncertainty compared with Example 3 which yielded limits of 38% and 553%. This is illustrated in graphically Figure 3.2.30.

Table 3.2.9. Comparison of Selected Quantiles with 90% Confidence Limits

1 in Y AEP	Quantile Estimate q_Y	Quantile Confidence Limits 5% Limit	Quantile Confidence Limits 95% Limit	Expected 1 in Y AEP for q_Y
10	3294	2181	4947	10.37%
50	9350	5778	16 511	2.09%

1 in Y AEP	Quantile Estimate q_Y	Quantile Confidence Limits 5% Limit	Quantile Confidence Limits 95% Limit	Expected 1 in Y AEP for q_Y
100	13 511	7785	27 687	1.08%
500	28 542	12 966	85 583	0.28%

Note that **Report File** presents the Expected AEP in 1 in Y years whereas Table 3.2.9 presents as the Expected AEP as a percentage.

This example highlights the significant reductions in uncertainty that historical data can offer. However, care must be exercised to ensure the integrity of the historic information – see Book 3, Chapter 2, Section 3 for more details.

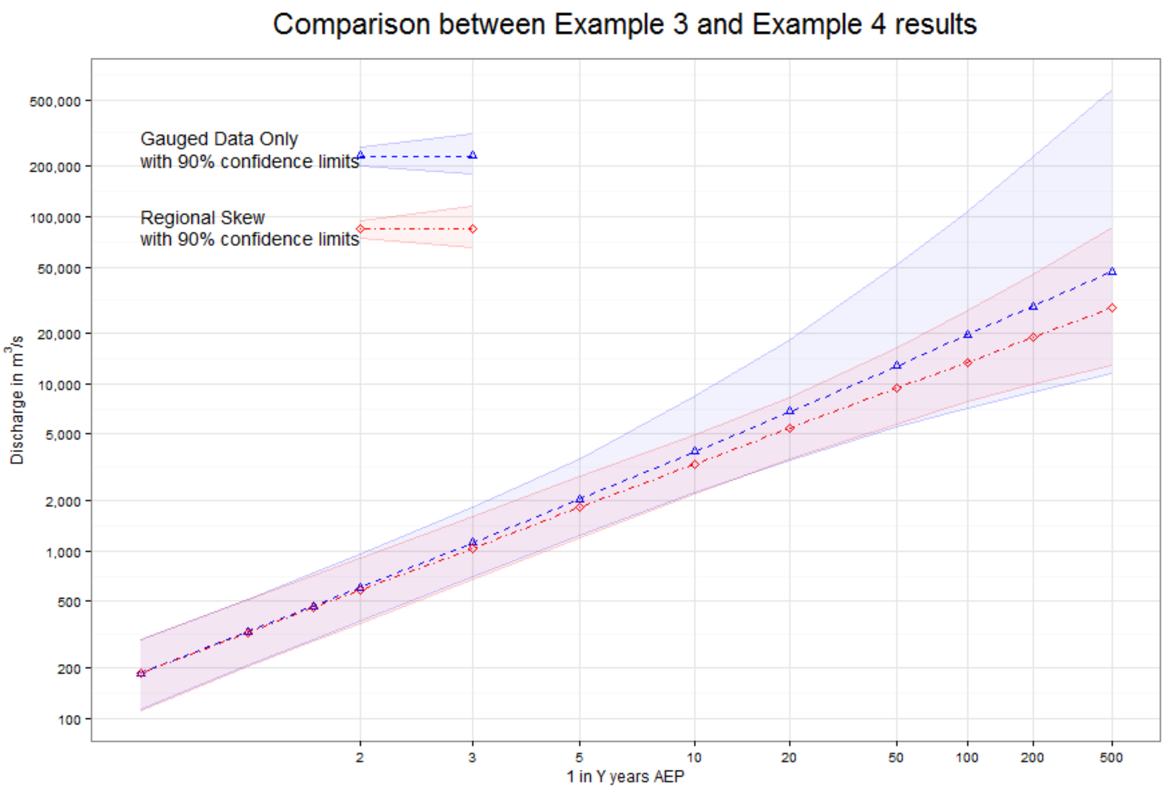


Figure 3.2.30. Probability plot of the Singleton data with historic information

2.8.5. Example 5: Use of regional information

In this example the use of regional parameter information is explored, building on Example 3. As was shown in Example 3, there was significant uncertainty in the skewness parameter. In that example, the posterior mean of the skewness was estimated to be 0.131 with a posterior standard deviation of 0.479. This led to significant uncertainty in the quantile estimates, for instance the 5% and 95% confidence limits for the 1% AEP quantile were 37% and 546% respectively of the 1% AEP quantile. This example shows how the use of regional information can reduce, sometimes significantly, the uncertainty of quantile estimates. Details on the use of regional information can be found in Book 3, Chapter 2, Section 3 and Book 3, Chapter 2, Section 6.

In this hypothetical example, a regional analysis of skewness has been conducted and the expected regional skew was found to be 0.00 with a standard deviation of 0.30. This information can be incorporated into the Bayesian analysis undertaken by TUFLOW Flike as shown in this example.

2.8.5.1. Launch TUFLOW Flike

The Singleton data from the previous examples will be used in this example, so as in Example 4 launch TUFLOW Flike and open the **.fld** file created in Example 3. Save the opened **.fld** as, say, **Example_5.fld**.

2.8.5.2. Enter Prior Information

The next step will be to enter the prior information, that is the regional information on skew. To do this, select **Edit data**from the **Options** menu. As before, this opens the **Flike Editor**. To enter the prior regional information, check the **Gaussian prior distributions** radio button and then click on the Edit button as shown in Figure 3.2.31. This will open the **Prior for Log-Pearson III** window as shown in Figure 3.2.32.

The regional skewness (0.00) is entered into the **Mean Skew of log Q** text box and the standard deviation of the regional skew (0.300) is entered into the **Standard Deviation Skew of log Q** as shown in Figure 3.2.32. Note in practice careful attention to the units being used is required.

Very large prior standard deviations are assigned to the **Mean of log Q** and **Standard deviation of log** Q parameters to ensure there is no prior information about theses parameters. If the *Log Pearson III* distribution has been selected, the option to import the prior information from the ARR Regional Flood Frequency Estimation method is available (Book 3, Chapter 3).

Select **OK** to return to the **Flike editor** window.

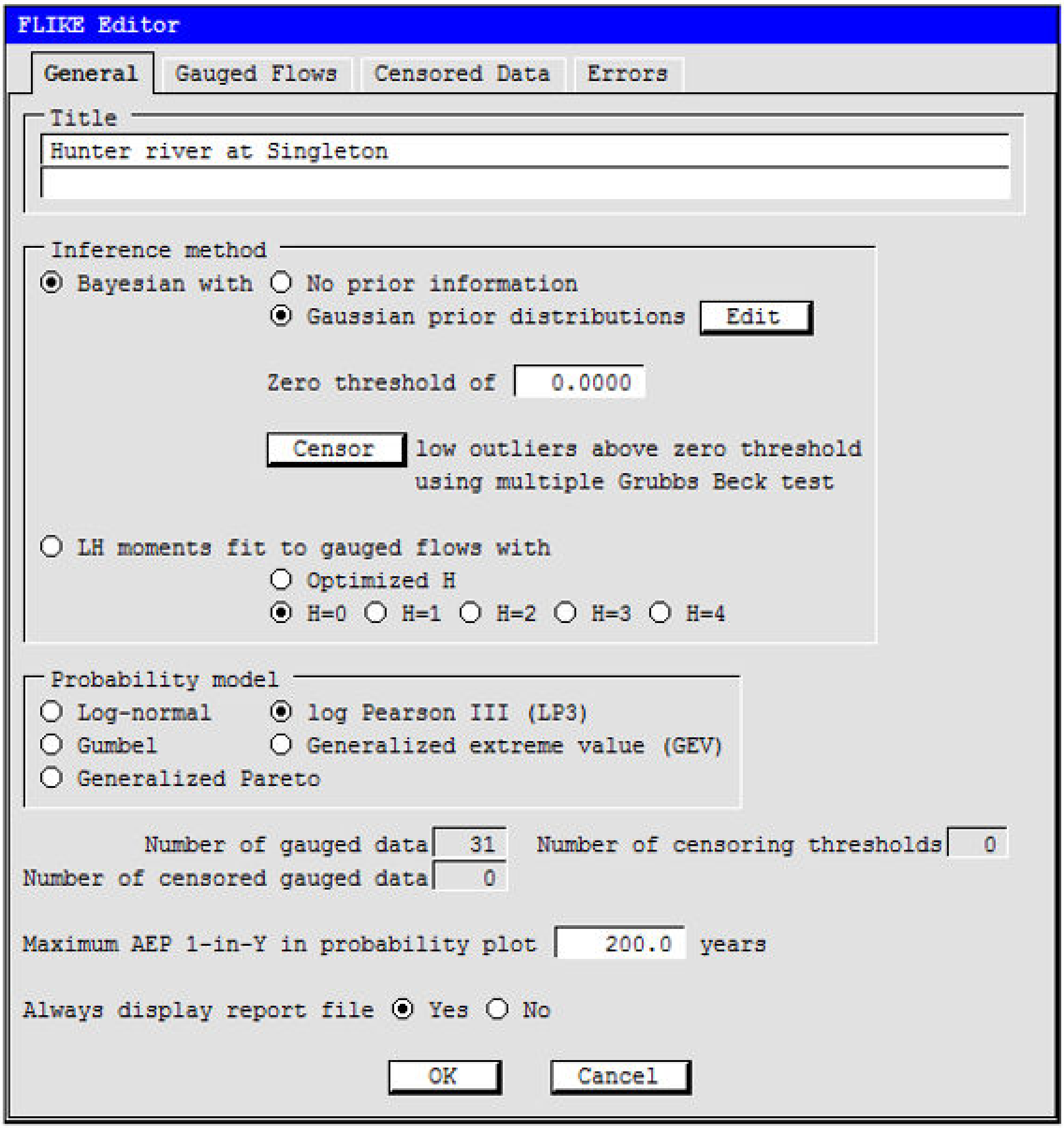


Figure 3.2.31. Gaussian prior distributions

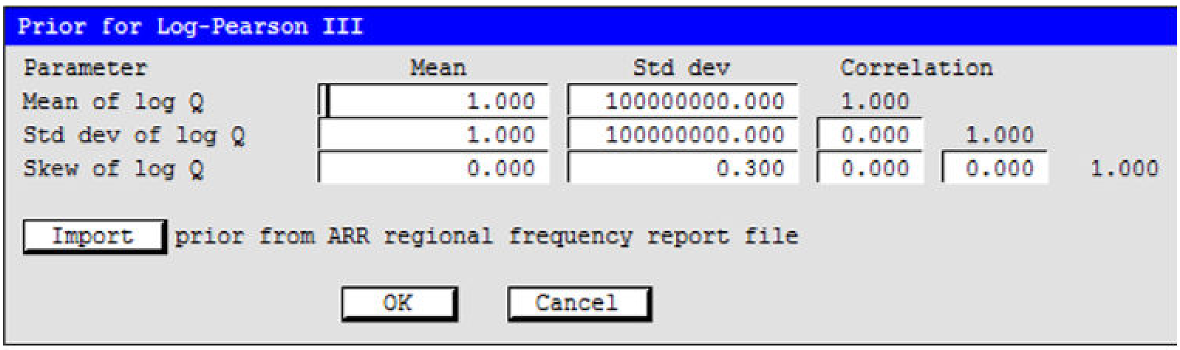


Figure 3.2.32. Prior for Log-Pearson III window

2.8.5.3. Run TUFLOW Flike with Regional Information

As in the previous examples select **OK**from the **Flike Editor** window to return to the main **TUFLOW Flike** window and select **Fit model** from the **Options menu** to run TUFLOW Flike. This should result in the Probability plot as shown in Figure 3.2.33.

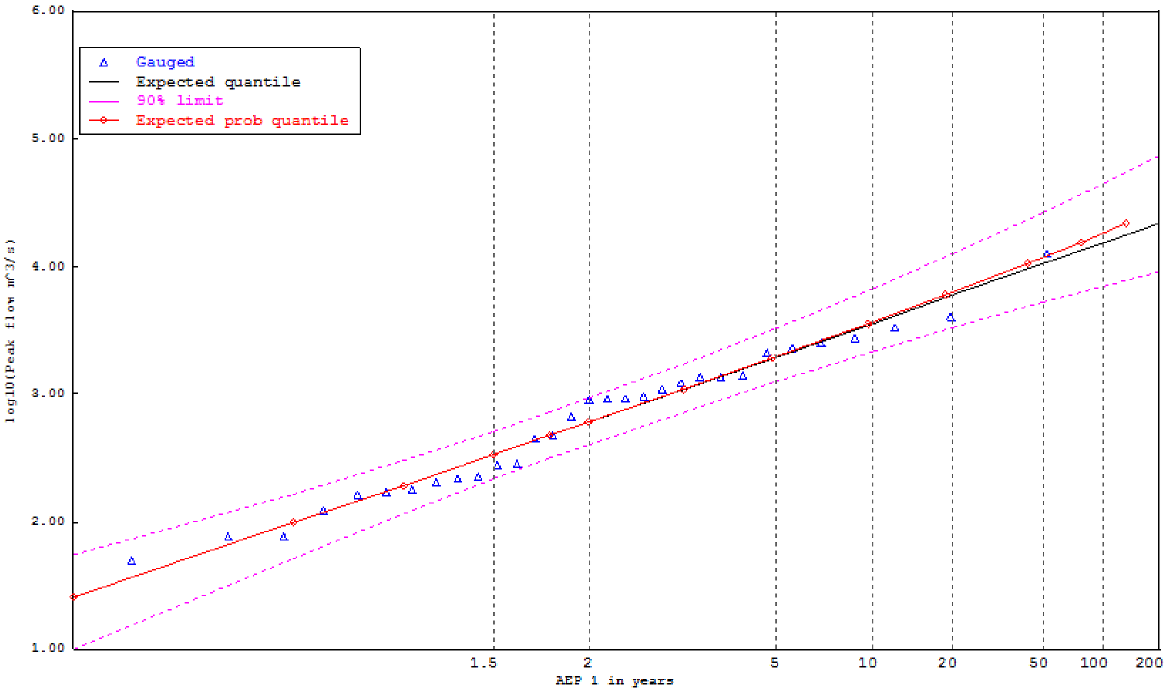


Figure 3.2.33. Probability plot of with prior regional information

Figure 3.2.33 presents the probability plot for the LP III model fitted to the gauged data with prior information on the skewness. Comparison of the results from this example with the results from Example 3 (see Figure 3.2.34) reveals substantially reduced uncertainty in the right hand tail.

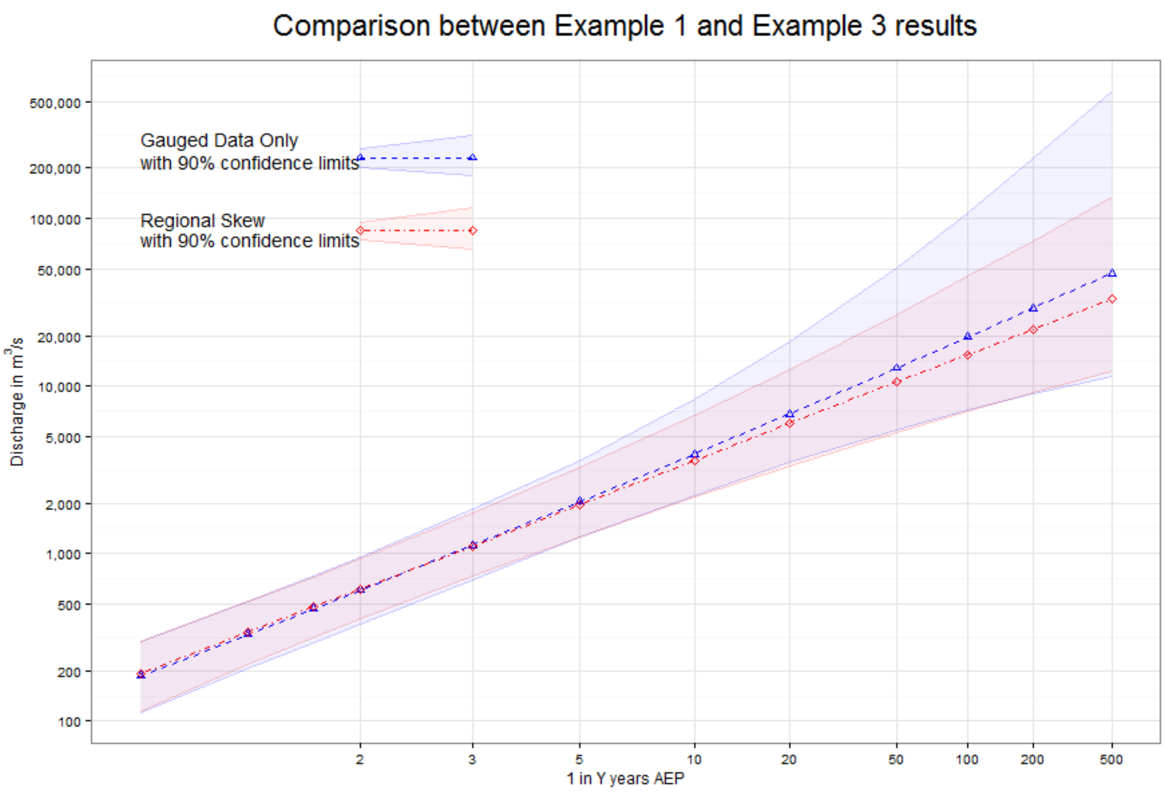


Figure 3.2.34. Comparison between the results from Example 3 and Example 5

Table 3.2.10. Comparison of LP III Parameters with and without prior information

LP III Parameter	No Prior Information		With Prior Information	
	Mean	Std. Deviation	Mean	Std. Deviation
m	6.433	0.262	6.421	0.251
$\log_e s$	0.353	0.144	0.320	0.131
g	0.131	0.479	0.019	0.261

Table 3.2.11 presents selected AEP quantiles qY and their 90% confidence limits. This table further illustrates the benefit of incorporating regional information. For example, for the 1% AEP flood the 5% and 95% confidence limits are respectively 37% and 546% of the quantile q1% when no prior information is used. These limits are reduced to 46% and 292%, respectively using prior regional information.

Table 3.2.11. Selected Results

AEP (%)	No Prior Information			With Prior Information		
	Quantile Estimate q_Y	Quantile Confidence 5% Limit	Quantile Confidence 95% Limit	Quantile Estimate q_Y	Quantile Confidence 5% Limit	Quantile Confidence 95% Limit
10%	3929	2229	8408	3598	2172	6702
2%	12 786	5 502	51 010	10 535	5 310	26 633
1%	19 572	7 188	107 122	15 413	7 093	45 087

AEP (%)	No Prior Information			With Prior Information		
	Quantile Estimate q_Y	Quantile Confidence 5% Limit	Quantile Confidence 95% Limit	Quantile Estimate q_Y	Quantile Confidence 5% Limit	Quantile Confidence 95% Limit
0.2%	47 034	11 507	570 635	33 365	12 244	134 107

2.8.6. Example 6: Censoring PILFs using multiple Grubbs-Beck test

In many Australian watercourses there are often years in which there are no floods. The annual maximum from those years are not representative of the population of floods and can unduly influence the fit of the distribution as discussed in Book 3, Chapter 2, Section 6. The flow values are referred to as Potentially Influential Low Flows (PILFs). It is recommended that in all flood frequency analyses the removal of these flows is investigated using the multiple Grubbs-Beck test to identify PILFs. The following example is taken from Pedruco et al. (2014) using data provided by the Wimmera Catchment Management Authority. The table at the end of this example lists 56 years of Annual Maximum discharges for the Wimmera River at Glynwylln. This data is included in the TUFLOW Flike download and was installed in the **data** folder in the install location of TUFLOW Flike. This location will be something similar to *C:\TUFLOW Flike\data\wimmeraGaugedFlows.csv*.

This example will examine the influence of PILFs and demonstrate how to use the multiple Grubbs-Beck test to safely remove them from the flood frequency analysis.

2.8.6.1. Launch TUFLOW Flike and Import Data

As in Example 3 launch TUFLOW Flike and create a new **.fld** file. Save the opened .fld as say, **Example_6.fld**. Import the Wimmera River data in the same way that the Singleton data was imported, ensuring that the structure of the data has been checked using the **View** button. The **Records** start in the second row (skip the first), **Years** are in column 1 and the **Gauged values** are in column 2. Configure the import options and import the data.

Once this has been done and the **Gauged values** have been ranked in descending order the Flike Editor window should look like Figure 3.2.35.

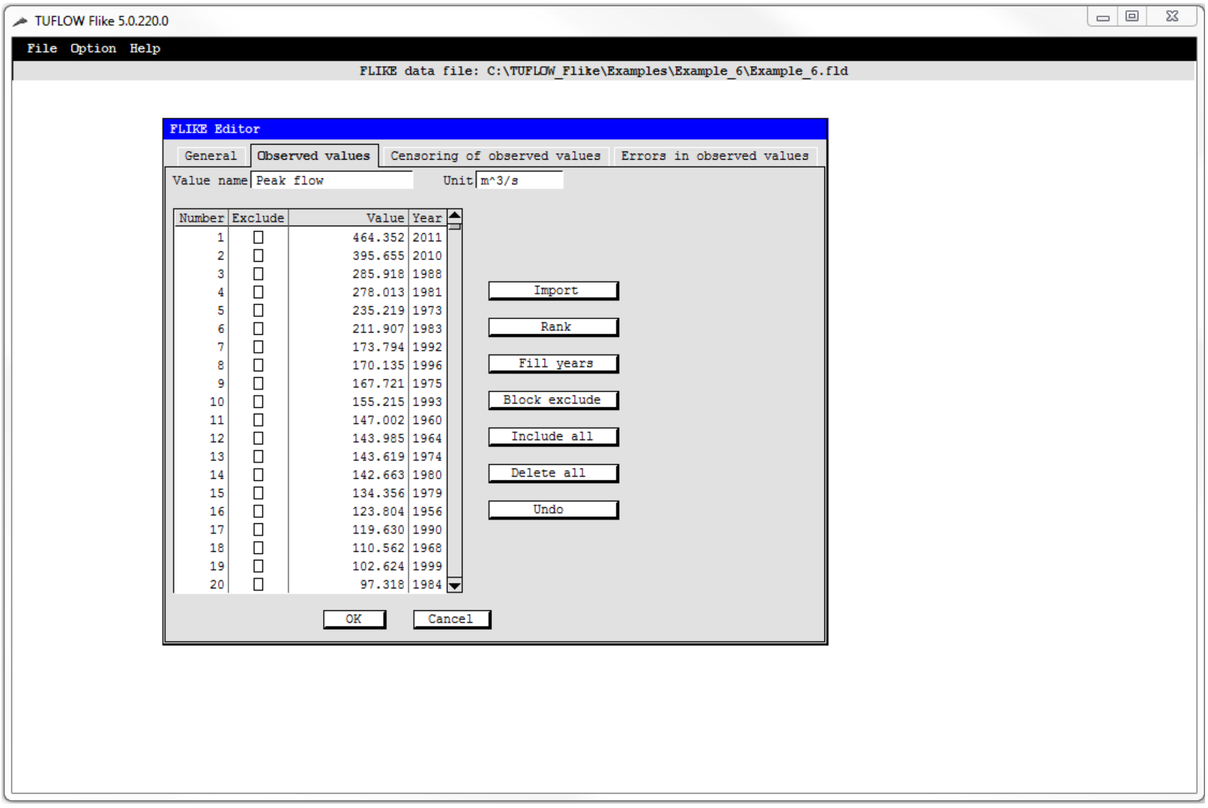


Figure 3.2.35. TUFLOW Flike editor window with Wimmera data

2.8.6.2. Fit Distribution

The Wimmera data will be fitted to a Generalised Extreme Value (GEV) distribution. To do this, return to the **Flike Editor General** tab and ensure that the following settings have been chosen:

- Bayesian inference method with **No prior information;**
- The **GEV** probability model; and
- The **Maximum AEP** is set to 200 years

Once these settings have been selected, select **OK** and run TUFLOW Flike in the usual way.

2.8.6.3. Initial Results

When TUFLOW Flike has run a new probability plot window will open. The plot will **not** look like Figure 3.2.36. To expose a better view of the distributions fit, the plot scale should be changed using the **Plot Scale** button from a **Gumbel** plot scale to a **Gumbel-log** plot scale and the y-axis rescaled using the **Rescale** button to have a minimum of 0.0 and a maximum of 4.0.

In Figure 3.2.36, the fit to the right-hand tail is not satisfactory. The expected quantiles are significantly greater than the gauged data, further the largest 3 data points fall outside of the lower 90% confidence limits.

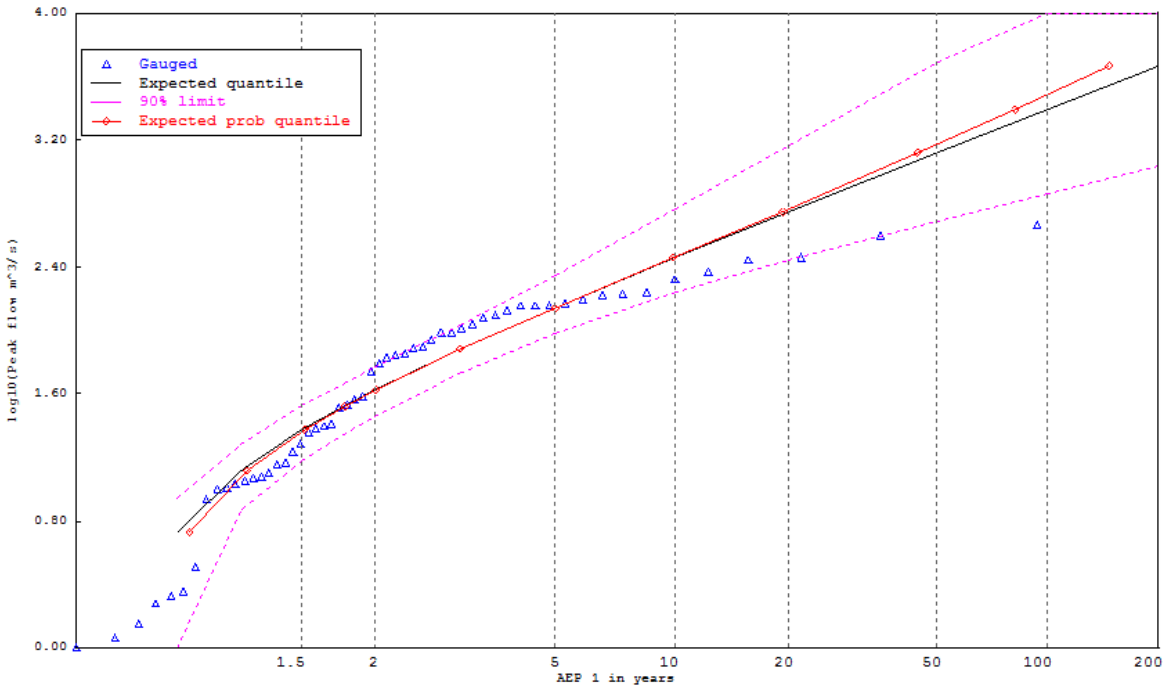


Figure 3.2.36. Initial probability plot for Wimmera data with GEV

2.8.6.4. Multiple Grubbs-Beck test

The fit of the distribution can be improved by removing PILFs. In TUFLOW Flike this can be done using the multiple Grubbs-Beck test, to do this, return to the **Flike Editor** window and select the **Censor** button. TUFLOW Flike will run the multiple Grubbs-Beck test on the Wimmera data and when finished it will return a window similar to the one shown in Figure 3.2.37. The multiple Grubbs-Beck test has detected 27 possible PILFs, select **Yes** to censor them.

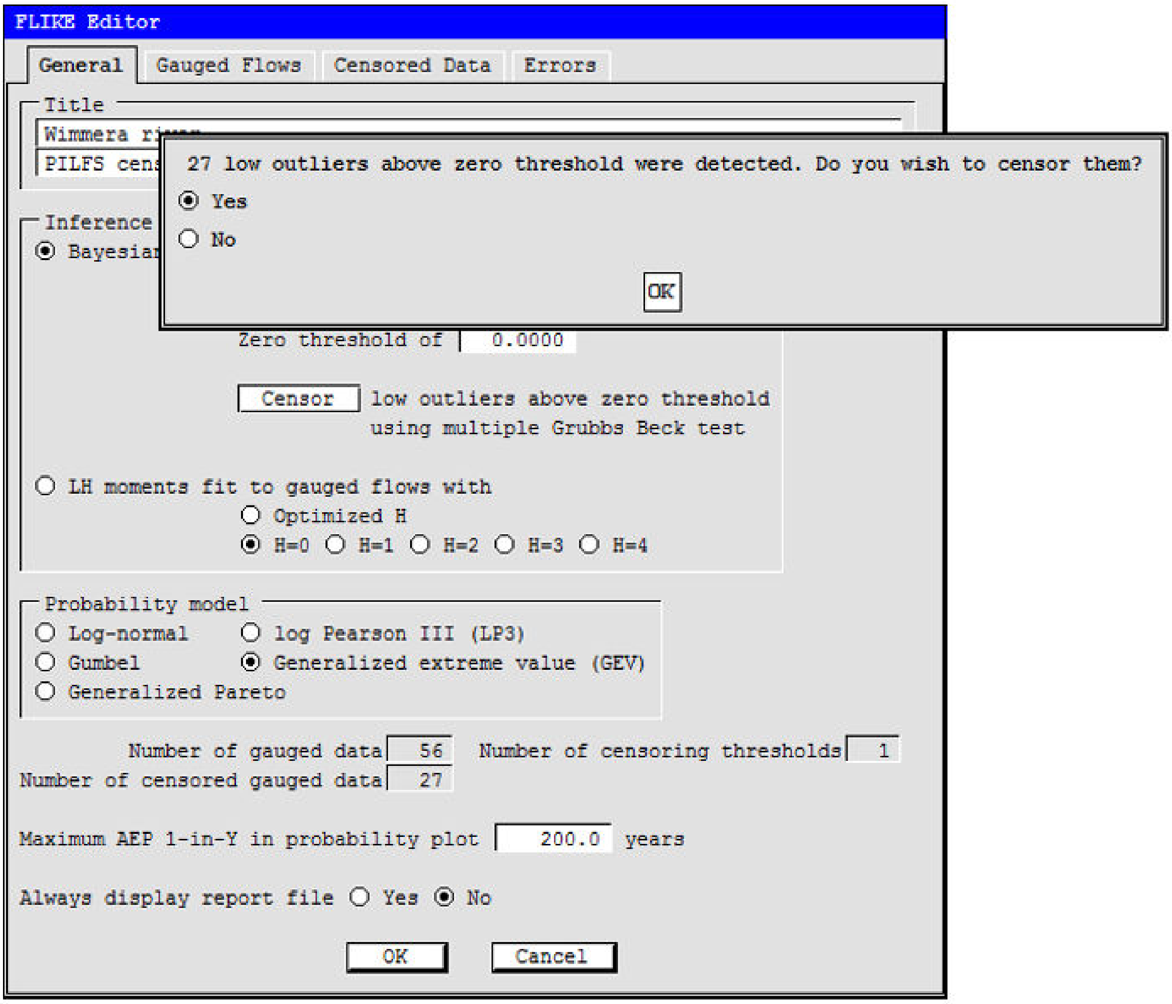


Figure 3.2.37. Results of the multiple Grubbs-Beck test

On agreeing to censor these flows, TUFLOW Flike automatically performs two changes to the inference setup:

1. The 27 lowest discharges are excluded from the calibration.

2. A censored threshold is added, with the information that there are 27 Annual Maximum discharges that lie below the threshold of 54.396m^3/s which corresponds the 28th ranked discharge.

These are further explained below

2.8.6.4.1. Excluded Data

The exclusion of the lowest 27 discharges can be seen in the **Observed Flows** tab of the **Flike Editor** as shown in Figure 3.2.38. In this tab all the values below the threshold have the **Exclude check** box crossed, this can be seen by scrolling down the window or by re-ranking the data and selecting **Ascending**. If you have re-ranked the data in ascending order re-rank it back into **Decesending** order.

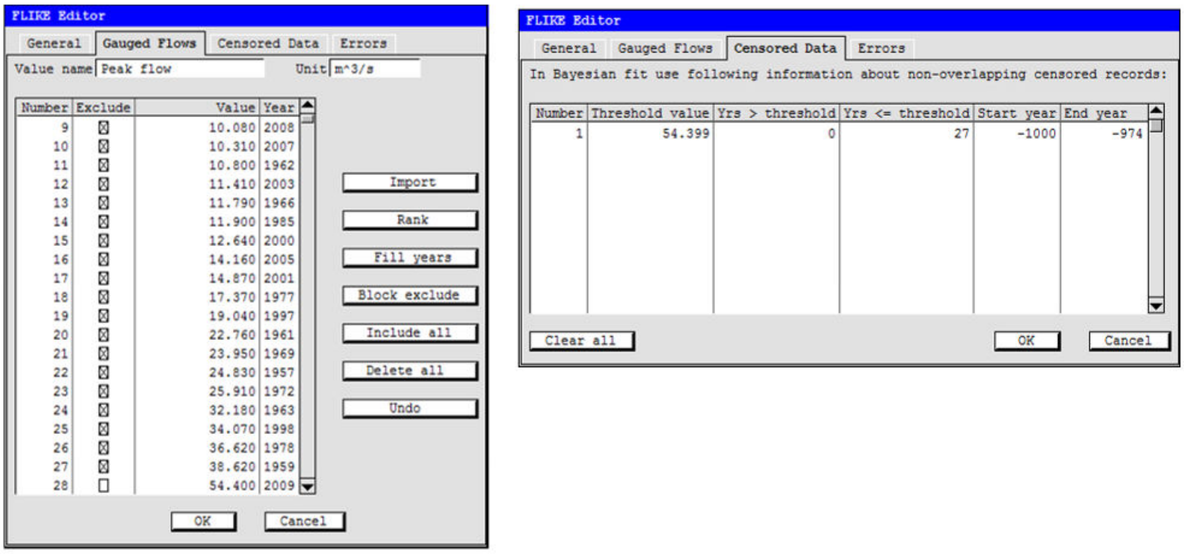


Figure 3.2.38. Excluded gauged values

2.8.6.4.2. Censoring Threshold

The addition of the censored threshold appears in the **Censoring of observed values** tab of the **Flike Editor** as shown in Figure 3.2.39. The **Threshold** value (54.396m^3/s) has been automatically populated together with the years that are greater than the threshold (0). The number of years less than the threshold (27) has also been populated. What this is telling TUFLOW Flike is that 27 years of discharges are less than the threshold are being censored; that is, gauged values are not considered but the frequency is. The **Start year** and **End year** are also populated with dummy year ranges beginning 1000BC. This is done to satisfy an automatic check in TUFLOW Flike designed to assist in the entry of historic data.

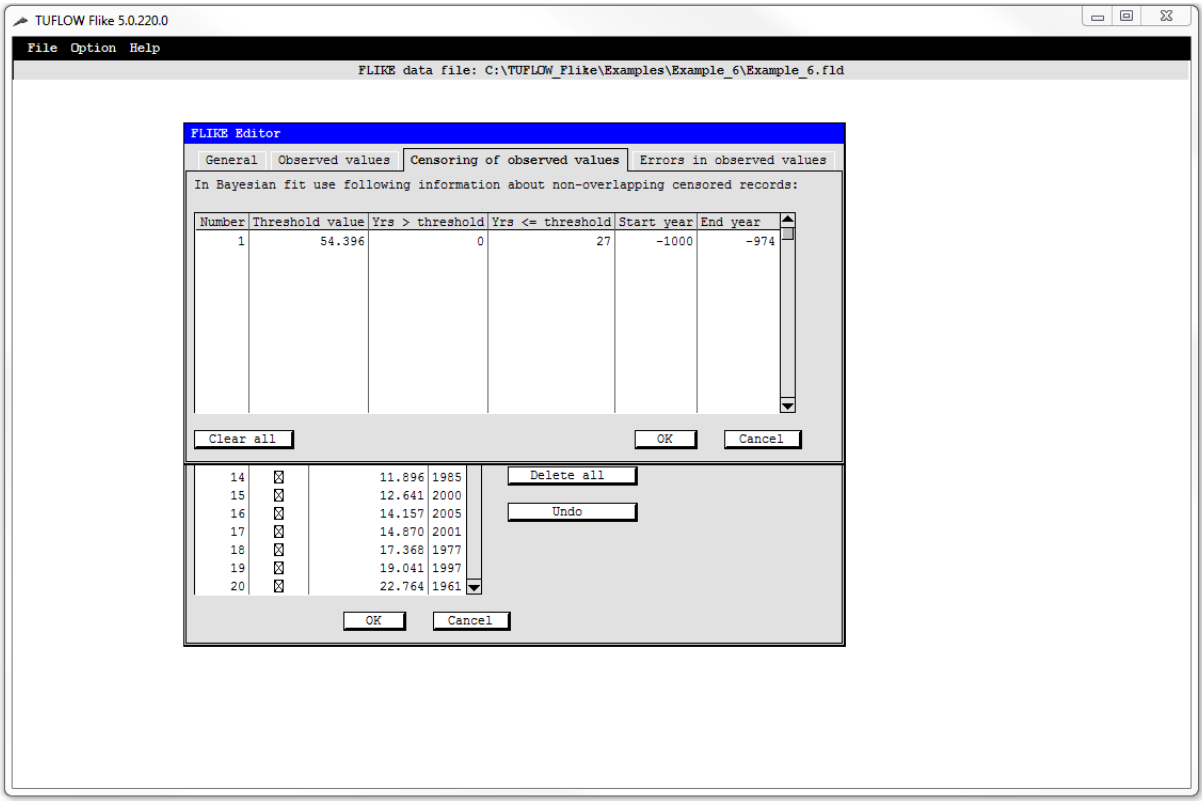


Figure 3.2.39. Censoring of observed values

2.8.6.5. Results using multiple Grubbs-Beck test

Return to the main **TUFLOW Flike** window and run TUFLOW FLIKE by selecting **Fit model**. As usual, a **Probability plot** window will automatically appear, as for the initial results change the plot scale to Gumbel-log and rescale the y-axis to have a minimum of 0.0 and a maximum of 4.0. The resulting plot will look like Figure 3.2.40.

A comparison of Figure 3.2.36 and Figure 3.2.40 shows the improved fit, in Figure 3.2.40 all of the gauged data points fall within the 90% confidence limits. Further, censoring the PILFs using the multiple Grubbs-Beck test has significantly altered the quantile estimates and reduced the confidence limits as shown in Table 3.2.12. For instance the quantile q1% when PILFs are excluded is around 21% of the initial estimate. The lower and upper confidence limits have been considerable reduced, initially they were 30% and 500% of the quantile q1% and following the removal of PILFs they became 68% and 220% of the quantile q1%.

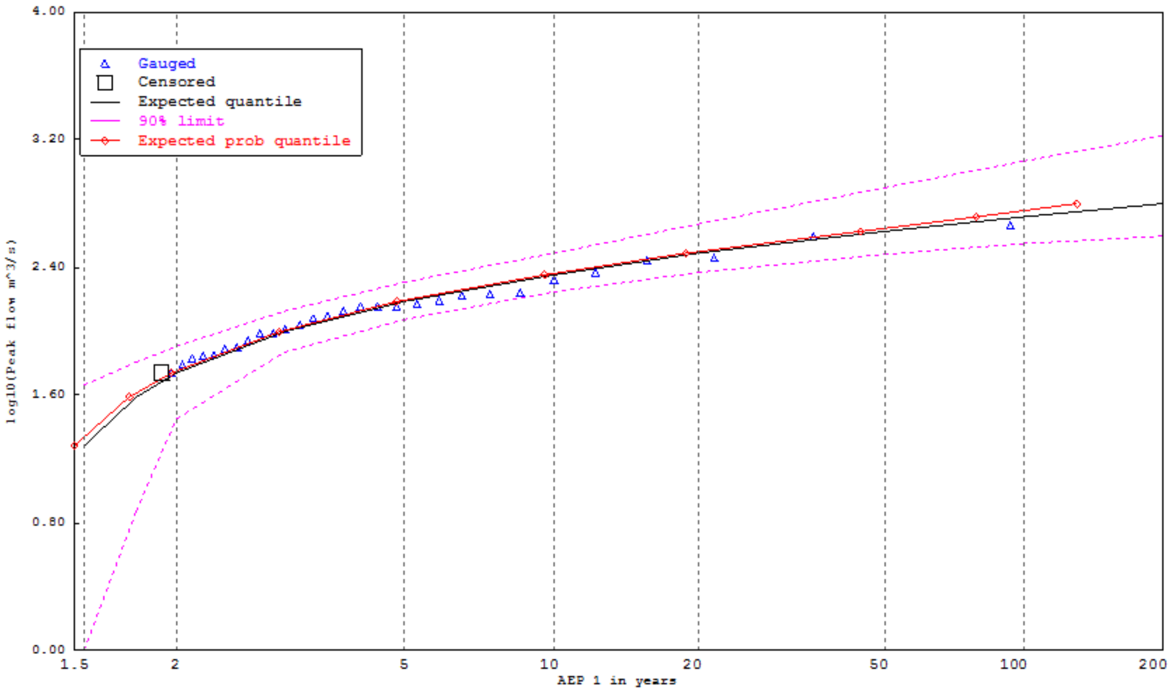


Figure 3.2.40. GEV fit - 56 years AM of gauged discharge - Using multiple Grubbs-Beck test

Table 3.2.12. Selected Results

AEP (%)	No Removal of PILFS			Removal of PILFS		
	Quantile Estimate q_Y	Quantile Confidence 5% Limit	Quantile Confidence 95% Limit	Quantile Estimate q_Y	Quantile Confidence 5% Limit	Quantile Confidence 95% Limit
10%	286	172	578	227	177	311
2%	1315	493	4975	423	304	784
1%	2481	737	12 398	521	354	145
0.2%	10696	1802	101 034	789	448	2813

Annual Maximum data for the Wimmera River at Glynwylln

464.35	167.72	119.63	71.4	32.18	14.16	8.52
395.65	155.22	110.56	69.67	25.91	12.64	3.22
285.92	147	102.62	67.49	24.83	11.9	2.28
278.01	143.99	97.32	61.64	23.95	11.79	2.13
235.22	143.62	96.78	54.4	22.76	11.41	1.9
211.91	142.66	87.98	38.62	19.04	10.8	1.43
173.79	134.36	79.15	36.62	17.37	10.31	1.16
170.13	123.8	77.03	34.07	14.87	10.08	0.01

2.8.7. Example 7: Improving poor fits using censoring of low flow data

The standard probability models such as GEV and LP III may not adequately fit flood data for a variety of reasons, for example Probable Influential Low Flow Flows (PILFs). In this example the censoring of data is used to censor low discharge data and improve the fit of the distribution to the data.

Often the poor fit of a distribution is associated with a sigmoidal probability plot as illustrated in Figure 3.2.41. In such cases a four or five-parameter distributions which have sufficient degrees of freedom can be used to track the data in both upper and lower tails of the sigmoidal curve. Alternatively a calibration approach that gives less weight to smaller floods can be adopted. The second approach is adopted in this example.

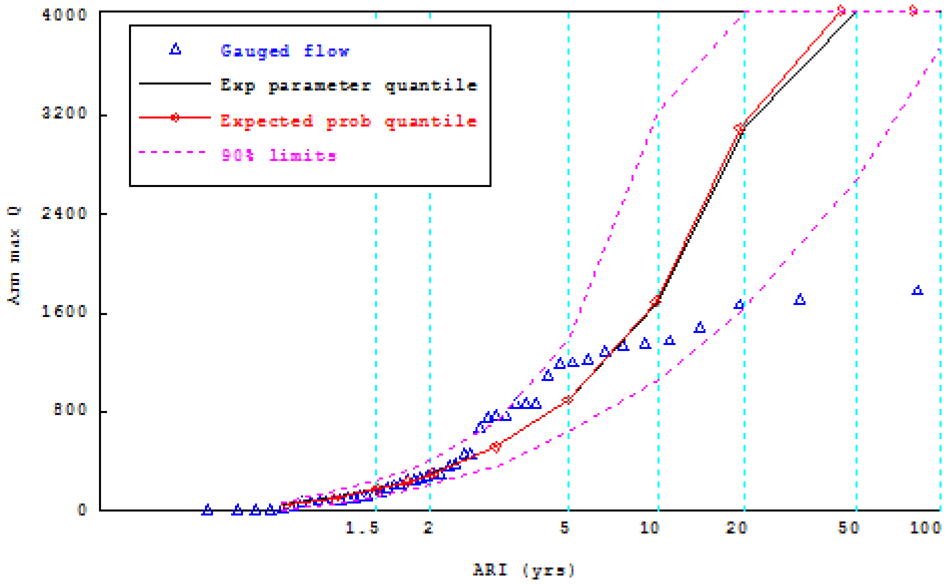


Figure 3.2.41. Bayesian fit to all gauged data Gumbel probability plot

2.8.7.1. Launch TUFLOW Flike and Import Data

As in previous examples launch TUFLOW Flike, create a new **.fld** file and import the Albert River at Bromfleet data (*albertRvGaugedFlows.txt)* file which was included in the TUFLOW Flike install in the **data** directory. Note the structure of this file and configure the **Import gauged values** window. The Albert River at Broomfleet data is included at the end of this example.

2.8.7.2. Fit GEV Distribution

To recreate Figure 3.2.41 fit a GEV distribution to the Albert River data and accept the defaults in the **General** tab of the **FLIKE Editor**. The plot in Figure 3.2.41 can be recreated by changing the plot scale to **Gumbel** and rescaling the y-axis to 0 and 4000.

Figure 3.2.41 displays the GEV Bayesian fit on a Gumbel probability plot. Although the observed floods are largely contained within the 90% confidence limits, the fit, nonetheless, is poor – the data exhibit a sigmoidal trend with reverse curvature developing for floods with an AEP greater than 50%. It appears that the confidence limits have been inflated because the GEV fit represents a poor compromise.

2.8.7.3. Use the multiple Grubbs Beck test to improve fit

The first step in improving the poor fit of this data is to use the multiple Grubbs Beck test to remove PILFs. Repeat the procedure outline in the previous example. This will result in the censoring of 5 data points with a threshold of 36.509m^3/s.

Now run TUFLOW Flike and fit the model. Changing the plot scale and rescale the y-axis as above will result in Figure 3.2.42.

Figure 3.2.42 displays the fit after censoring the 5 low outliers identified by the multiple Grubbs-Beck test. The improvement in fit is marginal at best over Figure 3.2.41.

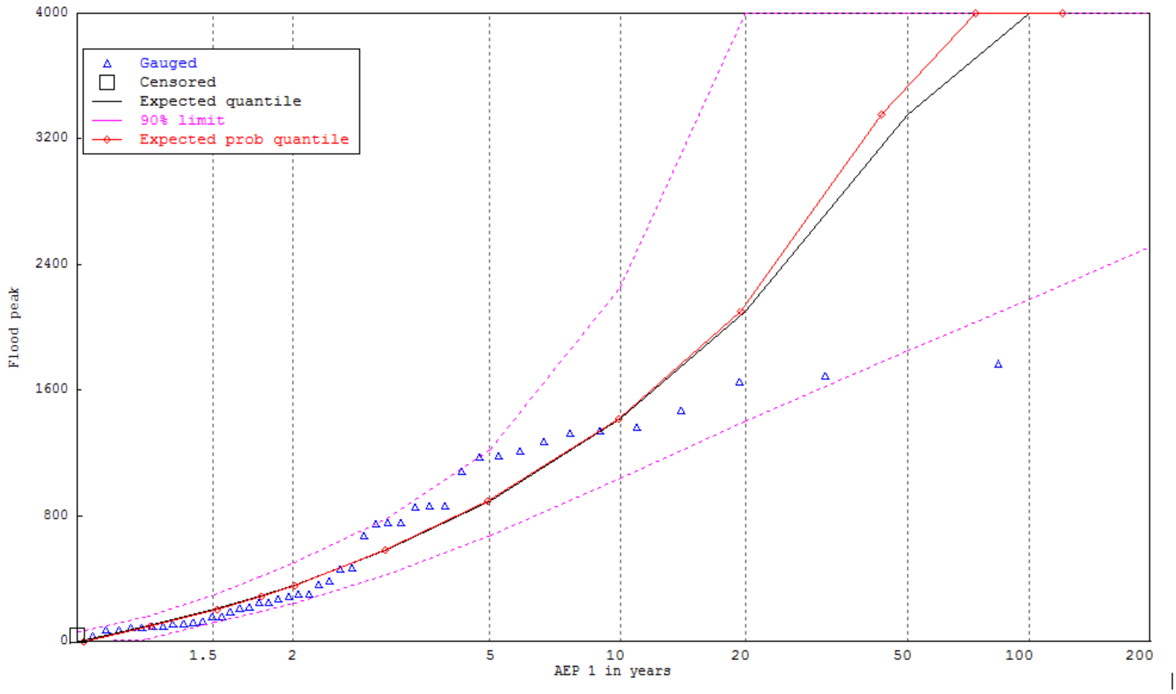


Figure 3.2.42. Bayesian fit with 5 low outliers censored after application of multiple Grubbs-Beck test

2.8.7.4. Trial and error approach

To deal with this poor fit, a trial-and-error approach to selecting the threshold discharge for the censoring low flows can be used to obtain a fit that favours the right hand tail of the distribution. This involves testing different threshold values until an acceptable fit is produced. Figure 3.2.43 illustrates one such fit. To de-emphasise the left hand tail the floods below the threshold of 250 m^3/s were censored. This means the GEV distribution was fitted to:

- A gauged record consisting of the 27 floods above 250m^3/s; and
- A censored record consisting of 23 floods below the threshold of 250m^3/s and 0 floods above this threshold.

To do this in TUFLOW Flike there are two steps, as in Example 6, these are:

- Exclude the flows below 250m^3/s
- Create a censoring threshold

This is essentially the same process that was undertaken to exclude flows in Example 6 except it needs to be done manually. This is outlined below.

2.8.7.4.1. Excluded data

The flows below 250m^3/s need to be excluded from the analysis. To do this select the **Observed values** tab of the **Flike Editor** and choose the **Block exclude** button. Enter 250 into **Value belowwhich values are to be excluded** text box and select **OK**. This will exclude all values below 250m^3/s which can be confirmed by scrolling down the table in the **Observed values** tab.

2.8.7.4.2. Censoring threshold

As in the previous example a censoring threshold needs to be entered into the Censoring of observed values tab. Populate the tab with the following information:

- **Threshold value**: 250
- Years greater than threshold (**Yrs > threshold**): 0
- Years less than or equal to threshold (**Yrs <= threshold**): 23
- **Start year**: 1000
- **End year**: 1022

2.8.7.5. Results of the trial and error approach

Run TUFLOW Flike in the usual way and a Probability plot similar to Figure 3.2.43 will be obtained.

The censored record provides an anchor point for the GEV distribution – it ensures that the chance of an Annual Maximum flood being less than 250m^3/s is about 23/50 without forcing the GEV to fit the peaks below the 250m^3/s threshold. The fit effectively disregards floods with a greater than 50% AEP and provides a good fit to the upper tail. Another benefit is the substantially reduced 90% confidence limits which can be reviewed by examining the results files.

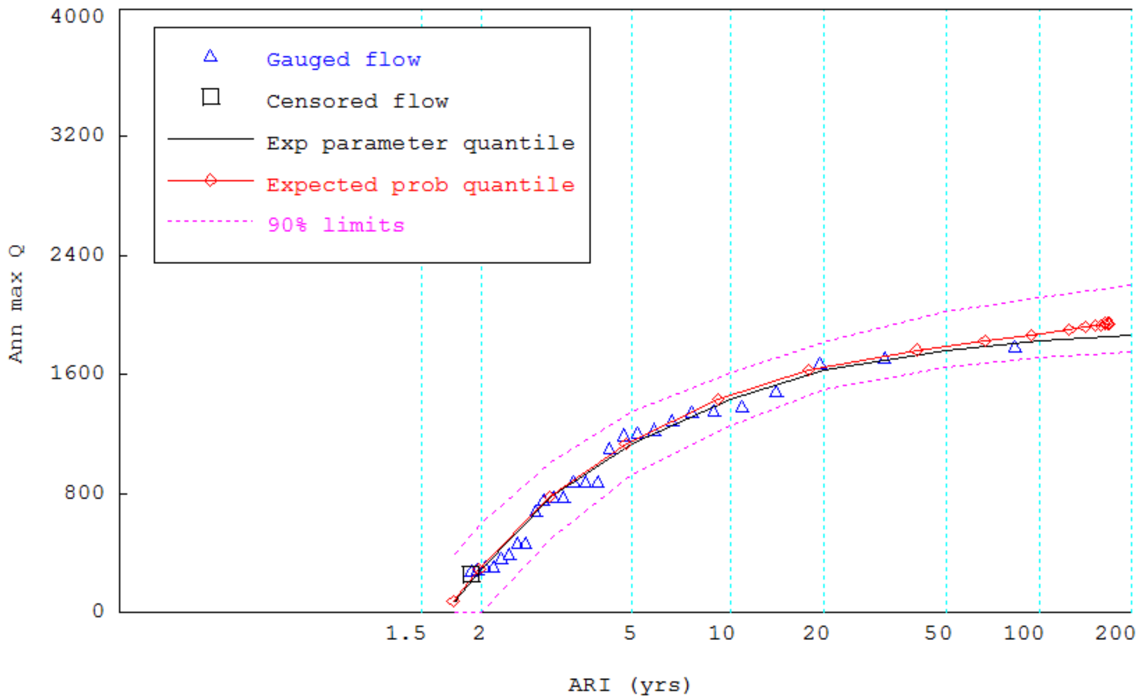


Figure 3.2.43. Bayesian fit with floods below 250 m^3/s threshold treated as censored observations

Annual Maximum data for the Albert River at Bromfleet data

1765.92	1689.51	1652.72	1468.77	1364.06	1341.42	1327.27	1273.5
1214.07	1185.77	1177.28	1086.72	865.98	863.15	860.32	761.27
761.27	752.78	676.37	466.95	461.29	384.88	362.24	305.64
302.81	285.83	271.68	294.61	249.61	220.74	210.55	190.74
156.5	156.22	131.03	124.52	116.88	113.77	99.9	95.65
88.3	87.73	78.11	72.73	36.51	22.36	16.7	15.85
15.57	13.02						

2.8.8. Example 8: A Non-Homogeneous Flood Probability Model

The work of Micevski et al. (2003) illustrates an example of a non-homogeneous model. An indicator time series based on the IPO time series (Figure 7) was used to create the exogeneous vector x

$$x = \{\mathrm{I}_t\ , \mathrm{t} = 1,\ldots,\mathrm{n}\} \tag{3.2.80}$$

where the indicator

$$I_t = \begin{cases} 1\ \mathrm{if IPO}_t \geq \mathrm{IPO}_{\mathrm{thresh}} \\ 0\ \mathrm{if IPO}_t < \mathrm{IPO}_{\mathrm{thresh}} \end{cases} \tag{3.2.81}$$

IPO_t is the IPO index for year t and IPO_{thresh} is a threshold value equal to -0.125.

At each of the 33 NSW sites considered by Micevski et al. the AM peak flows were stratified according to the indicator It. A 2-parameter log-Normal distribution was fitted to the gauged flows with indicator equal to 1 – this is the IPO+ distribution. Likewise, a 2-parameter log-Normal distribution was fitted to the gauged flows with indicator equal to 0 – this is the IPO- distribution. Figure 3.2.44 presents the histogram for the ratio of the IPO- and IPO+ floods for selected 1 in Y AEPs. If the IPO+ and IPO- distributions were homogeneous then about half of the sites should have a flood ratio < 1 – Figure 3.2.44 shows otherwise.

Figures Figure 3.2.45 and Figure 3.2.46 present log normal fits to the IPO+ and IPO- annual maximum flood data for the Clarence river at Lilydale respectively. Though the adequacy of the log normal model to fit high floods may be questioned, in the AEP range 1 in 2 to 1 in 10 years, the IPO- floods are about 2.6 times the IPO+ floods with the same AEP.

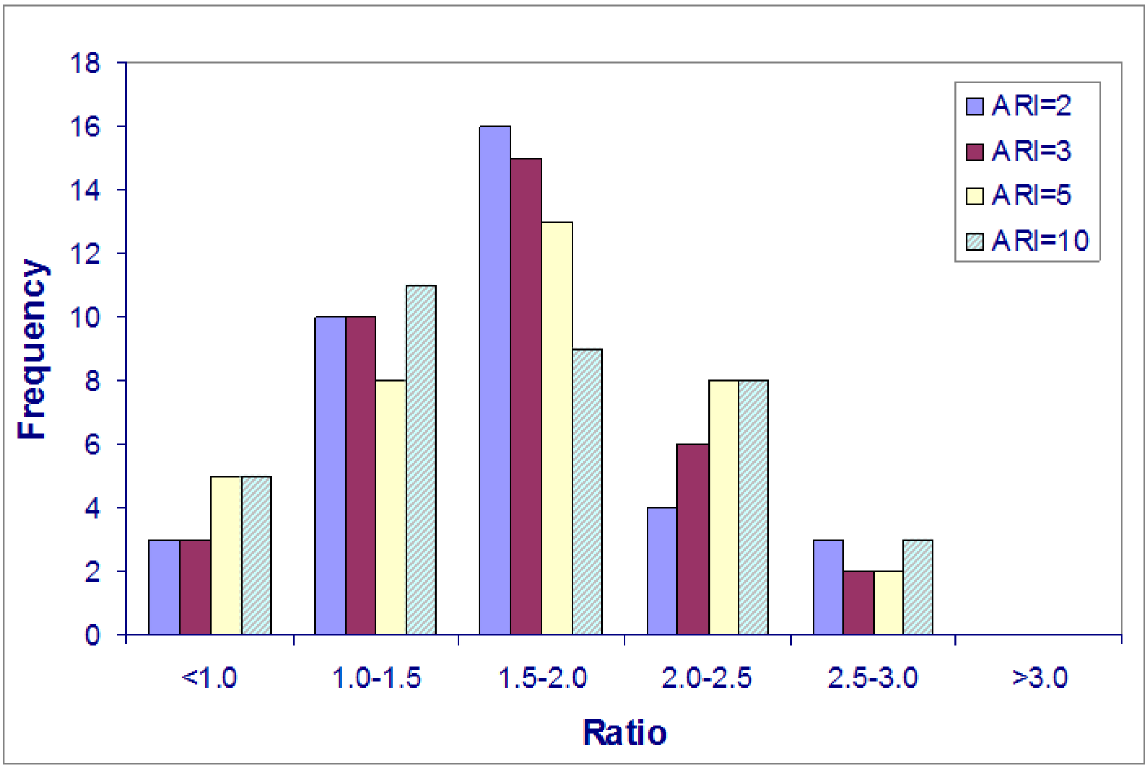


Figure 3.2.44. Histogram of IPO- and IPO+ flood ratios

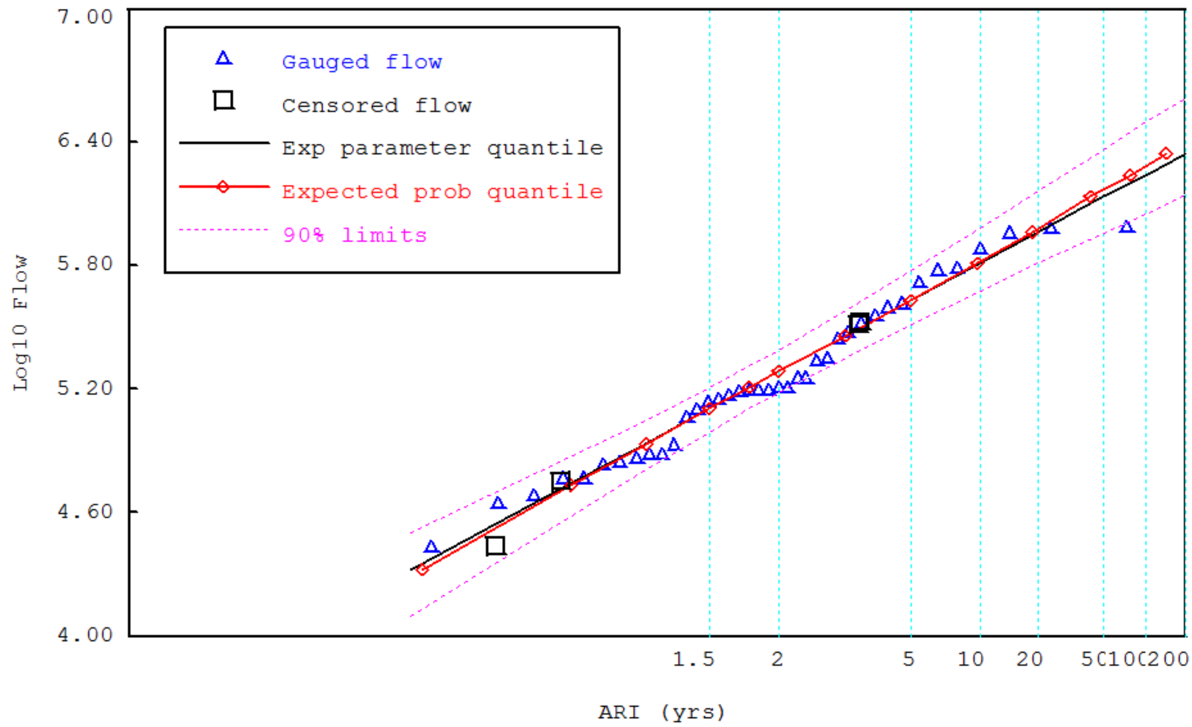


Figure 3.2.45. Log-Normal fit to 43 years of IPO+ data for the Clarence river at Lilydale (units ML/day).

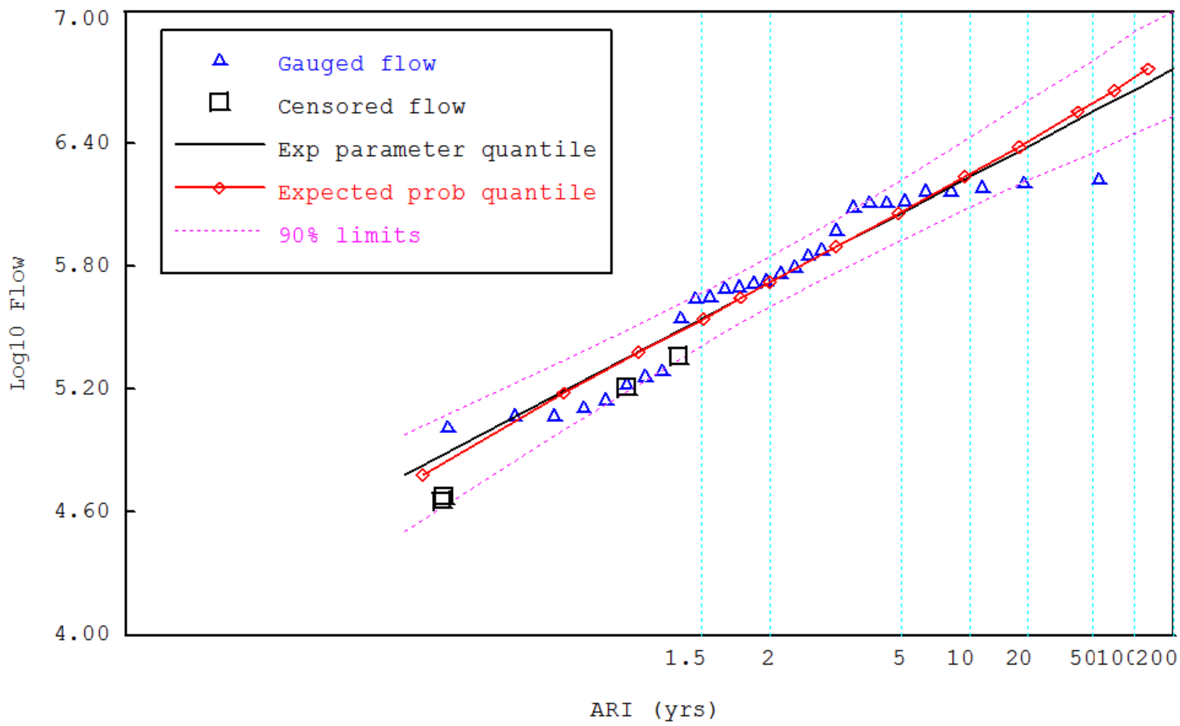


Figure 3.2.46. Log-Normal fit to 33 years of IPO- data for the Clarence river at Lilydale (units ML/day).

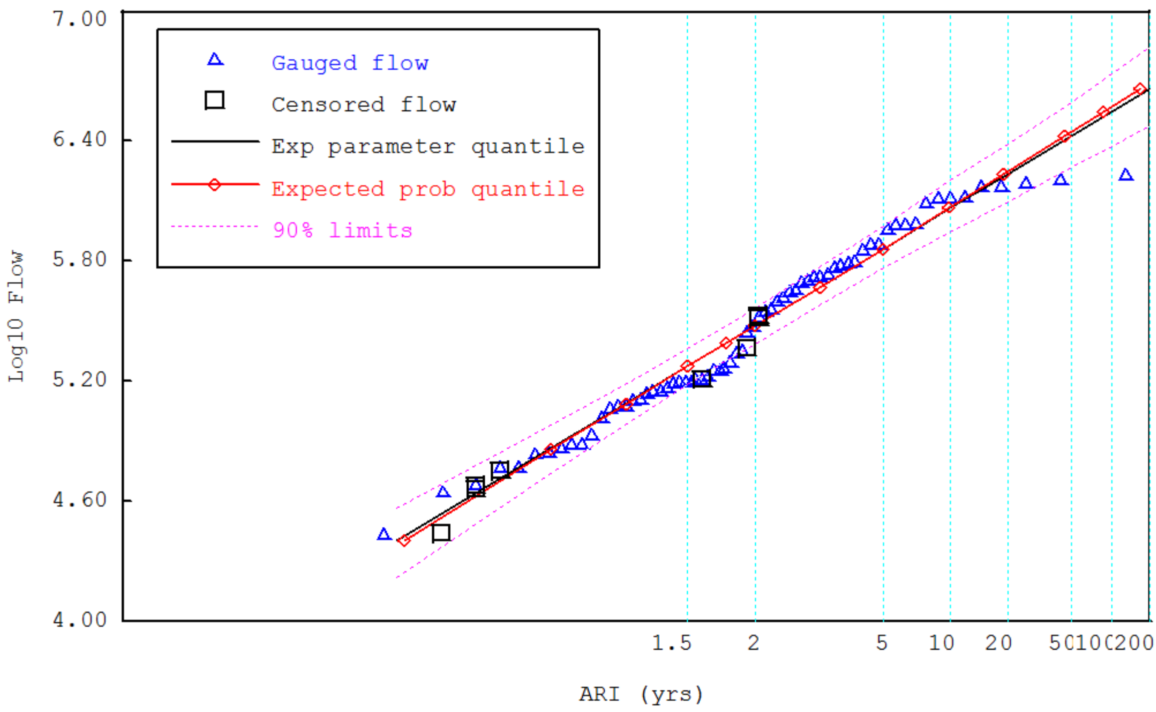


Figure 3.2.47. Log-Normal fit to 76 years of data for the Clarence river at Lilydale (units ML/day).

To avoid bias in estimating long-term flood risk it is essential that the gauged record adequately span both IPO+ and IPO- years. In this example, the IPO+ record is 43 years and the IPO- record is 33 years in length. With reference to Figure 7 this length of record appears to adequately sample both IPO epochs. This suggests that fitting to all the data will yield a largely unbiased estimate of the long-term flood risk. Figure 3.2.47 illustrates a log normal fit to all the data.

A better appreciation of the differences in flood risk can be gleaned by considering Figure 3.2.48 which presents the fitted log normal distributions to the IPO+, IPO- and total data. During an IPO+ period a flood peak of 100 m^3/s has a 1 in 20 AEP while during an IPO- period it has a 1 in 4 AEP. Likewise a flood peak of 200 m^3 /s has 1 in 100 and 1 in 10 AEPs for IPO+ and IPO- periods respectively. The differences in flood risk are considerable. If a short gauged record falling largely in the IPO+ period was used, a standard flood frequency analysis could seriously underestimate the long-term or marginal flood risk.

The marginal flood risk can be derived by combining the IPO+ and IPO- distribution using Equation (3.2.23) to give

$$P(Q \leq q) = P(x = 0)\int_0^q p(z|\theta(x=0))dz + P(x=1)\int_0^q p(z|\theta(x=1))dz \qquad (3.2.82)$$

The exogenous variable x can take two values, 0 or 1, depending on the IPO epoch. P(x=0), the probability of being in an IPO- epoch, is assigned the value 33/76 based on the observation that 33 of the 76 years of record were in the IPO- epoch. Likewise P(x=1), the probability of being in an IPO+ epoch, is assigned the value 43/76. It follows that $p(z|\theta,x=0)$ and $p(z|\theta,x=1)$ are the log normal pdfs fitted to IOP- and IPO+ data respectively.

The derived marginal distribution is plotted in Figure 3.2.48. It almost exactly matches the log normal distribution fitted to all the data.

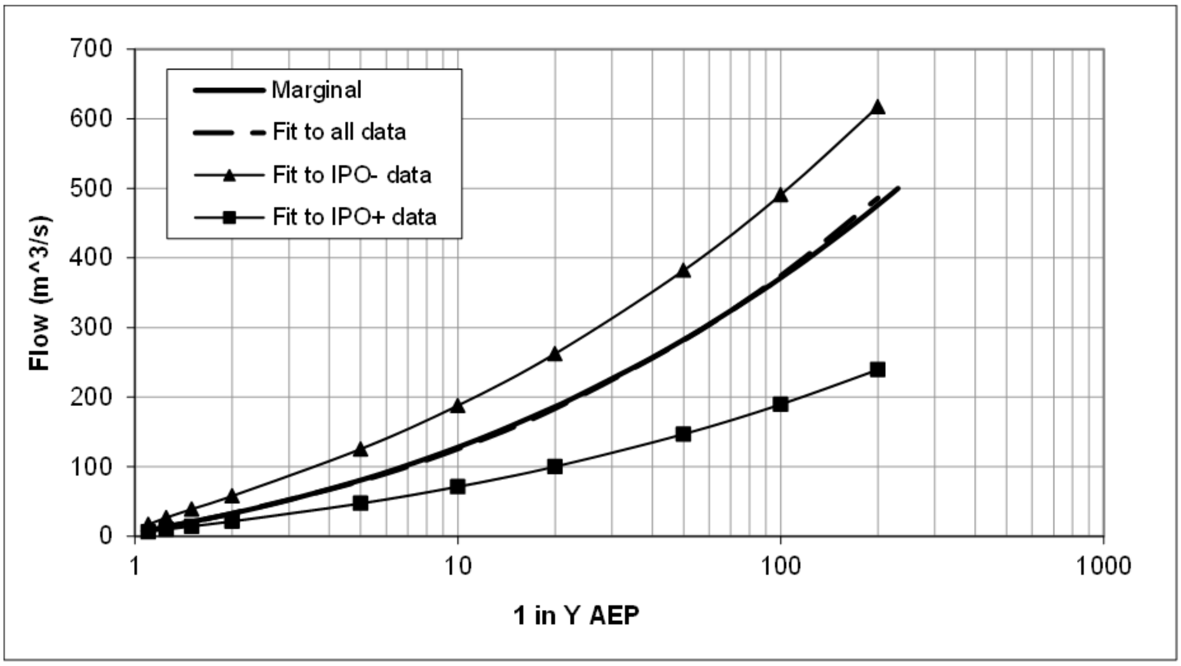


Figure 3.2.48. Marginal, IPO+ and IPO+ log-Normal distributions for the Clarence River at Lilydale

2.8.9. Example 9: L-moments fit to gauged data

This example illustrates fitting a GEV distribution to gauged data using L-moments. L-moments are a special case of LH-moments where there is no shift (H=0). The procedure to use L-moments to fit a distribution is set out in Book 3, Chapter 2, Section 6. In this example Annual Maximum flood data for the Styx River at Jeogla will be fitted using L-moments. The flood data are listed at the end of this example.

The procedure for fitting distributions by L-moments can be completed by hand, and also using TUFLOW Flike. Both of these techniques will be outlined in this example.

2.8.9.1. L-moments by Hand

The first four L-moments can be estimated by Equation (3.2.57) to Equation (3.2.60) and are reported in Table 3.2.13. The GEV parameter estimates can be calculated by substituting the L-moment estimates into the equations in Table 3.2.3 to estimate τ, K and α. The standard deviation and correlation were derived from 5000 bootstrapped samples following the procedure described in Book 3, Chapter 2, Section 6and Parametric bootstrap. Note standard deviation and correlation cannot be calculated by hand.

Table 3.2.13. L-moment and GEV Parameter Estimates

L-moment	L-moment Estimates	GEV Parameter	Parameter Estimate	Standard Deviation	Correlation	Correlation	Correlation
λ_1	189.238	τ	100.660	17.657	1.000		
λ_2	92.476	α	104.157	15.554	0.597	1.000	
λ_3	29.264	κ	-0.219	0.130	0.358	0.268	1.000

2.8.9.2. L-moments using TUFLOW Flike

L-moments and the distribution parameters can be estimated in TUFLOW Flike. To do this, create a new .fld file and import the Styx River at Jeogla data set. Return the **Flike Editor General** tab. Now set the Inference method to LH-moments fit to observed values with and check the H=0 radio box. This last option sets the shift to 0 (i.e. L-moments). The **Flike Editor** window should look like . Run TUFLOW Flike and examine the results file for the L-moments and GEV parameters.

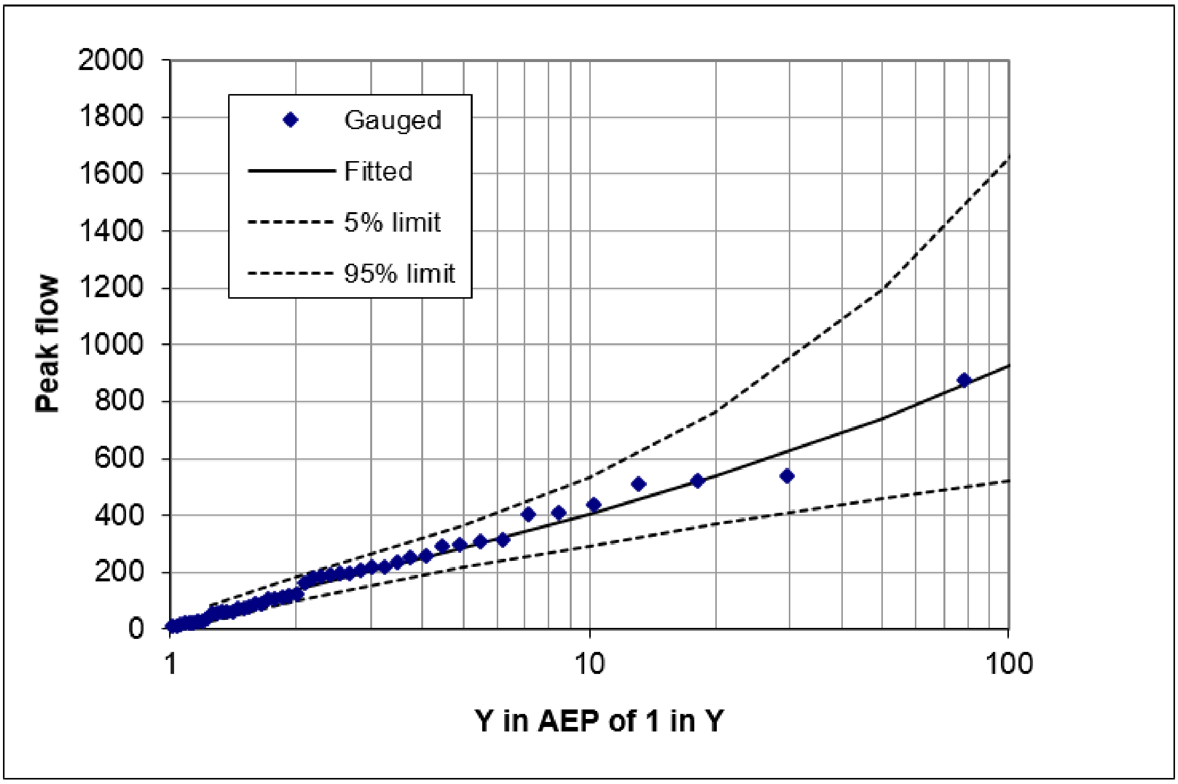


Figure 3.2.49. Flike Editor configured for L-moments

The following table lists 47 ranked flows for the Styx River at Jeogla.

878	541	521	513	436	411	405	315
309	300	294	258	255	235	221	220
206	196	194	190	186	177	164	126

117	111	108	105	92.2	88.6	79.9	74
71.9	62.6	61.2	60.3	58	53.5	39.1	26.7
26.1	23.8	22.4	22.1	18.6	13	8.18	

2.8.10. Example 10: Improving poor fits using LH-moments

In Example 5 the fit of the distribution to the Albert River flood series was improved by censoring low flows. In this example, LH-moments are used instead of censoring to improve the fit of the GEV distribution to the flood data.

2.8.10.1. Launch TUFLOW Flike

This example uses the same data as **Example 7** for the Albert River at Broomfleet, so the previous **Example 7.fld** file can be used. To do this, launch TUFLOW Flike and open **Example_7.fld** and save the opened **.fld** as **Example_10.fld**. Open the **Flike Editor** to configure the LH-moments fitting method. Note that the **Example_7.fld** file was configured with a Bayesian inference method.

2.8.10.2. Configure Inference Method

In **Example 7**, a Bayesian inference method was used with censored low flows, so a number of changes are required to **Example_7.fld** before the LH-moments inference method can be used. As low flows were censored in the previous example, these need to be included back into the analysis by:

- Removing the censoring threshold; and
- Including all the flood data.

Ensure that the **Bayesian with** button is still checked. If the **LH-moments fit to observed values with** radio button is checked the **Censoring of observed values** tab cannot be accessed.

To remove the censoring threshold, select the **Censoring of observed values** tab and select the **Clear all** button.

To include all the flood data, select the **Observed values** tab and select the **Include all** button. Scroll through the data to ensure that all the crosses (**x**) in the **Exclude** column have been removed.

2.8.10.3. Fit L-moments

To configure TUFLOW Flike to fit distributions using the LH-moments inference method, return to the **General** tab and check the **LH-moments fit to observed values with** radio button. In the first instance, select the **H=0** radio button. This will fit a distribution using L-moments, this is, LH-moments with no shift.

TUFLOW Flike will only fit LH-moments with H >= 1 for the GEV distribution, however it will fit L-moments (H = 0) for all distributions. Ensure that the GEV probability model has been selected.

The configured **Flike Editor** should look like Figure 3.2.50. Select OK and run TUFLOW Flike. As usual, a probability plot will appear together with the report file. Rescale the plot so it looks like Figure 3.2.51.

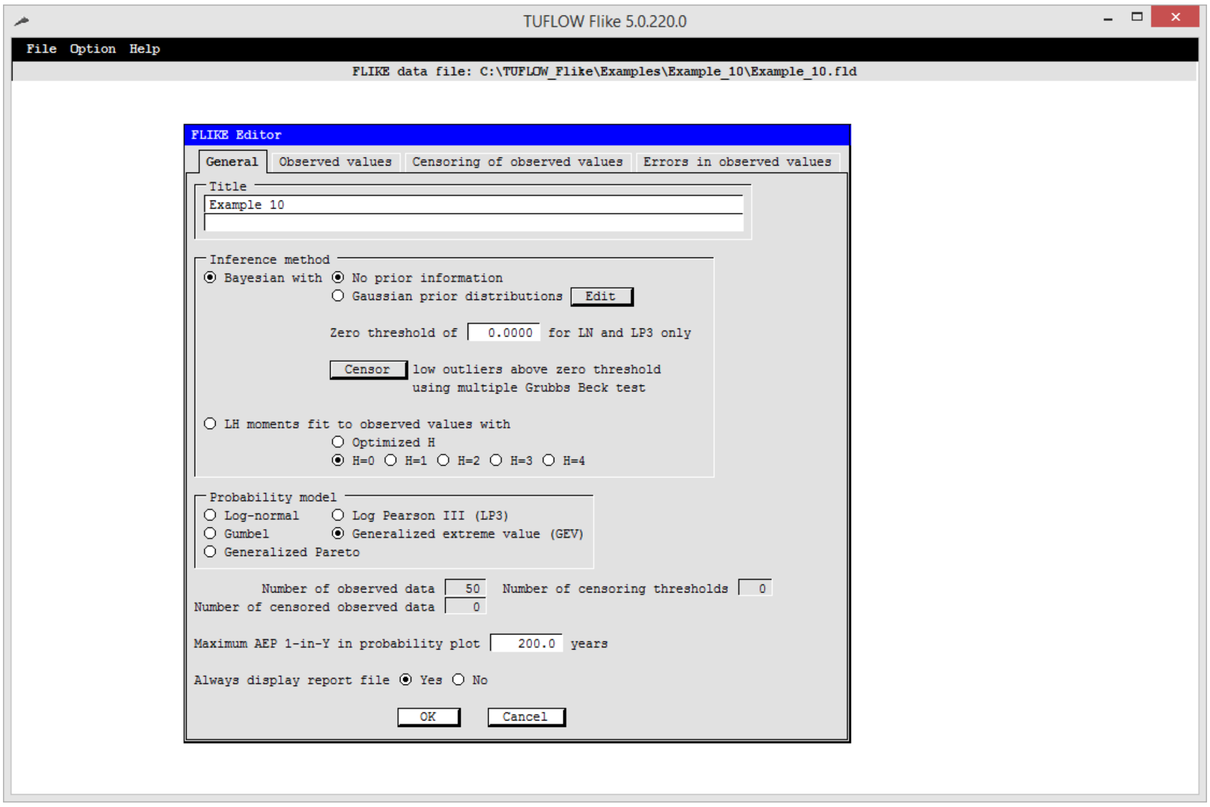


Figure 3.2.50. Configured Flike Editor

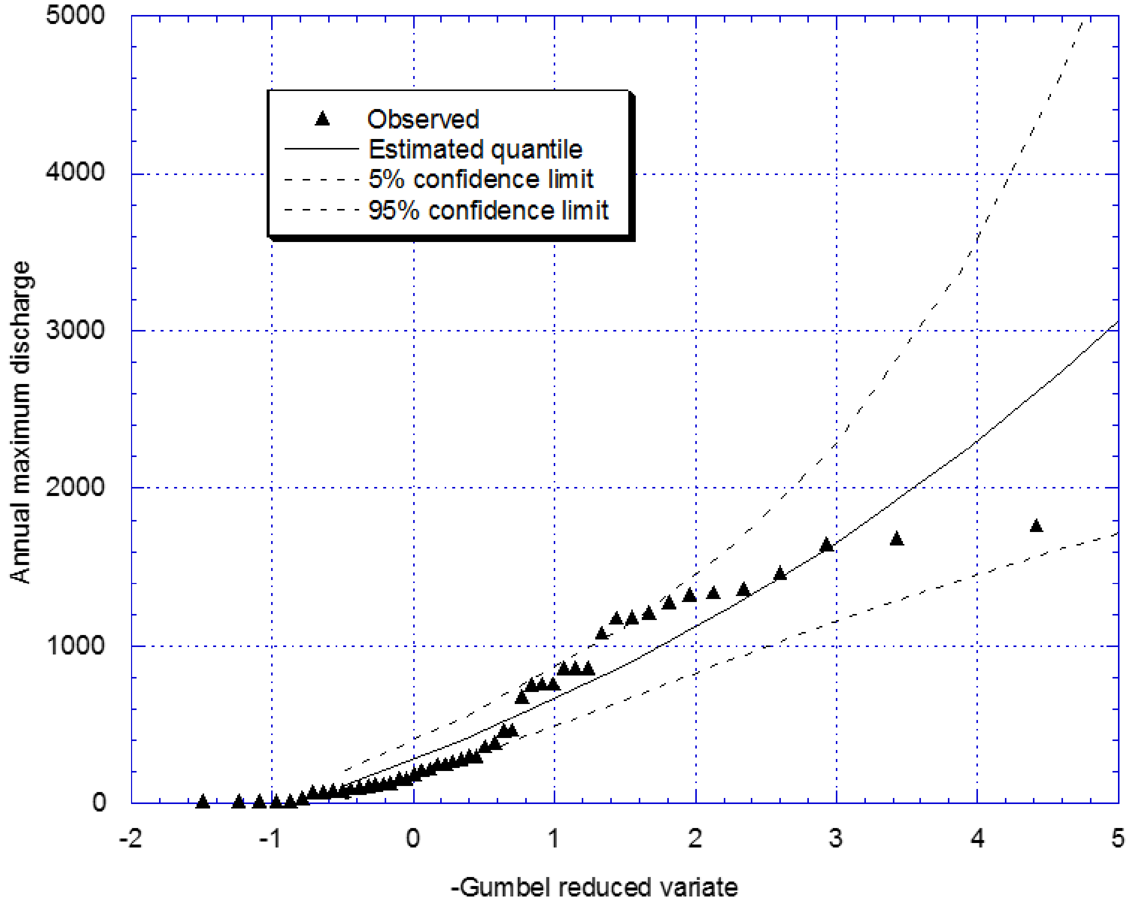


Figure 3.2.51. L-moment fit - Albert River at Broomfleet

Figure 3.2.51 displays the GEV L-moment fit on a Gumbel probability plot. Although the observed floods are largely contained within the 90% confidence limits, the fit, nonetheless, is poor with systematic departures from the data which exhibits reverse curvature.

2.8.10.4. Fit LH-moments

To deal with this poor fit, a LH-moment search was conducted to find the optimal shift parameter using the procedure described in Book 3, Chapter 2, Section 6. To do this in TUFLOW Flike check the **Optimized H** radio button and run TUFLOW Flike. The results file reveals that the optimal shift was found to be 4. Figure 3.2.52 presents the LH-moment fit with shift equal to 4. The fit effectively disregards floods more frequent than the 50% AEP (around 350m^3/s) and provides a very good fit to upper tail.

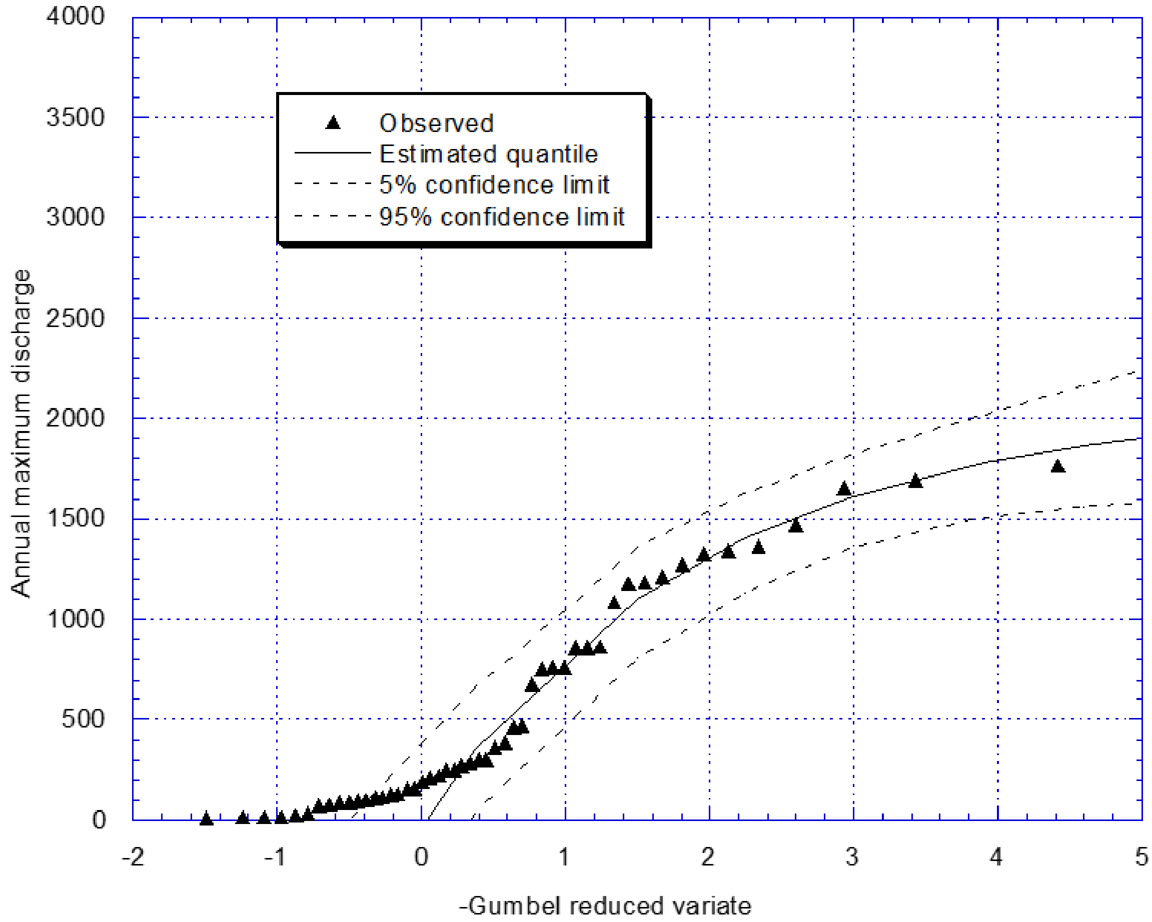


Figure 3.2.52. LH-moment fit with shift H=4

The very significant reduction in the quantile confidence intervals is largely due to the shape parameter *K* changing from –0.17 to 0.50. The L-moment fit in Figure 2 was a compromise; most of the small and medium-sized floods suggested an upward curvature in the probability plot which resulted in a negative GEV shape parameter (to enable upward curvature). In contrast, the LH-moment fit favoured the large-sized floods which exhibit a downward curvature resulted in a positive shape parameter. For positive *K* the GEV has an upper bound. In this case the upper bound is about 2070 m^3/s which is only 17% greater than the largest observed flood.

A comparison of the quantile derived from the Bayesian inference method with censoring of PILFs and those determined using Optimised LH-moments in presented in Table 3.2.14. The two different inference methods produce similar results in terms of the calculated quantiles; however, the confidence limits are smaller using the Bayesian framework. This highlights how LH-moment results could be used to inform the selection of the censoring threshold for PILFs in the Bayesian framework.

Table 3.2.14. Comparison of Quantiles using a Bayesian and LH-moments Inference Methods

AEP (%)	Bayesian with removal of PILFS	Optimised LH-moments				
	Quantile Estimate q_Y	Quantile Confidence 5% Limit	Quantile Confidence 95% Limit	Quantile Estimate q_Y	Quantile Confidence 5% Limit	Quantile Confidence 95% Limit
10%	1400	1249	1590	1406	1133	1634
2%	1720	1605	1931	782	1492	2021
1%	1782	1675	2003	1868	1546	2168
0.2%	1854	1757	2111	1982	1,99	2482

2.8.11. Example 11: Fitting a probability model to POT data

This example is a continuation of Example 9 which considers the Styx River at Jeogla. It illustrates fitting an exponential distribution to POT data. The table lists all the independent peak flows recorded over a 47 year period that exceeded a threshold of 74 m^3/s – the total number of peaks was 47. Comparison with the annual maximum flood peaks in Example 9 reveals that in 15 of the 47 years of record the annual maximum peak were below the threshold of 74 m^3/s.

878	541	521	513	436	411	405	315
309	301	300	294	283	258	255	255
238	235	221	220	206	196	194	190
186	164	150	149	134	129	129	126
119	118	117	117	111	108	105	98
92.2	92.2	91.7	88.6	85.2	79.9	74	

The first two L-moments were estimated as 226.36 and 79.2. Noting that the exponential distribution is a special case of the generalised Pareto when $\kappa = 0$, it follows from Table 3.2.3 that the exponential parameters are related to the L-moments by

$$\lambda_1 = q_* + \beta \quad \lambda_2 = \frac{\beta}{2} \tag{3.2.83}$$

which yields values for q* and β of 68.11 and 158.24 respectively. Therefore the probability of the peak flow q exceeding *w* in any POT event is

$$P(q > w) = \mathrm{e}^{\left(-\frac{w - q_*}{\beta}\right)} = \mathrm{e}^{\left(-\frac{w - 68.11}{158.24}\right)} \tag{3.2.84}$$

The second step obtains the distribution of annual maximum peaks. Using Equation (3.2.11), the expected number of peaks that exceed w in a year is

$$EY(w) = \nu P(q > w) = \nu e^{\left(-\frac{w - q_*}{\beta}\right)} \quad (3.2.85)$$

where v is the average number of flood peaks above the threshold q* per year.

For plotting purposes it is convenient to use a log transformation which yields

$$\log_e EY(w) = \log_e \nu + \frac{q_*}{\beta} - \frac{w}{\beta} \quad (3.2.86)$$

A plot of $\log_e EY(w)$ versus w should follow a straight line if the underlying POT distribution is exponential.

Given that 47 peaks above the threshold occurred in 47 years, v equals 1.0. The following figure presents a plot of the fitted POT exponential model against the observed POT series.

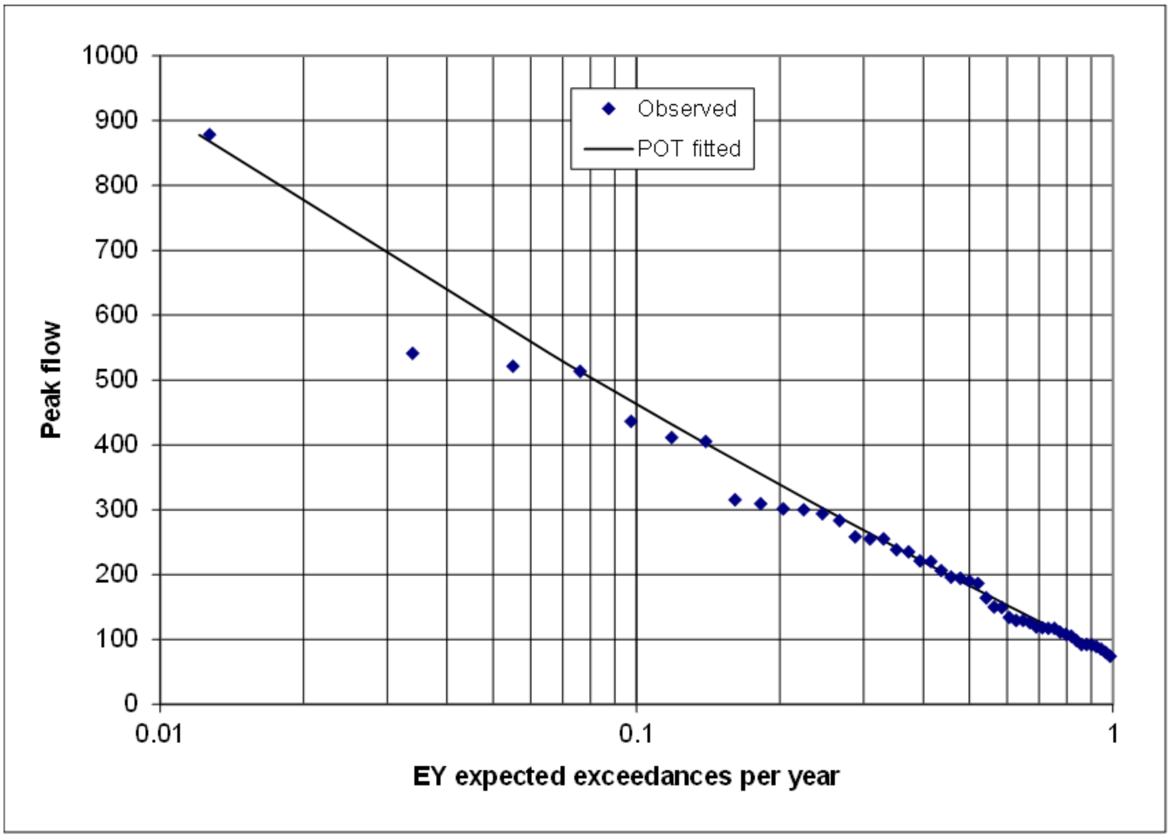


Figure 3.2.53. Plot of the fitted POT exponential model against the observed POT series

2.9. References

ASCE (American Society of Civil Engineers) (1949), Hydrology Handbook.

Adams, C.A. (1987), Design flood estimation for ungauged rural catchments in Victoria Road Construction Authority, Victoria. Draft Technical Bulletin.

Adams, C.A. and McMahon, T.A. (1985), Estimation of flood discharge for ungauged rural catchments in Victoria. Hydrol. and Water Resources Symposium 1985, Inst Engrs Aust., Natl Conf. Publ. No.85/2, pp: 86-90.

Alexander, G.N. (1957), Flood flow estimation, probability and the return period. Jour. Inst. Engrs Aust, 29: 263-278.

Allan, R.J. (2000), ENSO and climatic variability in the last 150 years, in El Nino and the Southern Oscillation, Multi-scale variability, Global and Regional Impacts, edited by H.F. Diaz and V. Markgraf, Cambridge University Press, Cambridge, uk, pp: 3-56.

Ashkanasy, N.M. and Weeks, W.D. (1975), Flood frequency distribution in a catchment subject to two storm rainfall producing mechanisms. Hydrol. Symposium 1975, Inst Engrs Aust, Natl Conf. Publ. (75/3), 153-157.

Ashkar, F. and Rousselle, J. (1983) Some remarks on the truncation used in partial flood series models. Water Resources Research, 19: 477-480.

Baker, V. (1984), Recent paleoflood hydrology studies in arid and semi-arid environments (abstract). EOS Trans. Amer. Geophys. Union, Volume 65, p: 893.

Baker, V., Pickup, G. and Polach, H.A. (1983) ,Desert paleofloods in central Australia. Nature, 301: 502-504.

Beard, L.R. (1974), Flood flow frequency techniques Univ. of Texas at Austin, Center for Research in Water Resources, Tech. Report CRWR119, October.

Beran, M., Hosking, J.R.M. and Arnell, N. (1986), Comment on 'Two-component extreme value distribution for flood frequency analysis.' Water Resources Research, 22: 263-266.

Blom, G. (1958), Statistical Estimates and Transformed Beta-Variables. Wiley, New York, p: 176.

Clarke-Hafstad, K. (1942), Reliability of station-year rainfall-frequency determinations. Trans. Amer. Soc. Civ. Engrs., 107(1), 633-652.

Cohn, T.A., England, J.F., Berenbrock, C.E., Mason, R.R., Stedinger, J.R. and Lamontagne, J.R. (2013), A generalized Grubbs-Beck test statistic for detecting multiple potentially influential low outliers in flood series, Water Resour. Res., 49(8), 5047-5058.

Conway, K.M. (1970), Flood frequency analysis of some N.S.W. coastal rivers. Thesis (M.Eng Sc.), Univ.NSW.

Costa, J.E. (1978), Holocene stratigraphy in flood frequency analysis. Water Resources Research, 4: 626-632.

Costa, J.E. (1983), Palaeohydraulic reconstruction or flashflood peaks from boulder deposits in the Colorado Front Range. Geol. Soc. America Bulletin, 94(8), 986-1004.

Costa, J.E. (1986), A history of paleoflood hydrology in the United States, 1800-1970. EOS Trans. Aoler Geophysical Union, 67(17), 425-430.

Cunnane, C. (1985), Factors affecting choice of distribution for flood series. Hydrological Sciences Journal, 30: 25-36.

Cunnane, C. (1978), Unbiased plotting positions - a review. Jour. of Hydrology, 37: 205-222.

Dalrymple, T. (1960), Flood-frequency analyses. Manual of Hydrology: Section 3. Flood-flow Techniques. U.S. Geological Survey Water Supply Paper 1543-A, 79 p.

DeGroot, M.H. (1970) Optimal statistcal decisions, McGraw-Hill

Doran, D.G. and Irish, J.L. (1980), On the nature and extent of bias in flood damage estimation. Hydrol. and Water Resources Symposium 1980, Inst. Engrs Aust, Natl Conf. Publ., (80/9), 135-139.

Duan, Q., Sorooshian, S. and Gupta, V. (1992), Effective and efficient global optimization for conceptual rainfall-runoff models, Water Resources Research, 28(4), 1015-1031.

Erskine, W. D. and Warner, R.F. (1988), Geomorphic effects of alternating flood and drought dominated regimes on a NSW coastal river, in Fluvial Geomorphology of Australia, edited by R.F. Warner, pp. 223-244, Academic Press, Sydney.

Fiering, M.B. (1963), Use of correlation to improve estimates of the mean and variance. U.S. Geological estimates of the mean and variance. U.S. Geological Survey Professional Paper 434-C.

Fiorentino, M., Versace, P. and Rossi, F. (1985), Regional flood frequency estimation using the two-component extreme value distribution. Hydrol. Sciences Jour., 30: 51-64.

Folland, C.K., Renwick, J.A., Salinger, M.J. and Mullan, A.B. (2002), Relative influences of the Interdecadal Pacific Oscillation and ENSO on the South Pacific Convergence Zone, Geophys. Res. Lett., 29(13), doi:10.1029/2001GL014201.

Franks, S.W. and Kuczera, G. (2002), Flood Frequency Analysis: Evidence and Implications of Secular Climate Variability, New South Wales, Water Resources Research, 38(5), 10.1029/2001WR000232.

Franks, S.W. (2002a), Identification of a change in climate state using regional flood data, Hydrol. Earth Sys. Sci., 6(1), 11-16.

Franks, S.W. (2002b), Assessing hydrological change: deterministic general circulation models or spurious solar correlation? Hydrol. Proc., 16: 559-564.

Gelman, A., Carlin, J.B., Stern, H.S. and Rubin, D.B. (1995), Bayesian data analysis, Chapman and Hall, p: 526.

Hosking, J.R.M. (1990), L-moments: Analysis and estimation of distributions using linear combinations of order statistics, J. Roy. Statist. Soc., Ser. B, 52(2), 105-124.

Hosking, J.R.M. and Wallis, J.R, (1986) Paleoflood hydrology and flood frequency analysis. Water Resources Research, 22: 543-550.

Houghton, J.C. (1978), Birth of a parent: The Wakeby distribution for modeling flood flows. Water Resources Research, 14: 1105-1109.

Interagency Advisory Committee on Water Data (1982), Guidelines for determining flood flow frequency. Bulletin 17B of the Hydrology Sub-committee, Office of Water Data Coordination, Geological Survey, U.S. Dept of the Interior.

Jayasuriya, M.D.A. and Mein, R.G. (1985), Frequency analysis using the partial series. Hydrol. and Water Resources Symposium 1985, Inst. Engrs Aust., Natl Conf. Publ. No. 85/2, pp: 81-85.

Kiem, A.S. and Franks, S.W. (2004), Multidecadal variability of drought risk - eastern Australia, Hydrol. Proc., 18, doi:10.1002/hyp.1460.

Kiem, A.S., Franks, S.W. and Kuczera, G. (2003), Multi-decadal variability of flood risk, Geophysical Research Letters, 30(2), 1035, DOI:10.1029/2002GL015992.

Kochel. R.C., Baker, V.R and Patton, P.C. (1982), Paleohydrology of southwestern Texas. Water Resources Research, 18: 1165-1183.

Kopittke, R.A., Stewart, B.J. and Tickle, K.S. (1976), Frequency analysis of flood data in Queensland. Hydrol. Symposium 1976, Inst Engrs Aust., Natl Conf. Publ. No. 76/2, pp: 20-24.

Kuczera, G. (1999), Comprehensive at-site flood frequency analysis using Monte Carlo Bayesian inference, Water Resources Research, 35(5), 1551-1558.

Kuczera, G. (1996), Correlated rating curve error in flood frequency inference, Water Resources Research, 32(7), 2119-2128.

Kuczera, G., Lambert, M.F., Heneker, T., Jennings, S., Frost, A. and Coombes, P. (2006), Joint probability and design storms at the crossroads, Australian Journal of Water Resources, 10(2), 5-21.

Laurenson, E.M. (1987), Back to basics on flood frequency analysis. Civ. Engg Trans., Inst Engrs Aust, CE29: 47-53.

Lee, P.M. (1989), Bayesian statistics: An introduction, Oxford University Press (NY).

Mantua, N. J., Hare, S.R., Zhang, Y., Wallace, J.M. and Francis, R.C. (1997), A Pacific interdecadal climate oscillation with impacts on salmon production, Bull. Amer. Meteorol. Soc., 78(6), 1069-1079.

Matalas, N.C. and Jacobs, B. (1964), A correlation procedure for augmenting hydrologic data. U.S. Geological Survey Professional Paper 434-E.

McDermott, G.E. and Pilgrim, D.H. (1982), Design flood estimation for small catchments in New South Wales. Dept of National Development and Energy, Aust Water Resources Council Tech. Paper No.73, p: 233

McDermott, G.E. and Pilgrim, D.H. (1983), A design flood method for arid western New South Wales based on bankfull estimates. Civ. Engg Trans., Inst Engrs Aust, CE25: 114-120.

McIllwraith, J.F. (1953), Rainfall intensity-frequency data for New South Wales stations. Jour. Inst Engrs Aust, 25: 133-139.

McMahon, T.A. (1979), Hydrologic characteristics of Australian streams. Civ. Engg Research Reports, Monash Univ., Report No.3/1979.

McMahon, T.A. and Srikanthan, R. (1981), Log Pearson III distribution- is it applicable to flood frequency analysis of Australian streams? Jour. of Hydrology, 52: 139-147.

Micevski, T., Kiem, A.S., Franks, S.W. and Kuczera, G. (2003), Multidecadal Variability in New South Wales Flood Data, Hydrology and Water Resources Symposium, Institution of Engineers, Australia, Wollongong.

Natural Environment Research Council (1975), Flood Studies Report, Vol.1, Hydrological Studies, London.

O'Connell, D.R., Ostemaa, D.A., Levish, D.R. and Klinger, R.E. (2002) Bayesian flood frequency analysis with paleohydrologic bound data, Water Resources Research, 38(5), 1058, DOI:10.1029/2000WRR000028.

Pedruco, P., Nielsen, C., Kuczera, G. and Rahman, A. (2014), Combining regional flood frequency estimates with an at site flood frequency analysis using a Bayesian framework: Practical considerations, Hydrology and Water Resources Symp., Perth, Engineers Australia.

Pilgrim, DH (ed) (1987) Australian Rainfall and Runoff - A Guide to Flood Estimation, Institution of Engineers, Australia, Barton, ACT, 1987.

Pilgrim, D.H. and Doran, D.G. (1987), Flood frequency analysis, in Australian Rainfall and Runoff: A guide to flood estimation, Pilgrim, D.H. (ed), The Institution of Engineers, Australia, Canberra.

Pilgrim, D.H. and McDermott, G.E. (1982), Design floods for small rural catchments in eastern New South Wales. Civ. Engg Trans., Inst Engrs Aust., CE24: 226-234.

Potter, D.J. and Pilgrim, D.H. (1971), Flood estimation using a regional flood frequency approach. Final Report, Vol.2, Report on Analysis Components. Aust. Water Resources Council, Research Project 68/1, Hydrology of Small Rural Catchments. Snowy Mountains Engg Corporation, April.

Potter, K.W. and Walker, J.F. (1981), A model of discontinuous measurement error and its effects on the probability distribution of flood discharge measurements, Water Resources Research, 17(5), 1505-1509.

Potter, K.W. and Walker, J.F. (1985), An empirical study of flood measurement error, Water Resources. Research, 21(3), 403-406.

Power, S., Tseitkin, F., Torok, S., Lavery, B., Dahni, R. and McAvaney, B. (1998), Australian temperature, Australian rainfall and the Southern Oscillation, 1910-1992: coherent variability and recent changes, Aust. Met. Mag., 47(2), 85-101.

Power, S., Casey, T., Folland, C., Colman, A. and Mehta, V. (1999), Inter-decadal modulation of the impact of ENSO on Australia, Climate Dynamics, 15(5), 319-324.

Rossi, F., Fiorentino, M. and Versace, P. (1984), Two-component extreme value distribution for flood frequency analysis. Water Resources Research, 20: 847-856.

Slack, J.R., Wallis, J.R. and Matalas, N.C. (1975), On the value of information in flood frequency analysis, , Water Resources. Research, 11(5), 629-648.

Stedinger, J.R. (1983), Design events with specified flood risk, Water Resources Research, 19(2), 511-522.

Stedinger, J.R and Cohn, T.A. (1986), Flood frequency analysis with historical and paleoflood information. Water Resources Research, 22: 785-793.

Stedinger, J.R., Vogel, R.M. and Foufoula-Georgiou, E. (1993), Frequency analysis of extreme events in Handbook of Hydrology, Maidment, D.R. (ed.), McGraw-Hill, NY.

Tavares, L.V. and Da Silva, J.E. (1983), Partial series method revisited. Jour. of Hydrology, 64: 1-14.

Wallis, J.R. and Wood, E.F. (1985), Relative accuracy log Pearson III procedures. Proc. Amer. Soc. Civ. Engrs. J. of Hydraul. Eng., lll(7), 1043-1056.

Wang, Q.J. (2001), A Bayesian joint probability approach for flood record augmentation, Water Resources Research, 37(6), 1707-1712.

Wang, Q.J. (1996), Direct sample estimators of L-moments, Water Resources Research, 32(12), 3617-3619.

Wang, Q.J. (1997), LH moments for statistical analysis of extreme events, Water Resources Research, 33(12), 2841-2848.

Wang, Q. J. (1998), Approximate goodness-of-fit tests of fitted generalized extreme value distributions using LH moments, Water Resources Research, 34(12), 3497-3502.

Chapter 3. Regional Flood Methods

Ataur Rahman, Khaled Haddad, George Kuczera, Erwin Weinmann

Chapter Status	Final
Date last updated	14/5/2019

3.1. Introduction

Estimation of peak flows on small to medium sized rural catchments is required for the design of culverts, small to medium sized bridges, causeways, soil conservation works and for various planning and regulatory purposes. Typically, most design flood estimates for projects on small to medium sized catchments are on catchments that are ungauged or have little recorded streamflow data. In these cases, peak flow estimates can be obtained using a Regional Flood Frequency Estimation (RFFE) approach, which transfers flood frequency characteristics from a group of gauged catchments to the location of interest. Even in cases where there is recorded streamflow data it is beneficial to pool the information in the gauged record with the RFFE information. A RFFE technique is expected to be simple, requiring only readily accessible catchment data to obtain design flood estimates relatively quickly.

The RFFE method described in this chapter ensures that design flood discharge estimates are consistent with the gauged records and with results for other ungauged catchments in a region. It is recognised that there will be considerable uncertainty in estimates for ungauged catchments because of the limited number of gauged catchments available to develop the method and the wide range of catchment types that exist throughout Australia.

In developing the RFFE technique, a number of criteria had to be satisfied. These criteria included:

- National consistency in approach;
- Smooth interfacing at the boundaries between areas;
- Use readily accessible data; and
- Utilise as much of Australia's streamflow database as possible.

The basis for the development of the RFFE technique recommended herein, therefore, is a national database consisting of 853 gauged catchments. These data were used to develop and test the RFFE technique presented in this chapter. Further details of the development of the database and RFFE technique are provided by the references noted in Book 3, Chapter 3, Section 16.

The following sections contain a description of the conceptual and statistical framework of the adopted RFFE technique, a computer-based application tool, referred to as ‘RFFE Model 2015’, that implements the adopted RFFE technique and a number of worked examples to demonstrate the application of the model.

Following the guidance provided in Book 1 and Book 3, Chapter 1, users of the RFFE technique are reminded that there are alternatives to the RFFE technique.

While the RFFE technique described in this chapter is regarded as a state-of-the-art approach for estimation of design flood peak discharges at ungauged catchments, the limitations of the method must be recognised. The RFFE technique has been developed using the best available database of gauged catchments throughout Australia, but the fact remains that only a small number of gauged catchments were available to represent the wide range of conditions experienced over an area of about 7.5 million km^2. Therefore, in accordance with the guidance in Book 1, Chapter 1 of ARR, designers and analysts have a duty to use an alternative technique if that technique can be shown to be superior to RFFE Model 2015 and to utilise any available local data, both formal and informal to assist in understanding local conditions and improve upon RFFE Model 2015 estimates. In comparing and selecting alternative methods, the uncertainty in the observed flood data due to factors such as limitations in record length and rating curve extrapolation should be recognised.

3.2. Conceptual Framework

3.2.1. Definition of Regional Flood Frequency Estimation

Regional Flood Frequency Estimation (RFFE) is a data-driven approach, which attempts to transfer flood characteristics from a group of gauged catchments to ungauged locations of interest (where design floods need to be estimated). A range of different methods are available to extract regional flood information from the pooled data and to transfer the relevant information to an individual ungauged catchment in the region (Sivapalan et al., 2013). All of these RFFE techniques use the results of at-site Flood Frequency Analysis (FFA, refer to Book 3, Chapter 2) as basic data. A RFFE technique essentially consists of two steps: (i) *Formation of Regions* - which involves identification of the regions for which flood data from the available streamflow gauging stations can be pooled for analysis; and (ii) *Development of Regional Estimation Equations* - which involves derivation of prediction equations to be used for design flood estimation within a region.

3.2.2. Formation of Regions

In RFFE techniques, the formation of regions can be based on geographic proximity or on similarity in catchment attributes. A region can be fixed, having a definite common boundary for all sites within it, or it can be formed around the ungauged catchment of interest (i.e. the location where flood quantile estimation is desired), using the nearest stations in geographic or catchment attributes space. Regions must satisfy explicitly or implicitly the assumption of ‘regional homogeneity’. The decision on what constitutes a homogeneous region for the purposes of Regional Flood Frequency Estimation depends on the methods used, more specifically on the extent to which differences in flood characteristics can be expressed through parameters in the regionalisation method. There have been many techniques developed which attempt to establish homogenous regions. For example, the recommended RFFE model for eastern New South Wales (NSW) and Victoria in Australian Rainfall and Runoff 1987 (Pilgrim, 1987), namely the Probabilistic Rational Method, used geographical contiguity as an indication of homogeneity; in other words the catchments which are closer to each other should have similar runoff coefficients (Pilgrim, 1987).

There has been little success in the identification of ‘acceptably homogeneous regions’ in Australia using statistical measures such as those proposed by Hosking and Wallis (1993). A common approach to defining a fixed region has been to base the region on political boundaries. However, these fixed regions based on state borders and other geographical boundaries have often been found to be highly heterogeneous (Bates et al., 1998; Rahman, 1997; Haddad, 2008).

As an alternative to fixed regions, Burn (1990a), Burn (1990b), and Zrinji and Burn (1994) proposed the Region Of Influence (ROI) approach where a location of interest (i.e. the catchment where flood quantiles are to be estimated) is allowed to form its own region by selecting a group of ‘nearby catchments’ in either a geographical or catchment attributes space (catchment attributes refer to the catchment characteristics that are influential in developing flood flows). The ROI approach attempts to reduce the degree of heterogeneity in a proposed region by excluding sites located remotely in geographical or catchment attributes space. The ROI approach often uses a statistical criterion to select the optimum size of the region, such as ‘minimum model error variance’ in the regression.

3.2.3. Development of Regional Flood Frequency Estimation Technique

According to Bates (1994), the most commonly adopted methods to develop Regional Flood Frequency Estimation techniques include various forms of the Rational Method, the Index Flood Method and regression based techniques. In ARR 1987, the Probabilistic Rational Method was recommended for general use in Victoria and eastern NSW (Pilgrim, 1987). The foundations of the Probabilistic Rational Method are presented by Pilgrim and McDermott (1982), Mittelstadt et al. (1987), and Adams (1984). The central component of the Probabilistic Rational Method, is the runoff coefficient and, in particular, the 10 year Average Recurrence Interval (ARI) runoff coefficient. It is worth noting that this runoff coefficient does not have a physical basis but rather is a parameter to ensure that the rainfall frequency is transferred to the flow frequency. Furthermore, this parameter has been assumed to vary smoothly over geographical space for purposes of extrapolating from known locations to locations for application. However, it has been found that the runoff coefficient may show sharp variation within a close proximity, reflecting discontinuities at many locations, particularly at catchment boundaries. Additionally, French (2002) noted that the isopleths of the runoff coefficient in ARR 1987 ignored the existence of watercourses.

Alternative methods to the Probabilistic Rational Method, such as the Index Flood Method heavily rely on the assumption of ‘regional homogeneity’, which as previously mentioned is satisfied poorly for Australian regional flood data. Studies on regression based RFFE techniques for Australia (e.g. (Hackelbusch et al., 2009; Haddad et al., 2008; Haddad et al., 2009; Haddad et al., 2011; Haddad et al., 2012; Haddad and Rahman, 2012; Micevski et al., 2015; Palmen and Weeks, 2009; Palmen and Weeks, 2011; Pirozzi et al., 2009; Rahman, 2005; Rahman et al., 2008; Rahman et al., 2009; Rahman et al., 2011a; Rahman et al., 2011b; Rahman et al., 2012; Rahman et al., 2015a; Rahman et al, 2015; Rahman et al., 2015b; Rahman et al., 2015c)) have demonstrated that these techniques are capable of providing quite accurate design flood estimates using only a few predictor variables. In particular, it has been found that the Generalised Least Squares (GLS) based regression technique offers a powerful statistical method which accounts for the inter-station correlation of annual maximum flood series and across-site variation in flood series record lengths in the estimation of the flood quantiles. Use of a GLS-based regression method also allows differentiation between sampling error and model error and thus provides a more realistic framework for error analysis. The GLS based quantile regression technique has been adopted in the US (see, for example, (Stedinger and Tasker, 1985; Tasker and Stedinger, 1989; Griffis and Stedinger, 2007)).

As an alternative to the quantile regression technique, the parameters of a particular probability distribution can be regressed against the catchment characteristics to develop prediction equations for the parameters of interest. This method is referred to as Parameter Regression Technique (PRT). It is this approach that is the basis of the recommended technique for design flood estimation in Australia using the RFFE model. For development of a PRT, the Bayesian GLS regression method is used to develop prediction equations for the model parameters for the three-parameter Log Pearson III (LP III) distribution; these model parameters are the mean, standard deviation and skewness of the natural logarithm of the annual maximum flood series. The PRT offers three significant advantages over the quantile regression technique:

1. It ensures flood quantiles increase smoothly with decreasing Annual Exceedance Probability (AEP), an outcome that may not always be achieved with quantile regression;

2. It is straightforward to combine any at-site flood information with regional estimates (see Book 3, Chapter 2) using the approach described by Micevski and Kuczera (2009) to produce more accurate quantile estimates; and

3. It permits quantiles to be estimated for any AEP in the range of interest.

3.2.4. Data Required to Develop Regional Flood Frequency Estimation Technique

The success of a RFFE technique largely depends on the quantity and quality of the available data and the capability of the adopted statistical techniques to transfer information from gauged to ungauged sites within the region. There are two basic types of data required for the development and application of RFFE techniques:

1. Flood data at gauged sites; and

2. Catchment characteristics relevant to production of floods in both gauged and ungauged catchments.

The quality and representativeness of the flood data determine to a large degree the accuracy and reliability of regional flood estimates. The challenge in collating a database for RFFE lies in maximising the amount of useful flood information, while minimising the random and systematic error (or ‘noise’) that may be present in some flood data.

In RFFE, various sources of errors in data and their effects on final flood estimates need to be recognised. The accuracy of flood quantile estimates at each individual gauged site depends largely on rating curve accuracy and record length at the individual site. Thus, the selection of a minimum record length at an individual site in the region is a very important step in any RFFE technique; the record length should be as long as possible while retaining enough sites in the region to make the results of RFFE useful. Also, the flood data at each site should satisfy a number of basic assumptions e.g. homogeneity, independence and stationarity. In the case where these assumptions are violated, appropriate measures should be taken; suitable techniques include GLS regression to account for the inter-station correlation (e.g. (Stedinger and Tasker, 1985; Griffis and Stedinger, 2007)), and non-stationary Flood Frequency Analysis to account for the impacts of climate change and changes in catchment conditions during the period of record. The data preparation issues for a RFFE technique are discussed in more detail by Haddad et al. (2010) and (Rahman et al., 2015a; Rahman et al, 2015; Rahman et al., 2015b; Rahman et al., 2015c). In addition to the consideration of the data available at individual gauging stations, it is important to also consider the representativeness of the gauged data. The available gauged catchments represent only a sample of the range of conditions that may occur throughout the regions where the RFFE method is developed and applied.

The transfer of flood information from gauged to ungauged catchments relies on the ability to identify a number of key catchment and climate characteristics which determine similarities and differences in the flood production of catchments. In the Probabilistic Rational Method similarity is assumed to exist on the basis of geographical proximity, but in other methods an appropriately small but informative set of predictor variables needs to be identified. The accuracy of a RFFE technique does not necessarily increase with the number of adopted predictor variables. For example, Rahman et al. (1999) used 12 predictor variables in an L-moments based Index Flood Method for south-east Australia. A subsequent study by Rahman (2005) showed that use of only 2 to 3 predictor variables can provide a similar level of accuracy.

Because of the difficulty of obtaining a sufficiently large number of gauged catchments in developing and testing a RFFE method, there is a chance that the available gauged catchments do not fully represent the range of conditions encountered in the region, Therefore where a catchment is judged to be atypical in some critical characteristic, the regional method may not be directly applicable and further analysis may be needed to estimate design flood quantiles. This issue is discussed further below.

3.2.5. Accuracy Considerations

All RFFE techniques are subject to uncertainty, which, generally, is likely to be greater than for at-site Flood Frequency Analysis when a good quality and long record of streamflow data set is available at the location of interest. A RFFE technique essentially represents a ‘transfer function’ that converts predictor variables to a flood quantile estimate. It is assumed that use of a limited number of predictor variables (e.g. catchment area and design rainfall intensity) combined with an optimised transfer function captures the general nature of the rainfall-runoff relationship for flood events and hence provides flood quantile estimates of ‘acceptable’ accuracy.

Because a RFFE technique typically has limited predictive power, design flood estimates produced by it are likely to have a lower degree of accuracy than those from a well calibrated catchment modelling system. From the investigations made by (Rahman et al., 2009; Rahman et al., 2012; Rahman et al., 2015a; Rahman et al, 2015), it may be stated that the relative accuracy of regional flood estimates using the RFFE model presented in this chapter is likely to be within ±50% of the true value; however, in a limited number of cases the estimation error may exceed the estimation by a factor of two or more (see Book 3, Chapter 3, Section 7). It is unlikely that any RFFE technique would be able to provide flood quantile estimates which are of much greater accuracy given the current availability of streamflow data (in terms of temporal and spatial coverage) and feasibility of the extraction of a greater number of catchment descriptors using simplified methods such as GIS based techniques. Because of the small sample of gauged catchments and limited availability of readily obtainable catchment descriptors, it is not possible to prepare an extremely detailed set of descriptor variables covering all possible conditions, so a sample must be selected that provides a suitable range to represent the critical parameters, but to limit the application of variables that do not contribute significantly to the overall performance of the RFFE technique.

For catchments having limited recorded streamflow data, the combination of at-site data with a RFFE technique is likely to provide more accurate flood quantile estimates than either the at-site or regional method alone. Details of how limited streamflow data can be combined with a RFFE technique are presented in Book 3, Chapter 2. Testing has shown this improves estimates in many cases.

An important assumption in all RFFE techniques is that the small set of predictor variables used in the regression equations is able to explain the differences in flood producing characteristics of the catchments in a region. Not all ungauged catchments located in the region satisfy this basic homogeneity assumption; some catchments may have characteristics that are substantially different from the gauged catchments in the region.

Book 3, Chapter 3, Section 13 contains further discussion on the limits of applicability of the RFFE technique, on what constitutes an atypical catchment and recommendations on how to derive flood estimates for such catchments.

3.3. Statistical Framework

3.3.1. Region Of Influence (ROI) Approach

In the formation of regions, the Region Of Influence (ROI) approach has been adopted for the parts of Australia where there are adequate numbers of gauged stations within close proximity to form ROI sub-regions. In the absence of a proven technique for the use of catchment characteristics as the basis for the ROI, the adopted ROI approach uses the geographical distance between stations as the distance metric and sets a maximum distance for inclusion of stations in the ROI sub-region. More details regarding development of ROI sub-regions are provided by (Rahman et al, 2015). Nonetheless, a summary of the process is presented. In applying the ROI approach, in the first iteration, a ROI sub-region consisting of the ten nearest stations to the site of interest is formed, the regional prediction equation is developed and its prediction error variance noted. At each of the subsequent iterations, the radius of the ROI sub-region is increased by 10 km and new stations are added to the previously selected stations. The final ROI sub-region for the location of interest is then selected as the one exhibiting the lowest prediction error variance.

One of the apparent limitations of the ROI approach is that for each of the gauged sites in the region, the regional prediction equation has a different set of model parameters; hence a single regional prediction equation cannot be pre-specified. To overcome this problem, the parameters of the regional prediction equations for all the gauged catchment locations are pre-estimated and integrated with the RFFE Model 2015 (see Book 3, Chapter 3, Section 14 for more details). To derive flood quantile estimates at an ungauged location of interest, the RFFE Model 2015 uses a natural neighbour interpolation method to derive quantile estimates based on up to the 15 nearest gauged catchment locations within a 300 km radius from the location of interest. This ensures a smooth variation of flood quantile estimates over the space.

3.3.2. Parameter Regression Technique

In the adopted RFFE technique for the Humid Coastal areas of Australia (see Book 3, Chapter 3, Section 4 for further details), the first three moments of the LP III distribution (i.e. the mean, standard deviation and skewness of the natural logarithms of the annual maximum flood series) were regionalised. This method is referred to as the Parameter Regression Technique (PRT). The LP III distribution is described by the following equation:

$$\ln Q_x = M + K_x S \tag{3.3.1}$$

where Q_x is the discharge having an AEP of X% (design flood or flood quantile),

M is the mean of the natural logarithms of the annual maximum flood series,

S is the standard deviation of the natural logarithms of the annual maximum flood series, and

K_x is the frequency factor for the LP III distribution of X% AEP, which is a function of the AEP and the skewness (SK) of the natural logarithms of the annual maximum flood series.

The prediction equations for M, S and SK were developed for all the gauged catchment locations in the Humid Coastal areas using Bayesian GLS regression and model parameters were noted. These model parameters are then integrated with the RFFE Model 2015.

3.3.3. Generalised Least Squares Regression

In developing the prediction equations, for the Humid Coastal areas, the Bayesian Generalised Least Squares (GLS) regression was adopted. The GLS regression assumes that the variable of interest (e.g. a moment of the LP III distribution) denoted by y_i for a location i can be described by a function of catchment characteristics (explanatory variables) with an additive error (Griffis and Stedinger, 2007):

$$y_i = \beta_0 + \sum_{j=1}^{k} \beta_j X_{\text{ij}} + \delta_i, i = 1, 2, \ldots, n \tag{3.3.2}$$

where X_{ij} (j = 1,…, k) are explanatory variables, β_j are the regression coefficients, δ_i is the model error which is assumed to be normally and independently distributed with model error variance σ_δ^2 , and n is the number of locations in the region. In all cases, an at-site estimate of y_i denoted as $\widehat{y_i}$ is available. To account for the error in the at-site estimate, a sampling error η_i must be introduced into the model so that:

$$\hat{y} = \mathrm{X}\upbeta + \eta + \delta = \mathrm{X}\upbeta + \varepsilon \text{where} \hat{y_i} = \gamma_i + \eta_i \text{: i} = 1,2 \ldots,\text{n} \tag{3.3.3}$$

Thus the observed regression model error ε is the sum of the model error δ and the sampling error η. The total error vector has a mean of zero and a covariance matrix:

$$\mathrm{E}[\varepsilon\varepsilon^T] = \Lambda(\sigma_\delta^2) = \sigma_\delta^2 \, \mathrm{I} + \Sigma(\hat{\mathrm{y}}) \tag{3.3.4}$$

where$\Sigma(\hat{\mathrm{y}})$ is the covariance matrix of the sampling error in the estimate of the flood quantile or the parameter of the LP III distribution and I is a (n x n) identity matrix. The covariance matrix for η_i depends on the record length available at each location and the cross correlation among annual maximum floods at different locations. Therefore, the observed regression model error is a combination of time-sampling error η_i and an underlying model error δ_i.

The GLS estimator of β and its covariance matrix for a known σ_δ^2 is given by:

$$\beta_{GLS} = \left[X^T \Lambda\left(\sigma_\delta^2\right)_e^{-} X\right]_e^{-} X^T \Lambda(\sigma_\delta^{-})\hat{y} \tag{3.3.5}$$

$$\Sigma[\beta_{GLS}] = \left[X^T \Lambda\left(\sigma_\delta^2\right)_e^{-} X\right]_e^{-} \tag{3.3.6}$$

The model error variance σ_δ^2 can be estimated by either generalised Method of Moments (MOM) or maximum likelihood estimators. The MOM estimator is determined by iteratively solving Equation (3.3.5) along with the generalised residual mean square error equation:

$$\left(\hat{y} - X\beta_i\left[\sigma_\delta^2 I + \Sigma(\hat{y})\right]_e^{-}\right)\left(\hat{y} - X\beta_{GLS}\right) = n - (k+1) \tag{3.3.7}$$

In some situations, the sampling covariance matrix explains all the variability observed in the data, which means the left-hand side of Equation (3.3.7) will be less than n – (k + 1) even if

σ_δ^2 is zero. In these circumstances, the MOM estimator of the model error variance is generally taken to be zero.

With the adopted Bayesian approach, it was assumed that there was no prior information on any of the β parameters; thus a multivariate normal distribution with mean zero and a large variance (e.g. greater than 100) was used as a prior for the regression coefficient parameters. This prior was considered to be almost non-informative, which produced a probability distribution function that was generally flat in the region of interest. The prior information for the model error variance σ_δ^2 was represented by a one parameter exponential distribution. Further description of the adopted Bayesian GLS regression can be found in Haddad et al. (2011) and Haddad and Rahman (2012).

3.3.4. Development of Confidence Limits for the Estimated Flood Quantiles

In developing the confidence limits for the estimated flood quantiles, a Monte Carlo simulation approach was adopted by assuming that the uncertainty in the first three parameters of the LP III distribution (i.e. the mean, standard deviation and skewness of the logarithms of the annual maximum flood series) can be specified by a multivariate normal distribution. Here the correlations among the three parameters for a given region were estimated from the residuals of the GLS regression models of the LP III parameters. The mean of the LP III parameter is given by its regional predicted value and the standard deviation of the LP III parameter is the square root of the average variance of prediction of the parameter at the nearest gauged location. Based on 10 000 simulated values of the LP III parameters from the multivariate normal distribution as defined above, 10 000 Q_x values are estimated in the RFFE Model 2015, which are then used to develop the 90% confidence intervals.

3.4. RFFE Techniques for Humid Coastal Areas

3.4.1. Data Used to Develop RFFE Technique

3.4.1.1. Flood data

Humid Coast areas had a relatively large number of recorded streamflow stations, as shown in Figure 3.3.1. An upper limit of catchment size of 1000 km² was generally adopted. However, in some states (such as the Northern Territory), a few larger catchments were included as the total number of catchments with areas less than 1000 km² was too small. The cut-off record length of 19 years was selected to maximise the number of eligible stations on the consideration that a higher cut-off would reduce that number.

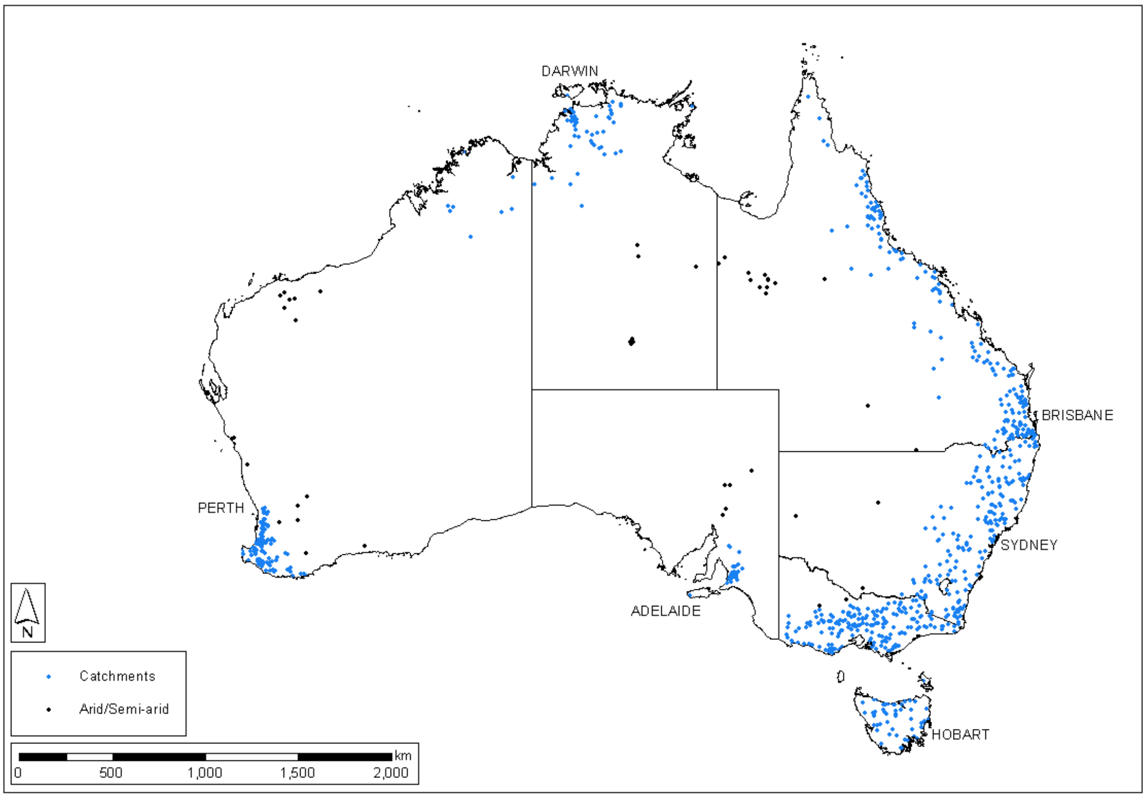


Figure 3.3.1. Geographical Distribution of the Adopted 798 Catchments from Humid Coastal Areas of Australia and 55 Catchments from Arid/Semi-arid Areas

The selected streams were unregulated since major regulation (e.g. a large dam on the stream) affects the rainfall-runoff relationship significantly by increasing storage effects. Streams with minor regulation, such as small farm dams and diversion weirs, were not excluded because this type of regulation is unlikely to have a significant effect on large annual floods. Gauging stations on streams subject to major upstream regulation were excluded from the data set. Catchments with more than 10% of the area affected by urbanisation were also excluded from the study data set. Catchments known to have undergone major land use changes, such as the clearing of forests or changing of agricultural practices over the period of streamflow records were excluded from the data set. Stations graded as 'poor quality' or with specific comments by the gauging authority regarding quality of the data were assessed in greater detail; if stations were deemed 'low quality' they were excluded.

The annual maximum flood series data may be affected by multi-decadal climate variability and climate change, which are not easy to deal with. The effects of multi-decadal climate variability can be accounted for by increasing the cut-off record length at an individual station. However, the impacts of climate change present a serious problem in terms of the applicability of the past data in predicting future flood frequency; this requires further research (Ishak et al., 2013). This is further discussed in Book 3, Chapter 3, Section 10.

The data sets for the initially selected potential catchments are further examined, as detailed in Haddad et al. (2010), Rahman et al. (2015a), Rahman et al (2015), and Rahman et al. (2015b): gaps in the annual maximum flood series was filled as far as could be justified, outliers were detected using the multiple Grubbs-Beck test (Lamontagne et al., 2013; Cohn et al., 2013), errors associated with extrapolation of rating curves were investigated and the presence of trends with the data were checked. From an initial number of approximately 1200 catchments, a total of 798 catchments were finally adopted from all over Australia (excluding catchments in the arid/semi-arid areas, where some of the above criteria were relaxed, as discussed in Section 5).

The record lengths of the annual maximum flood series of these 798 stations range from 19 to 102 years (median: 37 years). The catchment areas of the selected 798 catchments range from 0.5 km² to 4325 km² (median: 178 km²). Table 3.3.1 provides summary of the selected catchments from Humid Coastal areas of Australia.

Table 3.3.1. Summary of Adopted Catchments from Humid Coastal Areas of Australia

State	No. of Stations	Streamflow Record Length (years) (range and median)	Catchment Size (km^2) (range and median)
New South Wales & Australian Capital Territory	176	20 – 82 (34)	1 – 1036 (204)
Victoria	186	20 – 60 (38)	3 – 997 (209)
South Australia	28	20 – 63 (37)	0.6 – 708 (62.6)
Tasmania	51	19 – 74 (28)	1.3 – 1900 (158.1)
Queensland	196	20 – 102 (42)	7– 963 (227)
Western Australia	111	20 – 60 (30)	0.5 – 1049.8 (49.2)
Northern Territory	50	19 – 57 (42)	1.4 – 4325 (352)
TOTAL	798	19 – 102 (37)	0.5 – 4325 (178)

The at-site Flood Frequency Analyses were conducted using the FLIKE software (Kuczera, 1999). The Potential Influential Low Flows (PILFs) were identified using multiple Grubbs-Beck test (Lamontagne et al., 2013) and were censored in the Flood Frequency Analysis. A Bayesian parameter estimation procedure with LP III distribution was used to estimate flood quantiles for each gauged site for AEPs of 50%, 20%, 10%, 5%, 2% and 1%.

3.4.1.2. Catchment Characteristics Data

As discussed by Rahman et al. (2009), Rahman et al. (2012), over ten predictor variables were selected initially; however, it was found that the accuracy of a RFFE technique does not necessarily increase with the number of adopted predictor variables. A total of five candidate predictor variables were adopted finally in the RFFE technique, as outlined below:

(i) catchment area in km² (*area*);

(ii) design rainfall intensity at catchment centroid (in mm/h) for the 6 hour duration and 50% AEP (${}^{50\%}I_{6h}$);

(iii) design rainfall intensity at catchment centroid (in mm/h) for the 6 hour duration and 2% AEP (${}^{2\%}I_{6h}$);

(iv) ratio of design rainfall intensities of AEPs of 2% and 50% for duration of 6 hour (${}^{2\%}I_{6h}/{}^{50\%}I_{6h}$); and

(v) *shape factor*, which is defined as the shortest distance between catchment outlet and centroid divided by the square root of catchment area.

Design rainfall values were extracted from the new Intensity Frequency Duration (IFD) data for Australia as discussed in Book 2, Chapter 3 of ARR. Table 3.3.2 provides the distribution of shape factors for the selected catchments.

Table 3.3.2. Distribution of Shape Factors for the Selected Catchments

Percentile	1%	10%	50%	90%	99%
Shape Factor	0.32	0.51	0.76	1.06	1.51

3.4.2. Adopted RFFE Regions

Australia is divided into seven regions. There are five Humid Coastal regions (Table 3.3.5) and two arid/semi-arid regions (Table 3.3.4), as shown in Figure 3.3.2. There are seven fringe zones that are the interface between two regions, as discussed in Book 3, Chapter 3, Section 6.

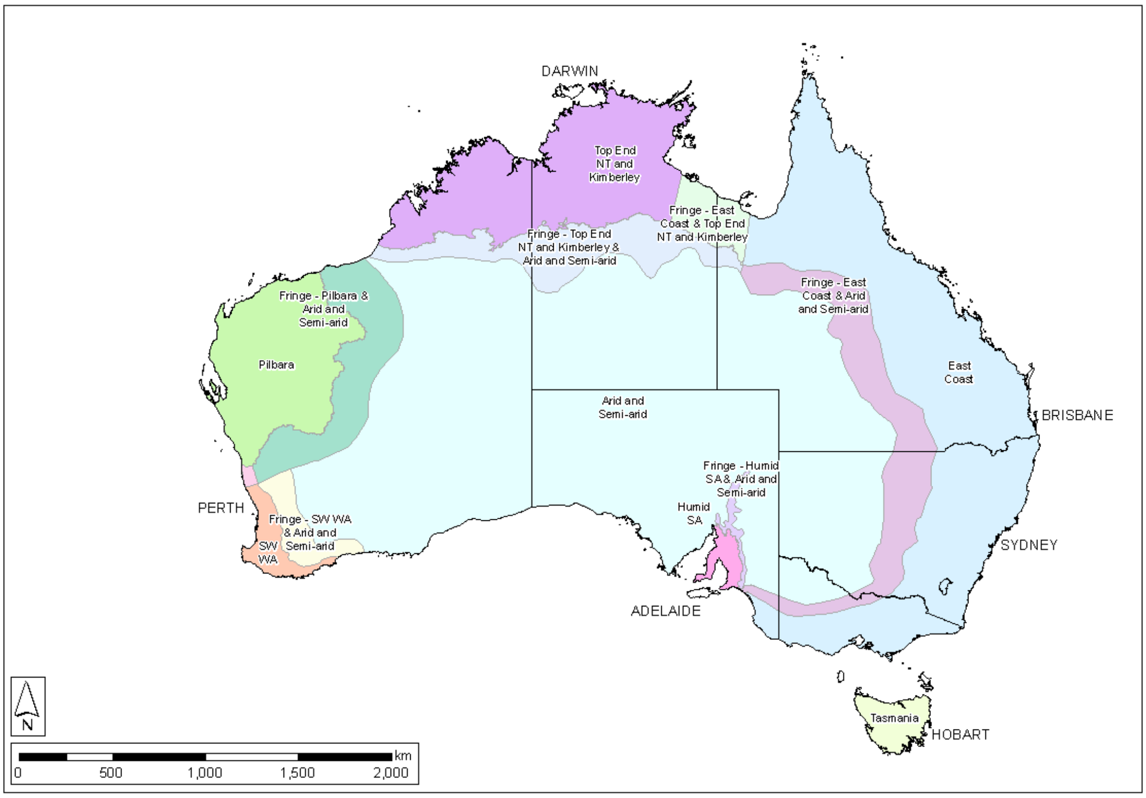


Figure 3.3.2. Adopted Regions for RFFE Technique in Australia

Table 3.3.3. Details of RFFE Technique for Humid Coastal Areas of Australia

Region	Method to form region	Number of stations	Estimation model
Region 1: East Coast	ROI (based on geographical proximity)	558	Bayesian GLS regression-PRT
Region 2: Tasmania		51	
Region 3: Humid SA		28	

Region	Method to form region	Number of stations	Estimation model
Region 4: Top End NT and Kimberley		58	
Region 5: SW WA		103	

3.4.3. Adopted Estimation Equations

For the five Humid Coastal regions described in Table 3.3.3, the adopted estimation equations for M, S and *SK* for the regional LP III model (Equation (3.3.1)) have the following general form:

$$M = b_0 + b_1(\ln(area)) + b_2\left(\ln\left(I_{6,50}\right)\right) + b_3(\ln(\text{shape factor})) \quad (3.3.8)$$

$$S = c_0 + c_1 \ln\left(\frac{I_{6,2}}{I_{6,50}}\right) \quad (3.3.9)$$

$$SK = d_0 + d_1 \ln(\text{area}) + d_2 \ln\left(\frac{I_{6,2}}{I_{6,50}}\right) + d_3 \ln\left(I_{6,2}\right) \quad (3.3.10)$$

where, area is the catchment area (km²);

$I_{6,50}$ is the design rainfall intensity (mm/h) at catchment centroid for 6 hour duration and 50% AEP;

shape factor is the shortest distance between catchment outlet and centroid/*area*$^{0.5}$; and

$I_{6,2}$ is the design rainfall intensity (mm/h) at catchment centroid for 6 hour duration and 2% AEP.

For Region 1 and 2, only the model intercepts were used in Equation (3.3.9) and Equation (3.3.10), which imply a regression equation without any predictor variable. Here, the weighted average values of S and SK were adopted, record lengths at the stations within the ROI sub-region were used as a basis for determining the weights.

The values of b_0, b_1, b_2, b_3, c_0, c_1, d_0, d_1, d_2 and d_3 at all the 798 individual gauged catchment locations (in the Humid Coastal areas) were estimated and embedded in the RFFE Model 2015. To derive flood quantile estimates at an ungauged location of interest, the RFFE Model 2015 uses a natural neighbour interpolation method to derive quantile estimates based on up to the 15 nearest gauged catchment locations within 300 km radius from the location of interest. This ensures a smooth variation of flood quantile estimates over space.

3.5. RFFE Techniques for Arid/Semi-Arid Areas

3.5.1. Data Used to Develop RFFE Technique

Most of Australia's interior falls into the arid/semi-arid areas, which are characterised by low mean annual rainfall in relation to mean annual potential evaporation. Rainfall events tend to be infrequent and their occurrence and severity are highly variable. Typically dry antecedent conditions may result in many rainfall events not producing any significant runoff. However, severe rainfall events can still result in significant flooding with serious consequences. Large transmission losses (i.e. losses occurring from flow in rivers and other drainage channels) may also result in discharge reducing in a downstream direction, particularly in the lower river reaches of larger catchments in arid areas. The special flooding characteristics of catchments in arid/semi-arid areas make it desirable to treat them separately from catchments in more humid areas. In arid/semi-arid areas, annual maximum flood series generally contain many zero values; these values were censored in flood frequency analysis.

In ARR 1987 (Pilgrim, 1987), only a few catchments were used from arid/semi-arid areas to develop RFFE methods. Since the publication of ARR 1987, there has been little improvement in terms of streamflow data availability in most of the arid/semi-arid areas of Australia. There are a number of reasons for poor stream gauging coverage and quality in arid/semi-arid areas, as follows:

1. Poorly defined water course catchments, meaning that the water courses are hard to define and therefore gauge.

2. Large floods may be outside the water course, meaning that the cross-section may be hard to measure and the flow may be difficult to gauge.

3. Infrequent flood events, meaning that the gauge may be operating for extended periods of time without any flow.

4. Remote location, making it difficult to take velocity measurements during the flood events.

5. Little incentive to gauge flows because of the perceived limited water resources and the limited demand for development of these resources.

In the preparation of the Regional Flood Frequency Estimation database, only a handful of catchments from the arid/semi-arid areas satisfied the selection criteria. To increase the number of stations from the arid/semi-arid areas to develop a RFFE method, the selection criteria were relaxed i.e. the threshold streamflow record length was reduced to 10 years and the limit of catchment size was increased from 1000 km² to 6000 km². These criteria resulted in the selection of 55 catchments from the arid/semi-arid areas of Australia (Figure 3.3.2 and Table 3.3.4). The selected catchments have average annual rainfall less than 500 mm. The catchment areas range from 0.1 to 5975 km² (median: 259 km²) and streamflow record lengths range from 10 to 46 years (median: 27 years).

Table 3.3.4. Summary of adopted stations from arid/semi-arid areas of Australia

Location	No. of stations	Streamflow record length (years) (range and median)	Catchment size (km^2) (range and median)
Pilbara	11	22 – 34 (28)	0.2 – 5975 (303)
Arid and Semi-arid	44	10 – 46 (27)	3 – 997 (209)
TOTAL	55	10 – 46 (27)	0.1 – 5975 (259)

3.5.2. Adopted Regions

The definition of regions in the arid/semi-arid areas in Australia is a difficult task, as there are only 55 catchments available over a vast area of Australia. There are two alternatives: (i) formation of one region with all the 55 stations; and (ii) formation of smaller sub-regions based on geographical proximity, noting that too small a region makes the developed RFFE technique of little statistical significance. Examination of a number of alternative sub-regions led to the formation of two regions from the 55 arid/semi-arid catchments: Region 6 (11 catchments from the Pilbara area of WA) and Region 7 (44 catchments from all other arid areas except Pilbara) (see Figure 3.3.2 for the extent of these two arid/semi-arid regions and Table 3.3.5 for other details).

Table 3.3.5. Details of RFFE technique for arid/semi-arid regions

Region	Method to form region	Number of stations	Estimation model
Region 6: Pilbara	Fixed region	11	Index flood method with Q_{10} as index variable
Region 7: Arid and Semi-arid		44	

3.5.3. Adopted Estimation Equations

Application of the ROI and PRT methods for arid/semi-arid regions was deemed inappropriate as the ROI due to an insufficient number of gauges. Hence a simpler RFFE method was considered more appropriate for the two arid/semi-arid regions. Here, an index type approach as suggested by Farquharson et al. (1992) is adopted. The 10% AEP flood quantile (Q_{10}) was used as the index variable and a dimensionless Growth Factor for X% AEP (GF_x) was used to estimate Q_x:

$$Q_x = Q_{10} \times \mathrm{GF}_x \tag{3.3.11}$$

A prediction equation was developed for Q_{10} as a function of catchment characteristics, and regional growth factors were developed based on the estimated at-site flood quantile. In the arid areas, significant storm events do not typically occur every year, and some of these events do not produce significant floods. The at-site Flood Frequency Analyses for the arid catchments was conducted using the FLIKE software (Kuczera, 1999). The Potential Influential Low Flows (PILFs) were identified using multiple Grubbs-Beck test (Lamontagne et al., 2013) and were censored in the Flood Frequency Analysis. A Bayesian parameter estimation procedure with LP III distribution was used to estimate flood quantiles for each gauged site for AEPs of 50%, 20%, 10%, 5%, 2% and 1%. It should be noted that the Flood Frequency Analysis procedure adopted in the Humid Coastal and arid areas was the same.

The Q_x/Q_{10} values were estimated at individual stations; the weighted average of these values (weighting was done based on record length at individual stations) over all the stations in a region then defined the Growth Factors (GF_x) for the region.

The adopted prediction equation for the index variable Q_{10} has the following form:

$$\log_{10} = b_0 + b_1 \log_{10}(\text{area}) + b_2 \log_{10}\left(I_{6,50}\right) \tag{3.3.12}$$

where b_0, b_1 and b_2 are regression coefficients, estimated using ordinary least squares regression; area represents catchment area in km², and $I_{6,50}$ is the design rainfall intensity (mm/h) at catchment centroid for 6 hour duration and 50% AEP. The values of b_0, b_1 and b_2 and the regional Growth Factors (GF_x) are embedded into the RFFE Model 2015.

3.6. Fringe Zones

The boundaries between the arid/semi-arid and Humid Coastal regions in Figure 3.3.2 were drawn (as a smoothed line) approximately based on the 500 mm mean annual rainfall contour. To reduce the effects of sharp variation in flood estimates for the ungauged catchments located close to these regional boundaries, seven fringe zones were delineated, as shown Figure 3.3.2. The boundary of the fringe zone with the Humid Coastal region was approximately defined by the 500 mm mean annual rainfall isohyet, while the other side was defined by the 400 mm mean annual rainfall isohyet to establish a fringe zone. In drawing the boundary, some minor adjustment was made to make the boundary as smooth as possible.

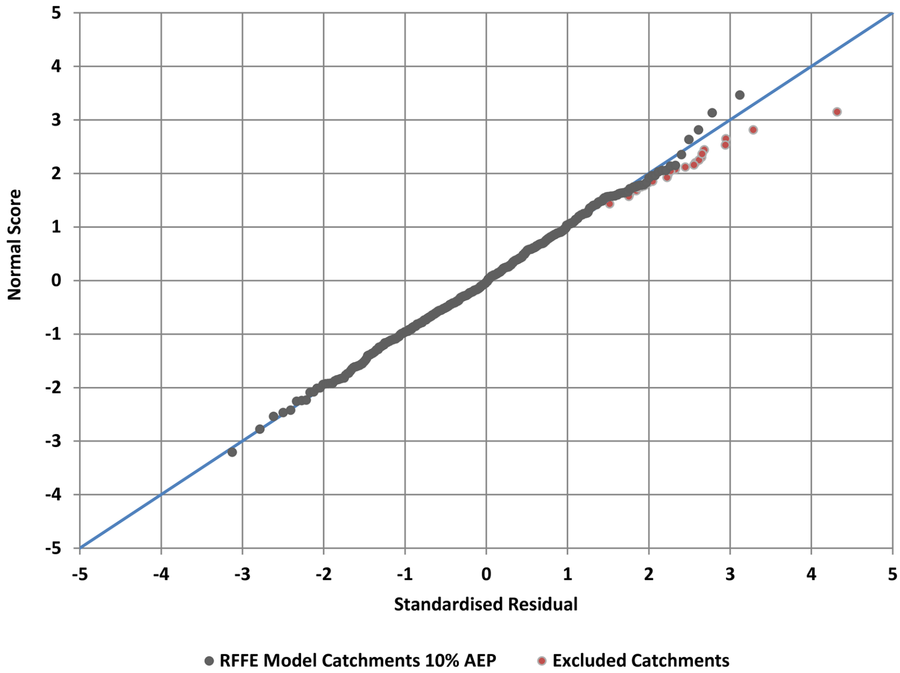
For these fringe zones, the flood estimate at an ungauged catchment location is taken as the inverse distance weighted average value of the flood estimates based on the two nearest regions. The method is embedded into the RFFE Model 2015.

3.7. Relative Accuracy of the RFFE Technique

The reliability and accuracy of the RFFE quantile estimates were assessed using leave-one-out (LOO) validation. In the LOO validation, one catchment was left out from the model data set and the RFFE technique is applied to the catchment that was left out. The flood quantiles estimated using the RFFE technique were then compared with the at-site flood frequency estimates obtained by FLIKE (Kuczera, 1999) as mentioned in Book 3, Chapter 3, Section 5. The procedure was repeated for each catchment in the regional data set to provide an overall assessment of the performance of the RFFE technique.

The reliability of the RFFE flood quantile confidence limits described in Book 3, Chapter 3, Section 3 was assessed empirically using standardised quantile residuals. The quantile residual is the difference between the logarithm of flood quantile estimates obtained using at-site Flood Frequency Analysis and the RFFE technique. The standardised quantile residual is the quantile residual divided by its standard deviation which is the square root of the sum of the RFFE predictive variance of the flood quantile and at-site quantile variance (Haddad and Rahman, 2012; Micevski et al., 2015). This accounts for both the model error (e.g. inadequacy of the RFFE model) and the sampling error (e.g. due to limiteations in streamflow record length). If the uncertainty in the log quantile estimates has been adequately described, the standardised quantile residuals should be consistent with a standard normal distribution.

Figure 3.3.3 shows the plots of standardised residuals vs. normal scores for Region 1 for AEPs of 10% and 5% (the plots for other AEPs for Region 1 are provided in Book 3, Chapter 3, Section 16). Plots for other regions can be seen also in Rahman et al (2015). These residuals were estimated without bias correction (details of bias correction are provided in Book 3, Chapter 3, Section 8). These plots include 558 catchments from Region 1 used in the RFFE model and some 28 catchments which were excluded from the final RFFE model data set based on the results of preliminary analysis and unusual characteristics such as significant natural floodplain storage. Figure 3.3.3 reveals that most of the 558 catchments closely follow a 1:1 straight line indicating that the assumption of normality of the residuals was not inconsistent with the evidence; this is supported by the application of the Anderson-Darling and Kolmogorov-Smirnov tests which showed that the assumption of the normality of the residuals cannot be rejected at the 10% level of significance. Under the assumptions of normality, approximately 90% of the standardised quantile residuals should lie between ± 2, which is largely satisfied. There were a few catchments with standardised residual values close to ± 3. These correspond to instances where the RFFE confidence limits may not be reliable; an example of such a catchment is provided in Book 3, Chapter 3, Section 15. The same conclusion applies to the other Humid Coastal regions.



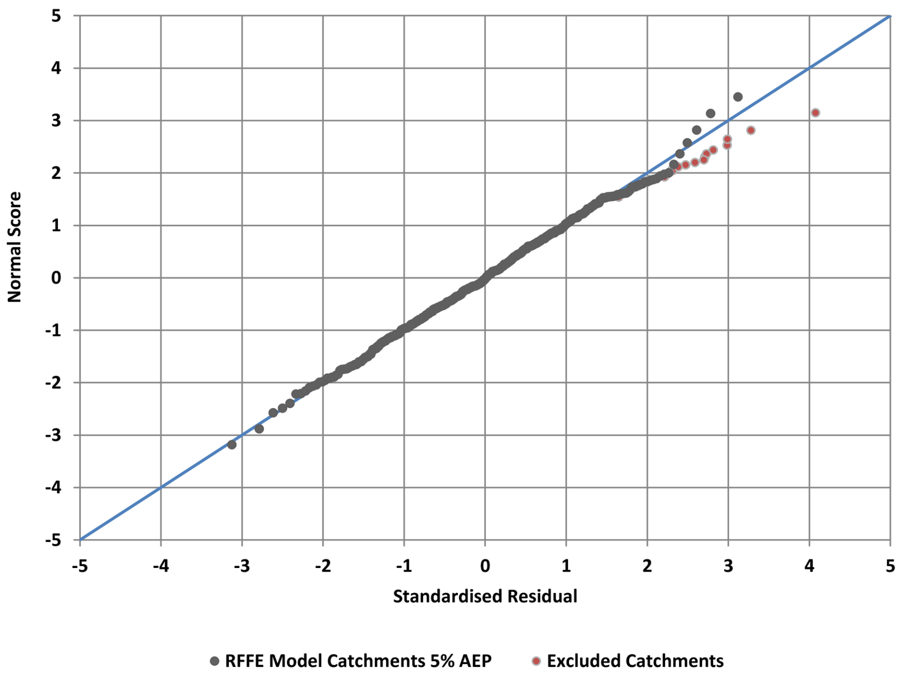


Figure 3.3.3. Standardised Residuals vs. Normal Scores for Region 1 Based on Leave One Out Validation for AEPs of 10% and 5%

The main conclusion from this analysis is that the quantification of uncertainty in the quantile estimates by the RFFE technique is reliable for the vast majority of the cases. Figure 3.3.3 and Book 3, Chapter 3, Section 15 serve as a reminder that some catchments may not be adequately represented by the catchments used in the RFFE analysis. Users of the RFFE Model 2015 should check that the catchment of interest is not atypical compared with the gauged catchments included in the ROI used to develop the RFFE estimate. To assist users in this regard the RFFE Model 2015 discussed in Book 3, Chapter 3, Section 14 and Book 3, Chapter 3, Section 13 lists the RFFE Model gauged catchments located nearest to the ungauged catchment of interest.

The accuracy of the flood quantile estimates provided by the RFFE technique is evaluated by using the Relative Error (RE) defined as:

$$\mathrm{RE}(\%) = \frac{Q_{\mathrm{RFFE}} - Q_{\mathrm{FFA}}}{Q_{\mathrm{FFA}}} \times 100 \tag{3.3.13}$$

where Q_{RFFE} is the flood quantile estimate for a given site for a given AEP by the RFFE technique; and Q_{FFA} is the flood quantile estimate from the at-site Flood Frequency Analysis (see Book 3, Chapter 3, Section 4 for details).

It should be noted that the Relative Error given by Equation (3.3.13) makes no allowance for the fact that the at-site flood frequency estimates are themselves subject to sampling error. Therefore, this error should be seen as an upper bound on the true relative error.

It should be noted here that LOO is a more rigorous validation technique compared with the split-sample validation where the model is tested on a smaller number of catchments (e.g. 10% of the total catchments). Hence, the relative error that is generated by LOO is expected to be higher than if split-sample validation were used. The medians of the absolute relative error values from the LOO validation for different regions are reported in Table 3.3.6. It can be seen that for the Humid Coastal regions, Region 5 (SW WA) has the highest relative error (59 to 69%) and Region 3 (Humid SA) has the smallest relative error (33 to 41%).

3.8. Bias Correction

In the spatial plots of the standardised residuals, a few cluster of notable underestimation and overestimation were detected. To overcome this problem, a bias correction was implemented, which attempeds to provide estimates of *M* (for Humid Coastal areas) (Equation (3.3.8)) and Q_{10} (for arid/semi-arid areas) (Equation (3.3.12)) closer to the observed *M* or Q_{10} at the location of each of the 853 model gauged catchments. This correction was based on an additive bias correction factor $M_{\mathrm{FFA}} - M_{\mathrm{RFFE}}$, (obtained at each of the model 798 gauged catchments in the Humid Coastal areas) and ${Q_{10,\mathrm{FFA}}}/{Q_{10,\mathrm{RFFE}}}$ (obtained at each of the model 55 catchments in arid/semi-arid areas). In practice, for an ungauged location of interest, an interpolated bias correction factor is calculated by the software using Natural Neighbour approach based on nearby 15 gauged catchments. It was found that this approach reduced the clusters of bias in the spatial plots of the standardised residuals and hence was adopted in the RFFE Model 2015.

Table 3.3.6. Upper bound on (absolute) median relative error (RE) from leave-one-out validation of the RFFE technique (without considering bias correction)

Median RE (%)						
	AEP					
Region	50%	20%	10%	5%	2%	1%
Region 1: East Coast	51	49	52	53	57	59
Region 2: Tasmania	53	46	46	46	46	45
Region 3: Humid SA	38	39	33	35	39	41

Median RE (%)						
Region 4: Top End NT and Kimberley	33	36	36	38	39	47
Region 5: SW WA	61	59	66	68	68	69
Region 6: Pilbara	35	37	35	42	37	43
Region 7: Arid and Semi-arid	63	67	67	61	57	49

3.9. Combining Regional and At-Site Quantile Estimates

Unless a station has a very long annual maximum flood series data (e.g. greater than 100 years), it is desirable to combine at-site and regional flood frequency quantile estimates to achieve a more accurate estimate. The RFFE Model 2015 (see Book 3, Chapter 3, Section 14 for the description of the model) provides the necessary parameter set and data in its output to combine the regional flood quantile estimates with the at-site data. This is discussed in more detail in Book 3, Chapter 2.

3.10. Impact of Climate Change

The adopted regional estimation equations include design rainfall intensity as a key input variable. As a result the impact of climate change as reflected in changes in design rainfall estimates (which are expected to be upgraded regularly) can be propagated through the RFFE Model 2015 to provide a first order estimate of the impact of climate change on regional design flood estimates. This approach assumes that that the contribution of the other catchment characteristics affecting flood production does not change with future climate state. Given that this assumption is unlikely to be valid (Ball, 2014) the use of potential future rainfall intensities in the RFFE Model 2015 will indicate the sensitivity of the design flood estimates with respect to climate change state. Further research on the impact of climate change on rainfall and flood in Australia will allow updating of the RFFE Model 2015 by incorporating the impact of climate change.

3.11. Progressive Improvement of RFFE Model 2015 v1

It is expected that the RFFE Model 2015 v1 will be updated in future when the streamflow record lengths at the selected 853 gauged catchments increase significantly and/or when additional catchments are available that satisfy the criteria of catchment selection (Book 3, Chapter 3, Section 4). Furthermore, additional predictor variables can potentially be included in the RFFE Model to enhance model accuracy, in particular if GIS based techniques can be adopted to more readily extract the additional predictor variables such as stream density and main stream slope.

3.12. RFFE Implementation and Limitations

3.12.1. Overview of RFFE Accuracy

3.12.1.1. Data Coverage

The RFFE Model has been developed from a detailed analysis of all appropriate streamflow gauging stations throughout Australia. The selected 853 gauges met the criteria for application to the development of the procedure.

It must be recognised however that this is a small number of gauges in the 7.7 million km^2 area of Australia. In addition to the sparse gauge coverage, there is a significant variation in catchment types across Australia. The RFFE Model must use the available data to develop a regional procedure that can estimate flood quantiles with the best possible accuracy. Relevant catchment characteristics therefore may not be represented sufficiently to allow inclusion in the regional relationship.

There are insufficient gauges to provide a representative coverage of all catchment types throughout Australia. This is a particular concern in the arid and semi-arid interior.

The regional relationship implemented in ARR has therefore used only characteristics that are sampled sufficiently. These characteristics are catchment area, rainfall intensity parameters and shape factor. Other factors, such as land use, slope, soils, geology or vegetation, are not sampled sufficiently to allow inclusion in the parameters used for the procedure, even though they are known to be important in the estimation of flood quantiles.

The RFFE Model therefore may be regarded as providing a “generic” estimate of flood quantiles for a range of typical catchment types. It may be expected that different flood estimates would be derived for other catchment types that have catchment characteristics that are dissimilar to those used in development of the method. If there were a larger sample of gauged catchments in Australia and they sampled a wider variety of catchment types, it is expected that the RFFE Model would incorporate a wider range of catchment characteristics and would therefore be capable of more reliable estimates for a wider range of catchments.

The procedure is based on the analysis of the selected catchment characteristics, but improvements in the procedure are possible with additional analysis.

3.12.1.2. Data Accuracy

While there is a relatively small and possibly unrepresentative distribution of gauged catchments used in the development of the procedure, it is also noted that there are inaccuracies in the base data used to develop the procedure. The gauges were selected to ensure at least a minimum quality standard. However all gauges have some level of inaccuracy, which could be caused by extrapolation of rating curves, variability with rating curves over time, effects of backwater at the gauge, changing catchment conditions, or flow diversions and overflows that affect flows. These factors particularly impact on the quality of flow records for the larger flows, which are the important records for Flood Frequency Analysis.

Imposing a higher data quality standard will reduce the number of gauges for analysis while lowering the standard will result in greater errors in the data used to develop the procedure.

For the development of the RFFE Model, the decision has been to impose a minimum data quality standard, to ensure a maximum number of gauges were included.

It must be noted though that such data inaccuracies were not be identified during the development of the RFFE Model as the analysis involved a large number of stations and detailed assessment of each of these was not feasible. The meta-data and other documentation provided by the water agencies was not sufficiently detailed to allow a routine assessment of data quality, and it was necessary to rely on local advice as to the suitability of the data used.

The inclusion of this data may impact on the quality of the regional results, but does improve the representativeness of the data. While it is impossible to quantify this impact, the possible impact needs to be borne in mind.

3.12.1.3. Representative Periods of Record

The selection of the gauged catchments for analysis was based on the period of record with a minimum record length of 20 years. This record length could have been for the most recent period for stations that are still operating or it could have been for an earlier period for closed stations.

Longer periods of record are more likely to represent the variability in flood magnitude and also more likely to sample large floods that occur infrequently. However, even with a record length of more than 20 years, it is possible that the record may not sample a wide range of flood sizes and there is still a potential source of inaccuracy when extrapolating to larger floods. The samples from different time periods may also affect the flood quantiles since it is well known that there are longer term variations in the distribution of flood maxima due to inter-decadel variability in climate. This may result in differences in the design flood quantiles that are purely a result of the sample period rather than a real difference in flood probability.

3.12.2. RFFE Implementation

The estimates of flood quantiles from the RFFE Model must be regarded as a first approximation of the required quantiles. In many cases, the estimates from the RFFE Model may provide an acceptable result for application.

However, given the accuracy considerations discussed above, some additional testing and review is recommended.

The additional testing and review can include the following processes:

- Review the catchment characteristics for the catchment being analysed and assess whether this catchment is typical for catchments that have been used in the development of the method: This review needs to consider the catchments in the local area, or elsewhere in Australia. The most relevant characteristics to review include the catchment shape, slope, soils and vegetation. The extent of floodplain storage, either natural or artificial needs to be reviewed. If the target catchment has features that are distinctly different from the range found in “typical” gauged catchments, the results from the RFFE Model can be either discarded or adjusted to allow for the local conditions.

- Review the nearby catchments listed in the RFFE Model: These are the nearest gauged catchments that have been used in the development of the procedure and are some gauges that will influence the results for the Region of Influence calculations.

  This review will need to consider whether these nearby catchments are similar to the target catchment or if there are any apparent outliers in this group. As with the review of catchment characteristics, the review of the nearby catchments in the RFFE Model could lead to an adjustment in the results, or the decision for the flood quantiles for the target catchment to be transposed directly (allowing for differences in catchment area) from a nearby catchment that is most similar to the target catchment.

- Consider an independent flood estimation procedure: such as the application of a runoff-routing model using regional parameter estimates that are appropriate for ungauged catchments in the local region. It must be remembered that these alternative procedures are still uncertain, but reconciliation of estimates from different sources provides valuable information with which to derive a “best estimate”. Consideration of catchment characteristics and similarities or differences need to be a part of this assessment. The RFFE Model results may be adjusted or neglected depending on the conclusions of this independent analysis.

- Review any local available data: This data may be as uncertain as a limited amount of anecdotal data, such as an observation of the frequency of overtopping of a bridge for example. Depending on the findings of the check of local observations, the RFFE Model results may need adjustment. The results of the RFFE Model calculation can be compared with the local observation to at least ensure that the calculated design flood quantiles are consistent with the local observations.

The conclusion of this additional analysis is that the calculated RFFE Model flood quantiles are not perfect, though they do provide the means to develop estimates that are consistent with a large body of gauged records. Further consideration of the results and possible adjustment will help to ensure a better estimate of the design flood quantiles needed for inclusion in the analysis.

3.13. Practical Considerations for Application of the RFFE Technique

The basis of the RFFE Model 2015 has been the analysis of all available streamflow gauging stations that meet the selection criteria described in this chapter, primarily that the stream gauge has an adequate length of record and the streamflow data are of “reasonable” quality and relate to relatively natural catchments. Most selected streamflow gauges have a catchment area less than 1000 km^2. The RFFE Model parameters were then developed based on regionalisation of the at-site flood quantiles for all of these gauges from a given region. All available and suitable streamflow data for all of Australia were adopted to develop the RFFE Model.

While the RFFE Model is appropriate for catchment types represented by the gauged catchments used to develop this model, there are catchments where the method either cannot be applied or where there may be gross error in the flood quantile estimates if the RFFE Model were applied without adjustments. These catchment types are described below.

3.13.1. Urban Catchments

One of the criteria for catchment selection in the development of the RFFE Model was that there should be essentially no urbanisation in the catchment, or at least such a small proportion that the Flood Frequency Analysis was not affected. It is well known that flooding is affected, sometimes significantly by urbanisation. There are insufficient gauged urban catchments to allow development of a RFFE Model for urban catchments.

Therefore, the RFFE Model 2015 cannot be applied for any catchment where urbanisation accounts for more than 10% of the catchment area, or where it is considered that there may be an impact of urbanisation on rainfall runoff relationship. In these cases an alternative approach for estimating design floods is needed, as no equivalent regional method to the rural RFFE Model is available.

3.13.2. Catchments Containing Dams and Other Artificial Storage

Dams or other artificial water storages will attenuate flood hydrographs and will therefore reduce the flood peak discharges at the catchment outlet. The selection of catchments used in the development of the RFFE Model excluded those with dams that were regarded as sufficiently significant to have an impact on flood discharges.

If a catchment has a dam where there will be an impact on flood discharges, the RFFE Model cannot be applied directly. In this case, the RFFE Model can be used to calculate design flood discharges for the “natural” catchment and then the effects of the dam can be included by the application of a runoff-routing model where the storage effects can be modelled directly.

3.13.3. Catchments Affected by Mining

Catchments where mining activity has affected a significant portion of the catchment area have also been excluded from those used to develop the RFFE Model. Catchments where mining activities are significant may produce lower flood peak discharges than natural catchments due to the presence of water quality ponds, tailings dams and other water management infrastructure and the mine pit itself. In addition, the runoff response from mining areas that include waste dumps and rehabilitated areas will be quite distinct from natural catchments.

In cases where mining is judged to impact a significant proportion of a catchment, the RFFE Model can only be applied on the natural portions of the catchment. The area affected by mining activities should be modelled by an alternative method. Assessment of the water balance affecting the volume of runoff is needed to ensure that the effects of water control in ponds and mine pits and infiltration in highly modified catchments is represented correctly. The catchment response is also affected by drainage works and diversions. Calculation of runoff from mining areas is a complex and specialised exercise and detailed understanding of individual conditions is necessary. In general, a detailed runoff-routing model is needed as well as a good understanding of the water balance in these situations. However, direct calibration of runoff routing models on ungauged catchments is not possible, which may result in grossly inaccurate design flood estimates.

3.13.4. Catchments with Intensive Agricultural Activity

While many of the catchments used in development of the RFFE Model have been located in agricultural regions, few are in areas of intensive agriculture, where the flood response may be affected by farm dams, soil conservation works or irrigation infrastructure. Many agricultural regions may be laser levelled to produce topography that is artificial and quite different from the natural catchments that are the primary catchment types used to develop the RFFE Model. The objectives of works on agricultural land is often to slow the rate of runoff and increase infiltration to the soil profile, thus the flood peak discharges will often be reduced on these catchment types as compared to the general catchment type as used to develop the RFFE Model.

Catchments affected by intense agricultural activities are similar to these affected by mining and similar analysis methods are needed to calculate flood discharges.

In this case the RFFE Model cannot be applied directly but additional analysis is needed to assess the catchment characteristics and then prepare a runoff-routing model to adjust the generic RFFE Model results using the catchment storage and channel characteristics.

3.13.5. Catchment Size

The RFFE Model has been developed using all suitable gauged catchments throughout Australia with catchment areas generally less than 1000 km^2.

The RFFE Model can be applied to small catchments with no lower limit though it is recognised that there are only a few gauged catchments smaller than 10 km^2 included in the database to develop the RFFE Model. Because of the limited available data, it is likely that there will be a greater degree of error in the quantile estimates for these smaller catchments.

The RFFE Model should not be applied for catchments larger than 1000 km^2 because these larger catchments were not generally used in the development of the method.

3.13.6. Catchment Shape

The distribution of the shape factors of the selected catchments in developing the RFFE Model is shown in Table 3.3.2. An ungauged catchment with shape factor beyond 10%-90% limit as shown in Table 3.3.2 will have lower accuracy in the estimated flood quantiles.

3.13.7. Atypical Catchment Types

The catchment characteristics adopted in the RFFE Model were limited to readily available/ easily obtainable catchment variables. The catchment characteristics used to calculate model parameters are limited and testing did not determine that more complex characteristics provide significant benefit in the regional relationship.

It is known that there are many other catchment characteristics that influence catchment response and flood discharges. These include factors such as:

- Catchment land use including vegetation coverage: The extent of clearing or forest cover can have a significant impact, especially in the south-west of Western Australia, for example.
- Soils and geology: These factors influence the rainfall losses and catchment flood response.
- Slopes: Steeper catchments respond more rapidly and therefore produce larger flood peaks.
- Channel types and floodplain storage: Well defined channels and smaller floodplain storage extents produce faster response and therefore larger flood peaks.

In cases where the catchment requiring the design flood estimate is judged to have characteristics significantly different from those used in the development of the RFFE Model, further hydrology and hydraulic analyses may be needed to refine the results from the generic RFFE Model results.

The RFFE Model has been developed from the analysis of gauged catchments, but in reality all catchments exhibit some differences. Gauges need to be installed at locations where there is a sensitive and stable control and where the flow is well constrained within a defined stream channel. This means that larger catchments with extensive floodplain storage of widely distributed flow for example may not be well represented in the gauged dataset. These catchments may also have lower data quality but may well be required for design purposes.

3.13.8. Catchment Location

Considering the large size of Australia and the relatively small number of gauged catchments used for the development of the RFFE Model, the average density of gauged catchments used in developing the model is quite low. In addition, it must be recognised that there are errors in the streamflow data (e.g. rating curve extrapolation error) used to develop the RFFE Model.

This means that catchments where design flood discharges are needed may be remote from the gauged catchments used to develop the RFFE Model and may have distinct differences from the adopted gauged catchments.

In these cases, the application of the RFFE Model may produce inaccurate results and hence additional review and checking are necessary to confirm that the catchment being analysed has similar characteristics to those used to develop the RFFE Model. Where there are significant differences, a similar assessment to that needed for atypical catchments should be considered, and an adjustment to the generic RFFE Model result may be needed.

3.13.9. Arid and Semi-arid Areas

The arid and semi-arid areas of Australia pose a particular problem. The RFFE Model divides these areas into two regions, the Pilbara and Arid and semi-arid. There are few gauged catchments in these regions with only 55 gauged catchments to represent a total area of about 5 million km^2.

The stream gauges in this region are often located at sites that may not be representative of the general area, since the gauge sites are selected because of access, confined channel and stable control, which may not be typical of the types of sites where design flood discharges are needed.

The Pilbara region is not as diverse as the remainder of arid and semi-arid areas and the RFFE Model estimates for catchments in the Pilbara can be adopted subject to the other limitations discussed here.

The general arid and semi-arid region though is more complex. The RFFE Model results for this region are based on the best available data from the gauges, but because of the diversity of conditions and the very low desnity of gauging, the RFFE Model results are very uncertain.

Because of this the recommended approach for the arid and semi-arid zone requires some additional analysis to refine the standard results. The RFFE Model data-base report (Rahman et al., 2015a) has a listing of all of the catchments that were used in the development of the method and the application of the RFFE Model software provides a map of the neighbouring catchments that were used in the development of the method.

3.13.10. Baseflow

The RFFE Model has been developed using at-site flood frequency analysis, so the estimated peak discharges include baseflow. This means that the results from the RFFE Model will not be consistent with those from runoff-routing models where only surface runoff is calculated. While baseflow is only a minor part of the total flood hydrograph in many areas of Australia, there are some areas where it is more significant and it becomes more important when considering smaller floods. When working with the two methods it is thus necessary to make appropriate allowance for baseflow contribution to the peak and volume of the design flood.

3.14. RFFE Model 2015

The RFFE technique presented in this chapter has been incorporated into a software tool referred to as RFFE Model 2015. The current version of the RFFE Model 2015 can be accessed from the ARR website[1]. Figure 3.3.4 presents a screen shot of the RFFE Model 2015 software landing page. The model requires the following basic inputs (to be entered by the user in the interface shown in Figure 3.3.5) for the catchment of interest to generate design flood estimates for six AEPs (50%, 20%, 10%, 5%, 2% and 1%).

i. Catchment name;

ii. Catchment outlet latitude in decimal degrees;

iii. Catchment outlet longitude in decimal degrees;

iv. Catchment centroid latitude in decimal degrees;

v. Catchment centroid longitude in decimal degrees; and

vi. Catchment area in km^2.

In the "Advanced" data input option, the user can also enter the following inputs:

i. Region name (can be selected from the dropdown list, see Table 3.3.7 for region name and Figure 3.3.2 for the extent of the regions);

Table 3.3.7. Region Names for Application of the RFFE Model 2015 (see Figure 3.3.2 for the Extent of the Regions)

Region name	Region code
Region 1: East Coast	1
Region 2: Tasmania	2
Region 3: Humid SA	3
Region 4: Top End NT and Kimberley	4
Region 5: SW WA	5
Region 6: Pilbara	6
Region 7: Arid and Semi-arid	7

ii. Design rainfall intensity at catchment centroid for 50% AEP and duration of 6 hour in mm/h; and

[1] www.arr.org.au

iii. Design rainfall intensity at catchment centroid for 2% AEP and duration of 6 hour in mm/h.

The RFFE Model 2015 output contains the following principal information (see Figure 3.3.6 as an example).

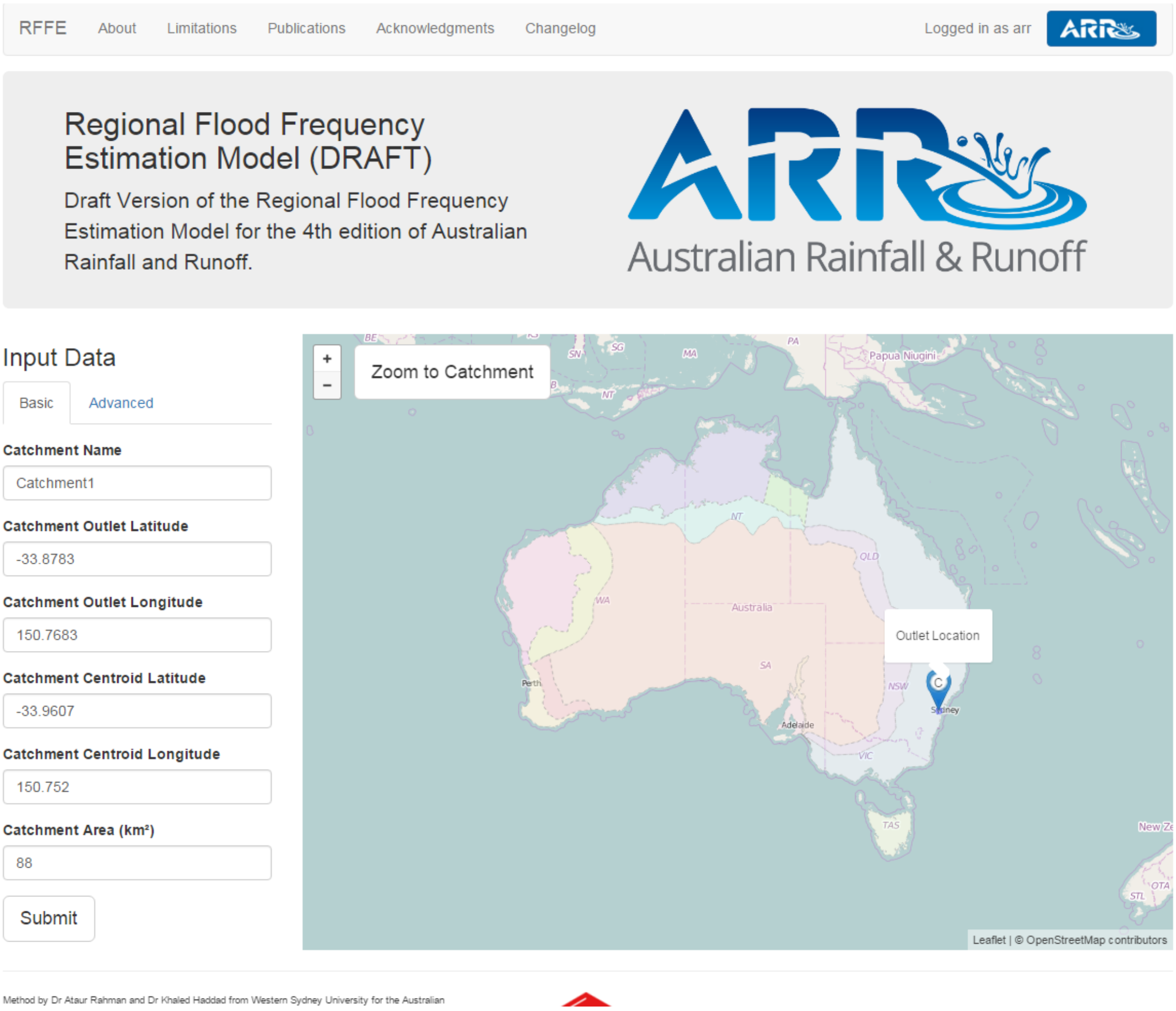


Figure 3.3.4. Screen Shot of RFFE Model 2015 (Landing Page)

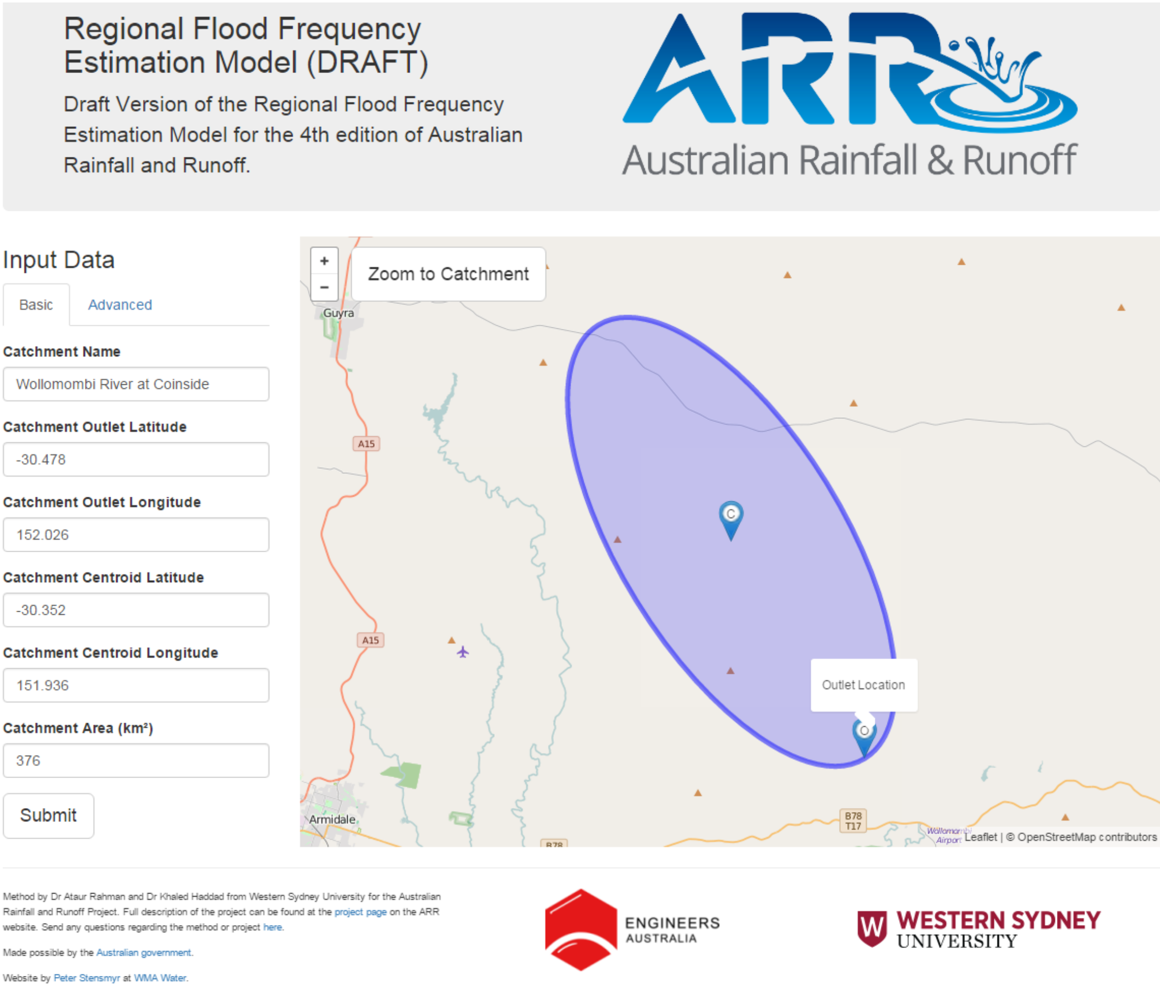
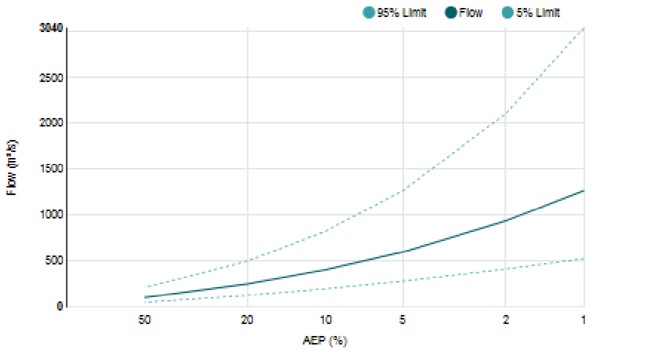


Figure 3.3.5. RFFE Model 2015 Screen Shot for Data Input for the Wollomombi River at Coinside, NSW

RFFE About Limitations Publications Acknowledgments Changelog Logged in as arr ARR

Results | Regional Flood Frequency Estimation Model



AEP (%)	Discharge (m³/s)	Lower Confidence Limit (5%) (m³/s)	Upper Confidence Limit (95%) (m³/s)
50	106	52.3	215
20	255	130	502
10	407	202	827
5	602	286	1270
2	940	415	2110
1	1270	528	3040

Input Data

Date/Time	2015-12-02 09:13
Catchment Name	Wollomombi River at Coinside
Latitude (Outlet)	-30.478
Longitude (Outlet)	152.026
Latitude (Centroid)	-30.352
Longitude (Centroid)	151.936
Catchment Area (km²)	376.0
Distance to Nearest Gauged Catchment (km)	0.07
50% AEP 6 Hour Rainfall Intensity (mm/h)	6.92
2% AEP 6 Hour Rainfall Intensity (mm/h)	15.93
Rainfall Intensity Source (User/Auto)	Auto
Region	East Coast
Region Version	RFFE Model 2015 v1
Region Source (User/Auto)	Auto
Shape Factor	0.85
Interpolation Method	Natural Neighbour
Bias Correction Value	0.129

Statistics

Variable	Value	Standard Dev
Mean	4.683	0.428
Standard Dev	1.027	0.205
Skew	0.099	0.028

Note. These statistics come from the nearest gauged catchment. Details.

Correlation		
1.000		
-0.330	1.000	
0.170	-0.280	1.000

Note. These statistics are common to each region. Details.

Download

TXT PDF Nearby JSON

Method by Dr Ataur Rahman and Dr Khaled Haddad from Western Sydney University for the Australian Rainfall and Runoff Project. Full description of the project can be found at the project page on the ARR website. Send any questions regarding the method or project here.

Made possible by the Australian government.

ENGINEERS AUSTRALIA

WESTERN SYDNEY UNIVERSITY

Figure 3.3.6. RFFE Model 2015 Screen Shot for Model Output for the Wollomombi River at Coinside, NSW (Region 1)

i. A table of AEP (in the first column), estimated flood quantiles labelled as discharge (m^3/s) in the second column, lower confidence limit (5%) (third column) and upper confidence limit (95%) (fourth column). The confidence limits represent the overall uncertainty with the estimated flood quantiles by the RFFE Model 2015.

ii. A second table labelled "Statistics", which shows the statistics for the regional LP III model at the catchment of interest, which are particularly useful to combine at-site and regional information to enhance the accuracy of flood quantile estimates (for details see Book 3, Chapter 2).

iii. A graph that shows estimated flood quantiles and confidence limits against AEPs.

iv. A graph that shows the catchment of interest with outlet and centroid locations and nearby gauged catchments that were included in the database to develop RFFE technique.

v. A download menu that allows the user to save the results and additional outputs generated by the model.

Three worked examples are provided in Book 3, Chapter 3, Section 15 to illustrate the use of RFFE Model 2015. The first two examples relate to catchments with no major regulation, no major natural or artificial storage and no major land use changes over time where the RFFE Model 2015 is directly applicable.

The third example relates to a catchment which has significant natural floodplain storage where RFFE Model 2015 is not directly applicable. For this case, the RFFE Model significantly overestimates the flood quantiles (as compared to at-site Flood Frequency Analysis). Here, the RFFE Model estimates need to be adjusted to account for the storage effect of the catchment by applying an appropriate technique (see Book 3, Chapter 3, Section 13 for further details).

3.15. Worked Examples

3.15.1. Application of RFFE Model 2015 to the Wollomombi River at Coinside, NSW (Region 1) (A Catchment Having No Major Regulation, No Major Natural or Artificial Storage and No Major Land Use Change)

The basic input data for the Wollomombi River at Coinside, NSW are provided in Table 3.3.8. The model screen shot for the data input is provided in Figure 3.3.5. The flood quantiles generated by the RFFE Model 2015 are provided in Figure 3.3.6. Figure 3.3.7 compares the RFFE Model and at-site FFA estimates, which shows that the RFFE Model estimates match the at-site FFA estimates well except for rarer AEPs where the RFFE Model estimates are higher compared with the at-site FFA estimates. Figure 3.3.7 also shows that the confidence band of the RFFE Model 2015 is much wider compared with at-site FFA confidence band, which is expected.

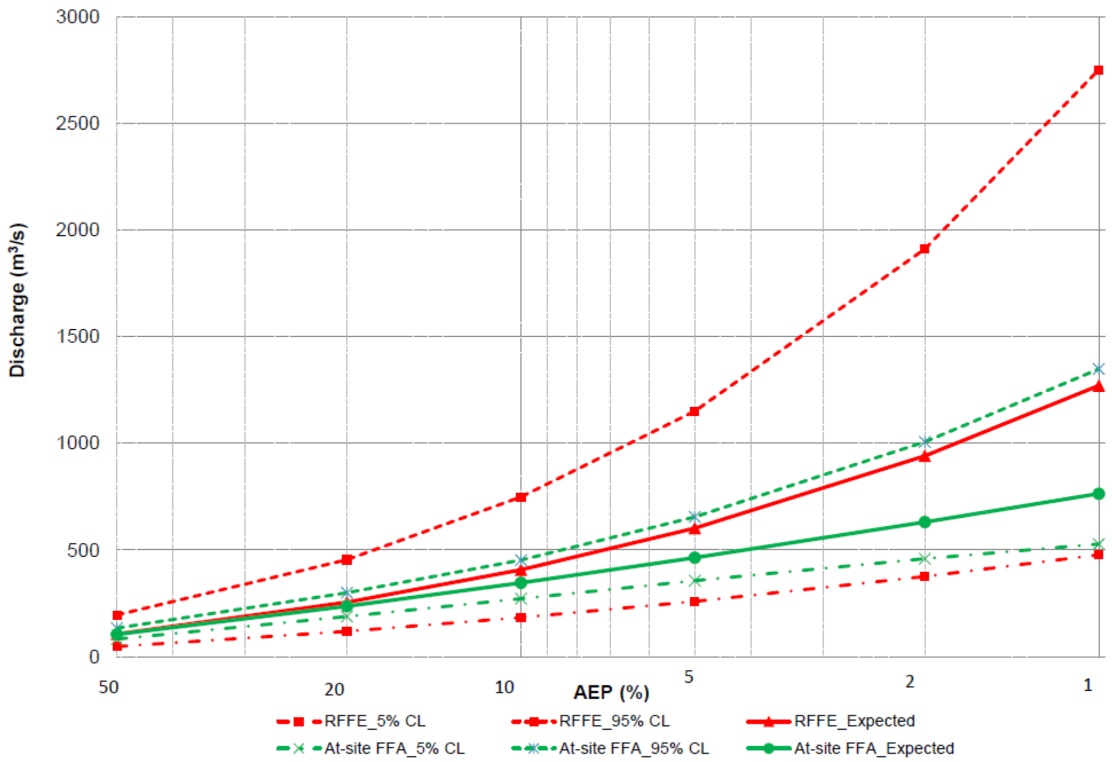


Figure 3.3.7. RFFE Model 2015 vs. At-site FFA Flood Estimates for the Wollomombi River at Coinside, NSW (Region 1)

Table 3.3.8. Application Data for the Wollomombi River at Coinside, NSW (Region 1) (Basic Input Data)

Menu	Input
Catchment Name	Wollomombi River at Coinside
Catchment Outlet Latitude in decimal degrees	-30.478
Catchment Outlet Longitude in decimal degrees	152.026
Catchment Centroid Latitude in decimal degrees	-30.352
Catchment Centroid Longitude in decimal degrees	151.936
Catchment Area in km^2	376

Table 3.3.9 shows a list of 15 gauged catchments, which is generated as part of the RFFE Model output. In this example, these gauged catchments are located closest to the Wollomombi River at Coinside, NSW and were used in the development of the RFFE model. The user should compare the characteristics of the ungauged catchment of interest with those of the nearest gauged catchments (as in Table 3.3.9) to ensure that the ungauged catchment is not atypical.

Table 3.3.9. Fifteen Gauged Catchments (Used in the Development of RFFE Model 2015) Located Closest to Wollomombi River at Coinside, NSW

Site ID	Dist. (km)	Area km^2	Lat. (outlet)	Long. (outlet)	Lat. (centroid)	Long. (centroid)	Record Length (years)	Mean Annual Rainfall (mm)	Shape Factor
206014	0.07	376	-30.478	152.0267	-30.352	151.936	57	871	0.8786
206001	18	163	-30.59	152.1617	-30.5244	152.279	33	1167	1.082
204030	24.29	200	-30.26	152.01	-30.1904	151.828	34	940	1.3946
206017	28.01	22	-30.478	152.3183	-30.4632	152.352	24	1167	0.8005
204008	31.64	31	-30.405	152.345	-30.4139	152.382	29	1249	0.6824
206026	35.67	8	-30.42	151.66	-30.4201	151.639	37	789	0.734
206025	37.68	594	-30.68	151.71	-30.6802	151.557	39	945	0.6194
206034	39.29	117	-30.7	151.7067	-30.7666	151.648	26	758	0.8859
418034	41.98	14	-30.3	151.64	-30.2758	151.65	29	818	0.7876
418014	63.83	855	-30.47	151.36	-30.5063	151.476	37	725	0.4171
206018	68.38	894	-31.051	151.7683	-30.9996	151.634	51	711	0.4846
204017	68.62	82	-30.306	152.7133	-30.3529	152.698	40	1995	0.6086
204037	72.28	62	-30.09	152.63	-30.1082	152.576	40	1586	0.7302
205002	72.50	433	-30.426	152.78	30.4537	152.566	29	1570	1.0277
206009	81.35	261	-31.19	151.83	-31.2597	151.757	57	910	0.6642

3.15.2. Application for the Four Mile Brook at Netic Rd, SW Western Australia (Region 5) (a Catchment Having No Major Regulation, No Major Natural or Artificial Storage and no Major Land Use Change)

The basic input data for the Four Mile Brook at Netic Rd, SW WA is provided in Table 3.3.10. The model screen shot for the data input is provided in Figure 3.3.8. The flood quantiles generated by the RFFE Model 2015 are provided in Figure 3.3.9. Figure 3.3.10 compares the RFFE Model and at-site FFA flood estimates which shows that RFFE Model estimates match the at-site FFA estimates quite well for all the six AEPs. Figure 3.3.10 also shows that the confidence band by the RFFE Model 2015 is much wider compared with at-site FFA confidence band, which is as expected.

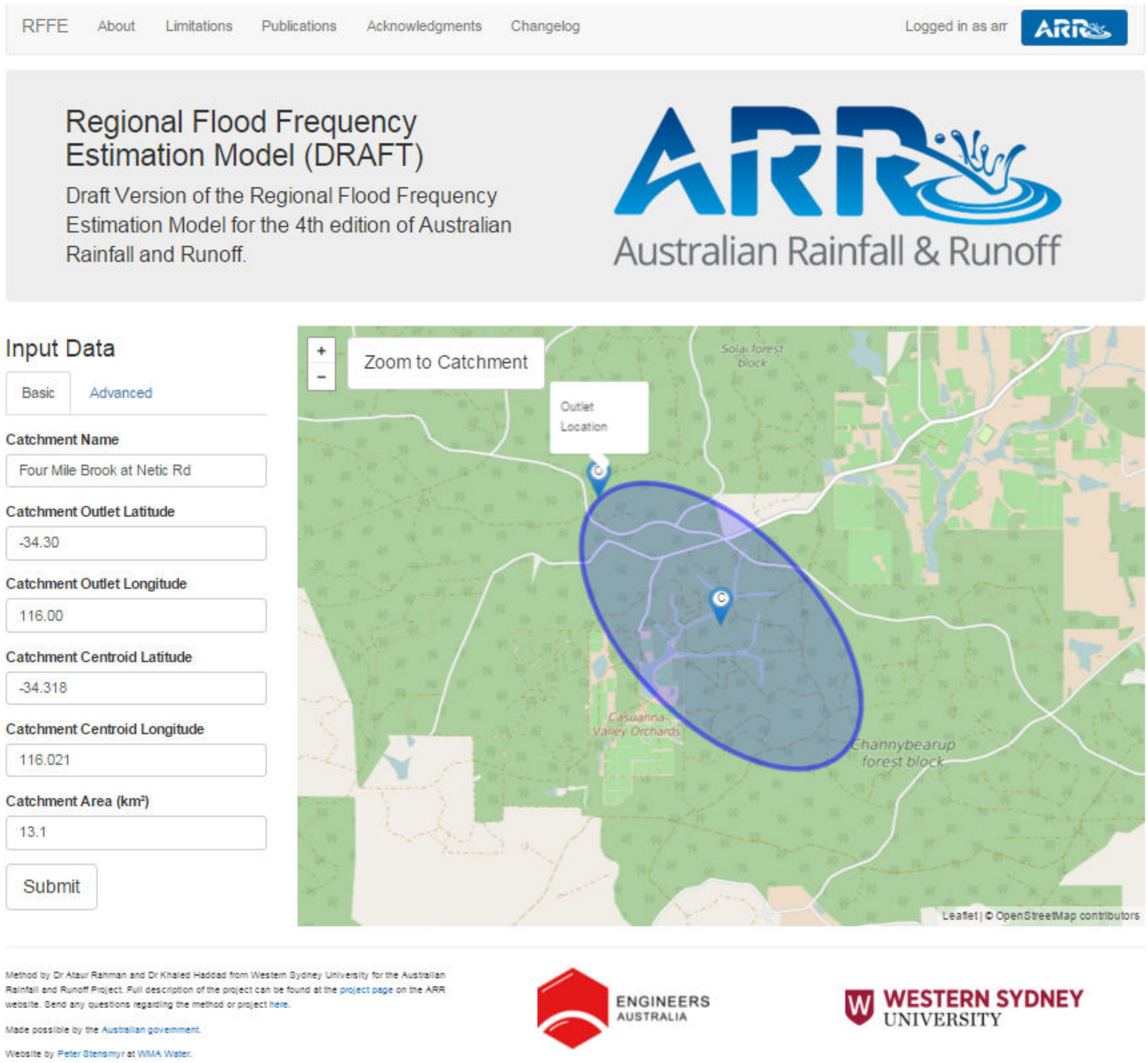
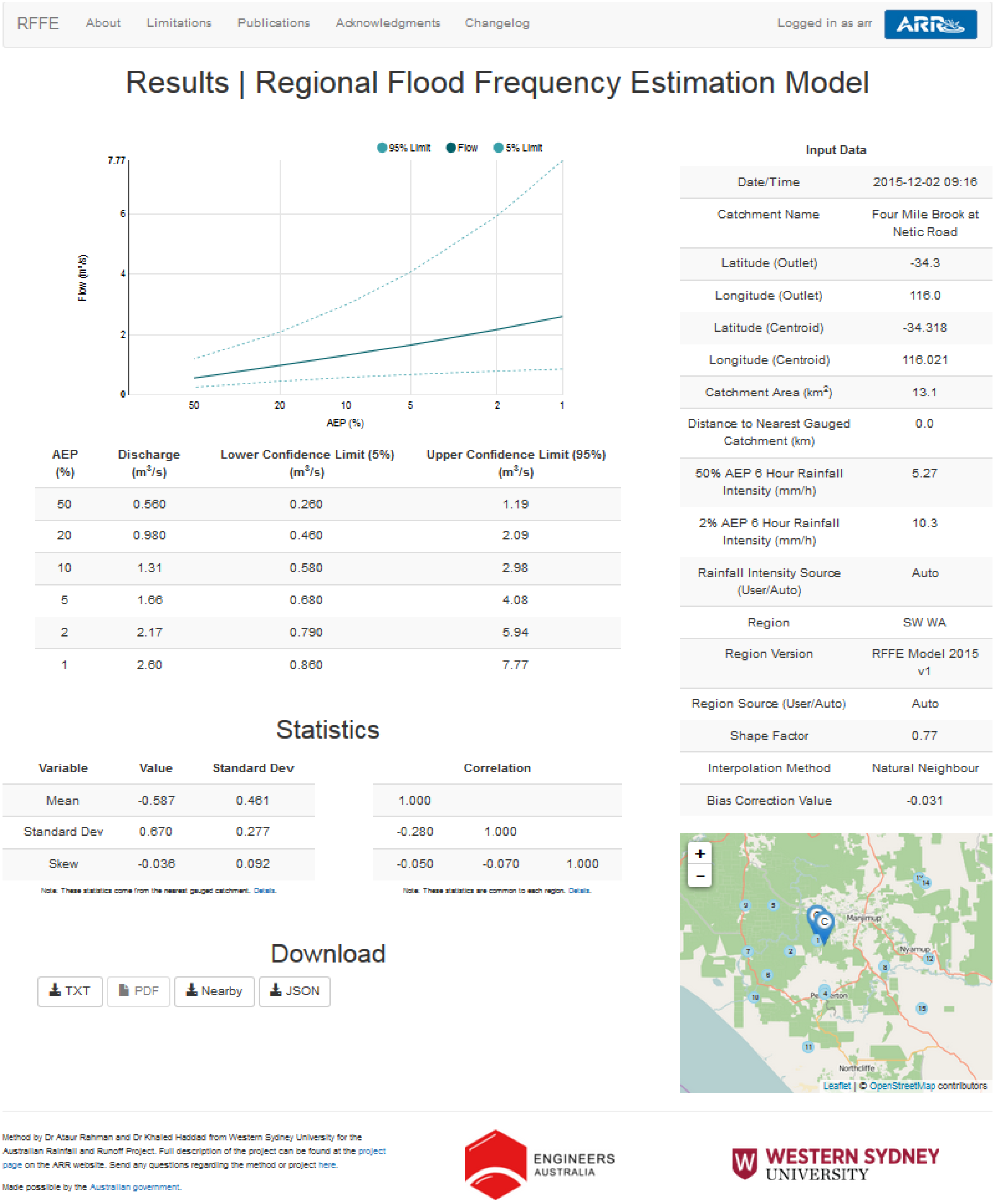


Figure 3.3.8. RFFE Model 2015 Screen Shot for Data Input for Four Mile Brook at Netic Rd, SW WA (Region 5)

Table 3.3.10. Application Data for Four Mile Brook at Netic Rd, SW Western Australia (Region 5) (Basic Input Data)

Menu	Input
Catchment Name	Four Mile Brook at Netic Rd
Catchment Outlet Latitude in decimal degrees	-34.30
Catchment Outlet Longitude in decimal degrees	116.00
Catchment Centroid Latitude in decimal degrees	-34.318
Catchment Centroid Longitude in decimal degrees	116.021
Catchment Area in (km^2)	13.1


RFFE About Limitations Publications Acknowledgments Changelog Logged in as arr ARR

Results | Regional Flood Frequency Estimation Model

AEP (%)	Discharge (m^3/s)	Lower Confidence Limit (5%) (m^3/s)	Upper Confidence Limit (95%) (m^3/s)
50	0.560	0.260	1.19
20	0.980	0.460	2.09
10	1.31	0.580	2.98
5	1.66	0.680	4.08
2	2.17	0.790	5.94
1	2.60	0.860	7.77

Statistics

Variable	Value	Standard Dev
Mean	-0.587	0.461
Standard Dev	0.670	0.277
Skew	-0.036	0.092

Note. These statistics come from the nearest gauged catchment. Details.

Correlation		
1.000		
-0.280	1.000	
-0.050	-0.070	1.000

Note. These statistics are common to each region. Details.

Download

TXT PDF Nearby JSON

Input Data	
Date/Time	2015-12-02 09:16
Catchment Name	Four Mile Brook at Netic Road
Latitude (Outlet)	-34.3
Longitude (Outlet)	116.0
Latitude (Centroid)	-34.318
Longitude (Centroid)	116.021
Catchment Area (km^2)	13.1
Distance to Nearest Gauged Catchment (km)	0.0
50% AEP 6 Hour Rainfall Intensity (mm/h)	5.27
2% AEP 6 Hour Rainfall Intensity (mm/h)	10.3
Rainfall Intensity Source (User/Auto)	Auto
Region	SW WA
Region Version	RFFE Model 2015 v1
Region Source (User/Auto)	Auto
Shape Factor	0.77
Interpolation Method	Natural Neighbour
Bias Correction Value	-0.031

Method by Dr Ataur Rahman and Dr Khaled Haddad from Western Sydney University for the Australian Rainfall and Runoff Project. Full description of the project can be found at the project page on the ARR website. Send any questions regarding the method or project here.

Made possible by the Australian government.

ENGINEERS AUSTRALIA

WESTERN SYDNEY UNIVERSITY

Figure 3.3.9. RFFE Model 2015 Screen Shot for Model Output for Four Mile Brook at Netic Rd, SW WA (Region 5)

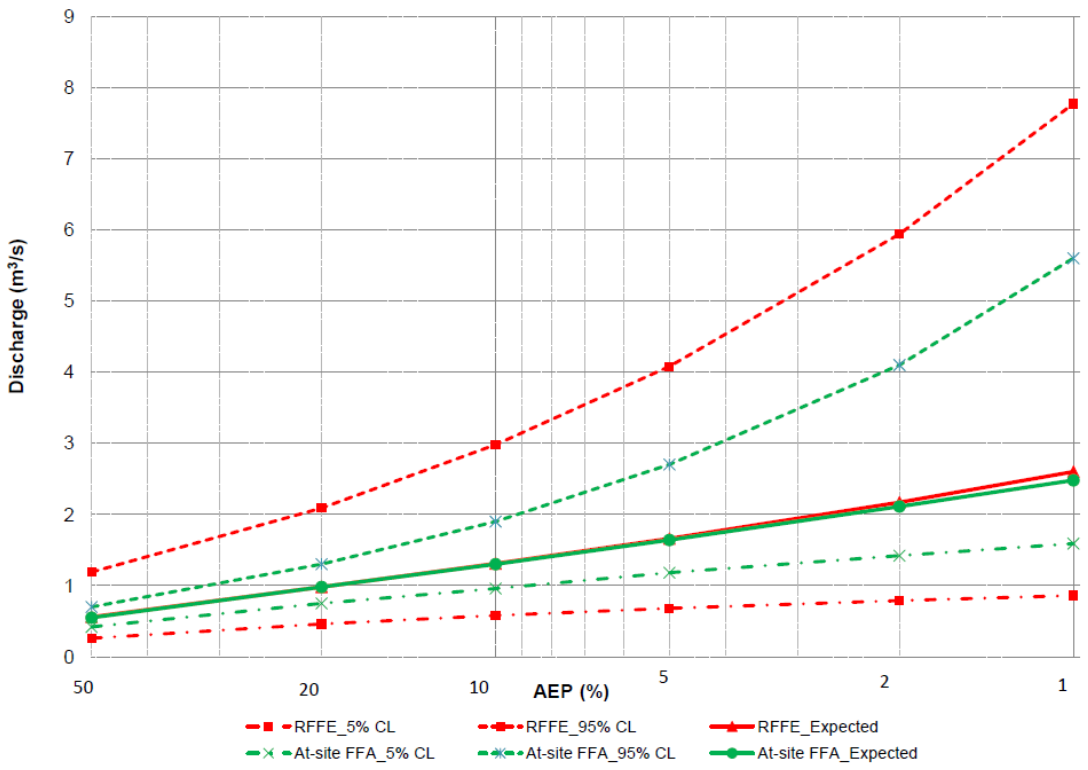


Figure 3.3.10. RFFE Model 2015 vs. At-site FFA Flood Estimates for Four Mile Brook at Netic Rd, SW WA (Region 5)

3.15.3. Application for the Morass Creek at Uplands, VIC (Region 1) (a Catchment Having Significant Natural Floodplain Storage Where RFFE Model 2015 Output is Not Directly Applicable)

This example (Morass Creek at Uplands, VIC) illustrates a catchment type where the RFFE Model is not applicable. The basic input data for this catchment is provided in Table 3.3.11. The model screen shot for the data input is provided in Figure 3.3.11. The flood quantiles generated by the RFFE Model 2015 are provided in Figure 3.3.12. Figure 3.3.13 compares the RFFE Model and at-site FFA flood estimates, which shows that RFFE Model estimates are much higher compared with at-site FFA estimates. This catchment has significant natural floodplain storage which is not typical for other catchments in the Region of Influence (ROI) on which the RFFE Model 2015 estimates are based. The RFFE Model 2015 estimates are thus not directly applicable. Here, the RFFE Model estimates would need to be adjusted downwards to account for the floodplain storage effect by applying an appropriate technique.

Table 3.3.11. Application Data for the Morass Creek at Uplands, VIC (Region 1) (Basic Input Data)

Menu	Input
Catchment Name	Morass Creek at Uplands
Catchment Outlet Latitude in decimal degrees	-36.87

Menu	Input
Catchment Outlet Longitude in decimal degrees	147.70
Catchment Centroid Latitude in decimal degrees	-36.88
Catchment Centroid Longitude in decimal degrees	147.84
Catchment Area in (km^2)	471

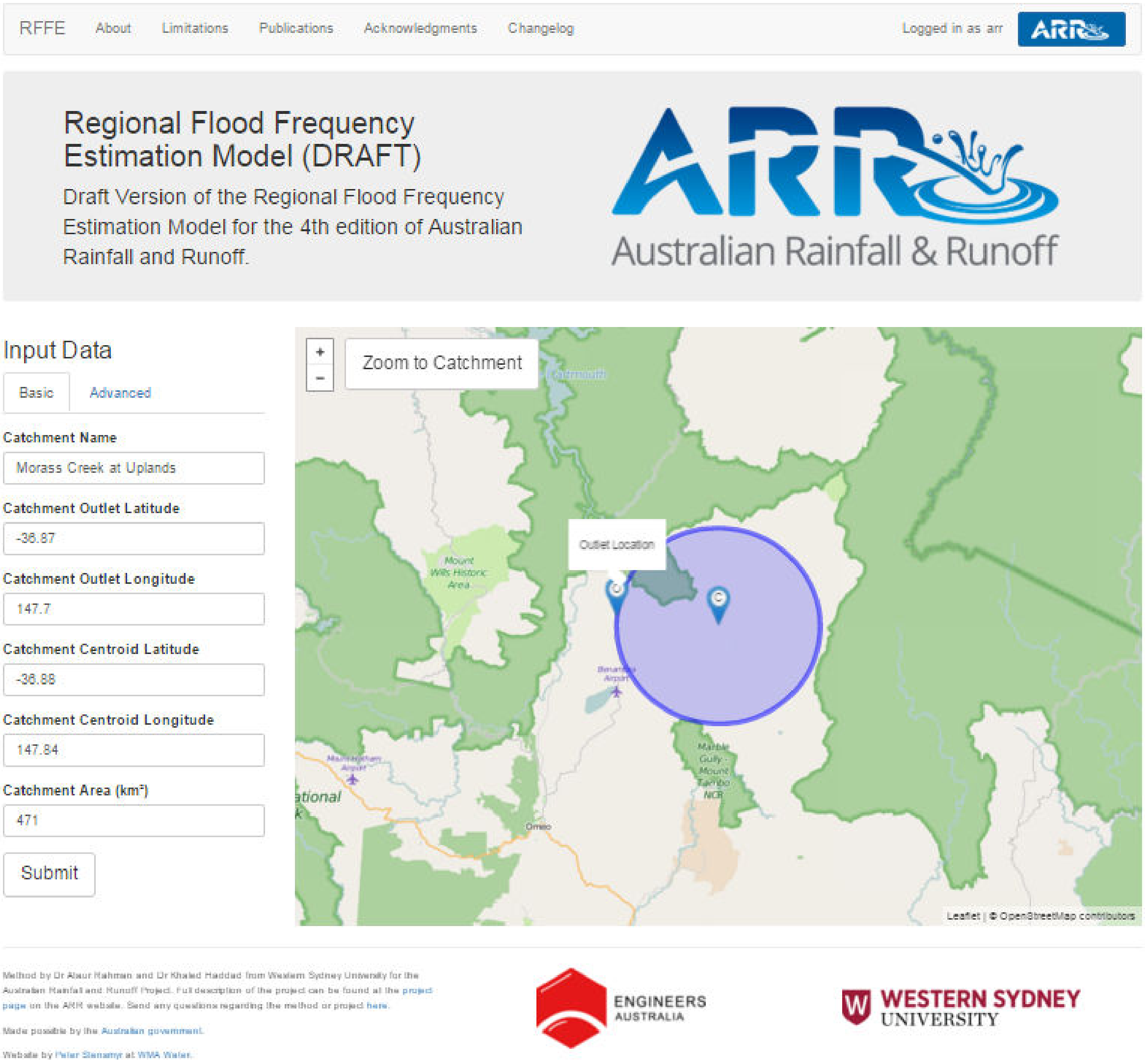
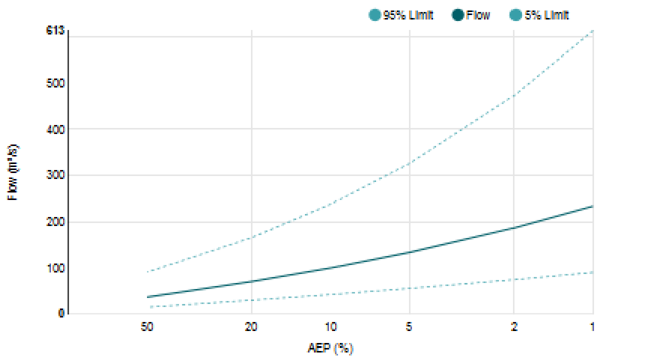


Figure 3.3.11. RFFE Model 2015 Screen Shot for Data Input for the Morass Creek at Uplands, VIC (Region 1)

RFFE About Limitations Publications Acknowledgments Changelog Logged in as arr ARR

Results | Regional Flood Frequency Estimation Model



AEP (%)	Discharge (m^3/s)	Lower Confidence Limit (5%) (m^3/s)	Upper Confidence Limit (95%) (m^3/s)
50	37.0	15.0	91.0
20	70.8	30.4	166
10	100	42.6	238
5	134	55.9	326
2	187	74.8	473
1	234	90.6	613

Input Data

Date/Time	2015-12-02 09:19
Catchment Name	Morass Creek at Uplands
Latitude (Outlet)	-36.87
Longitude (Outlet)	147.7
Latitude (Centroid)	-36.88
Longitude (Centroid)	147.84
Catchment Area (km^2)	471.0
Distance to Nearest Gauged Catchment (km)	13.37
50% AEP 6 Hour Rainfall Intensity (mm/h)	5.35
2% AEP 6 Hour Rainfall Intensity (mm/h)	10.96
Rainfall Intensity Source (User/Auto)	Auto
Region	East Coast
Region Version	RFFE Model 2015 v1
Region Source (User/Auto)	Auto
Shape Factor	0.58
Interpolation Method	Natural Neighbour
Bias Correction Value	-0.01

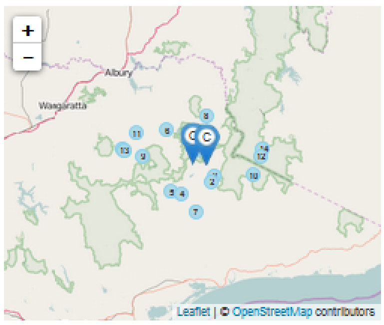
Statistics

Variable	Value	Standard Dev
Mean	3.644	0.576
Standard Dev	0.711	0.197
Skew	0.093	0.027

Note. These statistics come from the nearest gauged catchment. Details.

Correlation		
1.000		
-0.330	1.000	
0.170	-0.280	1.000

Note. These statistics are common to each region. Details.



Download

TXT PDF Nearby JSON

Method by Dr Ataur Rahman and Dr Khaled Haddad from Western Sydney University for the Australian Rainfall and Runoff Project. Full description of the project can be found at the project page on the ARR website. Send any questions regarding the method or project here.

Made possible by the Australian government.

ENGINEERS AUSTRALIA

WESTERN SYDNEY UNIVERSITY

Figure 3.3.12. RFFE Model 2015 Screen Shot for Model Output for the Morass Creek at Uplands, VIC (Region 1)

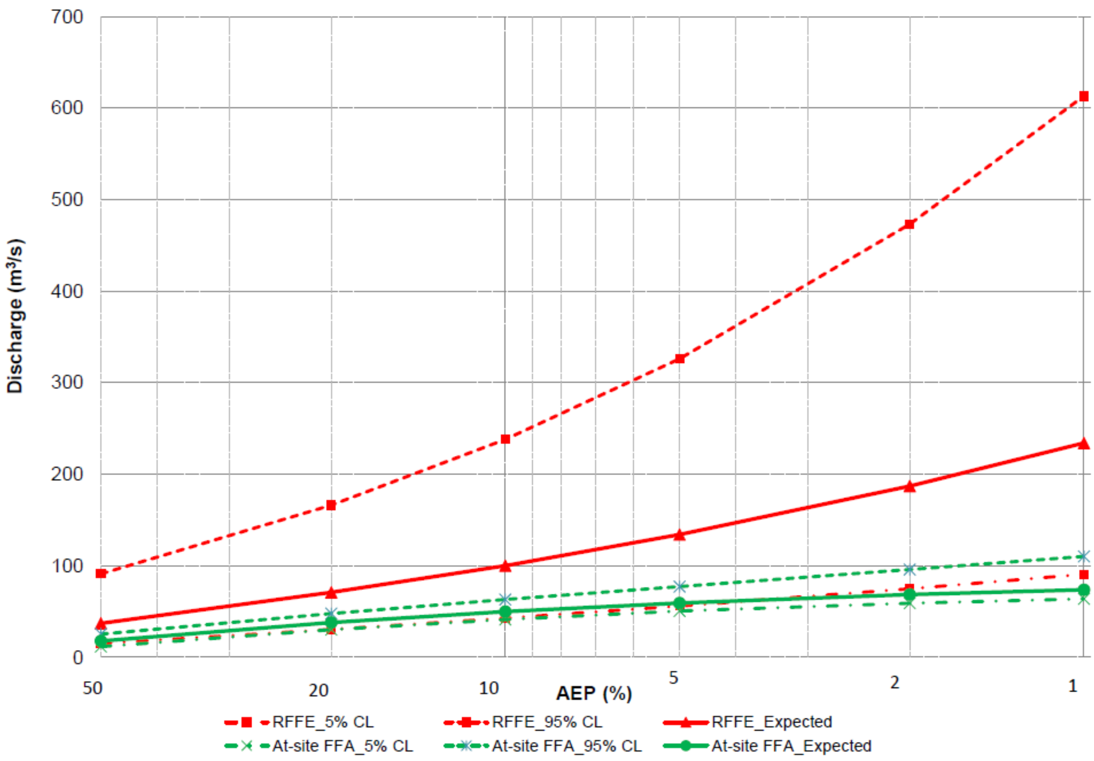


Figure 3.3.13. RFFE Model 2015 vs. At-site FFA Flood Estimates (for the Morass Creek at Uplands, VIC) (Region 1)

3.16. Further Information on the Development and Testing of RFFE Technique 2015

Table 3.3.12. Further Information on the Development and Testing of RFFE Technique 2015

Information	Source
Database consisting of 853 gauged catchments used to develop RFFE Technique 2015	Rahman et al. (2015a)
Aspects of streamflow data preparation	Haddad et al. (2010)
Comparison of Probabilistic Rational Method and quantile regression technique	Rahman et al. (2011b)
Comparison of ordinary and generalised least squares regression techniques	Haddad et al. (2011)
Comparison of fixed region and region-of-influence approaches for quantile and parameter regression techniques	Haddad and Rahman (2012)
Regionalisation of the parameters of the LP3 distribution	(Haddad et al., 2012; Micevski et al., 2015)
Development and testing of the RFFE technique	(Rahman et al., 2009; Rahman et al., 2012; Rahman et al, 2015)

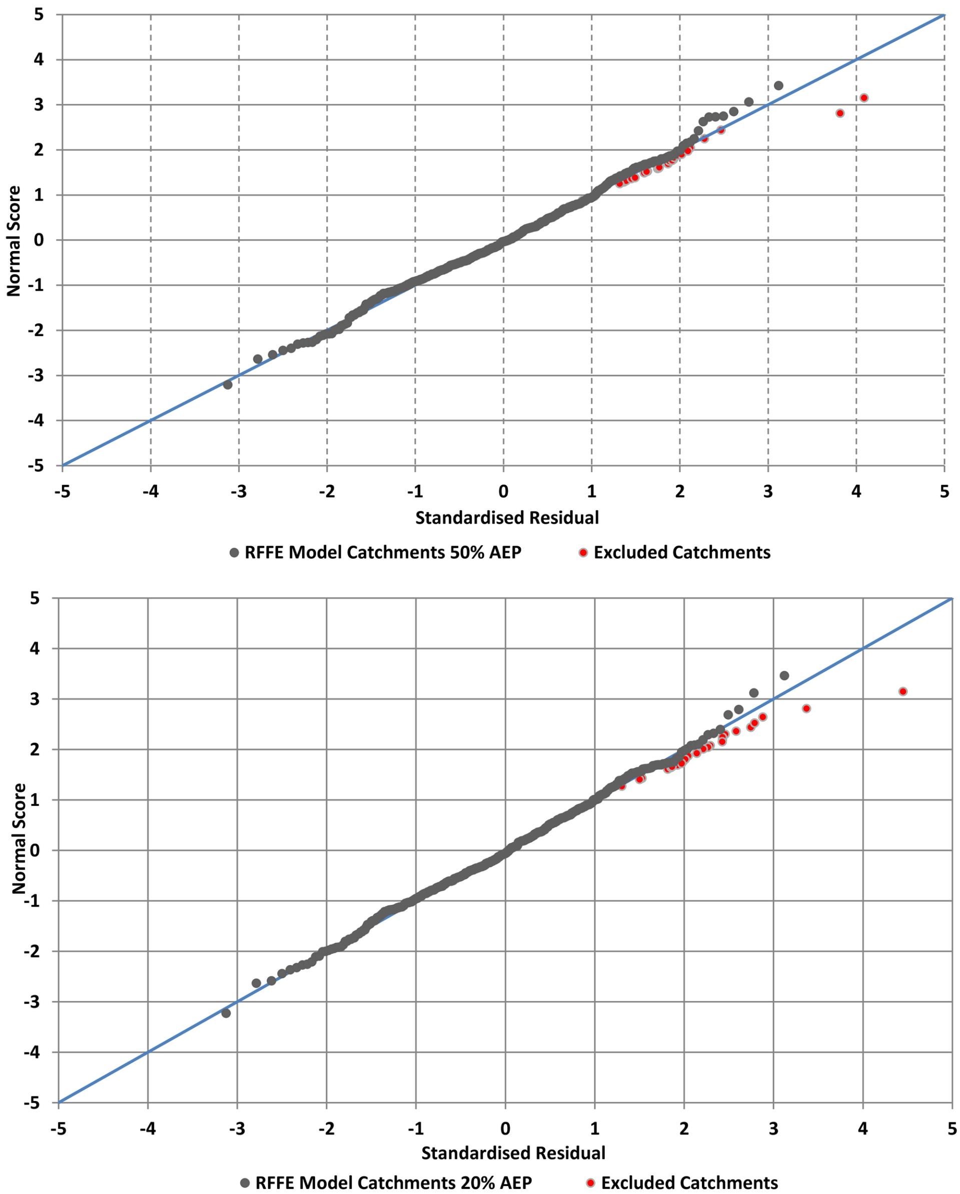
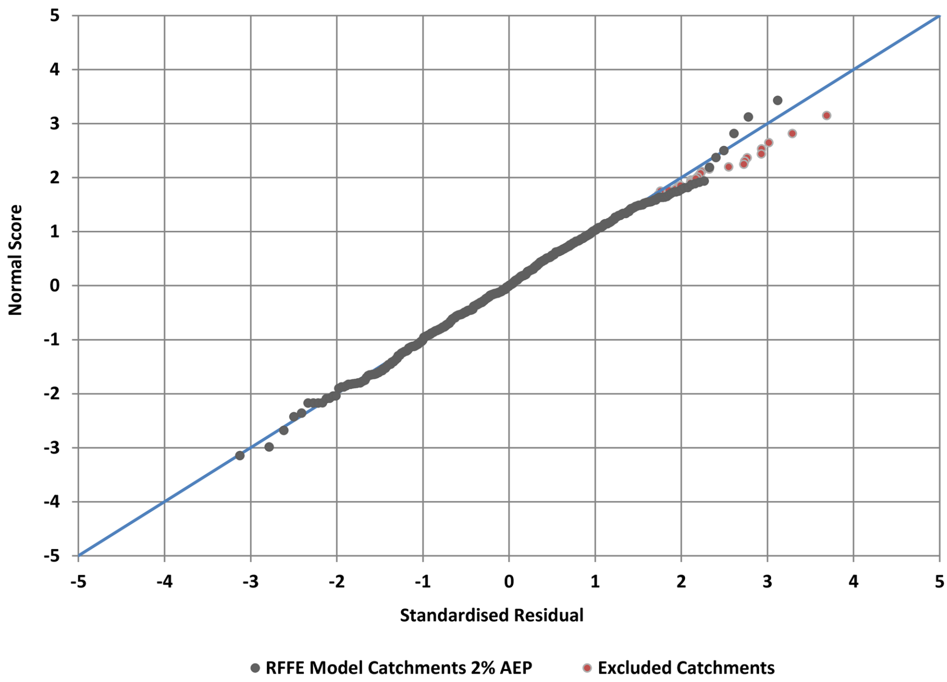


Figure 3.3.14. Standardised Residuals vs. Normal Scores for Region 1 Based on Leave-one-out Validation for AEPs of 50%, 20%, 2% and 1%



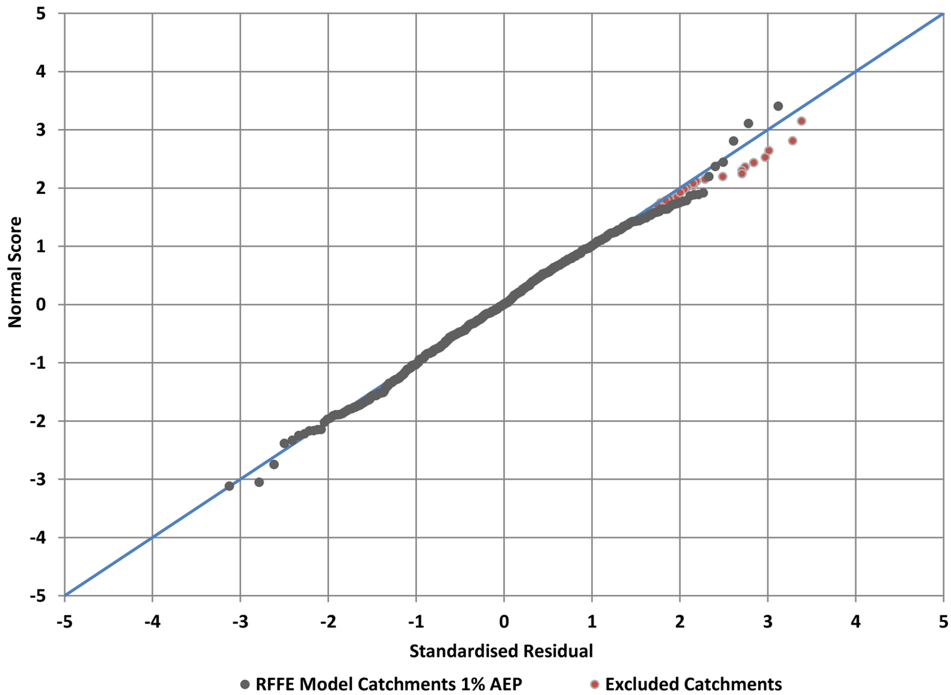


Figure 3.3.15. Standardised Residuals vs. Normal Scores for Region 1 Based on Leave-one-out Validation for AEPs of 2% and 1%

3.17. References

Adams, C.A. (1984), 'Regional flood estimation for ungauged rural catchments in Victoria', MEngSc Thesis, The University of Melbourne.

Ball, J.E. (2014), 'Flood estimation under climate change', 19th IAHR-APD Congress, Hanoi, Vietnam.

Bates, B.C. (1994), 'Regionalisation of hydrological data: A review', Report 94/5, CRC for Catchment Hydrology, Monash University.

Bates, B.C., Rahman, A., Mein, R.G. and Weinmann, P.E. (1998), Climatic and physical factors that influence the homogeneity of regional floods in south-eastern Australia, Water Resources Research, 34(12), 3369-3382.

Burn, D.H. (1990a), An appraisal of the region of influence approach to Flood Frequency Analysis, Hydrological Sciences Journal, 35(2), 149-165.

Burn, D.H. (1990b), Evaluation of regional Flood Frequency Analysis with a region of influence approach, Water Resources Research, 26(10), 2257-2265.

Cohn, T.A., England, J.F., Berenbrock, C.E., Mason, R.R., Stedinger, J.R. and Lamontagne, J.R. (2013), A generalized Grubbs-Beck test statistic for detecting multiple potentially influential low outliers in flood series, Water Resour. Res., 49(8), 5047-5058.

Farquharson, F.A.K., Meigh, J.R. and Sutcliffe, J.V. (1992), Regional Flood Frequency Analysis in arid and semi-arid areas, Journal of Hydrology, 138: 487-501.

French, R. (2002), 'Flaws in the rational method', Proc. 27th National Hydrology and Water Resources Symp., Melbourne.

Griffis, V.W. and Stedinger, J.R. (2007), The use of GLS regression in regional hydrologic analyses, Journal of Hydrology, 344: 82-95.

Hackelbusch, A., Micevski, T., Kuczera, G., Rahman, A. and Haddad, K. (2009), 'Regional Flood Frequency Analysis for eastern New South Wales: a region of influence approach using generalised least squares based parameter regression', Proc. 31st Hydrology and Water Resources Symp., Newcastle, pp: 603-615.

Haddad, K. (2008), 'Design flood estimation in ungauged catchments using a quantile regression technique: ordinary and generalised least squares methods compared for Victoria', Masters (Honours) thesis, School of Engineering, University of Western Sydney, New South Wales.

Haddad, K. and Rahman, A. (2012), Regional Flood Frequency Analysis in eastern Australia: Bayesian GLS regression-based methods within fixed region and ROI framework - quantile regression vs. parameter regression technique, Journal of Hydrology, pp: 430-431, 142-161.

Haddad, K., Rahman, A. and Weinmann, P.E. (2008), 'Development of a generalised least squares based quantile regression technique for design flood estimation in Victoria', Proc. 31st Hydrology and Water Resources Symp., Adelaide, pp: 2546-2557.

Haddad, K., Pirozzi, J., McPherson, G., Rahman, A. and Kuczera, G. (2009),'Regional flood estimation technique for NSW: application of generalised least squares quantile regression technique', Proc. 32nd Hydrology and Water Resources Symp., Newcastle, pp: 829-840.

Haddad, K., Rahman, A. and Kuczera, G. (2011), Comparison of ordinary and generalised least squares regression models in regional Flood Frequency Analysis: A case study for New South Wales, Australian Journal of Water Resources, 15(2), 59-70.

Haddad, K., Rahman, A. and Stedinger, J.R. (2012), Regional Flood Frequency Analysis using Bayesian generalised least squares: A comparison between quantile and parameter regression techniques, Hydrological Processes, 26: 1008-1021.

Haddad, K., Rahman, A., Weinmann, P.E., Kuczera, G. and Ball, J.E. (2010), Streamflow data preparation for regional Flood Frequency Analysis: Lessons from south-east Australia, Australian Journal of Water Resources, 14(1), 17-32.

Hosking, J.R.M. and Wallis, J.R. (1993), Some statistics useful in regional frequency analysis, Water Resources Research, 29(2), .71-281.

Ishak, E., Rahman, A., Westra, S., Sharma, A. and Kuczera, G. (2013), Evaluating the non-stationarity of Australian annual maximum floods, Journal of Hydrology, 494: 134-145.

Kuczera, G. (1999), Comprehensive at-site Flood Frequency Analysis using Monte Carlo Bayesian inference. Water Resources Research, 35(5), 1551-1557.

Lamontagne, J.R., Stedinger, J.R., Cohn, T.A. and Barth, N.A. (2013),'Robust national flood frequency guidelines: What is an outlier?', Proc. World Environmental and Water Resources Congress, ASCE, pp: 2454-2466.

Micevski, T. and Kuczera, G. (2009), Combining site and regional flood information using a Bayesian Monte Carlo approach, Water Resources Research, 45, W04405, doi: 10.1029/2008WR007173.

Micevski, T., Hackelbusch, A., Haddad, K., Kuczera, G. and Rahman, A. (2015), Regionalisation of the parameters of the log-Pearson 3 distribution: a case study for New South Wales, Australia, Hydrological Processes, 29(2), 250-260.

Mittelstadt, G.E., McDermott, G.E. and Pilgrim, D.H. (1987), Revised flood data and catchment characteristics for small to medium sized catchments in New South Wales. University of New South Wales, Department of Water Engineering, unpublished report.

Palmen, L.B. and Weeks, W.D. (2009),'Regional flood frequency for Queensland using the Quantile Regression Technique', Proc. 32nd Hydrology and Water Resources Symp., Newcastle.

Palmen, L.B. and Weeks, W.D. (2011), Regional flood frequency for Queensland using the quantile regression technique, Australian Journal of Water Resources, 15(1), 47-57.

Pilgrim, DH (ed) (1987) Australian Rainfall and Runoff - A Guide to Flood Estimation, Institution of Engineers, Australia, Barton, ACT, 1987.

Pilgrim, D.H. and McDermott, G. (1982), Design floods for small rural catchments in eastern New South Wales, Civil Engineering Transactions, I. E. Aust., CE24: 226-234.

Pirozzi, J., Ashraf, M., Rahman, A., and Haddad, K. (2009),'Design flood estimation for ungauged catchments in eastern NSW: evaluation of the Probabilistic Rational Method', Proc. 32nd Hydrology and Water Resources Symp., Newcastle, pp: 805-816.

Rahman, A. (1997),'Flood estimation for ungauged catchments: a regional approach using flood and catchment characteristics', PhD thesis, Department of Civil Engineering, Monash University.

Rahman, A. (2005), A Quantile Regression Technique to Estimate Design Floods for Ungauged Catchments in South-East Australia, Australian Journal of Water Resources, 9(1), 81-89.

Rahman, A., Haddad, K., Rahman, A.S., Haque, M.M., Kuczera, G. and Weinmann, P.E. (2015) Australian Rainfall and Runoff Revision Project 5: Regional Flood Methods: Stage 3, ARR Report Number P5/S3/025, ISBN 978-0-85825-869-3.

Rahman, A., Rima, K. and Weeks, W.D. (2008),'Development of Regional Flood Frequency Estimation methods using Quantile Regression Technique: a case study for north-eastern part of Queensland', Proc. 31st Hydrology and Water Resources Symp., Adelaide, pp: 329-340.

Rahman, A., Haddad, K., Kuczera, G. and Weinmann, P.E. (2009),'Australian Rainfall and Runoff Revision Projects, Project 5 Regional Flood Methods, Stage 1 Report', No.P5/S1/003, Nov 2009, Engineers Australia, Water Engineering, pp: 181.

Rahman, A., Haddad, K., Zaman, M., Ishak, E., Kuczera, G. and Weinmann, P.E. (2012),'Australian Rainfall and Runoff Revision Projects, Project 5 Regional flood methods, Stage 2 Report', No.P5/S2/015, Engineers Australia, Water Engineering, pp: 319.

Rahman, A., Bates, B.C., Mein, R.G. and Weinmann, P.E. (1999), Regional Flood Frequency Analysis for ungauged basins in south - eastern Australia, Australian Journal of Water Resources, 3(2), 199-207.

Rahman, A., Haddad, K., Zaman, M., Kuczera, G. and Weinmann, P.E. (2011a), Design flood estimation in ungauged catchments: a comparison between the Probabilistic Rational Method and Quantile Regression Technique for NSW, Australian Journal of Water Resources, 14(2), 127-139.

Rahman, A., Haddad, K., Rahman, A.S. and Haque, M.M. (2015a),'Australian Rainfall and Runoff Revision Projects, Project 5 Regional flood methods, Database used to develop ARR RFFE Model 2015', Engineers Australia, Water Engineering.

Rahman, A., Zaman, M., Fotos, M., Haddad, K., Rajaratnam, L. and Weeks, W.D. (2011b),'Towards a new Regional Flood Frequency Estimation method for the Northern Territory', Proc. 34th IAHR World Congress, Brisbane, pp: 364-371.

Rahman, A., Haddad, K., Haque, M.M., Kuczera, G., Weinmann, P.E., Stensmyr, P., Babister, M., Weeks, W.D. (2015b),'The New Regional Flood Frequency Estimation Model for Australia: RFFE Model 2015', 36th Hydrology and Water Resources Symp., 7-10 Dec, 2015, Hobart.

Rahman, A., Haque, M., Haddad, K., Rahman, A.S., Caballero, W.L. (2015c),'Database Underpinning the RFFE 2015 Model in the new Australian Rainfall and Runoff', 36th Hydrology and Water Resources Symposium, 7-10 Dec, 2015, Hobart.

Sivapalan, M., Blöschl, G. and Wagener, T. (2013),'Runoff Prediction in Ungauged Basins: Synthesis across Processes, Places and Scales', Cambridge University Press.

Stedinger, J.R. and Tasker, G.D. (1985), Regional hydrologic analysis 1 Ordinary, weighted, and generalised least squares compared, Water Resources Research, 21(9), 1421-1432.

Tasker, G.D. and Stedinger, J.R. (1989), An operational GLS model for hydrologic regression, Journal of Hydrology, 111(1-4), 361-375.

Zrinji, Z. and Burn, D.H. (1994), Flood frequency analysis for ungauged sites using a region of influence approach. Journal of Hydrology, 153: 1-21.

BOOK 4

Catchment Simulation for Design Flood Estimation

Catchment Simulation for Design Flood Estimation

Table of Contents

1. Introduction 1
    1.1. Simulation of Design Flood Hydrographs 1
    1.2. Difference Between Historic and Design Flood Simulations 2
    1.3. Scope 2
    1.4. Outline 2
2. Hydrologic Processes Contributing to Floods 4
    2.1. Introduction 4
    2.2. Runoff Generation 5
        2.2.1. Infiltration Excess Runoff 6
        2.2.2. Saturation Excess Runoff, Variable Source Areas and Partial Area Runoff 7
        2.2.3. Impervious Area Runoff 7
        2.2.4. Subsurface Storm Flow 7
        2.2.5. Runoff in Real Catchments 8
    2.3. Baseflow 8
    2.4. Losses 9
    2.5. Flow Routing 9
    2.6. Conceptualising Processes in Models 10
        2.6.1. Runoff Production 10
        2.6.2. Losses 11
        2.6.3. Baseflow 11
        2.6.4. Routing 12
        2.6.5. Spatial Representation of Hydrological Processes 13
    2.7. Examples 14
        2.7.1. South Creek - North Queensland 14
        2.7.2. Tarrawarra – Southern Victoria 15
        2.7.3. Runoff from Urban Areas 15
    2.8. References 17
3. Types of Simulation Approaches 19
    3.1. Introduction 19
    3.2. Event-Based Approaches 20
        3.2.1. General Concepts 20
        3.2.2. Simple Event 22
        3.2.3. Ensemble Event 23
        3.2.4. Monte Carlo Event 24
    3.3. Continuous Simulation Approaches 27
        3.3.1. General Concepts 27
        3.3.2. Stochastic Rainfall Data Generation 28
        3.3.3. Runoff- Routing Model 29
        3.3.4. Model Calibration 30
        3.3.5. Applications to Design Flood Estimation 30
    3.4. Hybrid Continuous Event-Based Simulation 31
    3.5. Performance of Methods 31
    3.6. Advantages and Limitations 33
    3.7. Example - Delatite River 34
    3.8. References 36
4. Treatment of Joint Probability 41
    4.1. Introduction 41
    4.2. Probability Concepts 41
        4.2.1. Variability and Uncertainty 41

4.2.2. Joint and Conditional Probabilities 42
4.2.3. Typical Joint Probability Applications 45
4.2.4. Typical Conditional Probability Applications 46
4.2.5. General Approach 48
4.2.6. Selection and Treatment of Factors 51
4.3. Monte Carlo Simulation 53
4.3.1. Introduction 53
4.3.2. Generation of Stochastic Inputs 55
4.3.3. Estimation of Exceedance Probabilities 61
4.4. Example 65
4.5. References 70

List of Figures

4.2.1. Catchment and Runoff Generation Processes 5

4.2.2. Simplified Description of the Process of Converting Rainfall to Runoff and Streamflow 6

4.2.3. Observed Hydrograph - Sum of the Baseflow Hydrograph and the Quickflow 9

4.2.4. Conceptual Relationship between Data Availability, Model Complexity and Predictive Performance (Grayson and Blöschl, 2000) 10

4.2.5. Spatial Representation of Physical Processes in Hydrologic Models 13

4.2.6. Comparison of Initial Loss in Urban and Rural Catchments 16

4.3.1. Elements of Three Different Approaches to Flooding 22

4.3.2. Simple Framework for Monte Carlo Simulation for Handling Joint Probabilities Associated with Both Losses and Temporal Patterns 27

4.3.3. Schematic Layout of Delatite River catchment and Calibration to December 2010 Event 35

4.3.4. Comparison of Design Flood Estimates with Flood Frequency Curve for the Delatite River at Tonga Bridge 36

4.4.1. Joint Probability Density for a Bivariate Normal Distribution 43

4.4.2. Difference in the Seasonal Likelihood of Large Long Duration Rainfall Events and Large Short Dduration Rainfall Events and their Concurrence with Catchment Losses 48

4.4.3. Generic Components that need to be Considered in Solution of Joint Probability Problems 50

4.4.4. Use of Standardisation to Derive a Regional Distribution Based on Catchment-specific analyses 52

4.4.5. Examples of Investigations into Dependence between Flood Producing Factors based on (a) Antecedent Seasonal Rainfall Data for Catchments over 1000 km^2 (Scorah et al., 2015), (b) Rainfall and Storm Surge Data (Westra, 2012), (c) and Temperature Coincident with Rainfall Maxima (Nathan and Bowles, 1997) 53

4.4.6. Examples of Difference in Correlation between Flow Maxima in the Namoi and Peel Rivers, Based on (a) Annual Maxima at Both Sites, and (b) Peel River Flows that are Coincident with Namoi River Maxima 53

4.4.7. General Framework for the Analysis of Stochastic Deterministic (Joint Probability) Problems using Monte Carlo Simulation 55

4.4.8. Inverse Transform Method 56

4.4.9. Histogram Obtained by Generating 1000 Random Numbers Conforming to a Normal Distribution with a Mean of 50 and Standard Deviation of 25, and the Resulting Distribution of variables Obtained by the Inverse Box Cox Transformation (with λ set to 1.20) 57

4.4.10. Generation of Variables with a Correlation of -0.7 based on (a) Uniform and (b) Normal Distributions 59

4.4.11. Conditional Empirical Sampling - Storage Volume in an Upstream Dam is Correlated with the Volume in a Downstream Dam 61

4.4.12. Manner in which Stratified Sampling is Applied to the Rainfall Frequency Curve 65

4.4.13. Schematic layout of example joint probability problem. 66

4.4.14. (a) Flood Frequency Curves for the Mainstream and Tributary gauging sites, and (b) Correlation between Historic Flood Peaks and Sample of Generated Maxima 66

4.4.15. Derivation of Deterministic Function Relating Upstream Flows to Downstream Levels 68

4.4.16. Derived Frequency Curve of Downstream Levels, with (b) Dependence of 1% Annual Exceedance Probability Level on Degree of Correlation between Flood Peaks . 70

List of Tables

4.4.1. Stochastic Generation of Correlated log-Normal Maxima 67
4.4.2. Derivation of Deterministic Function Relating Upstream Flows to Downstream Levels (a) 68
4.4.3. Calculation of Exceedance Probability of the Level Exceeding 10.4 m using the Total Probability Theorem 69

Chapter 1. Introduction

James Ball, Rory Nathan

Chapter Status	Final
Date last updated	14/5/2019

1.1. Simulation of Design Flood Hydrographs

There are many problems where design flood characteristics other than the peak flows are required. Most commonly this involves problems where the volume of the flood hydrograph has a dominant influence on the design objective of interest. Typical problems include the design of urban stormwater drainage systems where it is necessary to size on-site detention storages, the estimation of maximum surcharge levels in detention basins and dams, and the derivation of design flood levels in situations where the floodplain can store an appreciable proportion of the event. It may also be necessary to characterise the rate of rise of selected events for flood warning purposes. The assessment of such problems requires the simulation of complete design flood hydrographs, where it may be necessary to give particular attention to the rate of rise and the total volume of the hydrograph. If design interest is focused on a location in the catchment that is materially influenced by hydraulic controls (as in many urban catchments), then it is likely that it will be necessary to model the catchment as a "system" using an integrated combination of hydrologic and hydraulic modelling (therefore use of a catchment modelling system, Book 7). Conversely, if the point of interest is largely uninfluenced by hydraulic controls (as in many rural catchments or trunk drainage networks in urban areas) then it should be sufficient to model the catchment with a hydrologic model.

The simulation of design flood hydrographs is most easily undertaken using rainfall data. While the methods presented in Book 3 provide useful independent estimates on the peak of simulated hydrographs, additional information is required to check the shape and volume of design flood hydrographs. Rainfall-based methods involve the transformation of rainfalls into a selected flood characteristic:

- event-based models transform probabilistic bursts of rainfall to corresponding estimates of floods; and
- continuous simulation models transform a time series of rainfall into probabilistic flood estimates

Hydraulic models are then used to simulate flood levels from hydrologic (and/or rainfall) inputs. The challenge with these methods is how to achieve probability neutrality, that is how to ensure that the method used to transform rainfalls into design floods is undertaken in a fashion that minimises bias in the resulting exceedance probabilities.

The guidance in this Book is focused on the conceptual frameworks used to derive design flood hydrographs, rather than on the models used to transform rainfall into runoff. Detailed guidance on the different types of hydrologic models is provided in Book 5, and guidance on the hydraulic modelling of runoff through the catchment is provided in Book 6. Guidance on the application of catchment modelling systems and the interpretation of the results obtained is provided in Book 7.

1.2. Difference Between Historic and Design Flood Simulations

It is worth noting that there is a considerable difference in the modelling approaches required to simulate historic (or observed) and design floods. Although the same mathematical procedures (and software) may be involved in both, the simulation objectives and modelling considerations are markedly different.

Estimation of a design flood involves the derivation of the relationship between the magnitude and probability of a given flood characteristic. The objective of the analysis is to provide information for risk-based planning or design purposes. Such information can be extended to provide standards-based estimates such as the Probable Maximum Flood, but the simulation objective is still to assess the performance of a system under particular set of loading conditions.

In contrast, the modelling required to simulate floods using historic or forecast rainfalls involves quite different considerations. The challenge of selecting data inputs and analysis frameworks to estimate the exceedance probability of a particular outcome is replaced by the difficulty of preparing data to reflect conditions specific to a particular event in time. The focus of the analysis might be on simulating a particular historic flood, or it may involve assessment of the antecedent conditions and forecast rainfalls associated with a flood forecast required at a particular point in time.

Identical models might be used to simulate the response of the catchment under historic or design conditions. The essential difference is that design flood simulations are undertaken to derive the best estimate of the relationship between flood magnitude and exceedance probability, whereas simulation of actual floods represent the best estimate of flood characteristics for a particular point in time. The Guidance provided in Book 6 and Book 7 is equally applicable to models used for the simulation of design or actual floods. The guidance provided in this Book is specific to the issues involved in assigning an exceedance probability to flood characteristics.

1.3. Scope

The scope of this Book is largely on the simulation frameworks used to derive design floods. Particular attention is paid to those hydrologic processes that most influence flood magnitude, and on the approaches required to estimate the exceedance probability of a flood characteristic of interest. Guidance is included on the treatment of joint probability, as the explicit analysis of the way in which factors combine to influence the frequency of floods is an essential consideration in the estimation of design floods. Worked examples are provided to assist practitioners apply the guidance to typical real-world problems.

1.4. Outline

This book is structured as follows:

- Book 4, Chapter 2 provide a broad description of the runoff processes that contribute to streamflows (interception, depression storage, infiltration, interflow, and groundwater contributions), and identifies those components of most relevance to flood generation;
- Book 4, Chapter 3 describes three different approaches to event-based modelling (simple, ensemble, and Monte Carlo methods) and describes how increasing levels of sophistication can be used to minimise bias in the transformation of design rainfalls into

design floods; Book 4, Chapter 3 also describes the use of continuous (and hybrid) simulation approaches to flood estimation, and summarises the strengths and weaknesses of the various approaches;

- Book 4, Chapter 4 introduces the generic nature of joint probability requirements; it covers the factors involved in the transformation of rainfall to runoff, and other factors (e.g. initial reservoir level or tide levels) that may influence the design performance of interest; and
- Book 4, Chapter 3 and Book 4, Chapter 4 include worked examples that illustrate application of the techniques to practical problems.

Chapter 2. Hydrologic Processes Contributing to Floods

Anthony Ladson, Rory Nathan

Chapter Status	Final
Date last updated	14/5/2019

2.1. Introduction

This chapter outlines the hydrologic processes that contribute to floods including a review of runoff generation, baseflow contributions to flood flow, flow routing and losses. The chapter concludes with a discussion of the conceptualisation of these processes in models and case studies of floods in tropical and temperate rural catchments, and in urban areas.

Under Australian conditions, the ultimate cause of the large streamflows that result in floods is usually rainfall. Other causes, such as melting of snow and ice, are less important in our temperate climate. In places storm surge may combine with stream flows to cause flooding as discussed in Book 6, Chapter 5.

The link between rainfall and streamflow is mediated by a number of processes (Figure 4.2.1). Rainfall landing on the catchment surface can be converted to runoff in different ways that depend on infiltration capacity and whether soils are saturated. Four runoff processes are discussed in Book 4, Chapter 2, Section 2: those relating to infiltration excess, saturation excess, sub-surface stormflow and impervious area runoff. Typically, only a small proportion of rainfall will become streamflow with the rest being evaporated perhaps after being intercepted by vegetation, stored in surface depressions or infiltrated to become soil moisture or groundwater. Some groundwater may contribute to floods via baseflow (refer to Book 5, Chapter 4).

There are particular conditions that can lead to high streamflow, and flooding. A 'wet' catchment means reduced losses so that a greater proportion of rainfall will be converted to runoff. A catchment could be wet up by a long period of low intensity rainfall, particularly when evaportranspiration is low, such as in winter. A short burst of high intensity rainfall can lead to flooding if there are limited opportunities for rain to be lost. This is particularly the case in catchments where impervious surfaces and piped drainage systems link runoff to streams.

Figure 4.2.1 summarises the physical processes that can lead to floods, but floods can also be considered stochastic events caused by the random simultaneous occurrence of unusual conditions. The stochastic nature of flooding was illustrated in Book 1, Chapter 3 where it was shown that flood peaks resulting from 1% AEP rainfalls ranged in magnitude from 500 m^3/s to 2000 m^3/s. The cause of this disparity in response is random variation in catchment processes, such as interception and storage, and other factors such as the spatial and temporal patterns of rainfall. The series of peak flows at a gauge are manifestations of the joint probability of these random processes.

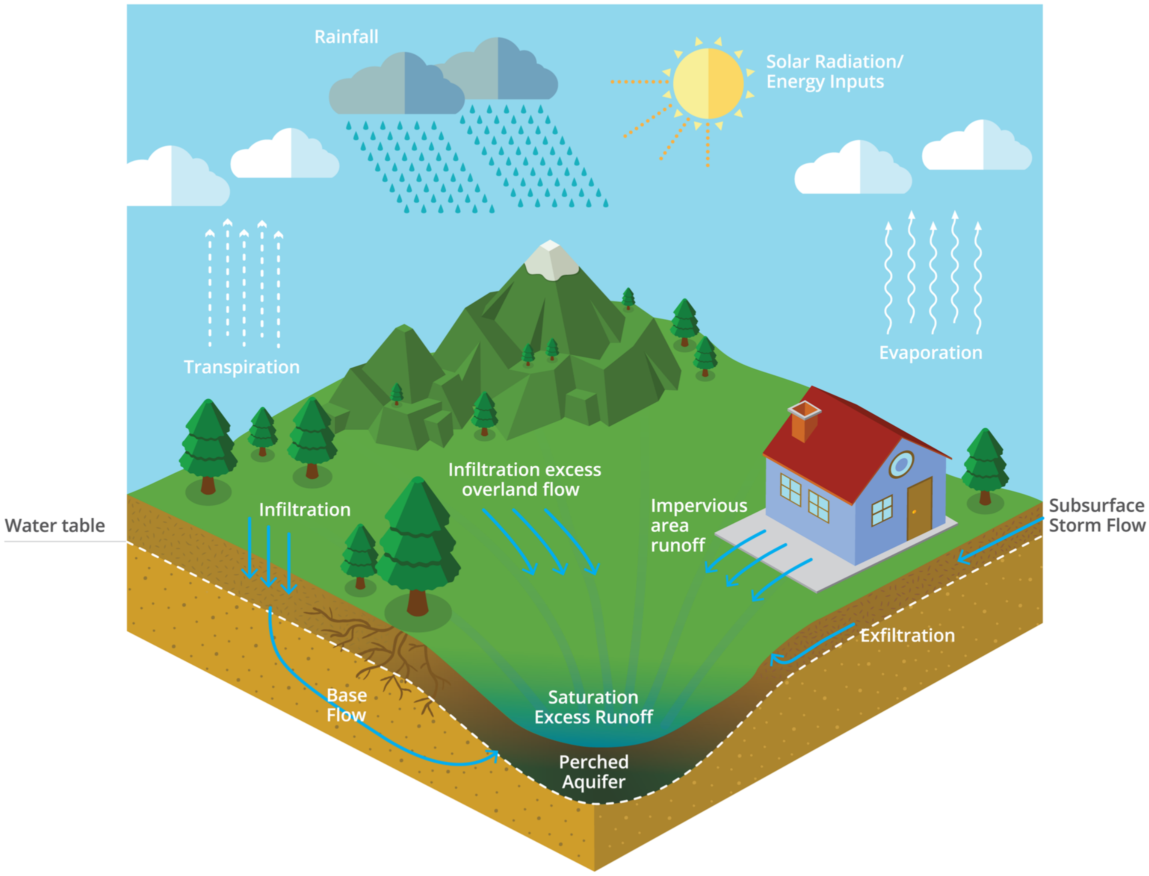


Figure 4.2.1. Catchment and Runoff Generation Processes

2.2. Runoff Generation

This section outlines some of the key runoff generation processes that can lead to floods. In particular, the following topics are addressed:

- Infiltration excess runoff;
- Saturation excess runoff;
- Variable source areas;
- Partial area runoff;
- Subsurface storm flow; and
- Impervious area runoff.

Here we are focussing on quickflow and the mechanisms that rapidly convert rainfall to streamflow and so cause a flood hydrograph. Book 4, Chapter 2, Section 3 briefly discusses the slower process of baseflow along with losses and flow routing (Figure 4.2.2).

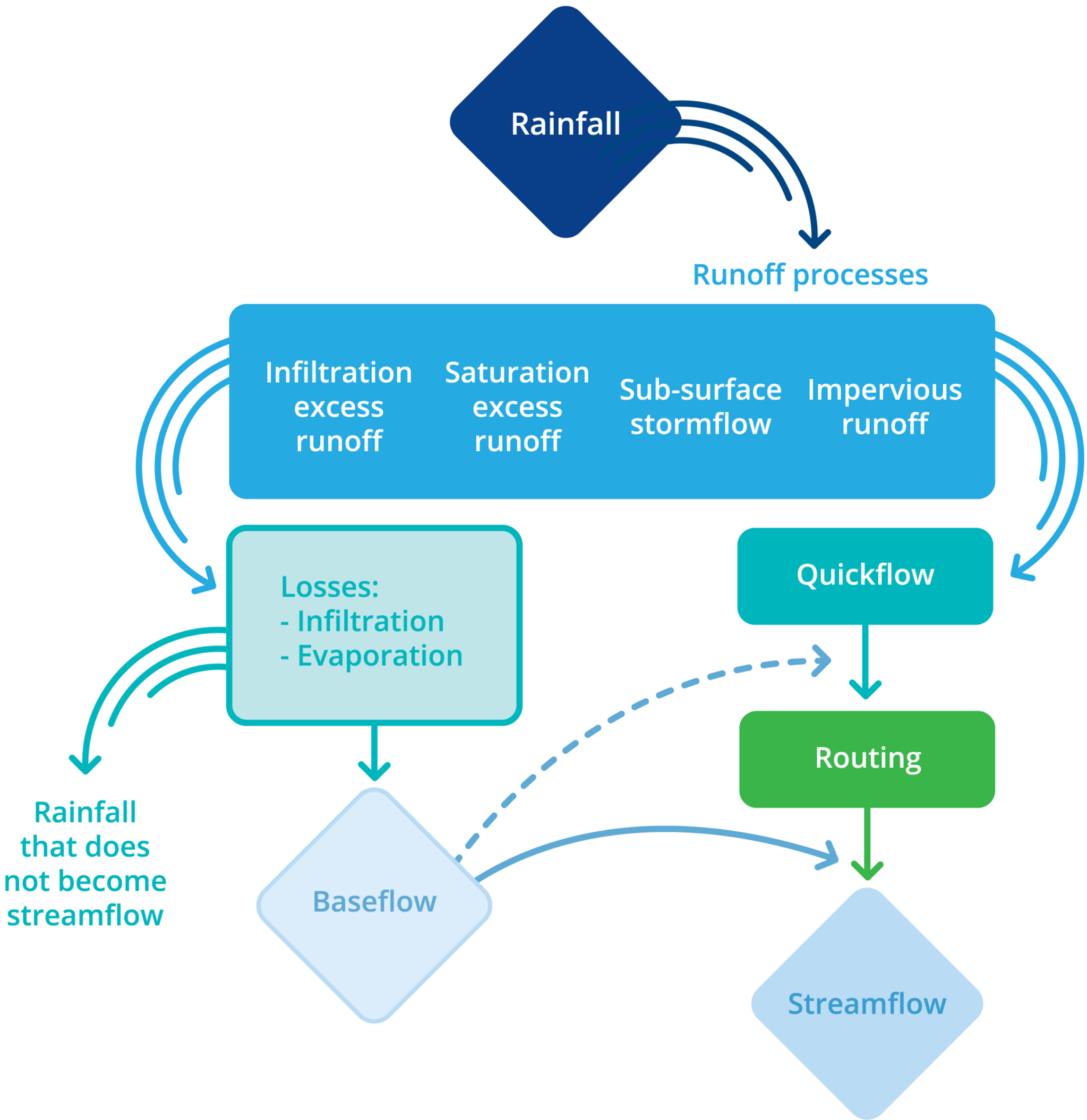


Figure 4.2.2. Simplified Description of the Process of Converting Rainfall to Runoff and Streamflow

2.2.1. Infiltration Excess Runoff

Once rainfall on a catchment reaches the soil surface, some will infiltrate into the soil. The infiltration rate, the rate at which water enters the soil, depends on:

- the rate at which water is supplied to the soil surface; and
- the infiltration capacity which is the maximum rate at which water can enter the soil.

If the rainfall rate (mm/hr) is greater than the infiltration capacity, water will pond at the soil surface; if the ground is sloping, then water will runoff. Runoff produced in this way is called infiltration excess overland flow, or Hortonian[1]overland flow. Hortonian overland flow can provide a rapid pathway for water to be converted from rainfall to runoff. Hortonian flow is likely to contribute to floods when catchment surfaces have low infiltration capacity, when there is intense rainfall and where there is a rapid mechanism for runoff to reach a stream.

2.2.2. Saturation Excess Runoff, Variable Source Areas and Partial Area Runoff

If soil becomes saturated, from rising soil moisture or because of flow from up-slope, then no additional rainfall can infiltrate. Any rainfall striking the saturated soil surface will be converted to saturation excess runoff. These saturated regions of a catchment are referred to as source areas.

Usually there are some areas within a catchment that are wetter than others. Areas along valleys and adjacent to streams may remain saturated for long periods with up-slope areas being dryer. Saturated areas enlarge and contract with the seasonal wetting and drying of a catchment. Saturated areas may expand during a storm and then shrink once rainfall ceases. As the amount of saturated area changes so does the source area contributing to runoff.

The concept of partial area runoff arises because only part of a catchment may be saturated and this area may be the only contributor to streamflow (Dunne and Black, 1970). Saturation excess runoff can contribute to floods when source areas are large and convert intense rainfall to runoff that flows directly to streams.

2.2.3. Impervious Area Runoff

Some natural catchments may contain impervious areas, such as rocky outcrops. Urbanisation leads to catchments being covered with roofs, roads, car parks and other impervious surfaces. A large proportion of rainfall landing on these surfaces is converted to runoff as there are few opportunities for rainfall to be intercepted and lost. Consequently, urbanisation leads to a large increase in runoff volume, flood frequency and magnitude. The hydrologic impacts of urbanisation have been quantified in a wide range of studies. Urbanisation causes up to a 10-fold increase in peak flows of floods in the range of 1 to 4 Exceedances per Year (EY), with diminishing impacts on larger floods (Tholin and Keifer, 1959; ASCE, 1975; Espey and Winslow, 1974; Hollis, 1975; Cordery, 1976; Ferguson and Suckling, 1990). Runoff in urban streams responds more rapidly compared to rural catchments (Mein and Goyen, 1988) and flow volumes increase (Harris and Rantz, 1964; Cordery, 1976; Ferguson and Suckling, 1990). Hydrologic impacts of urbanisation are discussed in Book 4, Chapter 2, Section 7 and in Book 9.

2.2.4. Subsurface Storm Flow

Subsurface flows can be an important source of flood runoff in areas with steep slopes, conductive soils and where the soil profile becomes saturated so that water can move through large pores. In many forested catchments surface runoff is rare. Soil infiltration rates are never exceeded by rainfall and confined streams limit opportunities for formation of saturated source areas. Instead, given appropriate soil conditions, water may be rapidly transferred down-slope as subsurface flow. This process is enhanced where there is an impeding soil layer that leads to the formation of perched water tables which cause soils to saturate and become highly conductive (Weiler et al., 2005).

[1]Hortonian runoff is named for Robert E. Horton (1875-1945); a pioneer of modern hydrology.

2.2.5. Runoff in Real Catchments

Although the distinctions between the various runoff mechanisms are useful and important, they may not be so clear cut in real catchments where runoff may be produced from a variety of mechanisms which vary between and during storms. Runoff processes may also differ compared to what would be expected. The runoff production that occurs during extreme events may not just be a variation on normal behaviour but the result of completely different processes. For example, infiltration excess processes may switch on during very intense rainfall in a catchment where runoff is normally contributed to by saturated source areas. In many cases, the catchment area can change if water flows across a drainage divide because of blockage or insufficient capacity of drainage structures. These issues are discussed further in the case studies in Book 4, Chapter 2, Section 7. Blockage issues are specifically addressed in Book 6, Chapter 6.

2.3. Baseflow

Streamflow is often divided into quickflow and baseflow. Quickflow is the characteristic rapid response of a stream to rainfall and catchment runoff while baseflow is contributed by slow release of stored water. Quickflow is often referred to as 'direct runoff' or as 'surface runoff' but, as noted above, can include subsurface stormflow. During floods, quickflow is of the greatest relevance but, particularly for modelling, baseflow must be considered where it provides a significant contribution to a flood hydrograph (Figure 4.2.3).

There are a range of processes that contribute to the conceptual baseflow hydrograph as shown in Figure 4.2.3. The initial baseflow represents the contribution from previous events; then as the hydrograph rises, baseflow can be depleted as water enters bank storage or is removed by transmission loss. Later, baseflow can increase as bank storage re-enters the stream, or through other processes such as interflow and discharge from groundwater (Laurenson, 1975).

Generally, quickflow will be explicitly modelled, by for example, a runoff-routing model, and then baseflow must be added to produce a flood hydrograph and unbiased estimate of the peak flow. Baseflow provides a significant contribution to peak flows in around 70% of Australian catchments (refer to Book 5, Chapter 4).

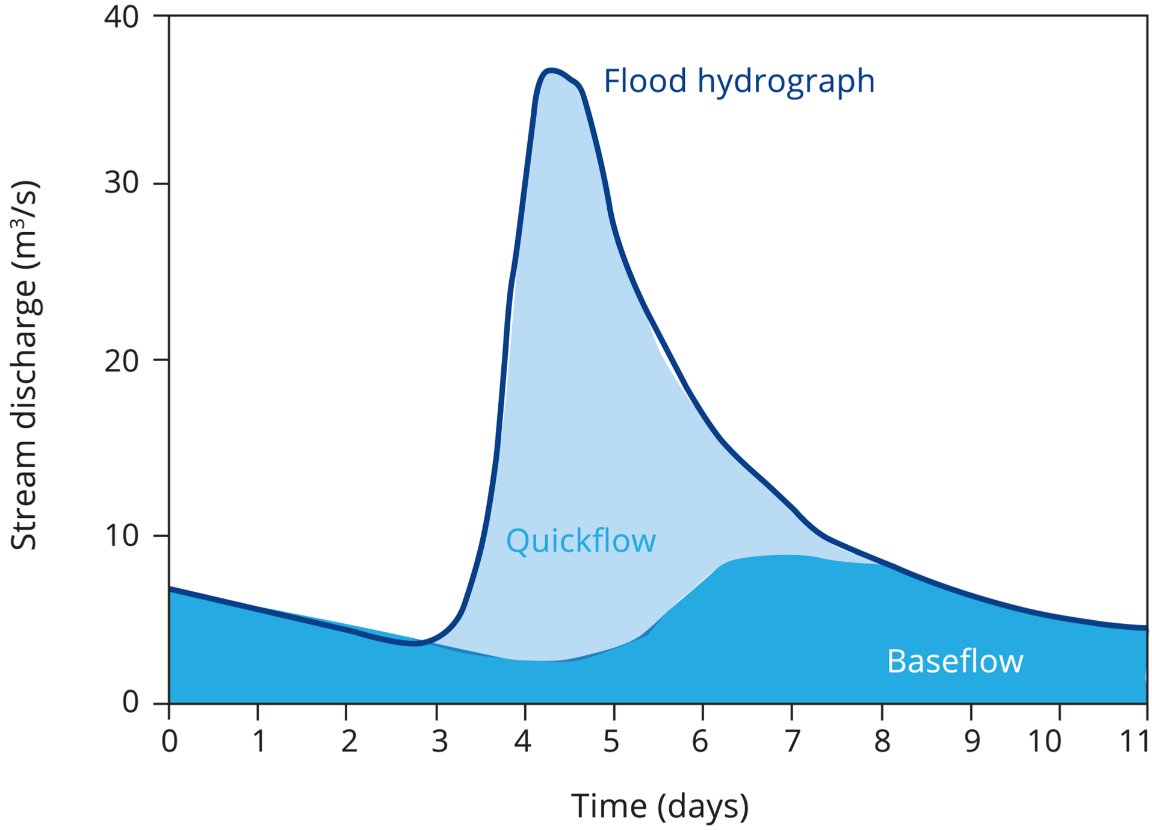


Figure 4.2.3. Observed Hydrograph - Sum of the Baseflow Hydrograph and the Quickflow

2.4. Losses

In flood hydrology, losses refer to any rainfall that is not converted to quickflow. The amount of loss is subtracted from storm rainfall to leave the “rainfall excess”, that is, quickflow is produced by the rainfall excess on the catchment. Some of the water accounted for in losses is evaporated, perhaps after being intercepted by vegetation or held in surface depressions. Some losses are infiltrated rainfall that may contribute baseflow to the stream.

Losses can be estimated for historic events. Where there are measurements of the volume of runoff, catchment area and rainfall depth, losses can be calculated as the difference between the volume of rainfall and the volume of the quickflow hydrograph (the flood hydrograph with the baseflow removed). This approach was used to estimate losses for a range of catchments as discussed in Book 5 and in earlier work on losses e.g. Hill et al. (1998).

Losses must also be predicted as part of flood forecasting and design values for losses are required as part of design flood estimation. A variety of loss models have been developed as discussed in Book 4, Chapter 2, Section 6 and in Book 5, Chapter 3, Section 2.

2.5. Flow Routing

During a flood, rainfall is converted to runoff and is transferred through a network of flow paths to the catchment outlet. These flow paths include overland flow on hill slopes, down tributaries, across floodplains, through natural and artificial storages and along main streams. Flow routing is the mathematical description of flow processes that model the attenuation and translation of hydrographs as water moves through this network. A variety of flood routing approaches are described in Book 5.

2.6. Conceptualising Processes in Models

The physical processes related to losses, runoff production, baseflow and routing need to be conceptualised and made mathematically explicit if they are to be used in modelling. This conceptualisation can vary in complexity as a function of the scales used for space and time and the representation of the underlying physics (Haan et al., 1982; Pilgrim and Cordery, 1993; Abbott et al., 1986; Beven, 2002; Beven, 2011; McDonnell, 2013; Wagener, 2003).

In general, the choice of model should depend on the amount of data that is available (Figure 4.2.4). Models that are too simple are not able to exploit the available data, while models that are too complex may suffer from 'over fitting' and have poor predictive ability. The enduring popularity of reasonably simple hydrologic models, such as RORB, is because they have been found to be of a complexity that matches the reasonably limited data that is available for most catchments.

This section briefly reviews the conceptualisation of hydrologic processes leading to floods and refers to other sections where more detail is available.

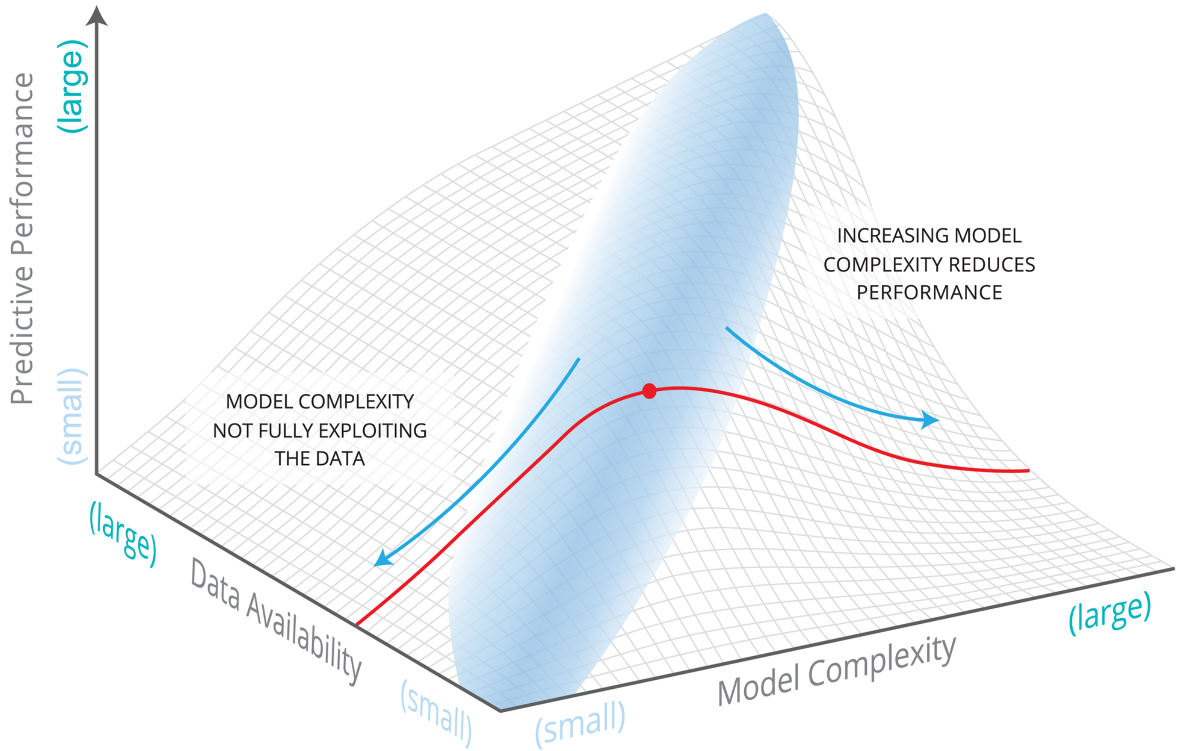


Figure 4.2.4. Conceptual Relationship between Data Availability, Model Complexity and Predictive Performance (Grayson and Blöschl, 2000)

2.6.1. Runoff Production

Models of runoff production usually require rainfall as an input, which is then allocated to surface runoff and possibly infiltration and evaporation. Rigorous approaches to modelling infiltration are available such as those based on the Richards Equation or the Green and Ampt approach (Mein and Larson, 1973; Dingman, 2002). Evaporation can be modelled as a function of meteorological drivers, soil properties and moisture content (Soil Vegetation Atmosphere Transfer (SVAT) models) (Dolman et al, 2001).

For flood modelling the physics of infiltration or evaporation, are seldom modelled explicitly, instead design or observed rainfall is converted to ‘rainfall excess’ by subtracting losses ie. the portion of rainfall that does not become direct runoff.

2.6.2. Losses

The loss models used in flood modelling are often simple, based on two parameters, one to characterise the Initial Loss (IL) (the water required to wet up the catchment) and one to characterise the Continuing Loss (CL). The output of these models is the rainfall excess that is then used to generate a direct flow hydrograph. Loss models can be standalone, ie. the rainfall excess can be calculated separately, or integrated within acatchment modelling system.

The current recommendation in ARR (Book 5, Chapter 3, Section 2) is that the IL/CL model is the most suitable for design flood estimation for both rural and urban catchments. This model uses a constant value of initial loss and constant value of continuing loss for a flood event.

For urban catchments, ARR (Book 5, Chapter 3, Section 5) provides IL and CL values for three hydrologically distinct surfaces:

- Effective Impervious Areas (impervious areas that are connected to streams by hydraulically efficient drainage);

- Pervious Areas - recommended loss values are the same as those for rural areas; and

- Indirectly Connected Areas (a combination of indirectly connected impervious and pervious areas). Recommended loss values are between those recommended for pervious and effective impervious areas.

Where losses must be estimated for flood forecasting, continuous simulation or other design problems, more complex loss model may be appropriate. Potential candidate models are discussed in Book 5, Chapter 3, Section 2.

2.6.3. Baseflow

For flood modelling, important aspects of baseflow that must be addressed are:

1. The removal of baseflow from measured hydrographs of historic flood events so that the quickflow hydrograph can be determined; and

2. The addition of a baseflow hydrograph to modelled direct flow so the total flood hydrograph, and particularly flood peak, can be correctly estimated.

Features of the baseflow hydrograph and the key processes are discussed in Book 5, Chapter 4.

When determining a design baseflow hydrograph, of particular relevance is the baseflow under the hydrograph peak as this provides a direct contribution to the maximum flood flow for an event. Procedures to estimate baseflow characteristics for design flood estimation are provided in Book 5, Chapter 4.

2.6.4. Routing

The purpose of flow routing in models is to provide a calculated estimate of the hydrograph at the downstream end of a reach given a hydrograph at the upstream end. This section briefly reviews catchment processes that are represented by routing methods in flood models. For further information on these methods, as discussed in Book 5, Chapter 5.

At any point in a stream, at a particular time during a flood event, the water flowing past will be contributed by a variety of pathways and processes that all come together to make up the flow at that instant. If we traced each drop of water within the flow, all would have originated as rainfall but have been on a variety of journeys through the catchment and travelled at different speeds: one drop of streamflow may have started as rainfall on the water surface a short distance upstream, another may have come from rain falling on saturated soil beside the river bank; yet another may have originated from a previous storm event and travelled to the stream via groundwater.

Streamflow derived from rainfall, passes through various storages. Groundwater represents long-term storage. There is also temporary storage, lasting as long as a flood event, consisting of water in transit in each element of the drainage system including water in the main stream, tributaries, hill slopes and overland flow. Water can be temporarily stored on floodplains and in retarding basins. There is also riverbank storage, water wetting up the bank profile at the start of an event and later flowing back into the stream as the water level drops.

This process description suggests routing models would need to be highly complex to represent the large number of pathways, flow speeds, and storage characteristics. However, surprisingly, simple mathematical approaches can be used to represent the movement of water along the different catchment pathways. Catchment response is usually highly damped so that short-term fluctuations in rainfall have little influence on the streamflow hydrograph and individual pathways do not need to be explicitly modelled. Instead, the dominant effect of routing is attenuation and translation which can be well represented by average response over longer time periods.

Routing of flows in a catchment may be achieved using hydrologic or hydraulic methods, and the various approaches to this are discussed in Book 5, Chapter 5. The simplest representation of routing in models is hydrologic routing which combines continuity with a relationship between storage and flow. With this approach, flow paths in a catchment are divided into a series of elements, where the volume of storage at any time is related to the discharge in each element. Differences between rural and urban streams may be represented by parameters which control the amount of water that is stored temporarily for a given flow rate. Hydrologic routing methods cannot easily accommodate backwater effects, and thus they are not well suited to situations which are influenced by tides and storm surges, or reaches in which waves propagate upstream due to the effects of large tributary inflows and waterway constrictions.

Hydraulic routing provides an increase in complexity and a reduction in the requirements for simplifying assumptions. Unsteady modelling of flows in two dimensions can be undertaken by solving the depth-averaged equations that describe the conservation of mass and momentum. These two dimesnaional (2D) models are described in more detail in Book 6, Chapter 4, Section 5 along with one dimensional (1D) unsteady models and coupled 1D/2D approaches. The limitations and appropriate use of these procedures, and others, are described in detail in Book 6.

It is possible to combine hydrologic and hydraulic routing. Hydrologic models have a short run time which facilitates the use of Monte Carlo approaches while hydraulic models are better able to represent complex routing situations. If a hydraulic model can be used to establish a storage discharge relationship, then this can be included in the hydrologic model which can then be run multiple times as part of an ensemble or Monte Carlo analysis. The use of 1D hydraulic models with short run time coupled with hydrologic models for Monte Carlo modelling is also possible.

2.6.5. Spatial Representation of Hydrological Processes

The sections above have outlined the conceptual representation of flood processes in models. Another key issue is how processes are represented spatially. In order of increasing complexity, models may be described as being lumped, semi-distributed, or distributed (Figure 4.2.5).

Lumped models (left panel of Figure 4.2.5) treat a drainage area as a single unit and use catchment averaged values of inputs and parameters. For example, spatially averaged rainfall is used as the main driver with single average values for initial and continuing loss. Simple routing approaches are used perhaps based on the passage of a hydrograph through a single storage or separate storages for surface water and groundwater. Lumped models are less common in design flood estimation or flood forecasting.

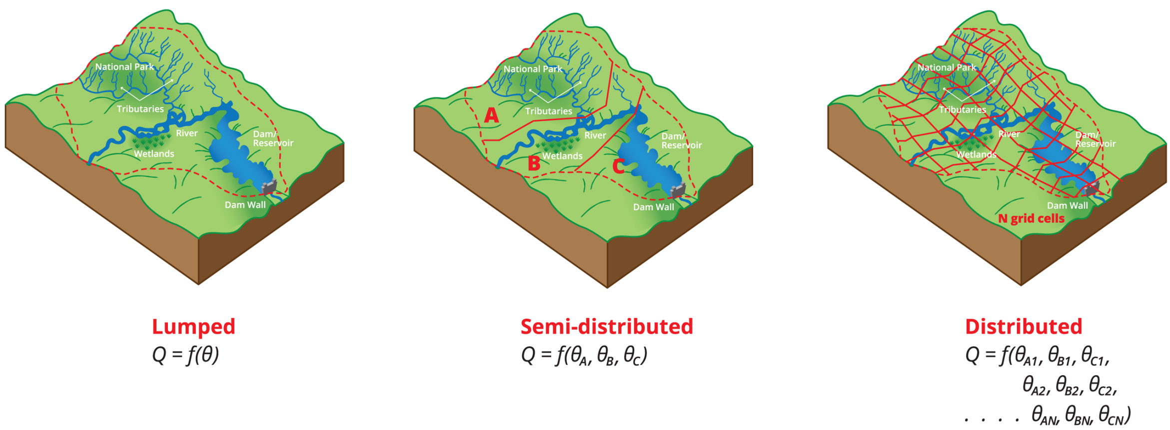


Figure 4.2.5. Spatial Representation of Physical Processes in Hydrologic Models

Semi-distributed models (middle panel of Figure 4.2.5) consider catchments as a number of reasonably large sub-areas. The spatial distribution of catchment rainfall is represented by the rainfall depth on each sub-catchment and losses and routing parameters can vary by sub-area. This approach is commonly used in design flood estimation to represents areal variations in rainfall and losses, and the effects of varying flow distance to the catchment outlet. Semi-distributed approaches can be used to create groups of hydrologic processes that are modelled in a consistent way. For example, the routing of flow down hill slopes can be modelled separately from flow routing in channels. Model setup then requires the explicit identification of hill slopes and channels that are to be modelled. The modelling equations, inputs and parameters for these areas must be provided. This group of models is discussed in detail in Book 5, Chapter 6, Section 4.

Distributed models (right panel of Figure 4.2.5) use a more spatially explicit approach, usually based on a grid that may be of a consistent size and shape across a study area or may be varied adaptively. Distributed models require inputs and parameters for each grid cell; the advantage is that results can then be produced for each grid cell. For example, two dimensional unsteady hydraulic routing approaches are commonly applied to grids to create spatially detailed information on flow depths, velocities and flood hazard in rural and urban areas.

Current approaches to design flood modelling are commonly based on a semi-distributed hydrologic model of an upper catchment area providing inputs to a distributed hydraulic model that generates outputs suitable for spatial flood mapping. The hydrologic model uses a semi-distributed approach to deal with losses and runoff generation. Hydrologic routing is used for flow down hill slopes and the upper reaches of the stream channel system. Hydraulic routing characterises flow both within channels and overbank areas where detailed information on depths and extents are required.

An alternative to combining a semi-distributed hydrologic model with a distributed hydraulic model is 'direct rainfall' or 'rainfall on grid' models. These types of models use a distributed approach to both hydrology and hydraulics by gridding an entire catchment and simulating the runoff-routing process for each grid cell. Rain falling on a grid cell is converted to runoff, after allowing for losses, and this is added to any existing flow and hydraulically routed downstream using an unsteady 2D approach. Some information on these models is provided in Book 6, Chapter 4, Section 7 with additional detail.

2.7. Examples

Three case studies are provided that outline flood runoff processes in:

- A tropical catchment (South Creek, North Queensland);
- A temperate catchment (Tarrawarra, Victoria); and
- Urban areas.

2.7.1. South Creek - North Queensland

The South Creek catchment provides a surprising example of flood runoff processes in a tropical environment with steep slopes and soils with high infiltration capacity. South Creek is 6 km east of Babinda, between Townsville and Cairns in north-east Queensland (17.35S, 145.98E) and has been well studied to determine key hydrological processes. The climate is tropical with high average annual rainfall compared to other regions of Australia. Cyclones produce rainfall intensities amongst the highest in Australia and daily rainfalls in excess of 250 mm have been reported. The catchment area is 25.7 ha with steep slopes (mean catchment slope 34%). The average saturated hydraulic conductivity of the surface soils is very high, mean value 1350 mm/hour which is higher than the rainfall intensity during the most extreme storms (the 1% Annual Exceedance Probability, 5 minute rainfall intensity is about 300 mm/hr). Saturated hydraulic conductivity decreases rapidly with depth to about 13 mm/hr below 0.2 m (Bonell et al., 1979).

At the time the South Creek catchment was instrumented, it was expected that there would be little or no overland flow. The steep, well drained and permeable slopes, along with high annual rainfall (> 4000 mm), and restricted layer at shallow depth, was expected to result in the upper layers of the soil profile becoming saturated, suggesting ideal conditions for lateral subsurface stormflow. However, this was found not to be the case.

Measurements showed that overland flow was the dominant runoff process. For example, during storms in January and March 1976, over 90% of runoff was produced by overland flow. Although rain infiltrated into the soils, the restricting layer at 200 mm depth led to a perched water table and caused saturation at the surface. Exfiltration and further rainfall landing on saturated areas, which covered most of the catchment, led to overland flow (Bonell and Gilmour, 1978).

The dominance of overland flow has implications for modelling of South Creek and similar catchments. The routing approach must be suitable for rapid flow down steep slopes that results in short lag times between rainfall and streamflow (Bonell et al., 1979). A constant continuing loss model may be suitable although this would need to be tested.

2.7.2. Tarrawarra – Southern Victoria

There has been extensive collection of hydrologic data at a small catchment (10.5 ha) at Tarrawarra 50 km ENE of Melbourne (37.66S, 145.42E), which demonstrates runoff processes in this agricultural environment (Western and Grayson, 1998). The climate is temperate with annual rainfall 820 mm and annual potential evaporation 830 mm. Evaporation exceeds rainfall in summer and surface soils dry out and crack.

During dry periods, runoff has not been observed, even during summer storms with rainfall intensities up to 50 mm/hr. When the catchment is dry, there may be local areas where rainfall intensity exceeds infiltration capacity, but any runoff that is produced enters the soil by running down surface cracks or infiltrating further down-slope and never reaches the catchment outlet.

Runoff only occurs after the catchment wets up, cracks close, and a zone of saturated soil provides a link to the catchment outlet. During wet periods, the soils at the bottom of swales saturate creating variable source areas that expand with additional rainfall. For example, during the storms of 29 and 30 July 1996 approximately half of the rainfall was converted to runoff (Western and Grayson, 2000).

The runoff production processes at Tarrawarra have implications for modelling of this type of catchment. For runoff-routing models, spatially explicit soil moisture accounting will be important as the water content of soils has a strong influence on losses (Western and Grayson, 2000). For event models, seasonal estimates of losses may be necessary. A proportional loss model may be more appropriate than one that relies on constant continuing loss.

2.7.3. Runoff from Urban Areas

Flood runoff from urban areas is larger than from rural catchments both because of catchment process and because of efficient drainage.

In an urban catchment, runoff is produced from impervious surfaces. On these surfaces: interception loss are low, because there is little vegetation; depression storage is small because the surfaces are smooth, and there is low infiltration. This means that even small amounts of rainfall will produce runoff.

In the analysis of events in urban areas, a significant feature is the small values of initial loss. Boyd et al. (1993) analysed 763 events in urban areas. For most of these, the initial loss was less than 1 mm. The average initial loss weighted by the number of events was 0.62 mm. Considering initial loss on individual catchments, information summarised in

Table 5.3.5 shows that 70% of catchments have an initial loss of 1 mm or less (refer also Book 5, Chapter 3, Section 5).

It is possible to compare the initial loss on rural and urbanised catchments. Data for Australian catchments is summarised in the Appendix to Book 5 (Book 5, Chapter 3, Section 8). A density plot of this data shows the substantially lower initial loss for urban catchments and the concentration near 0 mm. For rural catchments, the mean initial loss across all catchments is 32 mm but the high standard deviation (16.8 mm) means the density is spread across a wide range of values.

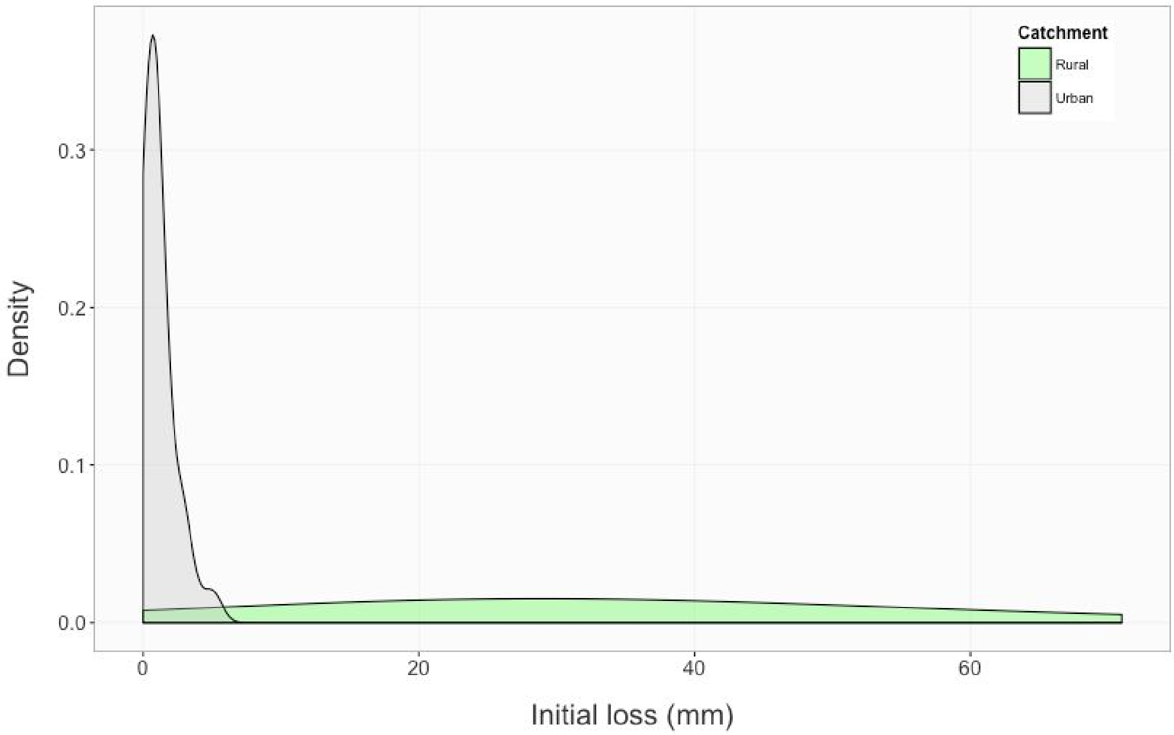


Figure 4.2.6. Comparison of Initial Loss in Urban and Rural Catchments

During larger events in urban areas, pervious surfaces also produce runoff either through infiltration excess or saturation excess processes. Many pervious surfaces in urban areas are compacted because they are walked or driven on, decreasing infiltration capacity and increasing the proportion of rainfall running off.

Along with these catchment processes, the piped drainage system in urban areas efficiently delivers water to streams. Piped drainage represents an extension of the drainage network so that even areas distant from the original natural waterways contribute flow to those waterways. In highly urbanised catchments every impervious surface will be drained to the stream.

In addition to catchment changes, the modification to urban streams also changes the transfer of flood flows. Modified urban streams have less attenuation, transmission losses are reduced and water travels more quickly. The results is a substantial increase in magnitude and frequency of flooding. Further details are provided in Book 9.

Modelling urban hydrology can be challenging because of the variety of different surface types and variation in connections between surfaces and drains. There are parallel flow paths with different routing characteristics. Some water will pass through the piped system while some will flow overland. Water can surcharge out of pipes or enter pipes at various locations in the catchment. Flow behaviour, and even catchment area, depends on flood magnitude. Modelling approaches for urban areas are discussed in Book 9.

2.8. References

ASCE (American Society of Civil Engineers) (1975), Aspects of hydrological effects of urbanisation. ASCE Task Committee on the effects of urbanisation on low flow, total runoff, infiltration, and groundwater recharge of the Committee on surface water hydrology of the Hydraulics Division. Journal of the Hydraulics Division, (5), 449-468.

Abbott, M.B., Bathurst, J.C., Cunge, J.A., O'Connell P.E. and Rasmussen, J. (1986), An introduction to the European hydrological system- Systeme Hydrologique Europeen, SHE, 1: history and philosophy of a physically-based, distributed modelling system. Journal of Hydrology, volume 87(1-2), pp.45-59.

Beven, K.J. (2002), Towards an alternative blueprint for a physically based digitally simulated hydrologic response modelling system, Hydrological Processes, 16: 189-206.

Beven, K.J. (2011), Rainfall-Runoff Modelling, Wiley (2nd Edition).

Bonell, M. and Gilmour, D.A. (1978), The development of overland flow in a tropical rainforest catchment. Journal of Hydrology, 39: 365-382.

Bonell, M., Gilmour, D.A. and Sinclair, D.F. (1979), A statistical method for modelling the fate of rainfall in a tropical rainforest catchment. Journal of Hydrology, 42: 251-267.

Boyd, M.J., Bufill, M.C. and Knee, R.M. (1993), Pervious and impervious runoff in urban catchments. Hydrological Sciences Journal, 38(6), 463-478.

Cordery, I. (1976), Some effects of urbanisation on streams. Civil Engineering Transactions, The Institution of Engineers, Australia, CE18(1), 7-11.

Cordery, I. and Pilgrim, D.H. (1983), On the lack of dependence of losses from flood runoff on soil and cover characteristics. Hydrology of Humid and Tropical Regions with Particular Reference to the Hydrological Effects of Agriculture and Forestry Practice (Proceedings of the Hamburg Symposium). Hamburg, IAHS Publication No.140.

Dingman, S.L. (2002), Physical hydrology. Prentice Hall.

Dolman, A.J., Hall, A.J., Kavvas, M.L., Oki, T. and Pomeroy, J.W. (2001), Soil-Vegetation-Atmosphere transfer schemes and large-scale hydrologic models. Proceedings on an international symposium held during the sixth Scientific Assembly of the International Association of Hydrological Sciences (IAHS) at Maastricht, The Netherlands 18-27 July 2001. IAHS Publication No.270.

Dunne, T. and Black, R.D. (1970), Partial area contributions to storm runoff in a small new England watershed. Water Resources Research, 6(5), 1296-1311.

Espey, W.H. and Winslow, D.E. (1974), Urban flood frequency characteristics. Journal of the Hydraulics Division. American Society of Civil Engineers, 100(HY2), 279-293.

Ferguson, B.K. and Suckling, P.W. (1990), Changing rainfall-runoff relationships in the urbanizing Peachtree Creek Watershed, Atlanta, Georgia. Water Resources Bulletin, 26(2), 313-322.

Grayson, R.B. and Blöschl, G. (2000), Spatial Patterns in Catchment Hydrology: Observations and Modelling. Cambridge University Press, p: 404

Haan, C.T., Johnson, H.P. and Brakensiek, D.L. (1982), Hydrologic modelling of small watersheds. American Society of Agricultural Engineers. St Joseph, Michigan.

Harris, E.E. and Rantz, S E. (1964), Effect of urban growth on streamflow regimen of Permanente Creek Santa Clara County, California: Hydrological effects of urban growth. Geological survey water-supply paper 1591-B. US Department of the Interior.

Hill, P.I., Mein, R.G. and Siriwardena, L. (1998), How much rainfall becomes runoff: loss modelling for flood estimation. Cooperative Research Centre for Catchment Hydrology.

Hollis, G.E. (1975), The effect of urbanisation on floods of different recurrence interval. Water Resources Research, 11: 431-435.

Laurenson, E.M (1975), Streamflow in catchment modelling. In: Chapman, T. G. and Dunin, F. X. (eds) Prediction in catchment hydrology. Australian Academy of Science, pp.149-164.

Laurenson, E.M. and Pilgrim, D.H. (1963), Loss rates for Australian Catchments and their Significance Journal, Institution of Engineers, Australia, 35(1-2), 9-24.

McDonnell, J.J. (2013), Are all runoff processes the same? Hydrological Processes, 27: 4103-4111.

Mein, R.G. and Goyen, A.G. (1988), Urban runoff. Civil Engineering Transactions, 30: 225-238.

Mein, R.G. and Larson, C.L. (1973), Modelling infiltration during steady rain. Water Resources Research, 9(2), 384-394.

Pilgrim, D.H. and Cordery, I. (1993), Flood runoff. In: Maidment, D.R. (ed) Handbook of Hydrology. McGraw-Hill, pp: 9.1-9.42.

Tholin, A.L. and Keifer, C.J. (1959), The hydrology of urban runoff. Journal of the Sanitary Engineering Division, 85(SA2), 47-106.

Wagener, T. (2003). Evaluation of catchment models, Hydrological Processes, 17.16:3375-3378.

Weiler, M., McDonnell, J.J., Tromp-van Meerveld, I. and Uchida, Taro. (2005), Subsurface Stormflow. In: Anderson, M.G. (ed.) Encyclopedia of Hydrological Sciences. John Wiley and Sons.

Western, A.W. and Grayson, R.B. (1998), The Tarrawarra data set: soil moisture patterns, soil characteristics and hydrological flux measurements. Water Resources Research 34(10), 2765-2768.

Western, A.W. and Grayson, R.B. (2000), Soil moisture and runoff processes at Tarrawarra. In: Grayson, R. B. and Blöschl G. (eds) Spatial patterns in catchment hydrology: observations and modelling. Cambridge University Press, pp: 209-246.

Chapter 3. Types of Simulation Approaches

Rory Nathan, Fiona Ling

Chapter Status	Final
Date last updated	14/5/2019

3.1. Introduction

Rainfall-based models are commonly used to extrapolate flood behaviour at a particular location using information from a short period of observed data. This can be done using either event-based or continuous simulation approaches.

Event-based approaches are based on the transformation of a discrete rainfall event into a flood hydrograph using a simplified model of the physical processes involved. It requires the application of two modelling steps, namely: a *runoff production* model to convert the storm rainfall input at any point in the catchment into rainfall excess or runoff at that location, and a *hydrograph formation* model to simulate the conversion of these local runoffs into a flood hydrograph at the point of interest. The rainfall event is described by a given depth of rainfall occurring over a selected duration, where it is necessary to specify the manner in which the rainfall varies in both time and space. The input rainfall may represent a particular observed event, or else it may represent the depth of rainfall with a specific Annual Exceedance Probability (ie. a design rainfall). The former approach is most commonly used for model calibration and flood forecasting, the latter approach is used to estimate flood risk for design and planning purposes. The defining feature of such models is that they are focused on the simulation of an individual flood event, and that antecedent (and baseflow) conditions need to be specified in some explicit fashion.

In contrast, continuous simulation approaches transform a long time series of rainfall (and other climatic inputs) into a corresponding series of streamflows. Such time series may span many weeks or years, and may represent behaviour that reflects the full spectrum of flood and drought conditions. Such models comprise simplified representation of catchment processes, and most usually involve the simulation of soil moisture and its control over the partitioning of rainfall into various surface and subsurface contributions to recharge and streamflow. Once simulated, information on the frequency and magnitude of flood behaviour needs to be extracted from the resulting time series using the same methods adopted for historical streamflow data.

The relative strengths and weaknesses of these approaches are outlined in Book 1, Chapter 3. The following sections provide information on simulation approaches relevant to each approach, where guidance on their calibration and application is presented in Book 7. Event-based models may be implemented in a variety of ways, and three approaches of increasing sophistication are described in Book 4, Chapter 3, Section 2 to Book 4, Chapter 3, Section 2. The Simple Event approach is first described in Book 4, Chapter 3, Section 2, and this includes discussion of the main elements that are common to all event-based approaches. The Ensemble Event approach (Book 4, Chapter 3, Section 2) provides a simple means to accommodate variability of a selected input, and this is followed by description of Monte Carlo approaches in Book 4, Chapter 3, Section 2, which provide a rigorous treatment of the joint probabilities involved in estimation of design floods. Continuous Simulation approaches are described in Book 4, Chapter 3, Section 3, and hybrid approaches based on a mixture of event- and continuous schemes are briefly described in Book 4, Chapter 3, Section 4. The performance, strengths and limitations of the different approaches are discussed in Book 4, Chapter 3, Section 5 and Book 4, Chapter 3, Section 6, and finally, the elements of a worked example are presented in Book 4, Chapter 3, Section 7.

3.2. Event-Based Approaches

3.2.1. General Concepts

Event-based approaches represent traditional practice in Australia and most overseas countries for derivation of design floods from design rainfalls. Typical hydrologic inputs to event-based models include:

- A *design storm* of preselected AEP and duration: historically it has been most common to only consider the most intense parts of complete storms (“design burst"), where the average intensity of the burst is determined from rainfall Intensity Frequency Duration (IFD) data (Book 2, Chapter 2). This information is generally available as a point rainfall intensity, and it is necessary to apply an Areal Reduction Factor (Book 2, Chapter 4) to correctly represent the areal average rainfall intensity over a catchment;

- *Temporal patterns* to distribute the design rainfall over the duration of the event, and this can include additional rainfalls before the start (and after the end) of the burst to represent complete storms (Book 2, Chapter 5);

- *Spatial patterns* to represent rainfall variation over a catchment that occurs as the result of factors such as catchment topography and storm movement (Book 2, Chapter 4); and

- *Loss parameters* that represent soil moisture conditions in the catchment antecedent to the event and the capacity of the soil to absorb rainfall during the event (Book 5, Chapter 5).

A range of event-based models are available to convert rainfalls into a flood hydrograph, though in generally these models provide highly simplified representations of the key processes relevant to flood generation:

- *A loss model* is used to estimate the portion of rainfall that is absorbed by the catchment and the portion that appears as direct runoff (Book 5, Chapter 3). This loss is typically attributed to a range of processes, including: interception by vegetation, infiltration into the soil, retention on the surface (depression storage), and transmission loss through the stream bed and banks; and

- *A hydrograph formation model or hydrologic routing model* (usually based on runoff-routing concepts, as discussed in Book 5, Chapter 6) is used to transform the patterns of rainfall excess into a design flood hydrograph. This flood hydrograph may include a baseflow component which initially represents the delayed contribution from previous rainfall events, and in the latter stages of the event may represent the contribution from earlier losses.

The most commonly applied event-based approach is the Design Event approach which assumes that there is a *critical rainfall duration* that produces the design flood for a given catchment. This critical duration depends on the interplay of catchment and rainfall characteristics; it is not known *a priori* but is usually determined by trialling a number of rainfall durations and then selecting the one that produces the highest flood peak (or volume) for the specific design situation.

An important consideration in the application of this approach is that the inputs defining the Design Event should be selected to be probability neutral. This involves selecting model inputs and parameter values such that the 1 in X AEP design rainfalls are converted to the corresponding 1 in X AEP floods. The task of defining a typical combination of flood producing factors for application in the 'Design Event' approach is made particularly difficult by the fact that flood response to rainfall is generally non-linear and can be highly non-linear. This means that average conditions of rainfall or loss are unlikely to produce average flood conditions. The probability neutrality of inputs can only be tested if independent flood estimates are available for comparison; for more extreme events, the adopted values of probability neutral inputs must be conditioned by physical and theoretical reasoning.

The following guidance presents three approaches to dealing with probability neutrality, namely:

- *Simple Event*, where all hydrologic inputs are represented as single probability neutral estimates from the central range of their distribution;
- *Ensemble Event,* where the dominant factor influencing the transformation is selected from a range of values representing the expected range of behaviour, and all other inputs are treated as fixed; and
- *Monte Carlo Event,* where all key factors influencing the transformation are stochastically sampled from probability distributions or ensembles, preserving any significant correlations between the factors, and probability neutrality is assured (for the given set of inputs) by undertaking statistical analysis of the outputs.

The key differences between these approaches is illustrated in Figure 4.3.1.Book 4, Chapter 3, Section 2 to Book 4, Chapter 3, Section 2 describe each of these procedures in turn, though it is worth noting here the essential similarities between the three methods as shown in Figure 4.3.1. It is seen that these three methods use the same source of design rainfalls and the same conceptual model to convert rainfall into a flood hydrograph. The process involved in calibrating a conceptual model to historic events is common to all three approaches, they differ only in how selected inputs are treated when deriving design floods.

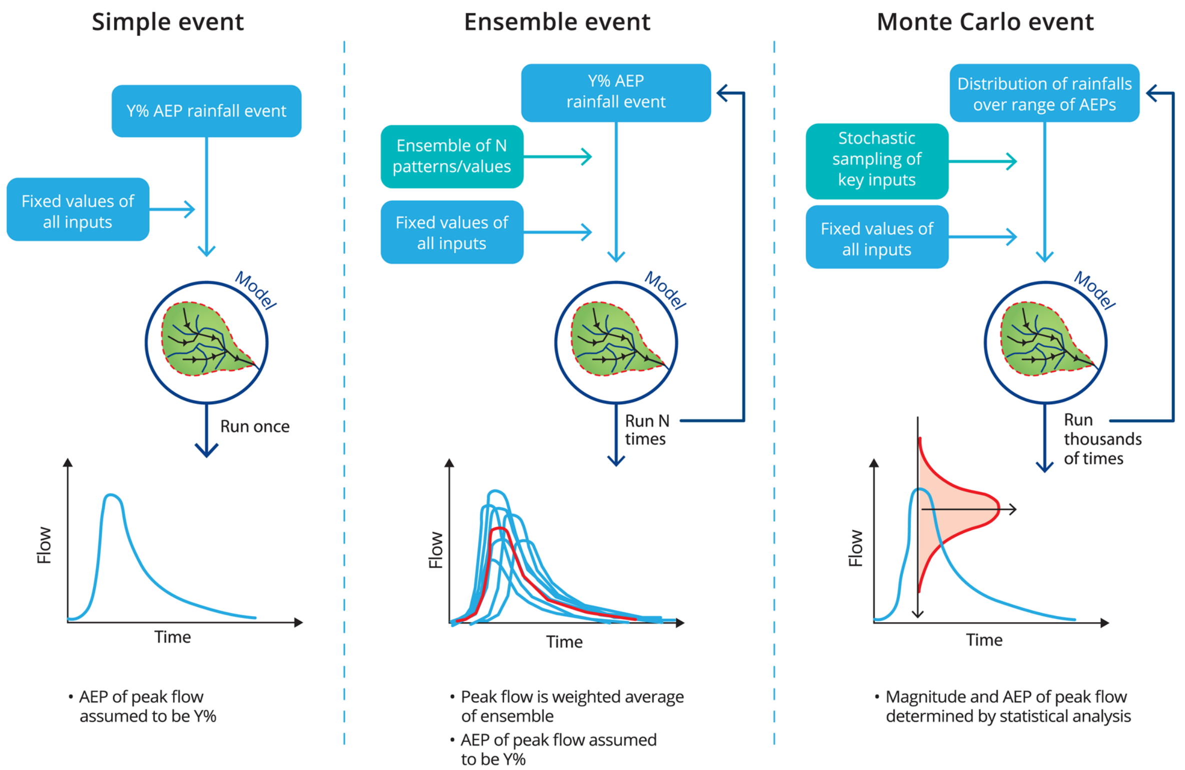


Figure 4.3.1. Elements of Three Different Approaches to Flooding

3.2.2. Simple Event

As shown in Figure 4.3.1, the first step in the Simple Event method is to estimate the average intensity or depth of rainfall corresponding to a given AEP for a selected duration using Intensity Frequency Duration (IFD) data, as provided in Book 2, Chapter 2. The next step is to select representative values of other factors that influence the transformation of rainfall to flood hydrograph. At a minimum, this involves selecting representative temporal and spatial patterns of rainfall, and selecting appropriate loss parameters.

Representative temporal patterns of rainfall may be obtained by applying the Average Variability Method to a sample of historic patterns (Pilgrim et al., 1969; Pilgrim and Cordery, 1975). The intent of this method is to derive a single temporal pattern which is representative of the average variability of intense rainfall relevant to the selected storm duration and severity. Their use is based on the assumption that such patterns should minimise the introduction of joint probabilities into the design flood model and aid in estimation of a flood with the same frequency as the design rainfall. However, there is good evidence that patterns of average variability do not ensure probability neutrality (e.g. Sih et al. (2008), and Green et al. (2003)), and it is possible that adoption of historical patterns selected from within the range of observed variability are as efficacious as synthetic ones derived using the Average Variability Method. Temporal patterns based on the Average Variability Method have been developed for point rainfalls up to the 1 in 500 AEP (Pilgrim (1987) Volume 2) and for areal Probable Maximum Precipitation estimates (Nathan, 1992; Green et al., 2003).

Spatial patterns of rainfall generally have a lower influence on flood characteristics than temporal patterns, and consequently simpler approaches are used to accommodate the joint probabilities involved. For most practical situations it is assumed sufficient to adopt a fixed non-uniform pattern that reflects the systematic variation arising from topographic influences (Book 2, Chapter 4).

For estimating losses, various types of models ranging from a simple loss model to complex conceptual runoff-routing models are available (Hoang et al., 1999; Hill et al., 2012). Loss models most suited for design purposes generally involve specification of a parameter (such as initial loss) that is related to soil moisture conditions in the catchment prior to the onset of the storm. They also generally involve specification of a loss term related to the infiltration of a proportion of storm rainfall during the event (e.g. continuing loss or proportional loss). The most comprehensive analyses of design loss values available to date have been undertaken by Kuczera et al. (2006) and Newton and Walton (2000), and guidance on suitable loss values to adopt is provided in Book 5, Chapter 5. The selected loss values can have a large influence on the resulting flood characteristic, and the adoption of regional estimates does not guarantee unbiased estimates of the resulting floods; for this reason it is also desirable to reconcile design values with independent flood frequency estimates where possible (as discussed in Book 5, Chapter 5).

The direct runoff simulated by the loss model is then routed through the catchment to generate the design flood hydrograph. The hydrograph corresponding to the rainfall burst duration that results in the highest peak (the critical rainfall duration) is taken as the design flood hydrograph, and it is assumed to have the same Annual Exceedance Probability as its causative rainfall. It needs to be stressed that probability neutrality is an untested assumption with the simple event approach, and without reconciliation with flood frequency estimates using at-site or transposed gauged maxima, there is no way of determining how the selected inputs may have biased the outcome.

In summary, the only probabilistic variable considered with the Simple Event approach is average rainfall intensity or depth, while other inputs (e.g. losses, rainfall temporal and spatial patterns) are represented by fixed values drawn from the central tendency of their distribution (Rahman et al., 1998; Nathan et al., 2002; Rahman et al., 2002a; Kuczera et al., 2006; Nathan et al., 2003).

3.2.3. Ensemble Event

The Ensemble Event approach is essentially an intermediate step between a Simple Event approach and Monte Carlo Event simulation. In its simplest implementation, a fixed factor with large influence on flood magnitude is replaced by a sample of values (an “ensemble”); each of these values is then input to the flood event model to derive a set of flood hydrographs. The magnitude of the design flood is then estimated from the weighted average of the hydrographs, where the weighting applied to each result reflects the relative likelihood of the selected input occurring. If a sample of observed temporal patterns is used instead of a single pattern of average variability, then studies have shown (Sih et al., 2008; Ling et al., 2015) that a simple arithmetic average based on a sample of 10 to 20 patterns provides a reasonably unbiased estimate of the design flood. The rationale for this approach is that each of the patterns selected for the ensemble is equally likely.

In concept the approach could be extended to take account of factors that are non-uniformly distributed, though here it would be necessary to carefully weight the outcome by the relative likelihood of the different values selected, or else select the input values in a way that reflects the form of their distribution. For example, if a sample of ten initial loss values were selected, then it would be necessary to weight each result by the probability of each loss value occurring, which could be determined (for example) from the cumulative distribution of losses presented in Book 5, Chapter 5; alternatively, the distribution of losses could be divided into ten equally likely exceedance percentile ranges, and the results then be given equal weighting.

It is expected that the approach is most suited to the consideration of temporal patterns, as suitable ensemble sets of patterns are readily available (as described in Book 2, Chapter 5). Flood magnitudes are generally very sensitive to temporal patterns and thus the ensemble approach provides a straightforward, if somewhat tedious, means of avoiding the introduction of bias due to this source of variability. Extending the ensemble method to consider other inputs, jointly or otherwise, would appear to introduce additional problems which are probably most easily handled by Monte Carlo approaches.

3.2.4. Monte Carlo Event

Monte Carlo methods provide a framework for simulating the natural variability in the key processes that influence flood runoff: all important flood producing factors are treated as stochastic variables, and the less important ones are fixed. The primary advantage of the method is that it allows the exceedance probability of the flood characteristic to be determined without bias (subject to the representativeness of the selected inputs).

In the most general Monte Carlo simulation approach for design flood estimation, *rainfall events of different duration* are sampled stochastically from their distribution. The simulated design floods are then weighted in accordance with the observed frequency of occurrence of rainfall events of different durations that produced them. This avoids any positive bias of estimated flood probabilities which may be associated with the application of the critical rainfall duration concept (Weinmann et al., 2000; Weinmann et al., 2002; Rahman et al., 2002b). The application of this generalised approach relies on the derivation of new design data for rainfall events that are consistent with a new probabilistic definition of storm 'cores' or complete storms (Hoang et al., 1999). Such design rainfall data is currently not available, thus limiting the application of the generalised approach. To obviate the need for this, Nathan et al. (2002) and Nathan et al. (2003) adapted the approach to separately consider different rainfall durations; the resulting peak flows are then enveloped to select the critical event duration, consistent with the 'critical rainfall duration' concept used in traditional design flood estimation practice. This is the approach further described herein. Whilst adherence to the 'critical duration' concept could possibly introduce systematic bias into the results, it has the advantage of ensuring consistency with existing design approaches and allows much of the currently available design data to be readily used.

Undertaking a Monte Carlo simulation requires three sets of key decisions, followed by a simulation step that involves construction of the derived flood frequency curve. The overall steps involved are as follows:

i. *Select an Appropriate Flood Event Simulation Model* - The criteria for selecting an appropriate model are similar to those used with the traditional Design Event approach and are described in Book 5. The selected model should be able to be run in batch mode with pre-prepared input files or be called from the Monte Carlo simulation application. Models with fast execution speeds are well suited to Monte Carlo simulation; complex models with slow run-times can still be utilised, though generally they need to be invoked within a stratified sampling scheme (Book 4, Chapter 4, Section 3) to ensure that the simulations times are within practical constraints.

ii. *Identify the Model Inputs and Parameters to be Stochastically Generated* - The stochastic representation of model inputs should focus on those inputs and parameters which are characterised by a high degree of natural variability and a non-linear flood response. Examples include rainfall temporal pattern, initial loss and reservoir storage content at the start of a storm event. If the assessment indicates limited variability and essentially linear system response, then there may be little to be gained from extending the Monte Carlo simulation approach to include such additional inputs or parameters.

iii. *Define the Variation of Inputs/Parameters by Appropriate Distributions and Correlations* - The considerations and methods applicable to joint probability aspects are described in Book 4, Chapter 4. The distributions used to generate the stochastic inputs can be defined by the use of specific theoretical probability distributions or else an empirical, non-parametric approach can be adopted. Schaefer and Barker (2002) and Schaefer and Barker (2004) adopts a strongly parametric approach to sampling a wide range of storm and catchment processes, Rahman et al. (2002b) and Rahman et al. (2002a) provides examples in which both losses and temporal patterns are defined using a Beta distribution. (Nathan et al., 2003) and (Nathan and Weinmann, 2004) adopt a more empirical approach that is more closely aligned to the nature of design information used in the traditional Design Event method. If any of the stochastic inputs exhibit significant correlations, their correlation structure needs to be defined, and the correlations included in the sampling scheme.

iv. *Undertake Monte Carlo Simulation* - The design inputs and parameters exhibiting significant variability are sampled in turn from their distributions allowing for significant correlations, and the resulting combination of inputs and parameters is then used in a simulation model run. Only those inputs that have a significant influence on the results need to be stochastically sampled, and other inputs can be treated as fixed (usually average or median) values. For Monte Carlo simulation involving several stochastic variables, many thousands of simulations are required to adequately sample the inherent variability in the system, and thus for most practical problems some thought is required to minimise disc storage space and simulation times.

v. *Construct the Derived Flood Frequency Curve* - Once the required number of runs has been undertaken, it is necessary to analyse the results to derive the exceedance probabilities of different flood magnitudes. Where very simple models are used or the probabilities of interest are not extreme – more frequent than, say, 1 in 100 Annual Exceedance Probabilities (AEP) – the simulation results can be analysed directly using frequency analysis (as described in Book 3, Chapter 2). Alternatively, in order to estimate rarer exceedance probabilities (or use more complex models with slow execution speeds) it is desirable to adopt a stratified sampling approach to derive the expected probabilities of given event magnitudes, as described in Book 4, Chapter 4.

An example flowchart for the last two steps is illustrated in Figure 4.3.2. This flowchart represents the high level procedure relevant to the consideration of the joint probabilities involved in the variation of loss parameters and temporal patterns. The starting point for this simple Monte Carlo simulation is the Step “A” in Figure 4.3.2. The loss and temporal patterns are then sampled and combined with fixed values of other inputs for simulation using a flood event model. Once many thousands of combinations of rainfall depth, losses and temporal patterns have been undertaken, the resulting flood maxima are analysed to derive unbiased estimates of flood risk (represented by Step “B”, Figure 4.3.2). Suitable sampling schemes and analyses relevant to these steps are described in Book 7, Chapter 7, where additional variables (such as reservoir level or rainfall spatial pattern) can be included as additional sampling steps as required.

Figure 4.3.2 also depicts the relationship between Monte Carlo schemes and the other simpler event-based methods discussed above. The blue-shaded shapes represent the steps involved in the traditional Simple Event (or Design Event) approach, where the flood characteristic obtained from a single simulation using the selected inputs (Step “C”) is assumed to have the same Annual Exceedance Probability as its causative rainfall. The ensemble approach is shown as an added loop: in this example the simulation would be repeated for each available temporal pattern, and the results would be averaged (at Step “C”) to yield the flood characteristic of interest, where again it is assumed that the Annual

Exceedance Probability of the calculated flood is the same as its causative rainfall. The 2nd and 3rd last shapes represent the additional steps required to implement a Monte Carlo scheme.

It should be noted that the steps involved between points A and B in Figure 4.3.2 represent the scheme required to consider the joint probabilities associated with the variability of selected inputs. It represents the characterisation of *aleatory uncertainty*, which is the (irreducible) uncertainty associated with variability inherent in the selected inputs. However, Monte Carlo schemes can also be used to consider *epistemic uncertainty*, and the additional steps involved in this are shown by the first and last steps in Figure 4.3.2 Epistemic (or reducible) uncertainty is due to lack of knowledge, and is associated with errors in the data or the simplifications involved in representing the real world by a conceptual model. In essence, the consideration of aleatory uncertainty allows the derivation of a single (probability neutral) “best estimate” of flood risk, and consideration of epistemic uncertainty allows the characterisation of confidence limits about this best estimate. The outer (dark blue-shaded) iteration loop shows extension of approach to estimate confidence limits. Figure 4.3.2 has inner (blue-shaded) shapes that show steps involved in Simple Event approach, where dashed lines indicate additional iteration required for Ensemble Event approach.

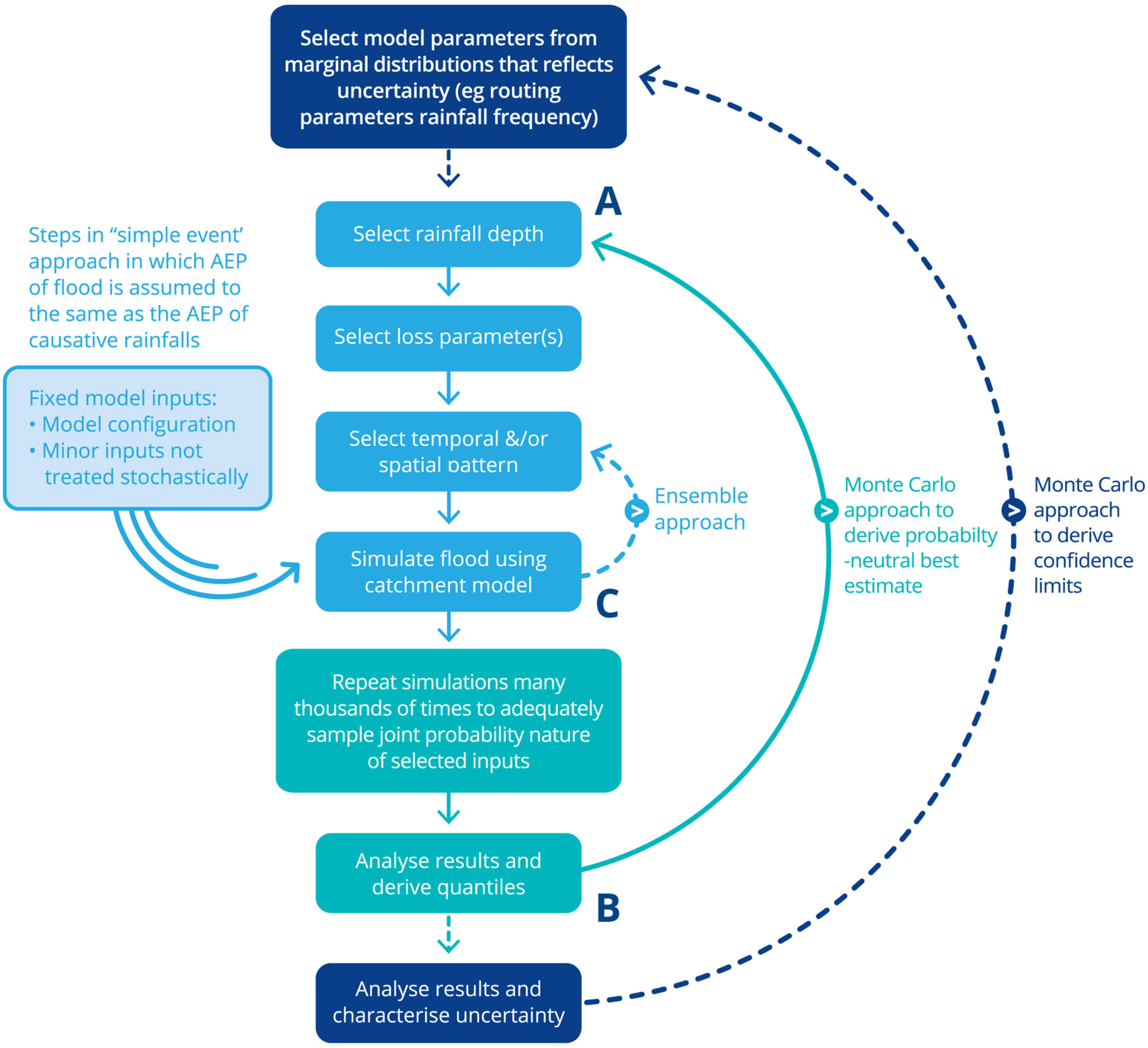


Figure 4.3.2. Simple Framework for Monte Carlo Simulation for Handling Joint Probabilities Associated with Both Losses and Temporal Patterns

In general, while the information required to characterise aleatory uncertainty can be readily obtained from the observed record, this is not the case with epistemic uncertainty. Indeed it is quite difficult to obtain information on the likely errors associated with input data or model parameterisation, and it is very difficult to characterise the uncertainty associated with model structure. Accordingly, the guidance presented here focuses on the assessment of aleatory uncertainty as it is considered that this approach can be readily understood and applied by practitioners with the appropriate skills. Thus, while it seems reasonable to regard the use of Monte Carlo procedures to accommodate hydrologic variability as “best practice” for many practical design problems, its use to derive confidence limits is expected to remain the domain of more academic specialists for the foreseeable future.

3.3. Continuous Simulation Approaches

3.3.1. General Concepts

The last few decades have seen considerable advances in computational power. This has allowed implementation of models that are more complex and that provide greater (and more elaborate) representation of the physical processes occurring in a catchment (Boughton and Droop, 2003). This has led to development of large numbers of runoff-routing models from the highly conceptualised Stanford Watershed Model (Linsley and Crawford, 1960) to more physically based models such as the Systeme Hydrologique Europeen Model (SHE; (Abbott et al., 1986)). Traditionally, rainfall based methods of estimating the design flood have predominately been event-based, while continuous simulation has been applied for yield estimation or flow forecasting. However, development of tools and methods that allow generation of long periods of synthetic rainfall data has led to increased interest in using continuous simulation for design flood estimation and the concept of using models traditionally developed for yield estimation for the estimation of design floods (Boughton and Droop, 2003).

The Continuous Simulation method of estimating the design flood is similar in intent to the event-based Monte Carlo approach discussed in Book 4, Chapter 3, Section 2. Both methods seek to adequately simulate the interactions between flood producing (rainfall and catchment characteristics) variables (Kuczera et al., 2006). Conceptually, the differences between the two methods arise in how wet and dry periods are sampled and incorporated into the process of estimating the design flood. In the event-based Monte Carlo method rrunoff-routing models are used to simulate the interactions occurring only during the storm (wet period) event. There is implicit consideration of the influence of dry periods in sampling the catchment-rainfall interactions (antecedent conditions, temporal patterns, storm durations) from exogenously derived distributions of initial conditions (Kuczera et al., 2006). The Continuous Simulation method, on the other hand, accounts for these interactions through direct simulation of the processes occurring in the catchment over an extended period (Kuczera et al., 2006; Boughton et al., 1999; Cameron et al., 1999). The Continuous Simulation method is also applicable in situations where the critical event duration extends over many weeks or months, as is the case for systems with large storage capacity but limited outflow capacity.

The Continuous Simulation method of estimating the design flood involves running a conceptual runoff-routing model for a long period of time such that all important interactions (covering the dry and wet periods) between the storm (intensity, duration, temporal pattern) and the catchment characteristics are adequately sampled to derive the flood frequency distribution. In general, pluviograph data of hourly resolution (or less) is used to drive the runoff-routing models. In most cases the period of record of pluviograph data rarely exceeds 20 years, therefore rainfall data is extended by using stochastic rainfall data generation. The runoff-routing model is calibrated using flow data, where available, and the calibrated model is then used to generate a long series of simulated flow. Finally the simulated flow is then used to extract the Annual Maximum Series and estimate the derived flood frequency curve. Important components of the Continuous Simulation approach are further discussed in the following sections:

- Stochastic Rainfall Data Generation; and
- Applications to Design Food Estimation.

3.3.2. Stochastic Rainfall Data Generation

The effectiveness of the Continuous Simulation method depends upon the availability of a sufficiently long rainfall data set to provide adequate information on extreme storm (and drought) events. In reality however, pluviograph data rarely extends beyond 50 years, and the inference of floods greater than 2% AEP is difficult (Boughton et al., 1999).

In such cases stochastic rainfall generation has been used to provide a long time series of synthetic rainfall (Boughton et al., 1999; Cameron et al., 1999; Droop and Boughton, 2002; Haberlandt and Radtke, 2013). The synthetic data set thus generated is designed to be statistically indistinguishable from observed rainfall data (Kuczera et al., 2006).

There are well established methods to generate stochastic data at a coarse time scale. However, generating fine resolution synthetic data that can reproduce the statistics of the observed rainfall series at various temporal scales (annual, monthly, daily and hourly) is challenging (Srikanthan and McMahon, 2001; Boughton and Droop, 2003; Kuczera et al., 2006). Therefore, a commonly used approach is to generate the synthetic rainfall data at a daily time step first, and then disaggregate to a sub-daily time step by using functional relationships between daily and sub-daily rainfall statistics. Boughton (1999) used the Transition Probability Matrix (TPM) model to generate thousands of years of daily rainfall data and then disaggregated the daily data to an hourly time-step using the sub-daily rainfall statistics derived from IFD curves and temporal patterns. Kuczera et al. (2006) tested the ability of the DRIP rainfall generating model Heneker et al. (2001) to reproduce observed rainfall statistics at different levels of aggregation (hourly to yearly) and found that the model was able to reproduce the observed rainfall statistics satisfactorily for the large storms.

Techniques are available for generating daily rainfalls at any site in Australia (Book 2, Chapter 7) thus the inputs required for continuous simulation models can be developed for catchments without adequate at-site rainfall data.

3.3.3. Runoff- Routing Model

Types of runoff-routing models used to simulate the flow can be varied and depend upon the complexity required to provide unbiased simulation of the hydrologic process in the catchment. For example, Boughton (1999) and Droop and Boughton (2002) used a simple lumped Australian Water Balance Model (AWBM) to simulate a long series of precipitation excess, for small to mid-sized catchments, which were then routed using an hourly hydrograph generation model. Haberlandt and Radtke (2013) used HEC-HMS (Feldman, 2000), a semi distributed rainfall-runoff model, in three medium sized catchments in Germany. Cameron et al. (1999) applied a semi-distributed conceptual runoff-routing model known as TOPMODEL (Beven et al, 1987) for design flood estimation in small sized catchments in the UK. For large catchments with large spatial heterogeneity, England (2006) recommends using a physically based distributed model to fully characterise the spatial distribution of the processes occurring in the catchment. Other commonly used continuous simulation models include SIMHYD (Chiew and McMahon, 2002), Sacramento Model (Burnash et al., 1973) and GR4H (Mathevet, 2005).

The three factors that need to be considered when selecting a continuous simulation model for flood estimation are:

1. The ability of the model to represent the physical processes occurring in the catchment (model complexity);

2. Adequate temporal resolution to simulate the embedded flood hydrographs; and

3. The amount of data and computational resources available to properly describe and calibrate the model (model parsimony).

Useful guidance on the trade-offs involved in matching model complexity with data availability is provided in (Vaze et al., 2012).

3.3.4. Model Calibration

Implementation of Continuous Simulation, and the use of synthetic data, is complicated by the need to calibrate both the rainfall data generation model and the runoff-routing model using the observed data set. Effective calibration depends upon the calibration method applied, the length and the quality of data used for calibration. Gupta and Sorooshian (1985) report that the benefit of using additional data (with similar information content) diminishes with the reciprocal of the square root of the number of data points used in the calibration. Therefore, while the length of data is an important factor, the data series should also contain a sufficient number of ‘unusual events’ (or extreme events) to enable estimation of the parameter values (Singh and Bárdossy, 2012).

The rainfall generation model is generally calibrated to storm events, as in alternating renewal models like DRIP, or to aggregation statistics (such as mean, skewness, coefficient of variation, auto correlations etc.) at various time scales (Kuczera et al., 2006). The runoff-routing models are calibrated to observed flow data, flow statistics (Boughton et al., 1999) and in some cases the flood frequency curve (Cameron et al., 1999). The alternative calibration strategies will result in different model parameter values, leading to differing representation of hydrographs and peak events.

Lack of observed data is a major problem for calibration of the rainfall generation model or the runoff-routing model. In the case of the rainfall generation model, for example, the short rainfall data sets generally available are unlikely to include extreme rainfall events caused by various rain producing mechanisms (for example cyclones vs. thunderstorms) and to sample the full range of natural variability.

3.3.5. Applications to Design Flood Estimation

Boughton et al. (1999) developed a Continuous Simulation System (CSS) for estimation of design floods, and applied this to a number of catchments of mid to small sizes in Victoria. The CSS comprised of a stochastic rainfall generator, the AWBM water balance model and a hydrograph model. The stochastic rainfall generator was based on Transition Probability Matrix model to generate daily rainfalls, and these were then disaggregated to hourly data. A multi objective calibration strategy was used to calibrate the runoff-routing model against the monthly runoff volume and maximum values of daily flow. To reduce the computational time, the model was run at daily time step during the long relatively dry periods and hourly time step during the storm event. They estimated the design flood values to 0.05% AEP and showed that the derived frequency curve calculated by the method was able to properly match the observed flood frequency curve for more frequent floods (5% AEP).

Newton and Walton (2000) further applied the CSS in a large (13 000 km^2), semi-arid catchment in Western Australia. They compared the design estimates produced by the CSS to the observed flood frequency curve and found that the design flood estimates overestimated the observed flood frequency curve for more frequent floods. They speculated that the discrepancy between observed flood frequency curve and the CSS result might be due to the sampling problem; the observed flood frequency curve was estimated based on a shorter period (31 years) of data, while the rainfall generation model was calibrated to longer (93 years) data. The observed streamflow data covered a relatively dry period and did not represent the total climatic variability over a longer period.

There have been other applications of Continuous Simulation approaches for estimation of the derived flood frequency curve, for example Haberlandt and Radtke (2013), Cameron et al. (1999) and Droop and Boughton (2002), to catchments of various sizes and characteristics. In all cases stochastic rainfall generators were used to extend the rainfall data. Although different rainfall generation and process models were used, all report that the derived distribution curve produced by the method was able to provide a satisfactory match to the observed flood frequency curves for large floods. However, in all cases described, the ability of the model to properly reproduce extreme flood events has not been confirmed, due to lack of data for extreme events.

3.4. Hybrid Continuous Event-Based Simulation

There is a range of “hybrid” approaches that do not fit neatly into the foregoing categories. Typically, hybrid approaches use statistical information on rainfall storms in combination with continuous simulation and event-based models. With this approach, long-term recorded (or stochastically generated) climate sequences might be used in combination with a continuous simulation model to produce a time series of catchment soil moisture and streamflows (which also may include simulation of snowpack conditions). This information is used to specify antecedent conditions for an event-based model, which is then used in combination with statistical information on rainfall storms to generate extreme flood hydrographs. For example, the SEFM model (MGS Engineering Consultants, 2009) undertakes soil moisture accounting and snowpack modelling for an extended period prior to the onset of an event to establish antecedent conditions, then uses a flood event model in combination with probabilistic design rainfall intensities to simulate the flood hydrographs.

SCHADEX (Paquet et al., 2013) is also an example of a hybrid approach. SCHADEX is a semi-continuous runoff-routing model in which a continuous hydrological simulation model is used to generate the possible hydrological states of the catchment, and floods are simulated on an event basis. The method incorporates a statistical model to characterise the distribution of rainfalls, where the observed rainfall series is split into several homogeneous sub-samples based on a classification of regional weather characteristics. The MORDOR hydrological model is used to convert rainfalls into floods; this is a conceptual, lumped, reservoir model with daily areal rainfall and air temperature as the driving input data. The principal hydrological processes represented are evapotranspiration, direct and indirect runoff, groundwater, snow accumulation and melt, and routing. Selected daily rainfalls are replaced by a synthetic generator for extreme rainfall estimation (Garavaglia et al., 2010), and the resulting daily discharge volumes are converted to peak flows using an empirical function derived from observed hydrographs. The results are fitted to a frequency distribution and used to derive flood quantiles typically out to 1 in 1000 AEP.

3.5. Performance of Methods

Ling et al. (2015) tested the Monte Carlo and Ensemble Event approaches using ten natural test catchments located in different areas of Australia, and the Continuous Simulation approach was applied to five of these catchments. [It should be noted that Ling et al. (2015) used the term “design event” to denote the use of an event model with a sample of temporal patterns, which corresponds to the Ensemble Event approach as described in Book 4, Chapter 3, Section 2; they did not test the deterministic “Simple Event” method as described in Book 4, Chapter 3, Section 2]. The catchments were selected to cover a range of climatic conditions, catchment sizes and catchment characteristics. Monte Carlo and Ensemble Event models were developed for each of the ten catchments and calibrated using observed rainfall and flow data. Three continuous simulation models were considered, the Australian Water Balance Model (AWBM, (Boughton and Droop, 2003)), SIMHYD (Chiew and McMahon, 2002) and GR4H (Mathevet, 2005).

The results of the event-based modelling showed that in general an initial loss-continuing loss model run using both the Monte Carlo and Ensemble approaches performed well in reproducing the at-site flood frequency curve over the range of catchments tested, over a range from 50% to 1% AEP. The exception to this was that the Monte Carlo model did not perform well for one catchment (located in the south-west of Western Australia) where the flow response to rainfall events varied widely. SWMOD (Water and Rivers Commission, 2003) was used as an alternative loss model for this catchment, and it was found that use of this model improved the results significantly over the initial loss-continuing loss model.

Sih et al. (2008) also evaluated the performance of Monte Carlo and Ensemble Event approaches, and they included comparison with the traditional Simple Event method. They tested the three methods on seven catchments covering the temperate and tropical regions of Australia, and considered both long duration (24 hours and longer) and short duration (less than 6 hour) storms. The Simple Event method was found to generally underestimate the peak flows for events. On the basis of the seven catchments considered, the Simple Event method underestimated the Monte Carlo solution by around 10% to 15%, although in some cases the method underestimated peak flows by between 50% to 70%. Sih et al. (2008) found much closer agreement between the Ensemble Event and Monte Carlo approaches, where generally the Ensemble Event method was found to underestimate the Monte Carlo solution by around 5%.

The results of the method testing on continuous simulation models by Ling et al. (2015) found that while it was possible to calibrate the models to reproduce the overall flow regime of the catchments, the highest flow peaks were markedly underestimated and the simulated flood frequency curve calculated from simulated Annual Maximum Series provided a very poor fit to the observed flood frequency curve. Weighting the calibration to the largest events in the series reduced the ability of the model to reproduce the overall flow regime, and provided only slight improvements in the accuracy of the derived frequency curves. It was found that the models could be calibrated directly to selected quantiles of the observed flood frequency curve, but this resulted in a very poor representation of hydrograph behaviour and large biases in flood volume. This testing clearly illustrated the multi-criteria nature of the calibration problem (Gupta et al, 1998), and showed that it is difficult to obtain a very good fit to both the flood frequency curve and hydrograph behaviour. Furthermore, comparison of the calibrated parameters resulting from the different calibration approaches also showed large differences in values, indicating a trade-off between reproducing the hydrograph and the best representation of the flood frequency curve.

Ling et al. (2015) investigated the effect of record length on model performance. The results from the two test catchments tested by Ling et al. (2015) found that even when twenty years of data is available at a site, the model results can vary significantly based on the period of record used in analysis. This is particularly evident when one period is noticeably drier or wetter than the other. This highlights the need to investigate how representative the available flow data is in the context of any available long-term rainfall records. Both the Monte Carlo and Ensemble Event approaches gave similar results.

Ling et al. (2015) also investigated the efficacy of applying the methods to ungauged catchments. The results of the investigation by Ling et al. (2015) illustrated that even when data is available from a neighboring gauged catchment, care must be taken in transposing inputs and parameters from similar gauged catchments. When parameters were transferred between models from dissimilar catchments, the results of both the Monte Carlo and Ensemble Event approaches were very poor. From these tests it is concluded that only catchments with similar climatic conditions, catchment sizes and catchment characteristics should be considered for providing model parameters for ungauged catchments.

3.6. Advantages and Limitations

An overview of the advantages and limitations of the different approaches to flood estimation is provided in Book 1, Chapter 3, though it is worth emphasizing some points here that are specific to the methods discussed in the Event Based Approaches and Hybrid Continuous Event-Based Simulation Sections above.

The Simple Event method has been the most commonly used approach to date in Australia. It is simple to apply, and information on the required design inputs - design rainfalls, single temporal patterns of average variability, and median design losses - are readily available for most locations in Australia. The probability neutrality assumption is maintained by selecting single “representative” values of the inputs; however, without independent information there is no way of knowing whether this assumption has been satisfied. Thus, while simple and easy to apply, the method is lacking in robustness and defensibility.

The Ensemble Event method represents a modest increase in complexity. Rather than undertaking a single run for each combination of event AEP and duration, it is necessary to undertake ten or so simulations and average the outcome; if single hydrographs are required for design purposes then these can be obtained by simple scaling of a hydrograph obtained from a representative event. The method does involve a little more tedium for practitioners, though most modelling software can be configured for batch processing, and the additional computation burden is of no consequence. The method is most readily suited to the consideration of temporal patterns, where testing has shown that in natural catchments it yields similar estimates to those derived from more rigorous approaches. While the approach represents an appreciable improvement over Simple Event methods, the approach does suffer from the limitation that it is not well suited to considering the influence of additional stochastic factors that may have an influence on the derived flood estimates. In natural catchments this includes the estimation of floods which are heavily influenced by the joint occurrence of highly variable losses and temporal patterns, catchments in which natural lake (or snowpack) levels are subject to variable antecedent conditions, or catchments where it necessary to consider seasonal variation in individual inputs. In disturbed catchments the method is unable to consider the influence of variable initial reservoir levels on dam outflows, the likelihood of debris blocking culverts and bridge waterway areas, or the influence of controlled discharges from infrastructure works that may be subject to some variability.

In contrast, Monte Carlo methods are well suited to the consideration of multiple sources of variability from natural or anthropogenic sources. Once the simulation scheme has been established, it is easily expanded to consider additional factors of importance. For example, the same sampling scheme can be used to accommodate the variability associated with seasonality of storm occurrence or temporal patterns, drawdown in a reservoir, or blockage factors. The information required to characterise aleatory uncertainty (ie. hydrologic variability) is often available in the historic record: if there is sufficient information available to simulate a process with a deterministic model, then the necessary information required to characterise variability can be readily obtained (or generated). Importantly, it is a simple matter to expand a simulation scheme to allow for correlations between the stochastic factors modelled. Thus, if there is information available that suggests that the dominant season is dependent on event severity, or that the available airspace in a reservoir decreases with event severity, then this is easily accommodated by using a conditional sampling scheme. The limitation of the method is that specialist modelling skills are required to develop bespoke Monte Carlo schemes, and that additional effort is required to ensure that the distributions used to characterise variability are appropriate for the conditions being simulated. The method can be expanded to include consideration of epistemic uncertainty (e.g. uncertainty in the routing parameters or in the design estimate of rainfall depth), but the necessary information for such schemes can be difficult to obtain and justify.

If the catchment is subject to complex interactions between stochastic factors and/or antecedent conditions, then consideration should be given to use of the Continuous Simulation approach. This method is particularly suited to the analysis of volume-dependent problems which are influenced by the interaction between multiple factors. For example, the analysis of peak levels at multiple points in a catchment that is influenced by hydraulic controls or which contain a cascade of storages. The use of Continuous Simulation approaches in these cases obviates the need to explicitly consider the manner in which factors combine, and if a long enough sequence is considered then it implicitly accounts for the joint probabilities involved. This approach also lends itself to the analysis of systems which are influenced by long duration events or sequences of flood events. Its limitation, however, is that the models most commonly used for Continuous Simulation are not well suited to representing the flood response in a catchment, particularly for rarer events. It is difficult to calibrate (then validate) a continuous model in a manner that adequately captures the sequencing and variability of streamflows while reproducing the behaviour that determines peak and volume of flood events. For estimating rare events, it is also necessary to calibrate and apply a suitable stochastic climate generator.

Hybrid models have the potential to combine the benefits of both continuous and event approaches, though at this stage insufficient investigations have been undertaken to determine whether such schemes provide demonstrable benefits over other approaches.

3.7. Example - Delatite River

The Delatite River is located in central Victoria and has a catchment area of 368 km^2. The catchment headwaters are located between Mount Buller and Mount Stirling in the Great Dividing Range. The river flows generally westwards through forests which become less dense as the river descends and then flows into Lake Eildon. The river descends a total of 1230 m over its 85 km length. A map of the catchment and its drainage network is shown in Figure 4.3.3 which also shows the schematic of a conceptual runoff-routing model developed for the catchment. Streamflow data is available at the Tonga Bridge gauging site (Gauge No. 405214) from March 1957 to date.

The runoff-routing model was fitted to three historic flood events, and the results for the largest event (September 2010) are also shown in Figure 4.3.3. The initial loss parameters fitted to the three events were 25, 10, and 15 mm, and the corresponding continuing loss parameters were 2.5, 1.5, and 2.5 mm/hr.

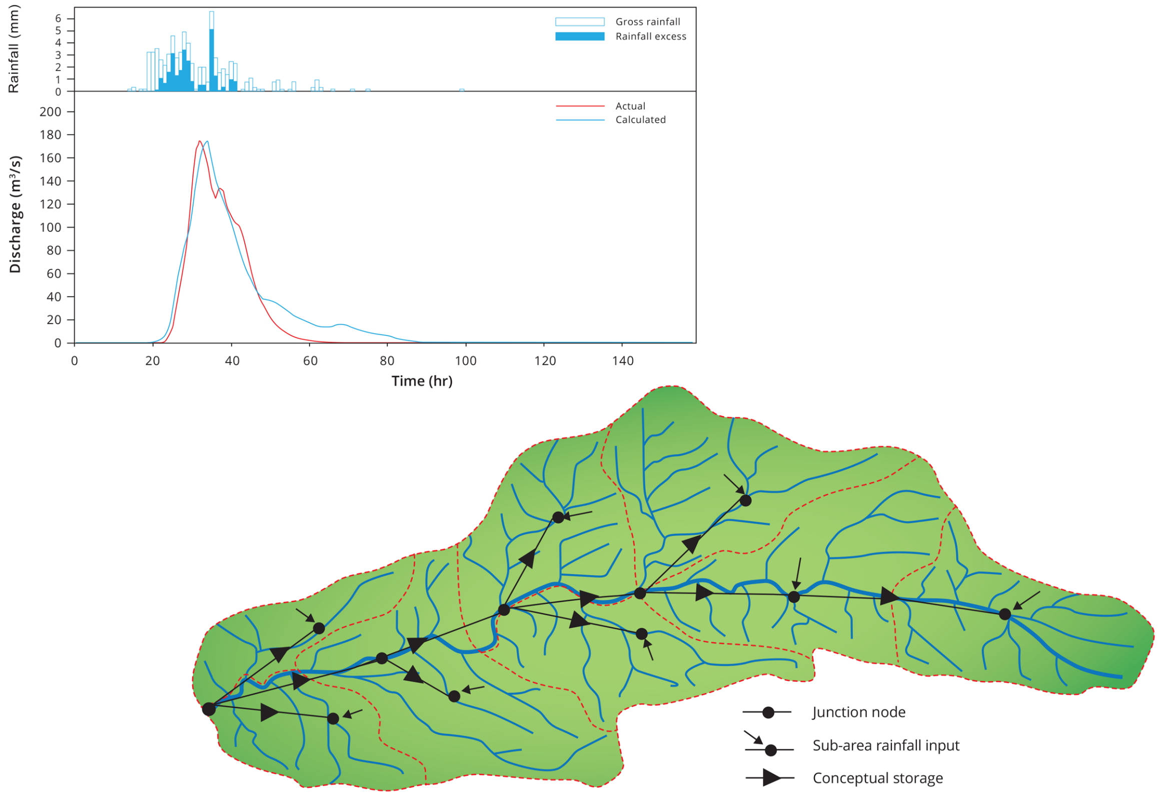


Figure 4.3.3. Schematic Layout of Delatite River catchment and Calibration to December 2010 Event

Three different approaches were used to derive design estimates using the calibrated runoff-routing model. The Simple Event approach used a single temporal pattern of average variability, along with a single set of loss parameters obtained from calibration to the three historic events. The Ensemble Event approach replaced the single temporal pattern with a sample of 19 patterns derived from rainfall events that have occurred in the inland region of south-east Australia, and used the same loss parameters as used in the Simple Event method. Monte Carlo results were obtained using the same set of temporal patterns as used in the Ensemble Event approach; the continuing loss parameter was held constant, and the initial loss was sampled from a non-dimensional distribution of initial losses (Hill et al., 2015) with a median loss value set equal to the value adopted for the Simple Event method. The results from these three approaches are shown in Figure 4.3.4 where it is seen that the Monte Carlo approach yields estimates that are very similar to the quantiles obtained from Flood Frequency Analysis. The Ensemble Event estimates are similar to but lower than those obtained using Monte Carlo analysis, and the Simple Event estimates are substantially higher. It is worth noting that all design flood estimates rarer than about 5% AEP lie within the confidence limits associated with the Flood Frequency Analysis.

Also shown in Figure 4.3.4 are the results obtained from Continuous Simulation. A number of conceptual models were trialled and the Sacramento model (Burnash et al., 1973) was found to provide the best results. Rainfall inputs to the model were obtained using gridded rainfall data (Jones et al., 2009) and mean monthly areal potential evapotranspiration inputs were obtained from the Bureau of Meteorology (Chiew et al., 2002). The model was initially calibrated to daily streamflows using 20 years of historic data, and then adjusted to reproduce the instantaneous peak flows over the same period. The model was used to derive 101 years of simulated streamflows using the gridded rainfall data, and a Generalised

Extreme Value distribution was then fitted to the annual maxima extracted from the time series. The results are shown in Figure 4.3.4, where it is seen that the design estimates are substantially lower than the results obtained from the event-based approaches. The derived flood frequency curve generally lies along the lower confidence limits of the frequency curve fitted using gauged maxima.

While no general conclusions should be drawn from this example about the relative efficacy of the different methods used, the results do illustrate the range of estimates obtained for a well gauged catchment. They indicate the degree of ‘model uncertainty’ that generally remains unknown when only a single simulation method is employed. The largest event used to fit the runoff-routing model occurred in December 2010 and has a peak similar in magnitude to the 2% AEP event determined from Flood Frequency Analysis. The period of record used to calibrate the Sacramento model spanned a representative range of climatic conditions. The data used in this example is more than is typically available, and nevertheless the design estimates vary by about a factor of two.

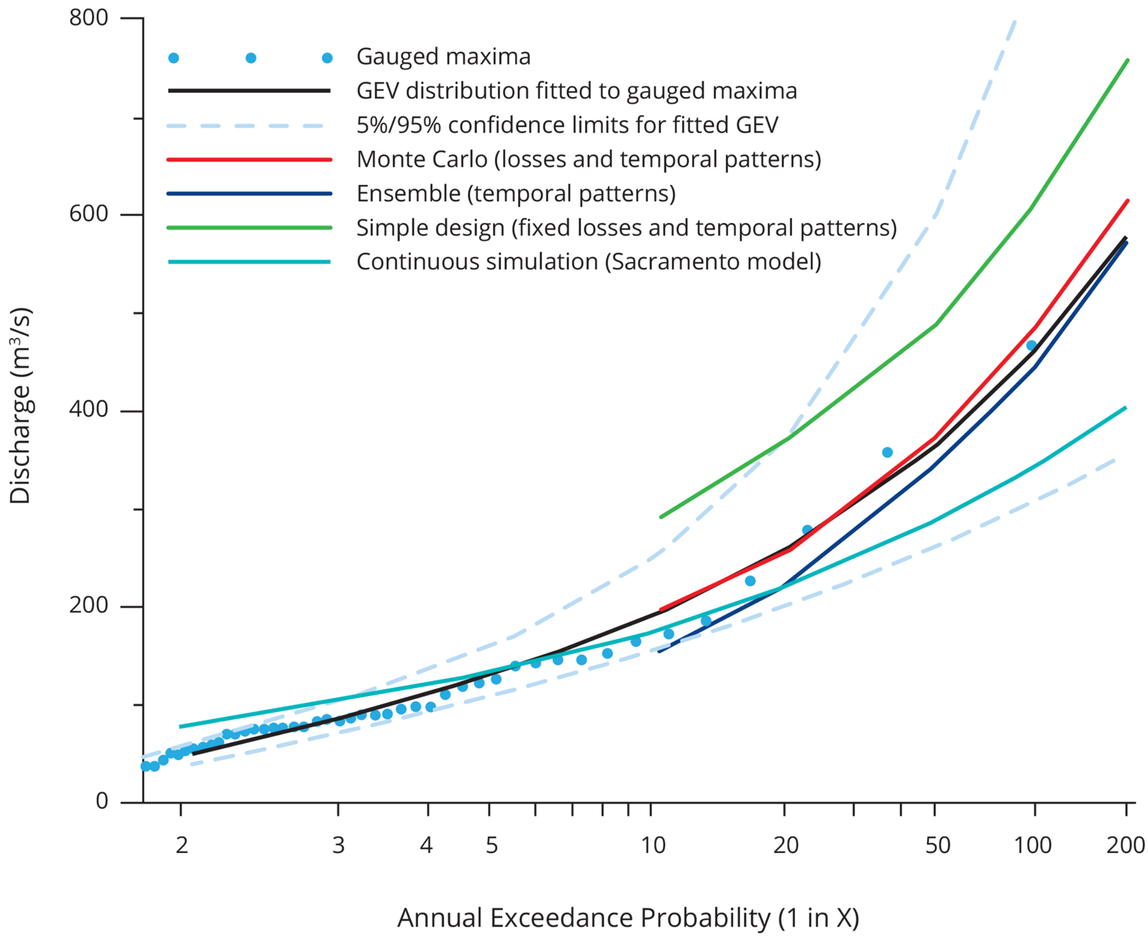


Figure 4.3.4. Comparison of Design Flood Estimates with Flood Frequency Curve for the Delatite River at Tonga Bridge

3.8. References

Abbott, M.B., Bathurst, J.C., Cunge, J.A., O'Connell P.E. and Rasmussen, J. (1986), An introduction to the European hydrological system- Systeme Hydrologique Europeen, SHE, 1: history and philosophy of a physically-based, distributed modelling system. Journal of Hydrology, volume 87(1-2), pp.45-59.

Beven K. J., Calver A. and Morris E. M. (1987). The Institute of Hydrology Distributed Model. Technical Report No. 98 Institute of Hydrology, Wallingford, UK.

Boughton, W.C. (1999), A daily rainfall generating model for water yield and flood studiesRep. Report 99/9, 21pp pp, CRC for Catchment Hydrology, Monash University, Melbourne.

Boughton, W. and Droop, O. (2003), Continuous simulation for design flood estimation-a review. Environmental Modelling & Software, 18(4), 309-318. Available at: http://linkinghub.elsevier.com/retrieve/pii/S1364815203000045 [Accessed October 3, 2013].

Boughton, W.C., Muncaster, S.H., Srikanthan, R., Weinmann, P.E., Mein, R.G. (1999) Continuous Simulation for Design Flood Estimation - a Workable Approach. Water 99: Joint Congress; 25th Hydrology & Water Resources Symposium, 2nd International Conference on Water Resources & Environment Research; Handbook and Proceedings; pp: 178-183.

Burnash, R.J.C., Ferral, R.L. and McGuire, R.A. (1973), A Generalised Streamflow Simulation System - Conceptual Modeling for Digital Computers. U.S. Department of Commerce, National Weather Service and State of California, Department of Water Resources.

Cameron, D.S, Beven, K.J, Tawn, J, Blazkova, S. and Naden, P. (1999), Flood frequency estimation by Continuous Simulation for a gauged upland catchment (with uncertainty). Journal of Hydrology, 219(3-4), 169-187. Available at: http://linkinghub.elsevier.com/retrieve/pii/S0022169499000578.

Chiew, F.H.S. and McMahon, T.A. (2002), Modelling the impacts of climate change on Australian streamflow. Hydrol. Processes, 16: 1235-1245.

Chiew, F.H.S., Wang, Q.J., McConachy, F., James, R., Wright, Q. and deHoedt, G. (2002), Evapotranspiration maps for Australia. Hydrology and Water Resources Symposium, Melbourne, May 20-23 2002.

Droop, O. and Boughton, W. (2002), Integration of WBNM into a continuous simulation system for design flood estimation. Research Report 2/02, Gilbert & Associates, Brisbane.

Feldman, A.D. (2000), Hydrologic Modeling System HEC-HMS. Technical Reference Manual. US Army Corps of Engineers (USACE), Hydrologic Engineering Center, HEC. Davis, CA, USA.

Garavaglia, F., Gailhard, J., Paquet, E., Lang, M., Garcon, R. and Bernardara, P. (2010), Introducing a rainfall compound distribution model based on weather patterns sub-sampling Hydrol. Earth Syst. Sci., 14: 951-964.

Green, J.H., Walland, D.J., Nandakumar, N. and Nathan, R.J. (2003), The Impact of Temporal Patterns on the Derivation of PMPDF and PMF Estimates in the GTSM Region of Australia. Presented at Hydrology and Water Resources Symposium, Wollongong, NSW, November 2003.

Gupta, V.K. and Sorooshian, S. (1985), The relationship between data and the precision of parameter estimates of hydrologic models. Journal of Hydrology, 81: 57-77.

Gupta, H.V., Sorooshian, S. and Yapo, P.O. (1998), Toward improved calibration of hydrological models: multiple and noncommensurable measures of information. Water Resour. Res., 34(4), 751–763.

Haberlandt, U. and Radtke, I. (2013), Derived flood frequency analysis using different model calibration strategies based on various types of rainfall-runoff data - a comparison. Hydrology and Earth System Sciences Discussions, 10(8), 10379-10417. Available at: http://www.hydrol-earth-syst-sci-discuss.net/10/10379/2013/ [Accessed October 13, 2013].

Heneker, T.M., Lambert, M.F. and Kuczera, G. (2001), A point rainfall model for risk-based design, J Hydrology, 247(1-2), 54-71.

Hill, P.I., Graszkiewicz, Z., Sih, K., Nathan, R.J., Loveridge, M. and Rahman, A. (2012), Outcomes from a pilot study on modelling losses for design flood estimation. Hydrology and Water Resources Symposium 2012, Sydney.

Hill, P.I., Graszkiewicz, Z., Loveridge, M., Nathan, R.J. and Scorah, M. (2015), Analysis of loss values for Australian rural catchments to underpin ARR guidance. Hydrology and Water Resources Symposium 2015. Hobart. 9-10 December 2015.

Hoang, T.M.T., Rahman, A., Weinmann, P.E., Laurenson, E.M. and Nathan, R.J. (1999), Joint Probability Description of Design Rainfalls. 25th Hydrology and Water Resources Symposium, Brisbane, 2009, Brisbane.

Jones, D. A., Wang, W. and Fawcett, R. (2009), High-quality spatial climate data-sets for Australia. Australian Meteorological and Oceanographic Journal, 5: 233-248.

Kuczera, G., Lambert, M., Heneker, T., Jennings, S., Frost, A. and Coombes, P. (2006), Joint probability and design storms at the crossroads. In 28th International Hydrology and Water Resources Symposium, Woolongong, pp: 10-14.

Ling, F., Pokhrel, P., Cohen, W., Peterson, J., Blundy, S. and Robinson, K. (2015), Australian Rainfall and Runoff Project 12 - Selection of Approach and Project 8 - Use of Continuous Simulation for Design Flow Determination, Stage 3 Report.

Linsley, R.K., Crawford, N.H. (1960), Computation of a synthetic streamflow record on a digital computer. In Int. Assoc. Sci. Hydrol., pp: 526-538.

MGS Engineering Consultants (2009), General Stochastic Event Flood Model (SEFM), Technical Support Model. Manual prepared for the United States Department of Interior, Bureau of Reclamation Flood Hydrology Group.

Mathevet, T. (2005), Which rainfall-runoff model at the hourly time-step? Empirical development and intercomparison of rainfall-runoff models on a large sample of watersheds. ENGREF, Paris, FRANCE.

Nathan, R.J. (1992), The derivation of design temporal patterns for use with generalized estimates of Probable Maximum Precipitation. Civil Engng. Trans., I.E. Aust., CE34(2), 139-150.

Nathan, R.J. and Weinmann, P.E. (2004), An improved framework for the characterisation of extreme floods and for the assessment of dam safety. Hydrology: Science & Practice for the 21st Century, Vol 1. Proc. British Hydrol. Soc., London, pp: 186-193.

Nathan, R.J., Weinmann, P.E. and Hill, P.I. (2002), Use Of A Monte Carlo Framework To Characterise Hydrological Risk, ANCOLD Bulletin, (122), 55-64.

Nathan, R., Weinmann, E., Hill, P. (2003), Use of Monte Carlo Simulation to Estimate the Expected Probability of Large to Extreme Floods, proc. 28th Int. Hydrology and Water Res. Symp., Wollongong, pp: 1.105-1.112.

Newton, D. and Walton, R. (2000), Continuous Simulation for Design Flood Estimation in the Moore River Catchment, Western Australia. In Hydro 2000 Hydrology and Water Resources Symposium. Canberra, ACT, pp: 475-480.

Paquet, E., Garavaglia, F., Gailhard, J. and Garçon, R. (2013), The SCHADEX method: a semi-continuous rainfall-runoff simulation for extreme flood estimation, J Hydrol., 495: 23-37.

Pilgrim, DH (ed) (1987) Australian Rainfall and Runoff - A Guide to Flood Estimation, Institution of Engineers, Australia, Barton, ACT, 1987.

Pilgrim, D.H. and Cordery, I. (1975), Rainfall temporal patterns for flood designs, Proc. Amer. Soc. Civ. Engrs., J Hydrauclis Div., 100(1), 81-95.

Pilgrim, D.H., Cordery, I. and French, R. (1969), Temporal patterns of design rainfall for Sydney, Institution of Engineers, Australia, Civil Eng. Trans., CE11: 9-14.

Rahman, A., Hoang, T M.T., Weinmann, P.E. and Laurenson, E.M. (1998), Joint probability approaches to design flood estimation: a review. CRC for Catchment Hydrology, Report 199808, Melbourne, Australia.

Rahman, A., Weinmann, P.E. and Mein, R.G. (2002a), The use of probability-distributed initial losses in design flood estimation. Australian Journal of Water Resources. 6(1), 17-29.

Rahman, A., Weinmann, P., Hoang, T.M. and Laurenson, E. (2002b), Monte Carlo simulation of flood frequency curves from rainfall. Journal of Hydrology, 256(3-4), 196-210. http://doi.org/10.1016/S0022-1694(01)00533-9.

Schaefer, M.G. and Barker, B.L. (2002), Stochastic Event Flood Model (SEFM), Chapter 20, Mathematical Models of Small Watershed Hydrology and Applications, Water Resources Publications, LLC, VJ Singh editor.

Schaefer, M.G. and Barker, B.L. (2004), Stochastic Event Flood Model (SEFM) - Users Manual, MGS Engineering Consultants Inc, prepared for US Bureau of Reclamation Flood Hydrology Group.

Sih, K., Hill, P.I., Nathan, R.J., (2008), Evaluation of simple approaches to incorporating variability in design temporal patterns. Water Down Under 2008 (incorporating 31st Engineers Australia Hydrology and Water Resources Symposium), pp: 1049-1059.

Singh, S.K. and Bárdossy, A. (2012), Calibration of hydrological models on hydrologically unusual events. Advances in Water Resources, 38: 81-91.

Srikanthan, R. and McMahon, T.A. (2001), Stochastic generation of annual, monthly and daily climate data: A review. Hydrology and Earth System Sciences, 5(4), 653-670, Available at: http://www.hydrol-earth-syst-sci.net/5/653/2001/.

Vaze, J., Jordan, P., Beecham, R., Frost, A., Summerell, G. (2012), Guidelines for rainfall-runoff modelling Towards best practice model application.

Water and Rivers Commission (2003), SWMOD Version 2 A Rainfall Loss Model for Calculating Rainfall Excess, User Manual (Version 2.11).

Weinmann, P.E., Rahman, A., Hoang, T.M., Laurenson, E.M. and Nathan, R.J. (2000), Monte Carlo Simulation of flood frequency curves. Proc., Hydro 2000, 3rd International Hydrology and Water Resources Symposium, IE Aust, pp: 564-569.

Weinmann, P.E., Rahman, A., Hoang, T.M.T., Laurenson, E.M. and Nathan, R.J. (2002), Monte Carlo Simulation of Flood Frequency Curves from Rainfall - The Way Ahead. Aust. Journal of Water Resources, 6(1), 71-79.

Chapter 4. Treatment of Joint Probability

Rory Nathan, Erwin Weinmann

Chapter Status	Final
Date last updated	14/5/2019

4.1. Introduction

In many applications of flood simulation it is necessary to understand and apply the basic probability concepts involved when a range of factors combine to produce a flood event or when different events occur jointly. Such applications range from the stochastic simulation of design flood events allowing for the joint probabilities of several key flood producing or flood modifying factors, to typical situations where flood risk results from various combinations of flood events that have different causes or occur at different locations.

Book 4, Chapter 4, Section 2 introduces basic probability concepts that are applied in flood simulation methods and in determining flood risks for situations where several factors or events interact. It then describes typical practical applications where the interaction of different factors or events need to be considered and points to other sections where individual applications are treated in more detail. Book 4, Chapter 4, Section 3 is devoted to introducing Monte Carlo simulation as the most practical and flexible method of deriving distributions that result from the interaction of several stochastic components. Book 4, Chapter 4, Section 4 illustrates the application of joint probability concepts to a typical flood estimation problem

4.2. Probability Concepts

4.2.1. Variability and Uncertainty

When considering the variabilities of different factors involved producing flood risk and in the assessment of joint probabilities, it is worth differentiating between the *temporal and spatial variability* of the climate and hydrologic factors being modelled (aleatory uncertainty), and the random variation resulting from unavoidable *uncertainty* in the model inputs, structure, and parameters (epistemic uncertainty). Similar solution methods can be used to consider both these sources of uncertainty and thus there is sometimes some confusion about what aspects are being considered. However, the nature of the information available for these two broad sources of uncertainty – and hence the defensibility of the analyses undertaken – is markedly different.

Aleatory uncertainty represents the *natural variability* inherent in most hydrologic systems. In the context of design flood estimation, this usually involves consideration of natural variability in the characteristics of storm rainfalls (depths, temporal and spatial patterns), antecedent conditions (as they relate to initial losses, water levels in natural lake systems and snowpack characteristics), coincident streamflows (or levels) at the confluence of two streams, and the influence of tide levels on estuarine flood behaviour. Aleatory uncertainty associated with anthropogenic causes is also commonly a factor that needs to be considered. Perhaps the most common factor to be considered in design flood estimation is initial reservoir levels in dams (either singly or in cascade), though this can include consideration of the reliability of operating equipment (e.g. spillway gates and other forms of outlet works), and debris blockage of waterway areas provided for spillways, drainage works and bridges. Factors which vary randomly over time are termed stochastic variables.

Epistemic uncertainty, on the other hand, relates to the uncertainty arising from *a lack of knowledge* about hydrologic factors and their governing processes. In the context of design flood estimation, epistemic uncertainty is commonly associated with errors involved in rating curves (ie. in the relationship used to estimate streamflows from gauged levels), in the estimation of catchment rainfalls from point observations, and the uncertainties involved in estimating model parameters from a limited number of relevant events. An important source of epistemic uncertainty arises from the need for extrapolation. That is, there may be an adequate amount of information available at a particular site for estimating the exceedance probability of frequent floods, but additional uncertainty is introduced when transposing such information to an ungauged location, or when extrapolating to events much larger than have occurred in the historic record. As the degree of extrapolation increases, so does the uncertainty in the appropriateness of the configuration, or indeed of the conceptual structure, of the model being used. Such uncertainties arise from lack of knowledge, and as such can be reduced over time with collection of relevant data and increases in our understanding.

This Chapter only considers the influence of aleatory uncertainty on joint probability, and consideration of epistemic uncertainty is discussed in Book 1, Chapter 2 and Book 7, Chapter 9. The focus of this chapter is on the use of techniques that minimise the introduction of bias in the exceedance probability of the final design estimate. Such estimates will always contain uncertainty due to lack of knowledge, but the methods presented here are intended to make best use of the information on natural variability that we do have.

4.2.2. Joint and Conditional Probabilities

The range of situations or applications when combinations of different factors or events need to be considered can be grouped on the basis of the different probability concepts being applied.

4.2.2.1. Joint Occurrence of Different Factors or Events

In flood hydrology there are many situations where a number of factors need to be considered jointly when determining the probability of a flood outcome, in other words when “Event A” *AND* “Event B” determine the flood outcome. This includes the joint influence of a number of factors in determining the magnitude of a design flood event, e.g. the average depth and spatial/temporal distribution of rainfall inputs, the magnitude and temporal distribution of losses and the influence of flood modifying factors, such as the initial conditions of natural and artificial storages in the catchment. The flood simulation process then needs to allow for the joint probability of the different factors, which may be correlated or independent of each other.

The interaction of these different factors can be described by a *joint probability distribution* (Benjamin and Cornell, 1970; Haan, 1974). A *bivariate probability distribution* describes the joint probability of two variates *x* and *y*, and this case is the simplest to visualise (refer to Figure 4.4.1). Each of the two variables has a marginal probability distribution, $p(x)$ and $p(y)$, which represents the probability distribution without considering the influence of the other variable. At a particular value of one variable, say at x_0, the distribution of the other variable *y* can be said to be conditioned on *x* and this is referred to as the *conditional probability distribution* of *y*:

$$p(y|x = x_0) \tag{4.4.1}$$

The marginal distributions are illustrated in Figure 4.4.1 for the probability densities of a bivariate normal distribution in *x* and *y* (with means of 70 and 50, respectively), where the conditional probability distribution is shown for *x* = 90.

It is clear from the figure that the marginal probability distribution of *y* can be obtained by integrating the conditional probability distributions of *y* for all values of *x*. For independent events, the distribution of one variable is not conditioned on the other, and all conditional distributions are thus identical to the marginal distributions of that variable.

The concepts of marginal and conditional probability distributions can be extended to *multivariate probability distributions* where several variables are involved.

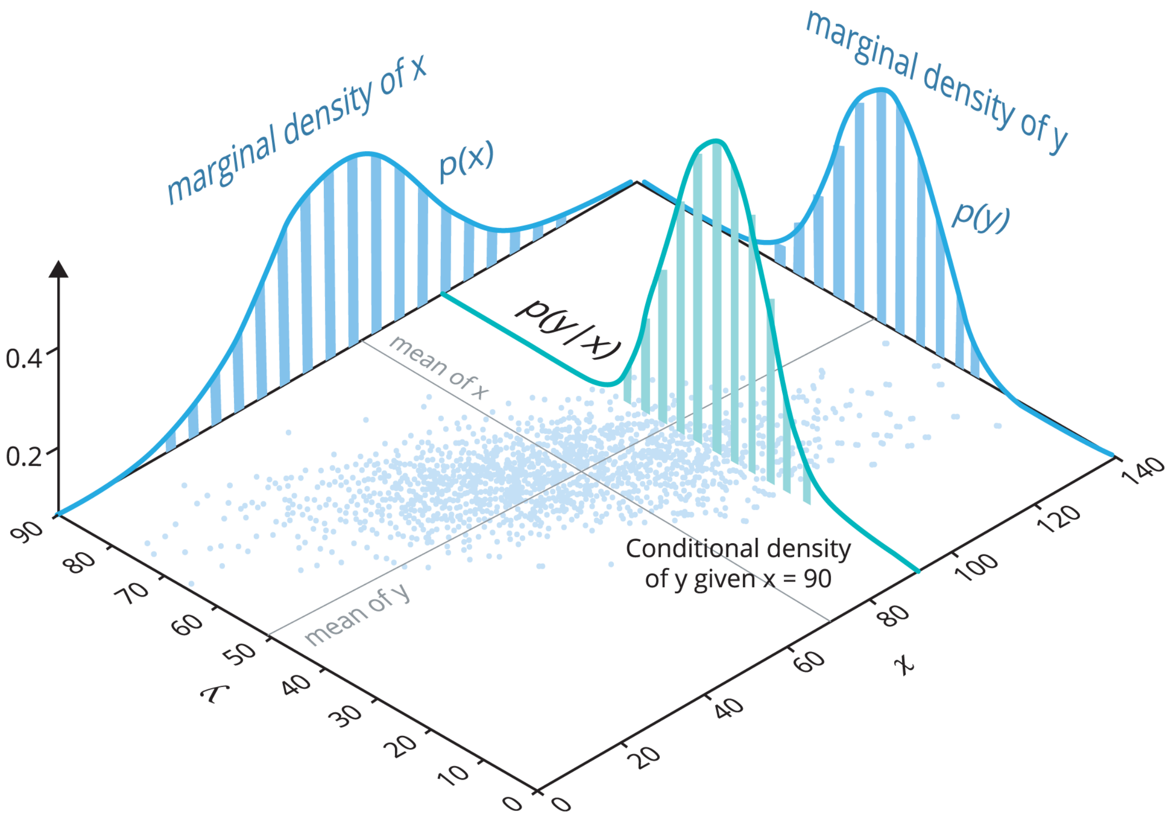


Figure 4.4.1. Joint Probability Density for a Bivariate Normal Distribution

The joint probability distribution concepts can also be applied to deal with the joint occurrence of events that are simulated separately. Examples of such applications include the interactions of riverine (or overland) flooding and sea level anomalies (Book 6, Chapter 5), the joint probability of reservoir inflows and initial storage contents, and the joint consideration of mainstream and tributary floods.

The general solution approach to joint probability problems and the selection of factors or events to be included in the joint probability framework are discussed in Sections Book 4, Chapter 4, Section 2 and Book 4, Chapter 4, Section 2 respectively.

Analytical approaches are available to deal with relatively simple joint probability applications. A special case is where component probability distributions can be considered to be independent of each other. In this case the joint probability can be evaluated simply by multiplying the component probabilities from the marginal distributions. However, in practice most joint probability applications are more complex and are most readily addressed by Monte Carlo simulation. In this approach the joint probability distribution is derived by randomly sampling from the (marginal) component distributions and simulating the system response a sufficient number of times to define the output distribution over the range of interest. The method can readily deal with several component distributions and correlations between them. This is the practical joint probability approach dealt with separately in Book 4, Chapter 4, Section 3 *Monte Carlo Simulation*.

Typical examples of practical problems are discussed in Book 4, Chapter 4, Section 2 *Typical Joint Probability Applications*, and this includes references to solutions that do not require practitioners to develop their own solution framework.

4.2.2.2. Combination of Conditional Occurrences

There are flood estimation applications where it is most practical or efficient to partition the total range of a key variable into a number of segments or intervals.

A typical example is to divide the range of rainfall input magnitudes into a number of intervals and then calculating the probability of a particular flood outcome conditional on this range of rainfall inputs. Key variables for other flood estimation applications may also be partitioned in a similar way.

The marginal exceedance probability of the flood outcome of interest *X* can then be calculated by the application of the Total Probability Theorem (Haan, 1974):

$$P(X > x) = \sum_i P\left[X > x \middle| C_i\right] p\left[C_i\right] \tag{4.4.2}$$

where the term $P\left[X > x \middle| C_i\right]$ denotes the conditional probability that the flood outcome *X* generated from this interval C_i exceeds *x* and the term $p\left[C_i\right]$ represents the probability that the conditioning variable falls within the interval *i*. For Equation (4.4.2) to be applicable, the set of conditioning events C_i needs to be mutually exclusive (meaning no overlap) and collectively exhaustive (meaning that the probabilities of the conditioning events have to add up to 1.0).

Typical applications of conditional probability concepts and the Total Probability Theorem are further discussed in Book 4, Chapter 4, Section 2.

4.2.2.3. Combination of Separate Independent Events

A specific flood outcome, such as flooding above the floor level of a building or flooding above a certain threshold level where access to a property is cut, may occur as a consequence of different events whose occurrences may be considered to be independent of each other. An example of such separate events is flooding as a result of high river levels (Event A) and flooding caused by overflows from a local drainage system (Event B). If the river flooding typically occurs from an extensive storm system over a large catchment and the drainage flooding from thunderstorms over a small local catchment, then these events can be considered to be essentially independent.

The combined exceedance probability of this specific flood outcome from either Event A *OR* Event B can then be calculated as:

$$P(A+B) = P(A) + P(B) - P(A)P(B) \tag{4.4.3}$$

where P(A)P(B) represents the exceedance probability of Events A and B occurring together. For events of relatively small AEPs this product is quite small and can generally be neglected. The combined exceedance probability of several events can be evaluated in an analogous fashion. An example involving several events is when flood frequency curves for different seasons are combined to determine the annual frequency curve.

It is important to note that for Equation (4.4.3) to be applicable, the different events being considered have to be defined in terms of the same magnitude (not exceedance probability).

In the example situations discussed above the interest is on the combined probability of occurrence of separate events. When the reliability of a linear structure such as a road or railway is being considered, the interest is not on the combined probability of exceedance of a given flood standard at different locations but on the combined *non-occurrence probability* or *survival probability*. Under the assumption of independent occurrences of damaging events at different locations, the overall reliability of the linear structure can be calculated as the product of the non-exceedance probabilities of a damaging event at different locations. The combined risk of failure of the structure can then be determined as the complement of the overall reliability.

Book 4, Chapter 4, Section 2 provides further discussion of this particular form of probability calculations.

4.2.3. Typical Joint Probability Applications

Floods by their nature are the result of the joint occurrence of different flood producing influences, and thus most practical problems require consideration of the joint probabilities involved. This section describes some typical examples of such problems, and provides references to some general and specific procedures for their solution.

It is commonly required to estimate flood risk downstream of a storage, where the outflow peak is dependent on the initial water level. If the variation in initial water level is small, such as in a retarding basin or small on-line storage, then it may appropriate to adopt a typical starting storage from the central range of conditions. However, the relationship between inflow and outflow can be highly non-linear, thus in general it cannot be expected that adoption of a mean initial water level will provide an unbiased estimate of outflows. A maximum water level could be used and justified on the basis that it provides a conservatively high estimate of flood risk, but introducing conservatism in intermediate steps of the analysis should generally be avoided as the compounding effects of such assumptions can undermine the validity of any risk-based decisions. If the initial water level does have an appreciable impact on the outflow flood, ie. when the available flood storage is large compared to the flood volume, then it will be necessary to give explicit consideration to the joint probabilities involved. Detailed guidance on this type of problem is provided in Book 8, Chapter 7, and worked examples using both analytical and numerical schemes are provided in Book 8, Chapter 8, Section 4. The general computational elements involved in the Monte Carlo solution to this type of problem are discussed in Book 4, Chapter 4, Section 3; particular attention is drawn to the need for conditional sampling (Book 4, Chapter 4, Section 3) as it possible that the storage level associated with a given exceedance probability tends toward a maximum value as the event magnitude increases.

Flood levels in estuarine regions may be dependent on the combined influence of storm surge and tide levels. The degree of influence depends on a number of factors, but the lower limits of such flood estimates are determined by assuming that fluvial flood levels are wholly independent of the ocean level; conversely, the upper limits of such flood estimates are derived using the assumption of complete dependence, that is, that fluvial floods will always coincide with ocean levels of the same exceedance probability. Book 6, Chapter 5 provides a practical approach to the solution of this class of problem. This guidance assists the practitioner determine whether consideration needs to be given to the dependence of flood levels on ocean conditions, and if so, then site-specific estimates for any location on the Australian coastline can be determined using a software tool (http://p18.arr-software.org/) based on the bivariate extreme value distribution. A Monte Carlo solution could be developed by generating correlated variates in combination with a stratified sampling scheme using the procedures described in Book 4, Chapter 4, Section 3 and the dependence parameters described in Book 6, Chapter 5. In concept, the spreadsheet based worked example presented in Book 4, Chapter 4, Section 4 is directly applicable to this type of problem, the only difference being that the correlation term relating to tributary flows replaces the dependence term governing coincident ocean levels. Regardless of the approach used, any solution of this type of problem will require the undertaking of deterministic modelling to obtain flood levels for different combinations of riverine flood and storm tides.

Another common problem arises when considering the influence of tributary flows at a confluence relevant to the region of interest. There are a number of solutions to this class of problem, and the degree of complexity required will dependent greatly on the sensitivity of the outcome to selected simplifying assumptions. If the focus is on mainstream flows, then it may be sufficient to estimate the tributary contribution by estimating the average flood inflow coincident with mainstream flow conditions; Book 8, Chapter 8, Section 5 presents a simple worked example for this based on the use of a bivariate log-Normal distribution. Conversely, if the focus is on tributary flows, then the assumption that there is an average flood in the mainstream that is coincident with local flooding is likely to yield a biased outcome. This is because any variation in mainstream floods may have a large influence on local flood levels, at least for the region susceptible to backwater influences. The worked example presented in Book 4, Chapter 4, Section 4 is directly applicable to this type of problem, the only difference in application is that levels computed using hydraulic modelling (final column of Table 4.4.2) relate to upstream levels in the tributary, rather than downstream of the confluence. It should be noted that the inputs to this worked example may be derived by either Flood Frequency Analysis or rainfall-based modelling. It would be expected that the deterministic relationship between mainstream flows and flood level is most easily obtained from some form of hydraulic modelling, but if gauged information is available for a range of historic events, then a suitable deterministic function may be obtained directly through analysis of the data, thus obviating the need for hydraulic modelling. An example of such an analysis is provided by Laurenson (1974).

The general form of solutions to the above problems all conform to the conceptual framework described in Book 4, Chapter 4, Section 3. The sub-sections following this framework provide for parametric and non-parametric approaches to characterising the input distributions, and allow for the additional consideration (if required) for dealing with conditional dependencies. The generic procedures covered here are intended to cover situations not specifically catered for in the methods presented elsewhere in ARR, as discussed above.

4.2.4. Typical Conditional Probability Applications

It is sometimes appropriate to estimate the probability of occurrence of a flood event subject to a restrictive range of conditions, such as the time of year or a specific range of rainfalls. If so, then additional steps are required to estimate the probability of exceedance for the complete range of conditions that might apply. It is common in hydrology to consider both conditional and unconditional probabilities, and care is required when interpreting and reporting such analyses to avoid confusion.

For example, conditional probability estimates are often required for the estimation of flood risk for construction activities. Flood risk varies seasonally throughout the year, and construction works may be scheduled to occur in a season of low flood risk. In this case it is appropriate to estimate conditional flood probabilities relevant to the particular season of interest; such analyses might involve undertaking Flood Frequency Analysis using flood maxima that have occurred over the months scheduled for construction, or else a rainfall-based approach might be used in which seasonal design rainfalls are used in combination with season-specific losses. The flood risk estimates derived from such analyses are *conditional* upon the season considered, and without additional analyses it is not possible to convert these estimates to annual risks.

The nature of the additional analyses required to derive unconditional estimates of annual risk depends on whether the conditioning events are *mutually exclusive* or not. Estimating annual flood risks based on seasonal analyses represents a mutually exclusive set of estimates, as clearly the annual maximum event cannot occur in two different seasons in the one year. Being mutually exclusive, the annual risk that a flood exceeds a given value is obtained by the simple addition of the individual seasonal exceedance probabilities.

It is often the case, however, that the conditioning events are not mutually exclusive. A common example of this is the estimation of flood immunity along a length of linear infrastructure, such as a major road or railway line. Here, the annual maximum event may well occur at multiple locations along its length, and thus the annual risk that access between two locations might be disrupted cannot be obtained by simply summing the estimates made at each individual crossing. Instead, some account must be given to the dependence of the factors that give rise to the individual floods. The probability of closure for an existing length of infrastructure is not simply equal to the exceedance probability of the most vulnerable crossing as this ignores the contribution of flood exceedance probabilities from rainfalls that may occur from other independent weather systems. Whether or not the degree of dependence needs to be considered depends on the significance of the outcome when the initiating events are considered to be wholly dependent or independent. The greater the difference between these two extremes, the greater is the need to complicate the solution by the explicit consideration of the dependencies involved.

Practitioners need to decide the appropriate level of complexity required to come up with a practical solution in a manner that is proportionate to the nature of the problem and the available resources. The simplest approach is to assume that the factors of most importance are highly correlated and that alternative combinations of conditions contribute little to the overall flood risk. With reference to Figure 4.4.2, it is seen that in temperate climates it might be expected that large long duration rainfall events occur at times when soil moisture is high and consequently catchment losses are low; conversely short duration (thunderstorm) events might occur when losses are high. As long as due care is given to matching the design inputs to match the dominant mechanism of interest, then it may be appropriate to derive estimates of rainfall-based flood estimates on an annual basis. Conversely, if the design loading of interest is sensitive to a mix of storm durations and catchment conditions, then it may be warranted to derive rainfall-based estimates on a seasonal basis and compute annual risks by summation of the seasonal exceedance probabilities.

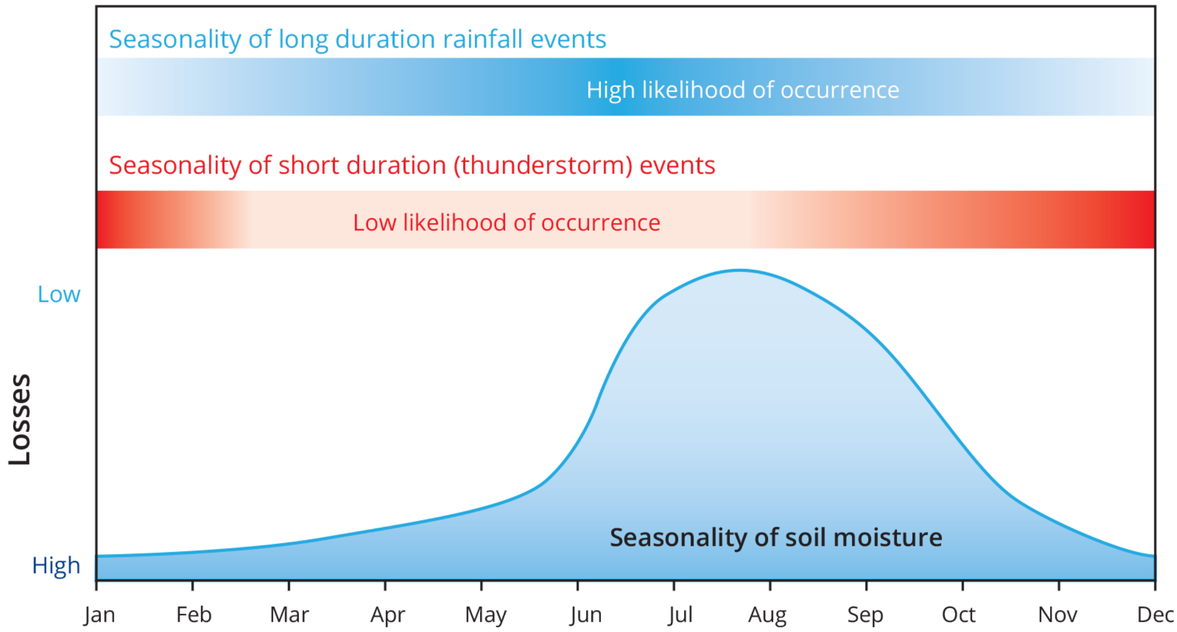


Figure 4.4.2. Difference in the Seasonal Likelihood of Large Long Duration Rainfall Events and Large Short Dduration Rainfall Events and their Concurrence with Catchment Losses

The analytical approach required to accommodate conditional probabilities when the events are not mutually exclusive is more complex. There are a number of different approaches that can be used, and in any given design situation the best approach to adopt depends on the nature and importance of the problem. Monte Carlo simulation in combination with evaluation of the Total Probability Theorem provides a general solution to problems involving conditional probabilities, and details on how to undertake such an approach is provided in Book 7, Chapter 9. However, different approaches are often available and the choice of solution does somewhat depend on the skills and experience of the practitioner. For example, while the assessment of flood immunity along a length of linear infrastructure could be solved by generating correlated rainfall inputs for use with event-based models, the use of gridded rainfall fields in combination with continuous simulation obviates the need to explicitly consider the joint probabilities involved (Jordan et al., 2015). Other approximate approaches that explicitly consider correlation in rainfall events have also been applied (Fricke et al., 1983), and a simple analytical example demonstrating a similar approach is provided in Book 8, Chapter 7, Section 3 and Book 8, Chapter 8, Section 5.

The techniques presented in this Book can also be applied to events which are mutually exclusive, however again it may be appropriate to adopted simpler approaches. For example, a discussion of the specific issues involved in computing annual risks from analyses undertaken on a seasonal basis is provided in Book 8, Chapter 7, Section 4; this approach is applicable to any design in which the conditional contributions are mutually exclusive, where the relative importance of the different factors may vary with event severity.

4.2.5. General Approach

Catchment Modelling Systems used to derive flood estimates can be considered to have stochastic and deterministic components. As discussed above, the stochastic components are related to factors (like rainfalls and losses) whose state at any given point in time is uncertain. The deterministic component represents processes that can be described mathematically and defines the manner in which inputs combine to yield a given output. This transformation is deterministic in the sense that the model will always yield the same outcome for a given set of inputs, antecedent conditions, and parameter values.

The general form of this concept is shown in Figure 4.4.3 for three different examples. In one example, the stochastic component represents the flood frequency distributions of two tributaries, where the deterministic component represents the manner in which the flows combine at their confluence. For a reservoir, the stochastic inputs might represent the frequency distribution of inflows and initial storage levels, where the deterministic component represents the relationship between inflows, storage and outflows. In hydraulic modelling, stochastic inputs may be used to represent inflows to a stream reach as well as the tide levels for a downstream boundary condition, where the deterministic component is governed by the hydraulic equations that predict flood level as a function of streamflow, reach characteristics and boundary conditions.

A variety of approaches are available for solving this general type of problem. Laurenson (1974) provides a general solution based on the matrix multiplication of a probability distribution of a stochastic input with a transition matrix derived from the deterministic operation of the system. The method is very general and suited to numerical solution. Careful effort is required to develop the elements of the transition matrix, and additional conditional probability terms need to be evaluated to allow for correlations in the inputs.

The joint occurrence of correlated stochastic factors can be evaluated using bivariate distributions, and there are numerous applications in the water resources literature where these have been used. The methodology used to assess the coincidence of catchment flooding and extreme storm surge for the coastline of Australia was developed using such an approach (Zheng et al., 2014), and is covered in detail in Book 6, Chapter 5. There are fewer examples where multivariate extreme distributions are used, and possibly the use of copula functions in combination with univariate distributions afford a more practical approach (Favre et al., 2004; Genest and Favre, 2007; Chen et al., 2012). Kilgore et al. (2010) reviews a range of methods and develop a general methodology for estimating joint probabilities of coincident flows at stream confluences based on the use of copulas which is intended for use by practitioners.

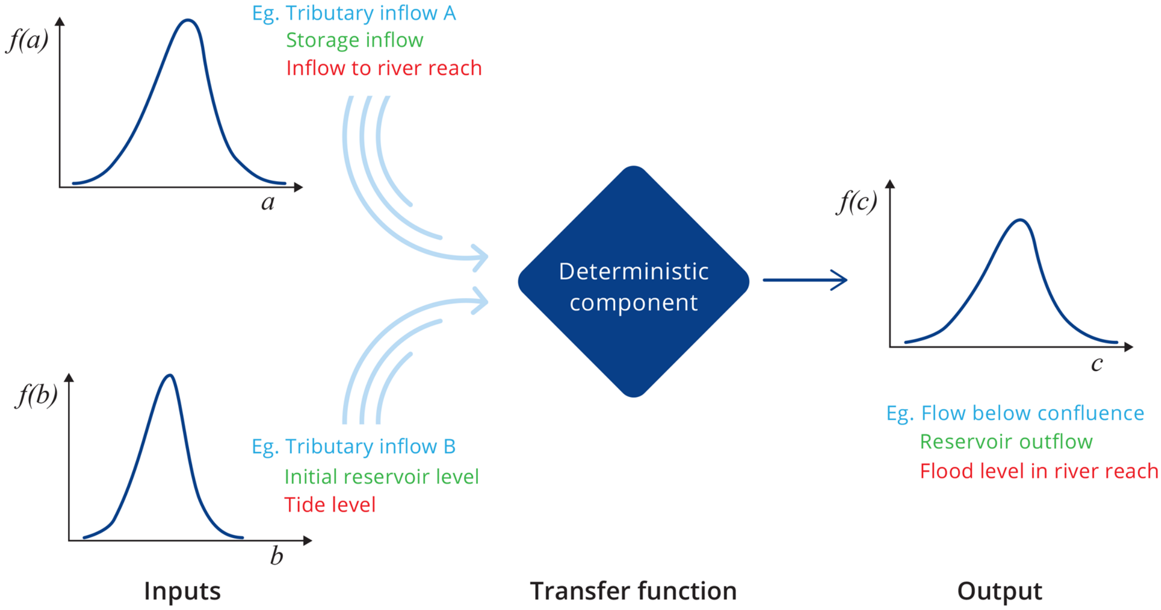


Figure 4.4.3. Generic Components that need to be Considered in Solution of Joint Probability Problems

However, the development and application of such approaches does require considerable statistical skill and they are not well suited for application by the majority of practitioners. Also, regardless of the methods used to characterise the extreme (possibly correlated) behaviour of the inputs, it is still necessary to model the deterministic component to determine how the various inputs combine to yield outputs of different magnitudes. Developing such response functions over the range of inputs required is itself a demanding task, and there is advantage if this can be done in such a way that leads directly to the exceedance probabilities of interest.

Monte Carlo techniques provide a structured means to generating outputs for a wide range of inputs, and if formulated correctly they represent a generic solution to the problem illustrated in Figure 4.4.3. With this approach, inputs are randomly sampled many hundreds, or thousands of times, and used in conjunction with a model of the deterministic component to obtain a distribution of the required outputs. Statistical analysis is then used to estimate the exceedance probability of the output variable of interest.

One of the main attractions of Monte Carlo methods is that the modelling tools and hydrologic concepts involved are essentially identical to those used in traditional approaches. Differences only arise in the manner in which the inputs are handled and the results analysed. Once the necessary framework has been developed, the factors of most importance can be modelled as stochastic inputs, and those of lesser importance can be set at fixed values. Many practitioners are used to developing automated means for running simulation models; such approaches can be adapted to Monte Carlo simulation by using simple probability models to generate the inputs, and straightforward statistics to analyse the outputs. The approach thus represents a powerful means of capturing the influence of variability on hydrologic systems in a manner that requires only a modest increase in the level of modelling sophistication.

4.2.6. Selection and Treatment of Factors

Any explicit analysis of joint probability should only focus on those factors which are characterised by a high degree of variability and which have a significant influence on flood response. Factors which have a small range of variation or a small influence on outcomes are best treated as fixed inputs to the model. The degree of importance of any factor can be assessed by simply undertaking a sensitivity analysis whereby the values of individual factors are varied systematically over the range of their expected variation and the factors with the largest stochastic influence are explicitly included in joint probability analysis. Some factors may have a large influence on the outcome (e.g. routing parameters) but are principally sources of epistemic uncertainty and thus do not need to be treated in a stochastic manner.

The common attribute of stochastic factors that influence flood response is that at any given point in time their state is uncertain. With sufficient data it is possible to estimate their average state and other characteristics related to their range and variability, and possibly the nature of their dependence on the magnitude of other factors. Often natural factors vary in a systematic fashion with the time of day or season, and they may be correlated. For example, initial loss might range between 0.1 and five times its median value but 70% of the time it might range between 0.5 and 1.5 times the median; average summer losses might also be expected to be twice the magnitude of winter losses, and because of the likelihood of rainfall occurring before intense rainfalls bursts, it might be that initial loss values vary inversely (ie. are negatively correlated) with rainfall depth.

4.2.6.1. Use of Regionalisation and Standardisation

Information on the variability and dependence of hydrological factors can be obtained from regional or catchment-specific (“at-site”) data. Physical reasoning should be used to determine what sources of data might be relevant to the catchment of interest. For example, information on the temporal variability of storm rainfalls is associated with storm types which may occur over a large region, and thus rainfall data collected over an extensive geographic area can be used to obtain information on the variability of temporal patterns that are relevant to a specific catchment (Book 2, Chapter 5). Conversely, the spatial variability of rainfalls across a catchment is subject to natural variability arising from storm behaviour, but it might be expected that there is a systematic component to this that is dependent on local topography and the dominant storm direction; accordingly, local rainfall data should be used to characterise catchment-specific spatial variability.

When considering the use of regional information it is often useful to standardise the data in some form to allow transposition from one site to another. An example of this relevant to flood estimation is the distribution of losses, as illustrated in Figure 4.4.4. While the typical magnitude of losses varies from one catchment to another, standardising these values (by simply dividing by the median value for the catchment) reveals that the likelihood that the catchment is wetter or drier relative to typical conditions is similar for a wide variety of catchment types (Hill et al., 2015). The representation of temporal pattern increments as a proportion of total burst depth rather than, say, as an absolute depth in mm, is another example of how regional information can be pooled to represent variability.

4.2.6.2. Dealing with Dependence

It is important to understand whether the variability in one factor might be correlated with another, or whether the nature of variation is dependent upon event magnitude. Again, judgment must be used to determine the appropriateness of data used to investigate such dependencies. If relationships are required on the nature of the dependence between selected hydrologic factors, then evidence can usually be found in meteorologically similar regions. Information on the variability of anthropogenic factors, such as reservoir levels or performance reliability, is also often available from the instrumented record, or from models used to simulate their operations.

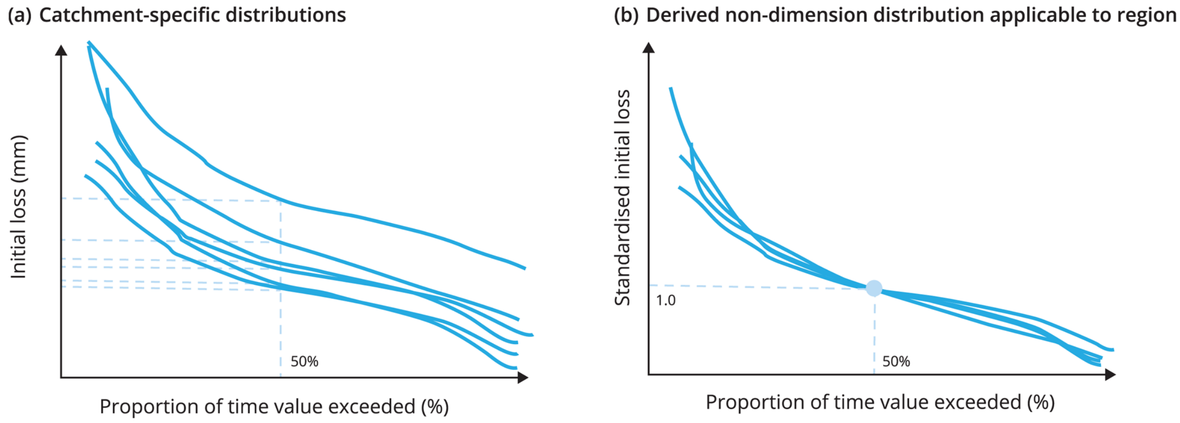


Figure 4.4.4. Use of Standardisation to Derive a Regional Distribution Based on Catchment-specific analyses

Variation with Event Severity

Investigation into how flood producing factors may vary with flood severity may be of particular importance as often the information at the location of interest may be limited. For example, it may be suspected that reservoir levels will be higher at the start of extreme rainfall events as these may be more likely to occur during wetter (La Niña) periods. Evidence for this might be obtained by examining historical correlations between initial reservoir levels prior to large rainfalls, but if such information is limited then it may be more appropriate to “trade space for time” by examining correlations between seasonal rainfalls and extreme storms over a wide region (once the data has been standardised to allow for systematic variation in rainfall depths). An illustration of this by Scorah et al. (2015) for south-eastern Australia is shown in Figure 4.4.5(a).

Two other examples of similar investigations are provided in Figure 4.4.5. The middle panel of Figure 4.4.5 shows the dependence of storm surge on rainfall maxima for an investigation into the interaction between coastal processes and severe weather events (Westra, 2012), and the right-hand panel illustrates the variation in temperature coincident with rainfall maxima for the consideration of the joint probabilities involved in rainfall-on-snow events (Nathan and Bowles, 1997).

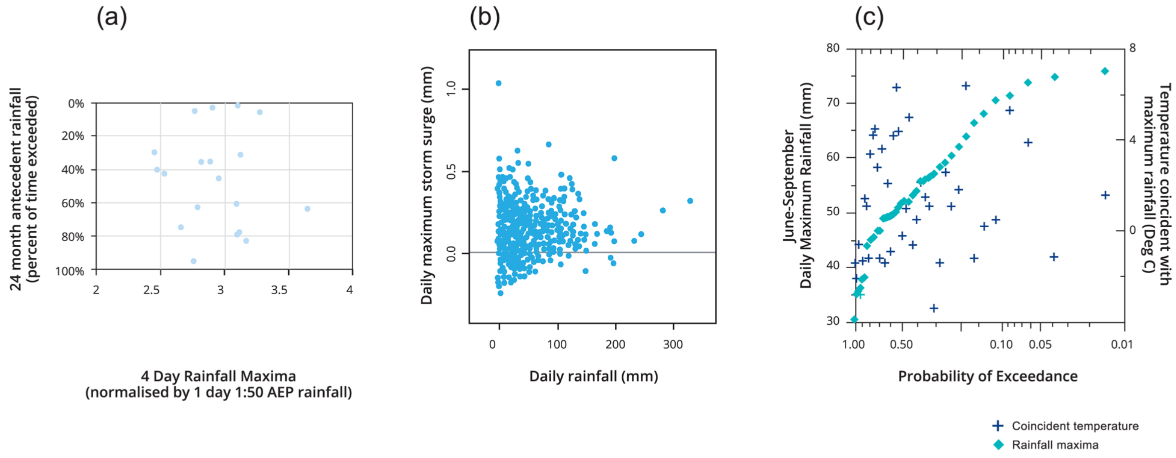


Figure 4.4.5. Examples of Investigations into Dependence between Flood Producing Factors based on (a) Antecedent Seasonal Rainfall Data for Catchments over 1000 km^2 (Scorah et al., 2015), (b) Rainfall and Storm Surge Data (Westra, 2012), (c) and Temperature Coincident with Rainfall Maxima (Nathan and Bowles, 1997)

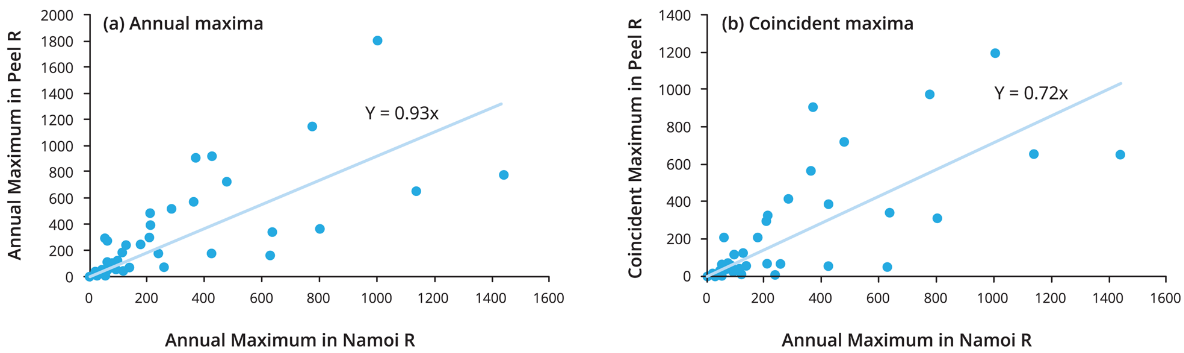


Figure 4.4.6. Examples of Difference in Correlation between Flow Maxima in the Namoi and Peel Rivers, Based on (a) Annual Maxima at Both Sites, and (b) Peel River Flows that are Coincident with Namoi River Maxima

4.2.6.3. Relevance of Sample

Lastly, it is worth stressing the importance of ensuring that the nature of the dependence being investigated is relevant to the design problem. For example, if it is desired to estimate the magnitude of a coincident flood at a downstream confluence to serve as a boundary condition for hydraulic modelling, then the dependency of interest is the flow in the tributary that is coincident with the flow in the mainstream of interest. As shown in Figure 4.4.6, this might well be a different relationship to, say, the correlation between annual maxima at the two sites.

4.3. Monte Carlo Simulation

4.3.1. Introduction

The following sections provide details on some core concepts used in Monte Carlo simulation. The focus of this material is to provide practitioners with sufficient understanding to be able to formulate a scheme that is suited to solving practical problems in flood estimation. A worked example is provided in Book 4, Chapter 4, Section 4 that demonstrates application of the techniques to a practical problem.

A general and very accessible introduction to Monte Carlo methods can be found in Burgman (2005), and more comprehensive and practical guidance is provided in Vose (2000) and Saucier (2000); the latter reference includes C++ source code for a collection of various distributions of random numbers suitable for performing Monte Carlo simulations. Hammersley and Handscomb (1964) provide a more advanced theoretical treatment of the subject, and useful discussion on the advantages of using Monte Carlo methods to estimate design floods can be found in Weinmann et al. (2002), Kuczera et al. (2003), and Weinmann and Nathan (2004).

It should be noted that while there are advantages to developing a simulation framework using high level computing languages such as Python, C++ and Fortran, it is quite feasible to initiate the required design runs and undertake the required statistical analyses using standard spreadsheet software. Robinson et al. (2012) applied such a framework to the solution of the joint probabilities involved in the simulation of extreme floods and reservoir drawdown. At its simplest, any practitioner familiar with the techniques required to prepare batched command scripts and use spreadsheet formulae will be able to implement the procedures described herein.

The following sections outline the main steps involved in developing a Monte Carlo solution of joint probability problems. The sections follow the sequence of steps shown in Figure 4.4.7, which refers to the stochastic deterministic components of the general catchment modelling system as illustrated in Figure 4.4.3. It should also be noted that this scheme is a generalisation of the Monte Carlo framework depicted in Figure 4.3.2 of Book 4, Chapter 3; specifically, the scheme shown in Figure 4.4.7 represents the treatment of natural variability in rainfall-based flood estimation, where no account is given to epistemic uncertainty in the data, parameters, or modelling components.

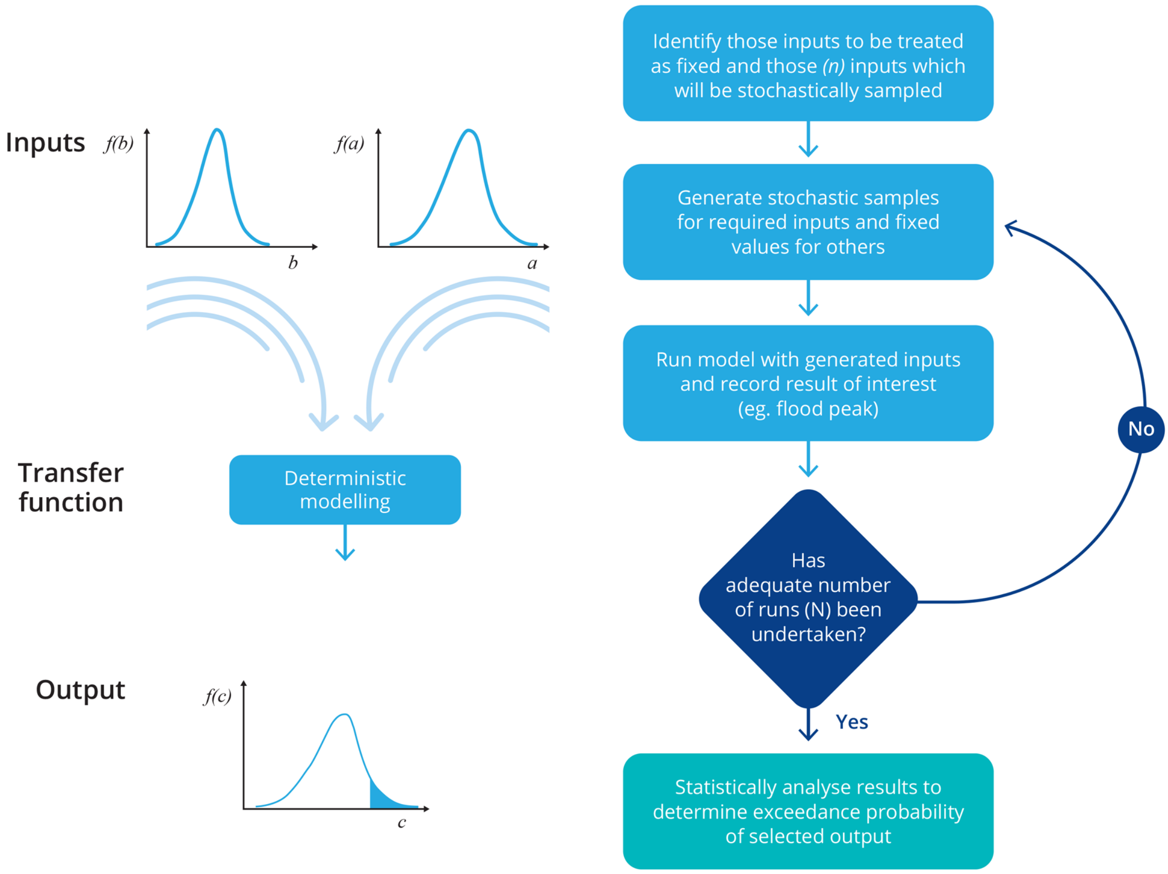


Figure 4.4.7. General Framework for the Analysis of Stochastic Deterministic (Joint Probability) Problems using Monte Carlo Simulation

4.3.2. Generation of Stochastic Inputs

4.3.2.1. Inverse Transformation Approach

The method used to stochastically sample from the input distributions is the core algorithm used in Monte Carlo simulation. Once a suitable framework has been established additional model inputs and/or parameter values can be added to the sampling procedure as required.

The generation scheme makes use of the *inverse transformation* approach. This can be applied to either formally defined probability models, or else to empirical “data-driven” distributions. The basis of the inverse transformation approach is to generate the required probability density function *f(x)* through uniform sampling of the inverse of the cumulative distribution function *F(x)* (ie. the function which gives the probability *P* of *x* being less than a specified value).

The two-step process for doing this is illustrated in Figure 4.4.6, and the algorithm can be summarised as follows:

1. Generate a random number (U) uniformly distributed between 0 and 1;

2. Calculate the value (*x*) of the inverse of the cumulative density function F^{-1}(U).

This process is illustrated in Figure 4.4.8 for three random numbers. The first random number generates a value near the tail of the distribution, and the next two yield values that are more centrally tended. For illustration purposes the input random numbers (*U*) in Figure 4.4.8 are shown as being equally spaced, but on exit the transformed numbers are unequally spaced, in conformance with the adopted distribution. Inverse functions of a number of useful distributions (Normal, log-Normal, Beta, Gamma) are provided in standard spreadsheet software (see example in Book 4, Chapter 4, Section 4). If an empirical distribution is used then values can be simply interpolated from a look-up table comprised of values of the cumulative density function (also see example in Book 4, Chapter 4, Section 4).

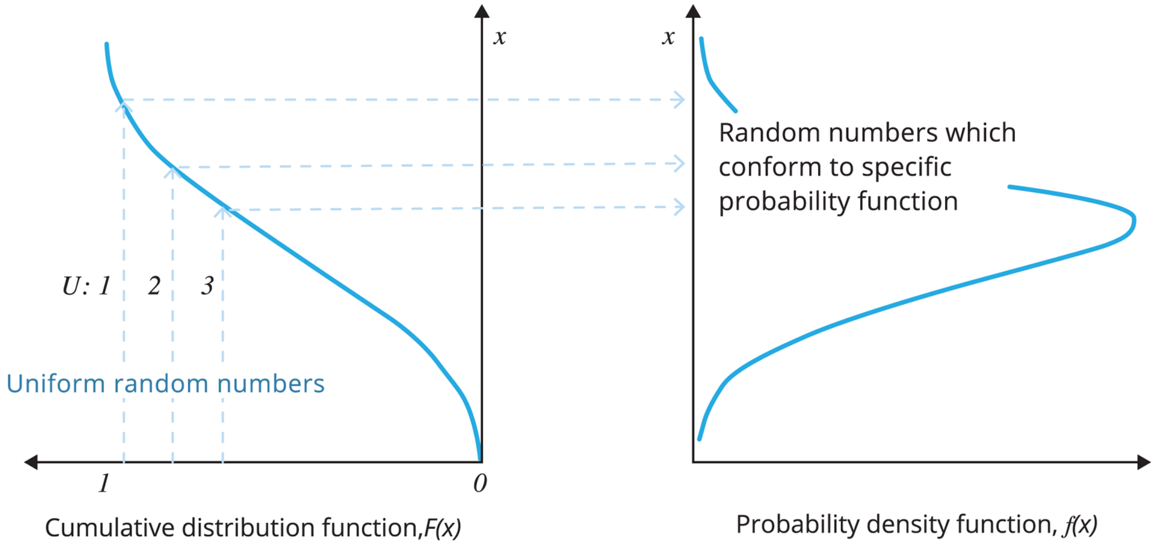


Figure 4.4.8. Inverse Transform Method

4.3.2.2. Parametric Sampling

There are a large number of statistical distributions that can be used to represent variability in different types of hydrological processes and input uncertainty. Information on a range of distributions of potential use can be found in Saucier (2000), Vose (2000), and (Maidment, 1993). Special mention is made here of the Normal distribution. This distribution is also of considerable practical utility as many stochastic processes in hydrology conform to the log-Normal distribution (that is they only take positive values and are skewed towards higher values), and transforming the data beforehand into the logarithmic domain is a simple means of taking direct advantage of the Normal distribution. In addition, many data sets can be transformed into the Normal domain by the Box-Cox transformation (Box and Cox, 1964); with this approach, a variate *X* can be transformed into the Normal domain (*Z*) by the following equations:

$$Z = \frac{X^{\lambda} - 1}{\lambda}, \text{ when } \lambda \neq 0;\ z = \ln(x), \text{ when } \lambda = 0 \tag{4.4.4}$$

where λ is a parameter determined by trial and error to ensure that the skewness of the transformed distribution is zero. A noteworthy special case of this transformation arises when λ is set to zero, then the transformation is equivalent to taking logarithms of the data. Fitting the parameter λ is most easily achieved by optimisation or the use of “solver” routines that are commonly available in spreadsheet programs. To illustrate the use of the inverse transformation method with a variable that has been transformed using a Box-Cox lambda of 1.2, where the resulting normally-distributed variates have a mean of 50 and a standard deviation of 25:

1. Generate a uniform random number (say, U = 0.548);

2. Derive the value of the inverse cumulative Normal distribution (z = 0.121);

3. Obtain the Normal variate, Z = 50.0+0.121*25 (=53.015);

4. Apply the inverse of the Box-Cox transformation (x = 32.257).

The above four steps can be repeated many hundreds (or thousands) of times as required for input to a model. The outcome of the above four steps repeated 1000 times is provided as a histogram in Figure 4.4.9.

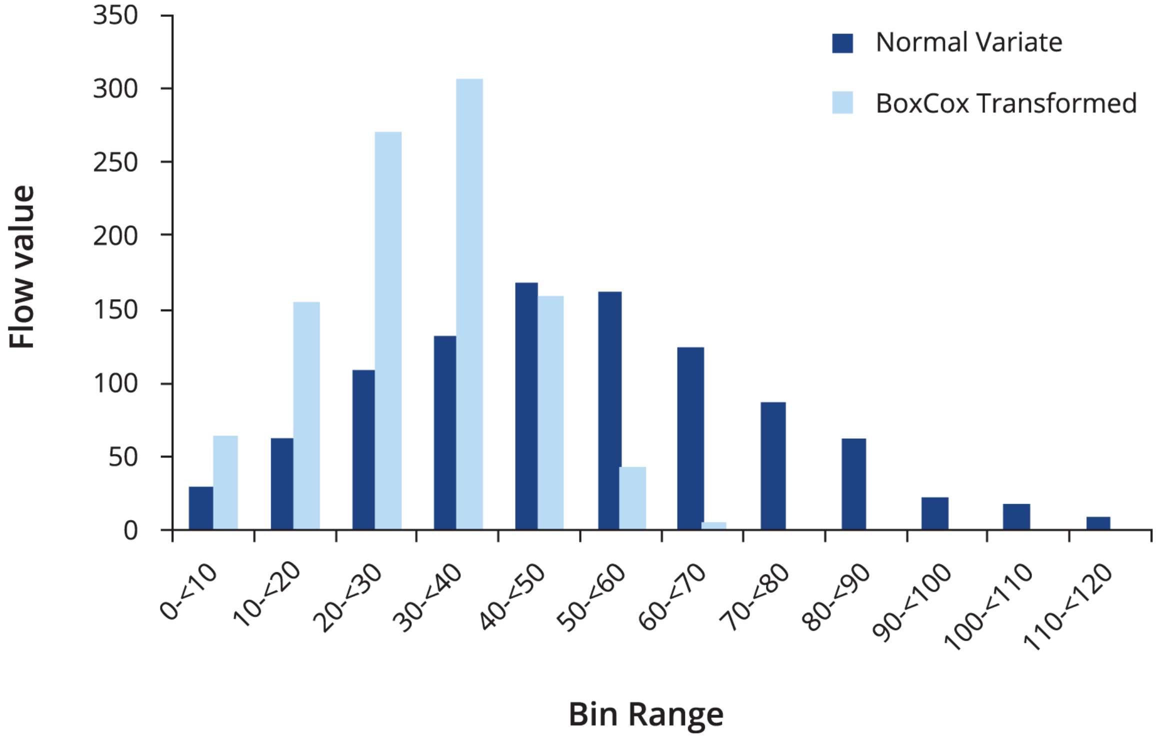


Figure 4.4.9. Histogram Obtained by Generating 1000 Random Numbers Conforming to a Normal Distribution with a Mean of 50 and Standard Deviation of 25, and the Resulting Distribution of variables Obtained by the Inverse Box Cox Transformation (with λ set to 1.20)

Details of the Normal distribution are provided in all statistics textbooks and thus further information will not be presented here. Source code for estimation of the cumulative Normal distribution is freely available (Press et al., 1993) and the function is available in spreadsheet software.

Lastly, it is worth noting that the uniform distribution is also of practical use in flood hydrology. A simple random number generator that varies uniformly between 0 and 1 can be directly applied to the sampling of temporal, or space-time, patterns of rainfall that are considered equally likely to occur.

4.3.2.3. Non-Parametric Sampling

One very practical way of undertaking a Monte Carlo simulation is to sample from a given set of data. This is a fast and simple technique that can be used to take advantage of empirical data sets (such as losses and reservoir drawdown) in a more defensible manner than simple adoption of a single best estimate or representative value. It is also useful for sampling from “pragmatic” distributions, such as rainfall frequency curves that extend beyond 1 in 2000 AEP and which are not based on a theoretical distribution function (Book 2, Chapter 2).

The algorithm to construct and sample from an empirical distribution is as follows:

1. Sort empirical data into either ascending or descending order as appropriate, and assign a cumulative probability value to each. If there are *n* data values, then the largest data value (x_1) is assigned an exceedance probability $F(x_1)$, the second largest (x_2) is assigned an exceedance probability $F(x_2)$, and so on till the last value, represented by x_n and $F(x_n)$;
2. Generate a uniform random number, U = U(0,1);
3. Locate interval *i* such that $F(x_i) \leq U < F(x_{i+1})$;
4. Return $X = x_i + \frac{U - F(x_i)}{F(x_{i+1}) - F(x_i)}(x_{i-1} - x_i)$;
5. Generate additional points by returning to Step 2.

While simple to implement, the use of empirical distributions in Monte Carlo simulation does require care. Most importantly, it is necessary to ensure that the data sample being used is relevant to the whole range of conditions being simulated. For example, if the data set is comprised of initial reservoir levels recorded over a short historic period, then these may not be relevant to the assessment of extreme flood risks under a different set of operating rules.

It is seen in Step 4 of the above algorithm that values within each interval are obtained by linear interpolation. This is normally quite acceptable, though obviously the less linear the relationship between the data values and their corresponding exceedance probabilities the less defensible is such an approach. Accordingly, in some cases it is best to first transform the data and/or the exceedance probabilities assembled for Sstep 1 of the algorithm. Many hydrological variables are approximately log-Normally distributed, and thus it is often desirable to undertake the interpolation in the log-Normal domain. To this end, the ranked data values are transformed into logarithms (it does not matter what base is used) and the exceedance probabilities are converted to a standard normal variate (that is, the inverse of the standard normal cumulative distribution). Step 2 of the above algorithm would thus need to be replaced with $U = U(z_{min}, z_{max})$ where $z_{min}$and z_{max} represent the standard normal deviates corresponding to $F(x_1)$ and $F(x_n)$, ie. the adopted limits of exceedance probability range.

Care is also required when sampling from the tails of the distribution. Empirical data sets are of finite size and, if the generated data are to fall between the upper and lower limits of the observed data, the cumulative exceedance probability of the first ranked value $F(x_1)$ should be zero, and that of the last ranked value $F(x_1)$ should be 1.0. Thus use of empirical data sets is appropriate for those inputs whose extremes of behaviour are not of great relevance to the output. Losses, for example, are zero bounded, and thus the difference in flood peak between a loss exceeded 95% of the time and that exceeded 99.999% of the time may well be of no practical significance. However, if an empirical approach is being used for the generation of rainfalls that are defined for between 1 in 2 and 1 in 100 AEP, then it is inevitable that more than half the random numbers generated in Step 2 of the above algorithm can be expected to lie outside the specified range of rainfalls. As long as the probability range of interest lies well within the limits specified, then rainfall values can be obtained by some form of appropriate extrapolation; however, if this approach is used then checks should be undertaken to ensure that the extrapolated values do not influence the results of interest.

4.3.2.4. Generating Correlated Variables

Many hydrologic variables are correlated and thus it is sometimes necessary to ensure that the adopted sampling scheme preserves the correlation structure of the inputs. A simple means of generating correlated variables is described by Saucier (2000). The approach is based on rotational transformation and the steps involved in generation of uniformly distributed variates can be stated as follows:

1. Independently generate two uniform random variates, *X= U(-1,1)* and *Z= U(-1,1)*;
2. Set $Y = \rho X + Z\sqrt{1-\rho^2}$ where *r* is the required correlation between *X* and *Z*;
3. Return:

$$x = \frac{x_{\min} + x_{\max}}{2} + X\left(\frac{x_{\max} - x_{\min}}{2}\right)$$

$$y = \frac{y_{\min} + y_{\max}}{2} + Y\left(\frac{y_{\max} - y_{\min}}{2}\right)$$

where x_{min} and x_{max} are the lower and upper bounds of the first variate and y_{min} and y_{max} are the corresponding bounds of the other.

Application of the above algorithm is illustrated in Figure 4.4.10(a). The bounds along the x-axis are 5 and 130, and those along the y-axis (for the mid-point of the x distribution) are 30 and 75. Figure 4.4.10 illustrates the results for the generation of 2000 correlated variates where the correlation coefficient (ρ) adopted is -0.7.

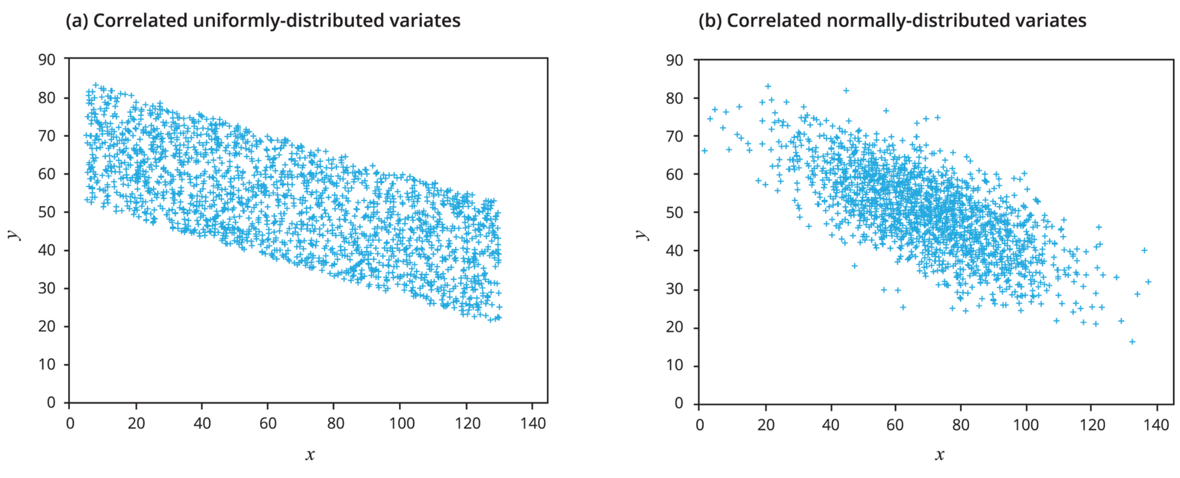


Figure 4.4.10. Generation of Variables with a Correlation of -0.7 based on (a) Uniform and (b) Normal Distributions

The above algorithm can easily be adapted to the generation of correlated variates that conform to some specified distribution. For the Normal distribution, the required algorithm is:

1. Independently generate two normal random variates with a mean of zero and a standard deviation of 1: *X= N(0,1) and Z= N(0,1)*;

2. Set $Y = \rho X + Z\sqrt{1 - \rho^2}$ where *r* is the required correlation between *X* and *Z*;
3. Return:

   $x = \mu_x + X\sigma_x$

   $y = \mu_y + Y\sigma_y$

   where μ_x and μ_y are the means of the two distributions and σ_x and σ_y are the required standard deviations.

Application of the above algorithm is illustrated in Figure 4.4.10(b). The input parameters to this example are $\rho = -0.7$, $\mu_x = 70$ and $\sigma_x = 10$, and $\mu_y = 50$ and $\sigma_y = 10$ and as before a total of 2000 correlated variates are generated. Any distribution could be used in lieu of the Normal distribution, or else the variates of interest could be transformed into the normal domain.

4.3.2.5. Conditional Sampling

The preceding two sections provide a means for generating "well-behaved" variables that can be fitted to a suitable function or distribution. However, many correlated hydrologic variables are awkwardly distributed and their variability is dependent on some (often non-linear) function of their magnitude. A typical example of this type of correlation is the manner in which the level in an upstream reservoir is weakly dependent on the level in a downstream reservoir. The nature of one such dependence is shown by the large solid symbols in Figure 4.4.11, which is derived from the behaviour of two reservoirs located in south-eastern Australia. Such data is difficult to normalise or fit to probability distributions, and thus an empirical sampling approach can be used.

The approach that can be followed to stochastically sample from such a data set can be described as follows:

1. Identify the "primary" variable that is most important to the problem of interest, and prepare a scatter plot of the two variables with the primary variable plotted on the x-axis (as shown in Figure 4.4.11);

2. Divide the primary variable into a number of ranges such that variation of the dependent variable (plotted on the y-axis) within each range is reasonably similar; in the example shown in Figure 4.4.11 a total of seven intervals has been adopted as being adequate. This provides samples of the secondary variable that are conditional on the value of the primary variable;

3. Stochastically generate data for the primary variable using the empirical approach as described in Book 4, Chapter 4, Section 3;

4. Derive an empirical distribution of the dependent data for each of the conditional samples identified in Step 2 above (that is, undertake Step 1 of the empirical approach as described in Book 4, Chapter 4, Section 3 for each of the intervals); thus, for the example shown in Figure 4.4.11 a total of seven separate empirical distributions of upstream storage levels are prepared;

5. For each generated value of the primary variable, stochastically sample from the conditional distribution corresponding to the interval that it falls within; for example, if a downstream storage level of 1500 ML was generated in Step 3 above, then the empirical

approach described in Book 4, Chapter 4, Section 3 is applied to the conditional distribution obtained from data occurring within the third lowest interval shown in Figure 4.4.11.

The results from application of the above procedure are illustrated in Figure 4.4.11 for 2000 stochastic samples (shown by the blue “+” symbols). The 2000 correlated values are stochastically generated based on information contained in 500 observations. It is seen that the correlation structure in the observed data set is preserved reasonably well by this procedure.

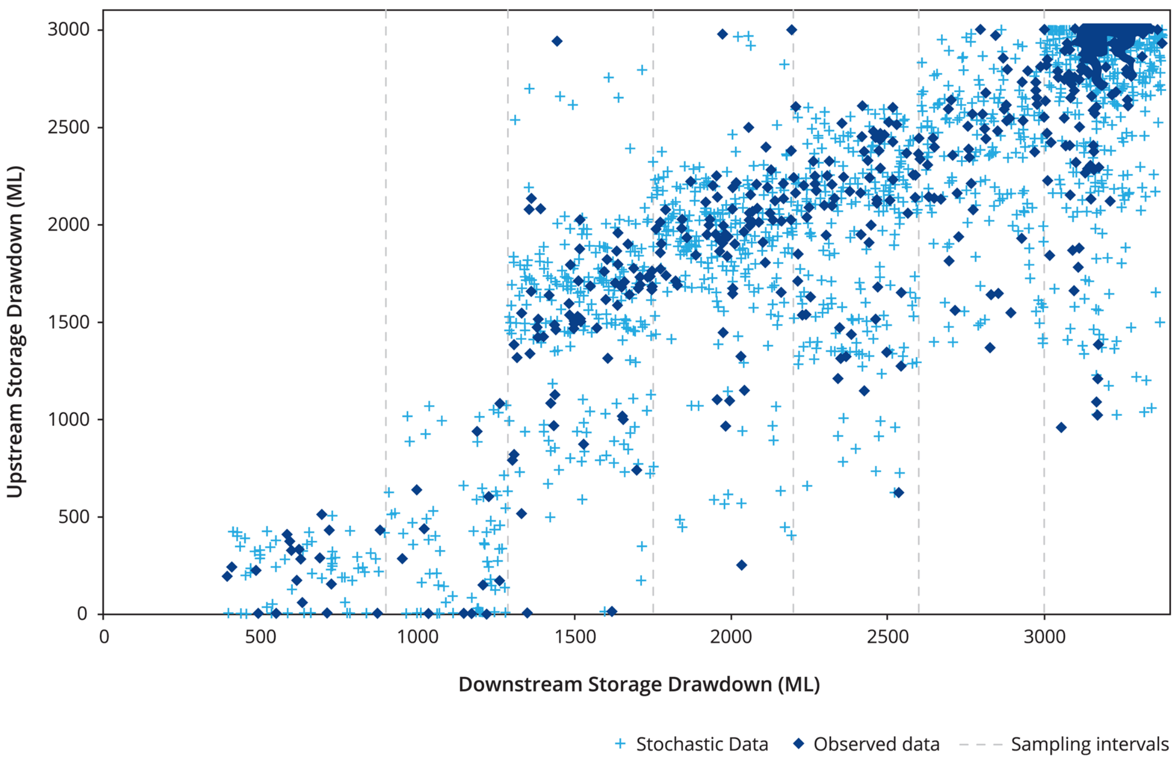


Figure 4.4.11. Conditional Empirical Sampling - Storage Volume in an Upstream Dam is Correlated with the Volume in a Downstream Dam

4.3.3. Estimation of Exceedance Probabilities

4.3.3.1. Selection of Method

Estimation of exceedance probabilities from Monte Carlo simulation results can be obtained by either “direct sampling” or “stratified sampling” approaches. With direct sampling, the results are analysed using either traditional frequency analysis or non-parametric methods; with stratified sampling, the results are analysed by application of the Total Probability Theorem. The decision regarding which approach to use is largely a practical one, though there are theoretical differences in the nature of the derived quantiles: application of the Total Probability Theorem yields *expected probability estimates* of a given flood magnitude, whereas traditional frequency analysis of the derived maxima based on Cunnane (and most other) plotting positions are formulated to yield *unbiased estimates of the flood magnitude* for a given exceedance probability, though adoption of the Weibull plotting position *i/(n+1)* should yield unbiased probability estimates (Book 3, Chapter 2, Section 6). It is always necessary to experiment with many different model parameters, model configurations, and design scenarios, and simulation times of more than an hour or so soon become impractical.

The first approach, based on direct sampling, is the most straightforward to implement. It is well suited to the analysis of problems that can be computed quickly, or else to more complex problems in which the probability range of interest is limited to reasonably frequent events. As a rule of thumb, the number of simulations required is around 10 to 100 times the largest average recurrence interval of interest. That is, if the rarest event of interest has an annual exceedance probability of 0.001, then it will be necessary to generate between 10 000 to 100 000 stochastic samples in order to derive a stable result.

The second approach, based on stratified sampling, does require more effort to implement. It can still be formulated using a “batch” file approach, though additional care needs to be taken with how the inputs are formulated and the results analysed. The benefit of this effort is that the number of runs required to estimate the exceedance probability of rare events is considerably fewer; indeed the algorithm can be designed so that a similar number of runs is required regardless of the range of probabilities of interest.

Further information on these two approaches is provided in the next two sections. It is worth noting that other approaches could be used; for example Diermanse et al. (2014) derive estimates using importance sampling, which is similarly efficient to the stratified sampling discussed below.

4.3.3.2. Direct Sampling

The results output from the Monte Carlo simulation are most easily analysed by non-parametric frequency analysis. Using flood peaks as an illustration, the steps involved can be summarised as follows:

1. Sort the *N* simulated peaks in order of decreasing magnitude;

2. Assign a rank (i) to each peak value; 1 to the highest value, 2 to the next highest, and so on, down to rank *N*;

3. Calculate the plotting position (*p*) of each ranked value using either the Weibull (Equation (4.4.5)) or the Cunnane (Equation (4.4.6)) formulae:

$$p = \frac{i}{N+1} \tag{4.4.5}$$

$$p = \frac{i-0.4}{N+0.2} \tag{4.4.6}$$

   If the design focus is on estimating the *probability* of a given flood magnitude then the Weibull formula (Equation (4.4.5)) should be used as this provides an unbiased estimate of the exceedance probability of any distribution. Alternatively, if the focus is on the *magnitude* associated with a given exceedance probability then the Cunnane formula (Equation (4.4.6)) is preferred as this provides approximately unbiased quantiles for a range of distributions.

4. Construct a probability plot of the ranked peaks against their corresponding plotting positions. The plot scales should be chosen so that the frequency curve defined by the plotted values is as linear as possible. In many hydrological applications the ranked values may be plotted on arithmetic or log scales and the estimated exceedance probabilities (the plotting positions) are plotted on a suitable probability scale. Most popular spreadsheet programs do not include probability scales and thus, for probability plots conforming approximately to the Normal or log-Normal distribution, it is necessary to

convert the probabilities to their corresponding standard normal cumulative distribution values. Alternatively, for probability plots conforming approximately to the exponential distribution, the reciprocal of the exceedance probabilities (the average recurrence interval) can be plotted on a logarithmic scale; and

5. The magnitude associated with a given exceedance probability (if the Cunnane plotting position is used) or else the exceedance probability associated with a given magnitude (if the Weibull plotting position is used) can be interpolated directly from the probability plot. For convenience, a suitable smoothing function (ie. polynomial equation) can be fitted to the plotted values in the region of interest to simplify the estimation of design values. The function is used merely to interpolate within the body of the plotted points and thus, as long as there is no bias in the fit, it matters little what function is used (polynomial functions are quite suitable).

If desired, the maxima can be fitted using a traditional probability model (Book 3, Chapter 2), but given that sufficient simulations need to be undertaken to yield a stable estimate, there is little point in doing so.

4.3.3.3. Stratified Sampling

While the above approach is straightforward, it is computationally inefficient as the vast majority of simulations undertaken provide little information on the extremes of interest. That is, the vast majority of computational effort is expended on deriving results for the range of exceedance probabilities that is of least interest. This inefficiency is of little concern when using simple models with sparing outputs and fast simulation speeds. However, as the data processing becomes more complicated and execution speeds increase, simulation times and data storage requirements quickly pose significant practical problems.

Adoption of a stratified sampling approach ensures that the computational effort is always focused on the region of interest and, if the simulation scheme is configured carefully, then it will usually be possible to apply Monte Carlo simulation to most practical problems.

The approach follows the same logic as represented in the flow chart of Figure 4.4.7, the only difference is that samples of the stochastic variable that is of most importance to the output are generated over specific probability ranges. It matters little how the ranges are defined and the ranges can be varied to suit the different ranges of interest. It is simplest to divide the domain into M intervals uniformly spaced over the standardised normal probability domain (Detail A in Figure 4.4.12). It should be noted that adopting this approach does not make any distributional assumption about the variable, it simply provides the means to distribute the simulations evenly across the probability domain. Typically 50 intervals should suffice, though care is required to ensure that there is adequate sampling over the region of most interest.

In the example illustrated in Figure 4.4.12, rainfall is used as the primary stochastic variable. Within each interval N rainfall depths are stochastically sampled and for each rainfall depth a model simulation is undertaken using an appropriate set of stochastic inputs (Detail B in Figure 4.4.12). The number of simulations specified in each interval (N) is dependent on the number of inputs being stochastically generated and their degree of variability, but in general it would be expected that between 50 and 200 simulations should be sufficient to adequately sample from the range of associated inputs.

The model results are recorded for all simulations taken in each interval (Detail C in Figure 4.4.12). These results are assessed using the Total Probability Theorem (Book 4, Chapter 4, Section 2) to yield expected probability estimates of the flood frequency curve. In all, if the rainfall frequency curve is divided into 50 intervals and 200 simulations are undertaken in each interval, a total of 10 000 runs is required. The same number of simulations could be used whether the upper limit of exceedance probability is 1 in 100 or 1 in 10^6, and it is merely necessary to ensure that a representative number of combinations is sampled within each rainfall range of interest. If the distribution of different rainfall durations is known, the Total Probability Theorem can also be used to give appropriate weighting to separate flood simulations for different rainfall duration intervals.

For the scheme illustrated in Figure 4.4.12, the expected probability that a flood peak (*Q*) exceeds a particular value *q* can be calculated from the Total Probability Theorem:

$$p(Q > q) = \sum_i p[Q > q|R_i]p[R_i] \tag{4.4.7}$$

where the term $p[R_i]$ represents the probability that rainfall occurs within the interval *i*, and the term $p[Q > R_i]$ denotes the conditional probability that the flood peak *Q* generated using a rainfall depth from within this interval R_i exceeds *q*. The term $p[R_i]$ is simply the width of the probability interval under consideration (this will be different for each of the *M* intervals considered), and $p[Q > R_i]$ can be calculated merely as the proportion of exceedances, *n*, in the sample of *N* simulations within interval *i* (ie. as *n/N*). A representative value of *R* can be used for all *N* simulations within the interval, though a smoother frequency curve can be obtained if *R* is sampled with the interval using a uniform distribution.

In order to ensure that the total probability domain is sampled, it is necessary to treat the first and last intervals differently from the intermediate ones. The issue here is that the full extents of the end intervals have to be adequately sampled, and on the assumption that these boundary intervals are distant from the probability region of interest, we can estimate their contribution to the total probability in a pragmatic fashion. For the last interval $p[R_1]$ is evaluated as the exceedance probability of its lower bound, and for the first interval it is evaluated as the non-exceedance probability of its upper bound. Also, for the first interval $p[Q > q|R_1]$ is replaced by the geometric mean of $p[Q > q|R_1*]$ and, say, 0.1 x$p[Q > q|R_1*]$, where R_1* is the rainfall value at the upper bound of the interval. Similarly, for the last interval the term $p[Q > q|R_\mathrm{N}]$ is replaced by the geometric mean of $p[Q > q|R_\mathrm{N}*]$ and 1.0, where R_N* is the rainfall value at the lower bound of the interval. Thus, we are assuming for the lowest interval that as the frequency of the rainfall event becomes very high the likelihood that the flow threshold is exceeded trends towards a very low value, in this case taken as one tenth the probability of $p[Q > q|R_1*]$; and for the uppermost interval we assume that the likelihood of the threshold being exceeded trends towards a value of 1.0 (ie. a certainty). The geometric mean is used in place of the arithmetic mean as here we are assuming a highly non-linear variation over the interval.

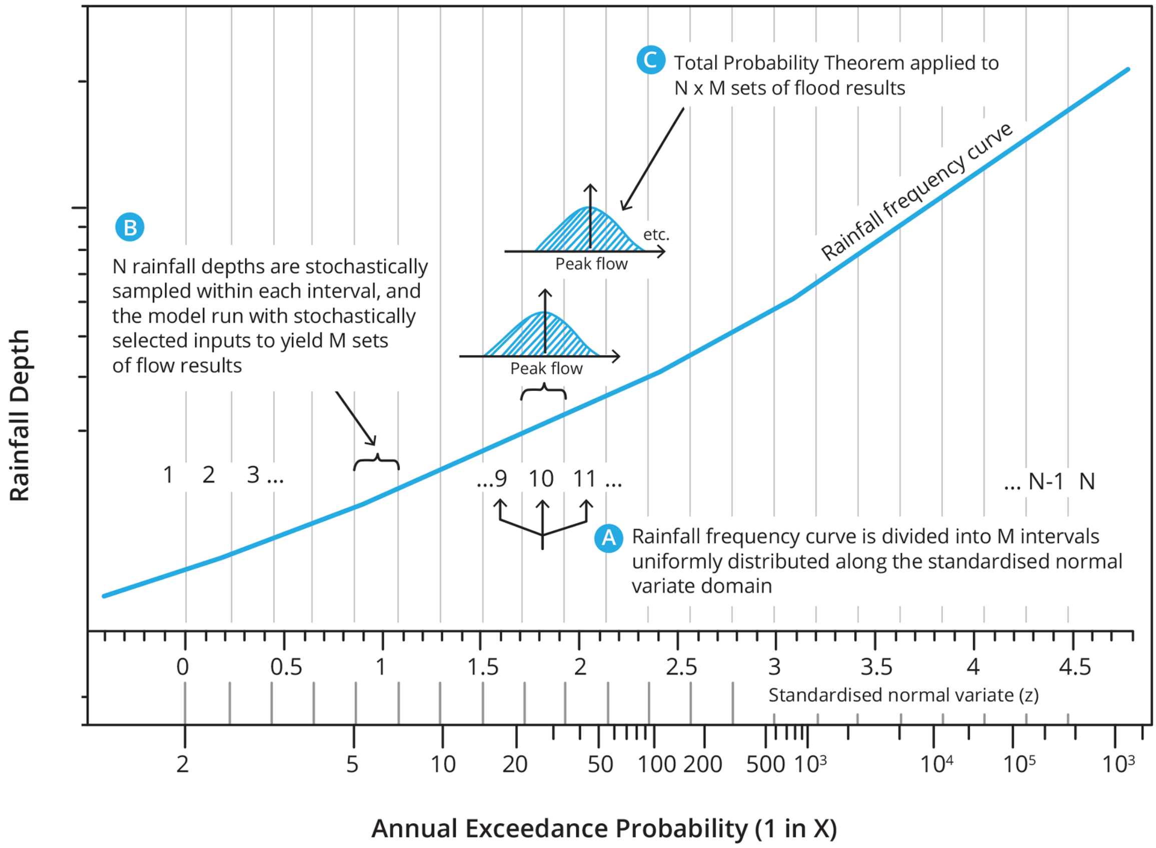


Figure 4.4.12. Manner in which Stratified Sampling is Applied to the Rainfall Frequency Curve

4.4. Example

The example below shows how the concepts described in this chapter may be used to solve a commonly encountered practical problem. The example is based on real data, but has been adapted somewhat to more easily illustrate the concepts involved.

The case study involves a township that is located below the confluence of two rivers (Figure 4.4.13). Both rivers are gauged, and one (referred to here as the “mainstream”) is larger than the other (the “tributary”). Flood frequency information has been derived for the two gauging sites, and the main focus of the study is to derive 1% AEP flood levels below the confluence, immediately upstream of the town. A one dimensional (HEC-RAS) model has been developed for the valley to allow flood levels to be determined throughout the town. The portion of the model of most relevance to this problem is shown by blueshading in Figure 4.4.13.

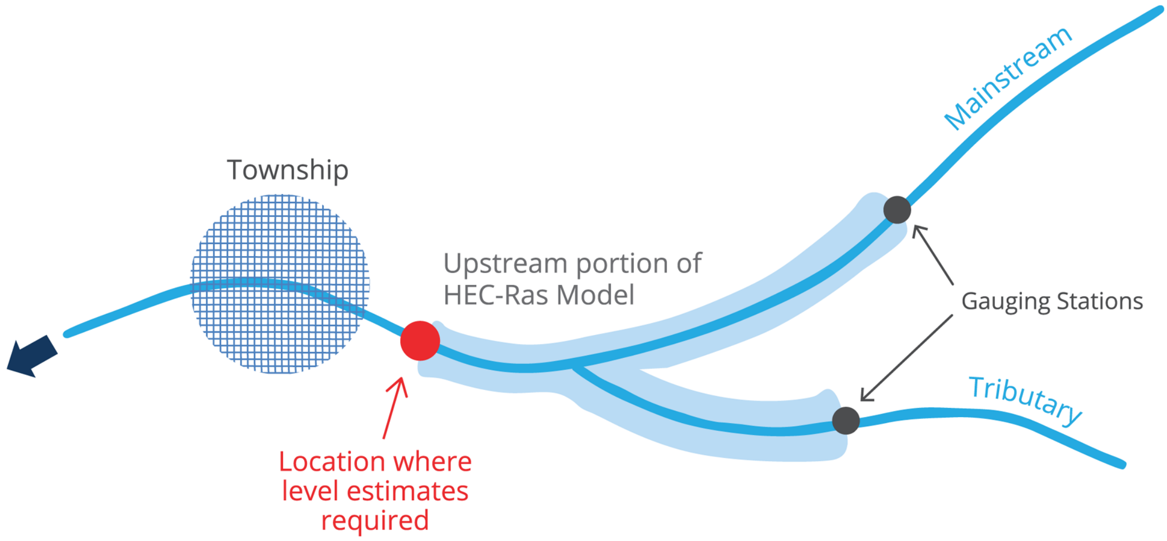


Figure 4.4.13. Schematic layout of example joint probability problem.

The analysis of this problem follows the components as outlined in Figure 4.4.7. Flood levels upstream of the town may be the result of a large flood in the mainstream with a small tributary flood, or a large flood in the tributary with average flow conditions in the mainstream; more commonly, it might be expected that the downstream levels are a function of different extremes of flooding in both contributing rivers. Flood Frequency Analysis was undertaken on the Annual Maxima Series derived at both gauges, and it was found that a log-Normal distribution provided an adequate fit to both (Figure 4.4.14a). An analysis of the coincident flow maxima at both sites indicated that the correlation between flood peaks was 0.6, and a scatter plot of the historic peaks used to make this inference is shown in Figure 4.4.14 b).

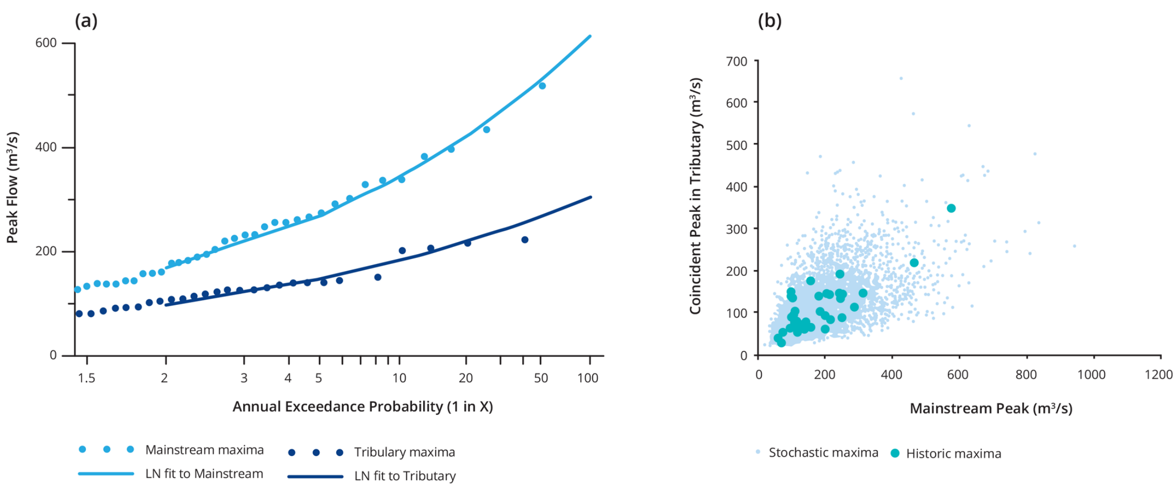


Figure 4.4.14. (a) Flood Frequency Curves for the Mainstream and Tributary gauging sites, and (b) Correlation between Historic Flood Peaks and Sample of Generated Maxima

The first step in the process is to generate the correlated stochastic inputs relevant to the two branches of the stream. This is done using the procedure outlined in Book 4, Chapter 4, Section 3 in conjunction with the inverse transform method (Book 4, Chapter 4, Section 3). The first ten rows of the simulation are shown in Table 4.4.1. Uniform random numbers are provided in Columns 2 and 3, and Columns 3 and 4 show the corresponding values of the inverse cumulative Normal distribution (the standard normal variates). Column 6 shows the correlated value of the standard normal variate, which is obtained from the procedure outlined in Book 4, Chapter 4, Section 3; however, as here a orrelated standard normal variate is generated rather than a correlated uniform variates, the two input variables are *X= N(0,1)* and *Z= N(0,1)*, ie. Columns 4 and 5, not columns 1 and 2. The corresponding maxima in the mainstream and the tributary are shown in Columns 7 and 8, and are obtained by scaling the N(0,1) variates by the relevant means and standard deviation of the log-Normal distribution, e.g. $x = \mu_x + X\sigma_x$ The mean and standard deviation for both streams are shown at the top of the table in Columns 4 and 5, and the results shown in Columns 7 and 8 have been transformed back into the arithmetic domain by taking the anti-log of *x*. The results of applying these steps 5000 times are shown in Figure 4.4.14(b).

Table 4.4.1. Stochastic Generation of Correlated log-Normal Maxima

			Mainstream	Tributary			Intercept	8.06727
		Mean	2.2146	1.9975			a	0.00402
		Std Deviation	0.2194	0.2228			b	0.00156
		Correlation	0.6				N	5000
Column 1	Column 2	Column 3	Column 4	Column 5	Column 6	Column 7	Column 8	Column 9
Count	U_x	U_y	X	Z	Y	Mainstream (m^3/s)	Tributary (m^3/s)	Level (m)
1	0.0608	0.3890	-1.5478	-0.2820	-1.1543	75.0	55.0	8.455
2	0.3928	0.3538	-0.2719	-0.3752	-0.4633	142.9	78.4	8.765
3	0.6415	0.3207	0.3625	-0.4659	-0.1552	196.9	91.4	9.003
4	0.1871	0.9256	-0.8887	1.4438	0.6218	104.6	136.8	8.702
5	0.5970	0.4625	0.2457	-0.0941	0.0722	185.6	103.2	8.975
6	0.6556	0.0662	0.4005	-1.5045	-0.9633	200.7	60.7	8.970
7	0.3334	0.1897	-0.4304	-0.8789	-0.9614	131.9	60.7	8.693
8	0.9805	0.6330	2.0647	0.3399	1.5107	465.2	215.8	10.277
9	0.1692	0.3399	-0.9572	-0.4128	-0.9045	101.1	62.5	8.572
10	0.2268	0.2388	-0.7494	-0.7100	-1.0177	112.3	59.0	8.611

The next step in the process is to derive the deterministic component of the system. To this end, representative flows were input into a HEC-RAS model of the stream and the results levels were obtained. Seven pairs of simulations were undertaken as shown in Figure 4.4.15 and Table 4.4.2. A multiple regression model was fitted to this information, and the resulting relationship is depicted in Figure 4.4.15. This function is used in Column 9 of Table 4.4.1 to obtain the flood level resulting from the stochastic maxima provided in Columns 7 and 8.

A probability plot of the ranked 5000 stochastic flood levels (using the Weibull plotting position formula) is depicted in Figure 4.4.16. The 1% AEP flood level may be found by simple linear interpolation of these results, and is found to be a level of 10.55 m. Also shown in Figure 4.4.16 is the dependence of this estimate on the degree of correlation between the mainstream and tributary peaks, where it is seen that if the peaks are assumed to be fully independent or dependent the flood level estimate varies between 10.40 and 10.73 m, respectively.

It is worth noting that trials were undertaken to determine how many simulations were required to yield stable estimates of the quantiles. In this example, there was no difference in results if 1000 or 5000 simulations were used, though below this number the estimates started to become unstable.

Table 4.4.2. Derivation of Deterministic Function Relating Upstream Flows to Downstream Levels (a)

Peak in Mainstream (m^3/s)	Peak in Tributary (m^3/s)	Flood Level (m)
248.1	286.0	9.54
320.0	283.2	9.75
393.6	274.1	10.05
424.8	260.8	10.22
444.6	242.1	10.33
458.7	196.0	10.12
464.4	0.1	9.95

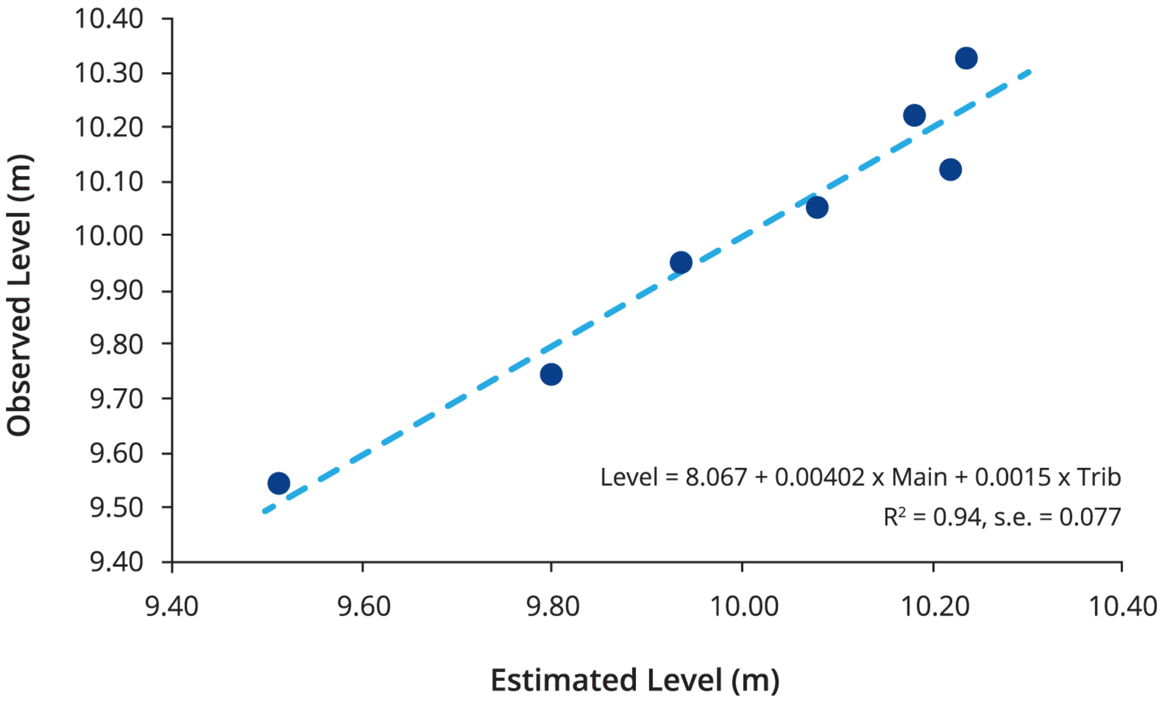


Figure 4.4.15. Derivation of Deterministic Function Relating Upstream Flows to Downstream Levels

Lastly, an estimate of the exceedance probability can be obtained using stratified sampling and use of the Total Probability Theorem. To this end, the probability domain was divided into 10 divisions, and 20 simulations were undertaken in each (totalling 200 simulations). The boundaries of the ten divisions are shown in Columns 2 and 3 of Table 4.4.3, where the limits have been uniformly distributed between standard normal variates of 1 and 4. The calculations are undertaken as described in Book 4, Chapter 4, Section 3 for the level threshold of 10.4 m, where the conditional probability terms are based on the exceedance probability of flows in the mainstream. The probability of an event occurring in each of the ten bins is shown in Column 4, and this is determined from the exceedance probabilities associated with each of the bins. For example, the probability that a flow in the mainstream lies within the first bin is simply the difference between 0.90320 and 0.84134 (= 0.06185), which are the probabilities of the normal distribution that correspond to the standard normal variates of 1.00 and 1.30. The number of times that a level exceeds 10.4 m in each bin is given in Column 5, and the corresponding conditional probability is shown in Column 6, which is computed by dividing by the number of samples in each bin (which in this case is 20). The product of the conditional probability term (Column 6) and the interval width (Column 4) is given in Column 7, and the summation is provided at the bottom of the table. It is thus seen that the exceedance probability of exceeding 10.4 m is estimated to be 0.0149 (or around 1 in 70). A comparison between three such estimates and the results obtained from simple simulation is shown in Figure 4.4.16, from which is seen that the results obtained are similar.

Table 4.4.3. Calculation of Exceedance Probability of the Level Exceeding 10.4 m using the Total Probability Theorem

Column 1	Column 2	Column 3	Column 4	Column 5	Column 6	Column 7
Bin	Z_{min}	Z_{max}	$p[M_i]$	Num [H>h]	$p[H>h\|M_i]$	$p[H>h\|M_i]*p[H>h]$
1	1.00	1.30	0.061855	0	0.00	0.000000
2	1.30	1.60	0.042001	0	0.00	0.000000
3	1.60	1.90	0.026083	0	0.00	0.000000
4	1.90	2.20	0.014813	4	0.20	0.002963
5	2.20	2.50	0.007694	15	0.75	0.005770
6	2.50	2.80	0.003655	20	1.00	0.003655
7	2.80	3.10	0.001588	20	1.00	0.001588
8	3.10	3.40	0.000631	20	1.00	0.000631
9	3.40	3.70	0.000229	20	1.00	0.000229
10	3.70	4.00	0.000076	20	1.00	0.000076
						0.014911

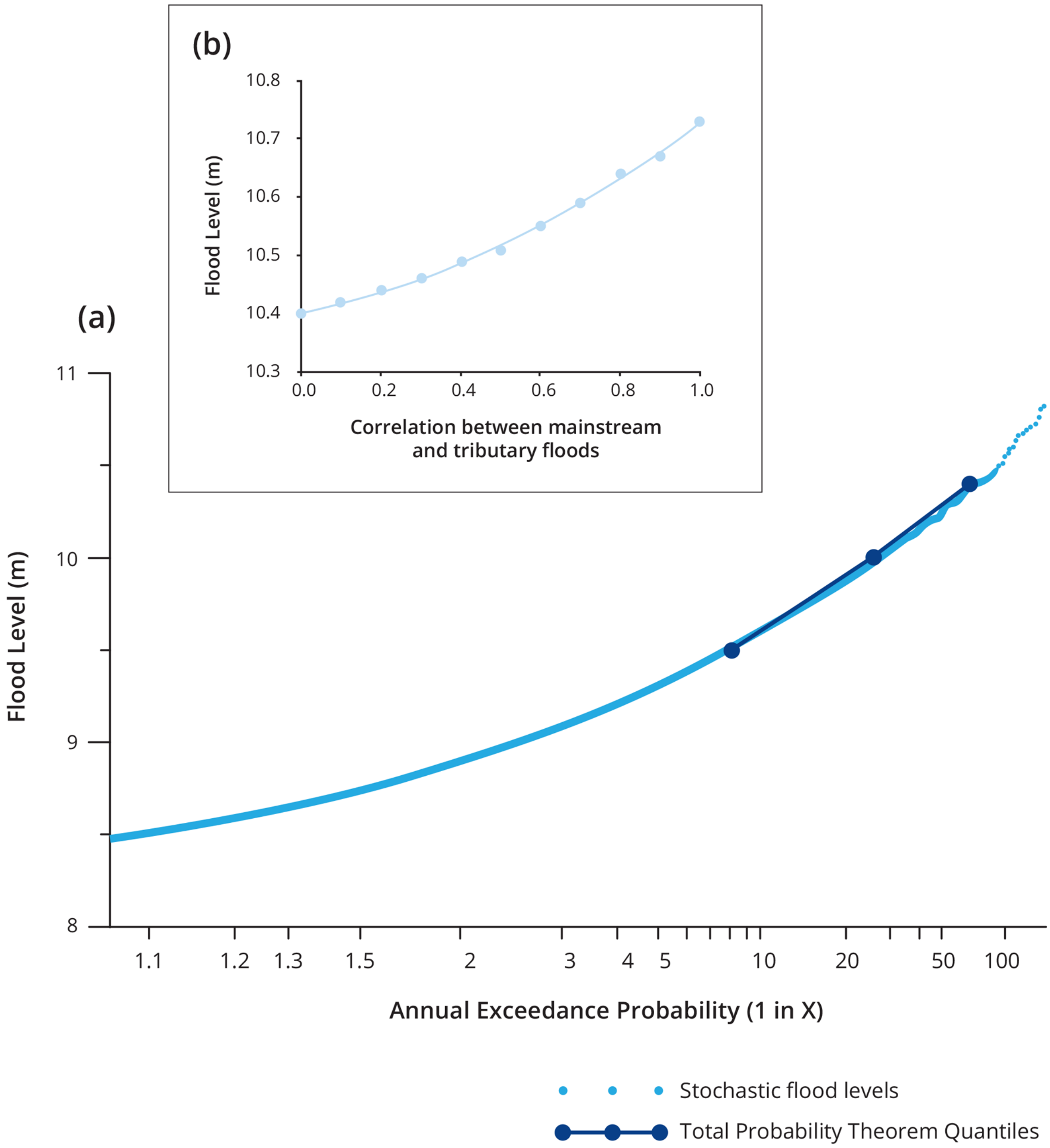


Figure 4.4.16. Derived Frequency Curve of Downstream Levels, with (b) Dependence of 1% Annual Exceedance Probability Level on Degree of Correlation between Flood Peaks

4.5. References

Benjamin, J.R. and Cornell, C.A. (1970), Probability, Statistics and Decisions for Civil Engineers. McGraw-Hill, New York.

Box, G.E.P. and Cox, D.R. (1964), An Analysis of Transformations, J Royal. Statistical Society, 211-243, discussion 244-252.

Burgman, M. (2005), Risks and Decisions for Conservation and Environmental Management. Cambridge University Press.

Chen, L., Singh, V.P., Shenglian, G., Hao, Z. and Li, T. (2012), Flood Coincidence Risk Analysis Using Multivariate Copula Functions, J. Hydrol. Eng., pp: 742-755, Diermanse, F.L.M., Carroll, D.G., Beckers, J.V.L., Ayre, R. and Schuurmans, J.M. (2014), A Monte Carlo framework for the Brisbane river catchment flood study. Hydrology and Water Resources Symposium 2014. Barton, ACT: Engineers Australia.

Favre, A.C., El Adlouni, S., Perreault, L., Thiémonge, N. and Bobée, B. (2004), Multivariate hydrological frequency analysis using copulas. Water Resour. Res., 40(1).

Fricke, T.J., Kennedy, M.R. and Wellington, N.B. (1983), The Use of Rainfall Correlation in Determining Design Storms for Waterways on a Long Railway Line. In: Hydrology and Water Resources Symposium (15th : 1983 : Hobart, Tas.). National conference publication (Institution of Engineers, Australia), (83/13), 215-219.

Genest, C., and Favre, A. (2007), Everything You Always Wanted to Know about Copula Modeling but Were Afraid to Ask. Journal of Hydrologic Engineering, 12(4), 347-368.

Haan, C.T. (1974), Statistical Methods in Hydrology, Second Edition. Iowa State Press.

Hammersley, J.M. and Handscomb, D.C. (1964), Monte Carlo Methods, Methuen and Co, London; John Wiley and Sone, New York.

Hill, P.I., Graszkiewicz, S., Loveridge, M., Nathan, R. and Scorah, M. (2015), Analysis of loss values for Australian rural catchments to underpin ARR guidance. Proc. 36th Hydrology and Water Resources Symposium, Hobart.

Jordan, P., Nathan, R., Weeks, B., Waskiw, P., Heron, A., Cetin, L., Rogencamp, G., Stephens, C. and Russell, C. (2015), Estimation of flood risk for linear transport infrastructure using continuous simulation modelling. Proc. 36th Hydrology and Water Resources Symposium, Hobart.

Kilgore, R.T., Thompson, D.B. and Ford, D.T. (2010), Estimating Joint Probabilities of Design Coincident Flows at Stream Confluences. National Cooperative Highway Research Program Report 15-36, Washington D.C.

Kuczera, G., Lambert, M., Heneker, T., Jennings, S., Frost, A., Coombes, P. (2003), Joint probability and design storms at the crossroads. Proc, 28th Int. Hydrology and Water Resour. Symp. 10-14 Nov, Wollongong.

Laurenson, E.M. (1974), Modeling of stochastic-deterministic hydrologic systems. Water Resources Research, 10(1), 955-961.

Maidment, D.R. (Ed) (1993). Handbook of Hydrology, McGraw-Hill.

Nathan, R.J. and Bowles, D.S. (1997), A probability-neutral approach to the estimation of design snowmelt floods. Hydrology and Water Resources Symposium: Wai-Whenua, November 1997, Auckland, pp: 125-130.

Press, W.H., Teukolsky, S.A., Vetterling, W.T., Flannery, B.P. (1993), Numerical recipes in Fortran: the art of scientific computing. Cambridge University Press.

Robinson, K., Ling, F., Peterson, J., Nathan, R., Tye, I. (2012), Application of Monte Carlo Simulation to the Estimation of Large to Extreme Floods for Lake Rowallan Proceedings of the 34th Hydrology & Water Resources Symposium. ISBN 978-1-922107-62-6. Engineers Australia.

Saucier, R. (2000), Computer generation of statistical distributions. Army Research Laboratory ARL-TR-2168.

Scorah, M., Lang, S., and Nathan, R. (2015), Utilising the AWAP gridded rainfall dataset to enhance flood hydrology studies. Proc. 36th Hydrology and Water Resources Symposium, Hobart.

Vose, D. (2000), Risk analysis: a quantitative guide. John Wiley and Sons Ltd, Chichester.

Weinmann, P.E. and Nathan, R.J. (2004), The continuing challenge of estimating extreme floods from extreme design storms. Advances in Hydro-Science and Engineering, Volume VI, Proc. of the 6th Intern. Conf. on Hydro-Science and Engineering, Brisbane May 31 - June 3, 2004.

Weinmann, P.E., Rahman, A., Hoang, T.M.T., Laurenson, E.M. and Nathan, R.J. (2002), Monte Carlo Simulation of Flood Frequency Curves from Rainfall - The Way Ahead. Australian Journal of Water Resources, 6(1), 71-79.

Westra, S. (2012), Australian Rainfall and Runoff - Revision Project 18: Interaction of Coastal Processes and Severe Weather Events. ARR Report Number P18/S2/010, Engineers Australia.

Zheng, F., Westra, S., Sisson, S. and Leonard, M. (2014), Flood risk estimation in Australia's coastal zone: modelling the dependence between extreme rainfall and storm surge. Hydrology and Water Resources Symposium, Perth 2014.

BOOK 5

Flood Hydrograph Estimation

Flood Hydrograph Estimation

Table of Contents

1. Introduction 1
    1.1. Flood Hydrograph Modelling 1
        1.1.1. Overall Flood Hydrograph Estimation Process 1
        1.1.2. Conceptual Representation of Flood Formation 2
    1.2. Scope 5
    1.3. Book Contents 5
2. Catchment Representation 6
    2.1. Introduction 6
    2.2. Lumped Models 7
    2.3. Semi-Distributed (Node-Link Type) Models 8
        2.3.1. Conceptual Model Elements 11
        2.3.2. Catchment Sub-Division 11
        2.3.3. Modelling of Runoff from 'Hill Slopes' (Overland Flow) 12
        2.3.4. Routing Through Network of Stream/Channel/Floodplain Elements 12
        2.3.5. Routing Trough Special Storages 14
    2.4. Grid-based (Distributed) Models 14
    2.5. References 15
3. Losses 16
    3.1. Introduction 16
    3.2. Conceptual Loss Models 16
        3.2.1. Loss Processes 16
        3.2.2. Types of Loss Model 19
        3.2.3. Empirical Models 19
        3.2.4. Simple Models 23
        3.2.5. Process Methods 26
    3.3. Approach to Selection of Loss Model and Values 26
        3.3.1. Selection of loss model 26
        3.3.2. Design Rainfall Bursts v Complete Storms 28
        3.3.3. Approaches to Estimating Loss Values 29
    3.4. Estimation of Effective Impervious Area 32
        3.4.1. Overview 32
        3.4.2. Estimating Effective Impervious Area 35
        3.4.3. Additional Considerations 48
    3.5. Regional Loss Information 49
        3.5.1. Introduction 49
        3.5.2. Rural Catchments 49
        3.5.3. Urban Catchments 57
    3.6. Distribution of Loss Values 67
        3.6.1. Non Parametric Approaches 67
        3.6.2. Parametric Approaches 70
        3.6.3. Correlation between Initial and Continuing Loss 70
    3.7. Other Considerations for Selecting Loss Values 71
        3.7.1. Variation of Loss with Event AEP 71
        3.7.2. Reduction of Continuing Loss for Long Events 72
        3.7.3. Influence of Timestep on the Estimation of Continuing Loss 73
        3.7.4. Antecedent Rainfall and Soil Moisture 76
        3.7.5. Seasonality 79
        3.7.6. Influence of Vegetation 82
        3.7.7. Interaction with Routing Parameters 83
        3.7.8. Influence of Snowpack 84

3.7.9. Link to Climate Drivers and Change ... 84
3.8. Appendix ... 85
3.9. References ... 92
4. Baseflow Models ... 99
4.1. Introduction ... 99
4.2. Guiding Principles ... 100
4.3. Baseflow Characteristics ... 101
4.4. Selection of Approach ... 102
4.4.1. Preliminary Assessment of Baseflow ... 103
4.4.2. Suitability of Stream Flow Data ... 104
4.5. Quantifying Baseflow Contribution to Design Flood Estimates ... 105
4.5.1. Estimating Baseflow Using Streamflow Data ... 105
4.5.2. Estimating Baseflow in the Absence of Streamflow Data ... 106
4.5.3. Generating the Total Streamflow Hydrograph ... 110
4.6. Example ... 111
4.6.1. North Maroochy River at Eumundi, Queensland ... 112
4.6.2. Dirk Brook, Western Australia ... 114
4.7. Appendix - Calculation of the Timing of the Baseflow Peak ... 117
4.8. References ... 118
5. Flood Routing Principles ... 120
5.1. Introduction ... 120
5.2. Fundamental Concepts ... 120
5.3. Hydrograph Translation (Lag) ... 122
5.4. Storage Routing ... 123
5.4.1. Basic Equation ... 123
5.4.2. Reservoir (Level Pool) Routing ... 123
5.4.3. Muskingum Hydrologic Routing ... 126
5.4.4. Muskingum-Cunge Storage Routing ... 134
5.4.5. Limitations of the Muskingum Method ... 138
5.5. Non-linear Storage Routing ... 138
5.5.1. Introduction ... 138
5.5.2. Different Forms of Non-linearity ... 139
5.5.3. Non-linear Distributed Storage ... 141
5.6. Hydraulic Routing Approaches ... 143
5.6.1. Introduction ... 143
5.6.2. Kinematic and Diffusion Wave Routing ... 144
5.7. References ... 145
6. Flood Hydrograph Modelling Approaches ... 147
6.1. Introduction ... 147
6.2. Time-Area approaches ... 148
6.2.1. Time-Area Theory ... 148
6.2.2. Limitations of Time-Area Approaches ... 151
6.2.3. Worked Example ... 151
6.3. Unit Hydrograph Approaches ... 154
6.3.1. Introduction ... 154
6.3.2. Unit Hydrograph Theory ... 154
6.3.3. Calculating Direct Runoff Hydrographs Using Unit Hydrographs ... 156
6.3.4. Derivation of Unit Hydrographs from Rainfall and Streamflow Data ... 158
6.3.5. Synthetic Unit Hydrographs ... 160
6.3.6. Limitations of the Unit Hydrograph Approach ... 161
6.4. Runoff Routing Approaches ... 162
6.4.1. Introduction ... 162

6.4.2. Representing the Distributed Nature of Flood Formation 163
6.4.3. Modelling of Hillslope (Overland Flow) Phase 170
6.4.4. Routing Through Channel, Stream and Floodplain Reaches 175
6.4.5. Routing Through Special Storages 177
6.5. Rainfall on Grid Modelling Approaches 178
6.5.1. Introduction 178
6.5.2. Modelling of Runoff from Individual Cells 179
6.5.3. Advantages and Limitations of the Rainfall-on-Grid Approaches 180
6.6. References 181

List of Figures

5.1.1. Conceptual Representation of Flood Formation Processes in the Most Commonly used Event-based Flood Hydrograph Estimation Models (courtesy R Nathan) 4

5.2.1. Conceptual Representation of the Runoff Routing Process in a Node-Link Type Model 10

5.3.1. Physical Processes which Contribute to Rainfall Loss 17

5.3.2. Initial Loss – Constant Continuing Loss Model 20

5.3.3. Initial Loss - Proportional Loss Model 21

5.3.4. The model structure of the Australian Representative Basins model based on the work of Chapman (1968). Diagram obtained from Black and Aitken (1977) and Mein and McMahon (1982). 25

5.3.5. Distinction between Storm and Burst Initial Loss 28

5.3.6. Example of a Directly Connected Impervious Surface (Left) and an Indirectly Connected Impervious Surface (Right) 32

5.3.7. Schematic of Rainfall Depth v Runoff, from Boyd et al. (1993) 34

5.3.8. Cumulative Discharge Plot from Giralang (ACT), showing Cumulative Rainfall, Observed Runoff and Estimated EIA runoff (Phillips et al, 2014) 35

5.3.9. Overview of Regression Analysis Approach 36

5.3.10. Example Regression Analysis for Albany Drain Catchment in Western Australia ... 38

5.3.11. Representation of Dayaratne (2000) Relationship for DCIA 41

5.3.12. Re-Analysis of Boyd et al. (1993) Data 42

5.3.13. Australian data from Phillips et al (2014) and Boyd et al. (1993) 42

5.3.14. Compilation of Available EIA Data 45

5.3.15. Example Sample Area Analysis for Residential and Commercial Land Use for the Giralang Catchment (ACT) 47

5.3.16. Regions Adopted for Loss Prediction Equations 50

5.3.17. Seasonality of Average Gridded Soil Moisture in Each Defined Region (Using Gridded Data) 51

5.3.18. Recommended Median IL_s (mm) 54

5.3.19. Recommended Median CL (mm/hr) 55

5.3.20. Storm Magnitude from Phillips et al (2014) 59

5.3.21. Summary of Initial Losses for Urban Catchments (from Phillips et al (2014)) 60

5.3.22. Indirectly Connected Area Continuing Loss Estimates (from Phillips et al (2014)) 62

5.3.23. Indirectly Connected Area Proportional Loss Estimates (from Phillips et al (2014)) 64

5.3.24. Horton Loss Model with Different Soil Classifications & AMC Numbers 66

5.3.25. Variation in Location but Not Shape of Initial Loss Distribution Nathan et al. (2003) 68

5.3.26. Regional average standardised loss distributions (Hill et al., 2014a) 69

5.3.27. Correlation between 1 hr Peak Rainfall Intensity and Continuing Loss (left Giralang (ACT) and right Powells Creek (NSW)) (from Phillips et al (2014)) 72

5.3.28. Example of Continuing Loss Varying with Modelling Timestep (February 1977 event at Currambene Creek at Falls Creek in NSW) 74

5.3.29. Variation of Continuing Loss with Modelling Timestep for Rural Catchments 75

5.3.30. Relationships of Urban Continuing Loss with Timestep 76

5.3.31. Proportion of Variance Explained (r^2) between Storm Initial Loss and API (Hill et al., 2014a) 77

5.3.32. Plot of Antecedent Rainfall Versus Initial Loss for Indirectly Connected Area (Phillips et al, 2014) 78

5.3.33. Seasonal Distribution of Events Analysed by Hill et al. (2014a) 80

5.3.34. Seasonality of Standardised Storm Initial Loss Values for Different Regions in Australia 81

5.3.35. Regional Runoff Coefficient Curves for South-West Western Australia (Pearce, 2011) 82

5.3.36. Comparison of Adopted Routing Parameters for IL/CL and SWMOD 84

5.4.1. Key Characteristics for Calculation in a Flood Hydrograph 102

5.4.2. Decision Tree for Method to Estimate Baseflow Contribution to Design Flood 103

5.4.3. Preliminary Assessment of Baseflow Peak Factor for a 10% AEP Event 105

5.4.4. AEP Factors, F_{AEP} to be applied to the 10% AEP Baseflow Volume Factor to determined the Baseflow Volume Factor for events of various AEPs 108

5.4.5. Map of Baseflow Peak Factor for a 10% AEP 109

5.4.6. Map of Baseflow Volume Factor for a 10% AEP 109

5.4.7. Total flow hydrograph generation approach, where (a) the data values calculated through the baseflow estimation process are plotted; (b) linear interpolation between the baseflow data points and matching the area under the curve to the baseflow event volume is used to estimate the baseflow time series, which is plotted on the hydrograph in green; and (c) the total streamflow time series is generated by summing the surface runoff and baseflow time series values, with the streamflow hydrograph plotted in dark blue. . 111

5.4.8. Streamflow Hydrograph Approximating the 10% AEP Event for the North Maroochy River at Eumundi 112

5.4.9. Surface Runoff Hydrograph for the 10% AEP Design Flood event at Eumundi 113

5.4.10. Surface Runoff, Baseflow and Total Streamflow Hydrographs for the 10% AEP Event at Eumundi 114

5.4.11. Surface Runoff Hydrograph for the 1% AEP Design Flood Event at Dirk Brook ... 115

5.4.12. Surface Runoff, Baseflow and Total Streamflow Hydrographs for the 1% AEP Event at Dirk Brook 117

5.4.13. Comparison of Average Time to Surface Runoff Peak and Time to Baseflow Peak, Based on Analysis of more than 30,000 Flood Events from Catchments across Australia. 118

5.5.1. Effects of Storage on Transforming Inflow Hydrograph 121

5.5.2. Effects of Reservoir on Transforming Inflow Hydrograph 124

5.5.3. Prism Storage and Wedge Storage in a River Reach on the Rising Limb of the Hydrograph 127

5.5.4. Werribee River Example – Inflow Hydrograph and Observed and Calculated Outflow Hydrographs 130

5.5.5. Werribee River Example – Impact of Routing Through Concentrated Storage (X = 0) or Fully Distributed Storage (X = 0.5) 131

5.5.6. Graphical Estimation of X and K (after Laurenson (1998)) 133

5.5.7. Werribee River Example – Application of the Kalinin-Miljukov Method (Two Routing Reaches of 10 km Length) 137

5.5.8. Storage-Discharge Relationships with Different Degrees of Non-linearity 141

5.5.9. Effect of Non-linearity of Storage-Discharge Relationship on Routed Hydrographs (after Pilgrim (1987)) 142

5.6.1. Isochrones and Time-Area Curve 149

5.6.2. Time-Area Relationship for Example 2 152

5.6.3. Calculation of Surface Runoff Hydrographs using Time-Area Approach 154

5.6.4. Unit Hydrograph Calculation – example with 3 periods of rainfall excess and unit hydrograph with 5 ordinates (after Laurenson (1998)). 157

5.6.5. Poor Averaging of Unit Hydrographs 160

5.6.6. (a) Clark and (b) Nash models of runoff routing 164

5.6.7. Clark Runoff Routing Example- resulting hydrographs for the five different durations 165

5.6.8. Original Laurenson Runoff Routing Model (South Creek catchment) (a) Isochrones of storage delay time (b) Time-area diagram 166

5.6.9. Node-link type representation of a catchment in runoff routing models: map view and schematic representation of node-link network in RORB and WBNM 167

5.6.10. Components of a runoff routing model 169

5.6.11. Hillslope representation in kinematic wave routing models (a) actual catchment (b) model representation (from HEC-HMS Manual) 174

5.6.12. Routing of rainfall excess hydrograph through a series of nonlinear storages (after Laurenson et al. (2010)) 177

5.6.13. Conceptualisation of generation of runoff hydrograph from a grid cell 179

List of Tables

5.3.1. Summary of Different Approaches for Estimating Loss Values 30
5.3.2. Summary of Effective Impervious Areas Results 39
5.3.3. Overview of International Literature on EIA/TIA 43
5.3.4. EIA Results of Alley and Veenhuis (1983) for 19 Catchments in Denver, as summarised in Shuster et al. (2005) 44
5.3.5. Range of Values Used in developing IL_s Prediction Equations 53
5.3.6. Range of Values Used in Developing CL Prediction Equations 53
5.3.7. Total Storms identified for Analysis Phillips et al (2014) 58
5.3.8. Comparison of Initial Loss Literature Values with Rural Recommended Values 61
5.3.9. Urban Continuing Loss Values Compared with Rural Continuing Loss Values 62
5.3.10. Soil Classifications in Horton Model 65
5.3.11. Antecedent Moisture Condition Number 65
5.3.12. Horton Loss Model Parameters 66
5.3.13. Standardised Loss Factors (Hill et al., 2014a) 70
5.3.14. Median Loss Values for Rural Catchments 85
5.3.15. Summary of EIA Results from (Boyd et al., 1993) 90
5.3.16. EIA Initial Loss Estimates from Various Studies 91
5.4.1. AEP Scaling Factors, F_{AEP}, to be applied to the 10% AEP Baseflow Peak Factor and the Baseflow Volume Factor to determine the Baseflow Peak Factor for events of various AEPs 107
5.4.2. Key Surface Runoff Characteristics for the 10% AEP Design Flood Event at Eumundi 113
5.4.3. Key Surface Runoff Characteristics for the 1% AEP Design Flood Event at Dirk Brook 115
5.4.4. Calculation of Baseflow Factors for the 1% AEP Design Event for the Dirk Brook 115
5.4.5. Calculation of Baseflow and Total Streamflow Characteristics for the 1% AEP Event for the Dirk Brook Catchment 116
5.5.1. Calculations for the Muskingum Routing Example 129
5.6.1. Conversion Factors 151
5.6.2. Calculation of Surface Runoff Hydrograph using Triangular Time-Area Curve 153
5.6.3. Methods used in different runoff routing modelling systems to derive the overland flow hydrograph 171
5.6.4. Routing models used in different runoff routing modelling systems to route flows through channel, stream and floodplain reaches 175

Chapter 1. Introduction

James Ball, Erwin Weinmann

Chapter Status	Final
Date last updated	14/5/2019

1.1. Flood Hydrograph Modelling

From the alternative flood estimation approaches introduced in Book 1, Chapter 3, the methods and models covered in this book of Australian Rainfall and Runoff focus on the event-based simulation approach. This approach simulates only the time period covering a single storm event, given the initial conditions for the event, but the storm may consist of several separate rainfall bursts, resulting in a multi-peaked flood hydrograph.

The more general aspects of catchment simulation for design flood estimation are covered in Book 4, and the chapters in this book deal specifically with the models and design inputs required to transform the event-based design rainfall inputs from Book 2 into design flood hydrographs at catchment locations of interest.

This book is an extension of the material covered in Book 3, which deals with the calculation of design flood peak discharges. While peak discharges (without flood hydrographs) are adequate for many applications, such as calculating bridge or culvert capacity, flood hydrographs are essential for many other applications. These applications include those where floodplain storage or artificial storage is an important issue or where the movement and modification of flood events through a catchment is of interest. With the increasing implementation of more advanced hydrological modelling systems and more complex analysis requirements, guidance on flood hydrograph modelling is becoming increasingly important.

The flood hydrograph methods described here provide an alternative method to the flood peak discharge methods covered in Book 3 and allow cross checking between the two methods. There is a place therefore for both peak flow and flood hydrograph estimation for different applications.

1.1.1. Overall Flood Hydrograph Estimation Process

The process of developing and applying an event-based flood hydrograph estimation model involves the following steps:

1. Definition of the flood estimation problem and the model requirements;

2. Assessment of data requirements and data availability, data collation and checking;

3. Study of catchment data and flood information to develop an understanding of the catchment behaviour during floods and to identify important features that need to be represented in the model - Book 5, Chapter 2;

4. Conceptualised representation of the runoff generation phase of flood formation (loss model and baseflow model) - Book 5, Chapter 3 and Book 5, Chapter 4;

5. Conceptualised representation of the flood hydrograph formation phase (the routing elements of the catchment) - Book 5, Chapter 2, Book 5, Chapter 5 and Book 5, Chapter 6;

6. Determination of model parameters by calibration to observed events, from experience values in regions with similar flood producing characteristics or from links with measured catchment characteristics - Book 7, Chapter 5;

7. Validation of the calibrated model to ensure that it is fit for the intended purpose – Book 7, Chapter 6;

8. Application of the model with design rainfalls (Book 2), design losses (Book 5, Chapter 3) and design baseflows (Book 5, Chapter 4) to estimate design flood hydrographs - Book 7, Chapter 7;

9. Interpretation and presentation of model results, including determination of uncertainty – Book 7, Chapter 9 and Book 7, Chapter 8; and

10. The modelled design flood hydrographs will generally form the inputs to a hydraulic model of the study area.

The following chapters of Book 5 introduce the important hydrologic modelling principles that are applied in Steps 3 to 5 of the overall process. Book 5, Chapter 3 (Losses) and Book 5, Chapter 4 (Baseflow Models) also provide guidance on the design values required in Step 7. Detailed application guidance relating to the other steps is provided in Book 7.

1.1.2. Conceptual Representation of Flood Formation

The complex hydrologic processes involved in the formation and modification of flood hydrographs are represented in flood hydrograph estimation models in a highly conceptualised form. The processes involved in the *runoff generation phase*, described in more detail in Book 4, Chapter 2, are represented in *conceptual loss models* (Book 5, Chapter 3, Section 2) in a simplified fashion. These conceptual loss models divide the rainfall inputs into rainfall excess and loss (without modelling what happens with the loss component). As the name implies, the rainfall excess reflects only the surface runoff component that is directly attributable to the event rainfall. The additional component of streamflow originating from recession flows from previous rainfall events or groundwater inflows is referred to as baseflow. This baseflow component is represented in *conceptual baseflow models* (Book 5, Chapter 4). Baseflow contributions to runoff are added to the rainfall excess component either at the sub-catchment scale or, more commonly, as a total baseflow hydrograph at the catchment outlet.

In the *hydrograph formation phase*, the routing of flood contributions from subareas through the various stream reaches, floodplains and natural or artificial storages is modelled by hydrologic or hydraulic routing models of different complexity (Book 5, Chapter 5).

Some flood hydrograph modelling approaches represent the catchment only as a single unit (lumped models). However, the models now typically applied in the event-based simulation approach are semi-distributed in nature; they represent the catchment being modelled by a number of sub-catchments or subareas, where the degree of spatial resolution used typically varies between around 10 to 100 subareas. The processes involved in the *runoff generation phase* are modelled at the sub-catchment or subarea scale, and the resulting runoff hydrographs are then routed along the different stream reaches and storages in the catchment to the point of interest. Node-link type runoff-routing models are the most common form of these models, where the nodes represent the subareas and stream junctions, and the links the routing reaches (Book 5, Chapter 6). In addition to providing a more detailed and physically based approach to hydrological modelling, distributed models allow the assessment of flood hydrographs for points within the main catchment as well as at the outlet, whereas lumped models allow calculation of hydrographs only at the catchment outlet.

Figure 5.1.1 depicts a schematic representation of how the flood formation processes are conceptualised in event-based flood hydrograph estimation models.

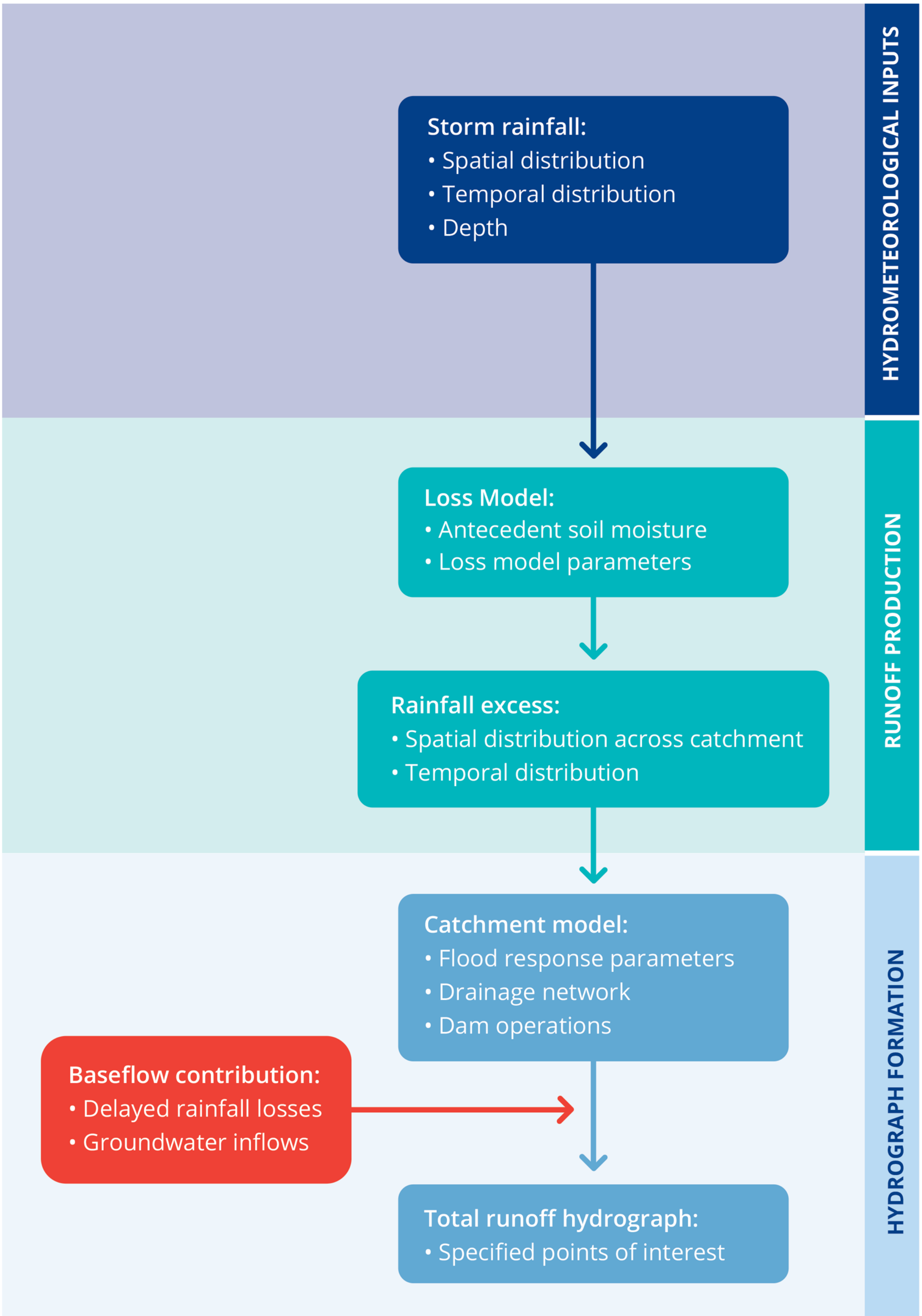


Figure 5.1.1. Conceptual Representation of Flood Formation Processes in the Most Commonly used Event-based Flood Hydrograph Estimation Models (courtesy R Nathan)

1.2. Scope

This book of Australian Rainfall and Runoff provides background information on the basic elements that make up event-based flood hydrograph estimation models, and an overview of the modelling systems most commonly used in Australia. This introductory information is intended to equip practitioners with a clearer understanding of the simplifications and assumptions involved in different model components. Book 5, Chapter 3 and Book 5, Chapter 4 also give guidance on the design loss and baseflow inputs for use with event-based flood hydrograph estimation models. Detailed guidance on other aspects of applying these models to practical flood hydrograph estimation problems is provided in Book 7.

As in other books, the guidance provided here should not be interpreted as being prescriptive, as unusual catchment conditions may require special considerations. Importantly, the application of the models and design data for flood hydrograph estimation should be informed by a good understanding of general hydrologic principles and concepts relevant to flood estimation as well as specific interpretation of local flood data.

1.3. Book Contents

After this introductory chapter, Book 5, Chapter 2 introduces the basic concepts and approaches used in representing catchments for event-based modelling of floods. Book 5, Chapter 3 gives details of the loss models applied in generating surface runoff, and the models used to represent the contribution of baseflow are dealt with in Book 5, Chapter 4. Both chapters are based on research undertaken as part of the ARR Revision Projects funded by the Commonwealth of Australia. The chapters give the background to the selection of the adopted models, then describe the sources of information for the derivation of design values of losses and baseflow, and finally provide guidance for the practical application of loss models and baseflow models. Book 5, Chapter 5 introduces important flood routing principles applied in modelling the movement of flood hydrographs through the stream and floodplain system, linking them with the hydraulic principles covered in Book 6. Finally, Book 5, Chapter 6 describes the most important conceptualisations and approaches used in runoff-routing modelling systems to derive complete flood hydrographs at points of interest.

Chapter 2. Catchment Representation

Erwin Weinmann

Chapter Status	Final
Date last updated	14/5/2019

2.1. Introduction

The representation of a catchment in a flood hydrograph estimation model is, by necessity, highly conceptualised and aims to represent those features and characteristics that are most influential in determining the overall flood response of the catchment. The distribution of storm rainfall over different parts of the catchment and the flood response to it may vary considerably, depending on the details of topography, vegetation cover, land use and drainage network characteristics. However, as the hydrograph inputs from different parts of the catchment are progressively combined on their way to the catchment outlet, only some of these differences in response characteristics are directly reflected in the combined hydrographs at downstream points of interest. Different catchment modelling approaches have therefore evolved to find an appropriate compromise between required model complexity and spatial resolution on the one hand, and desirable modelling efficiency on the other. These different modelling approaches can be applied with different degrees of complexity, and often more simple methods may be quite appropriate.

As is explained in more detail in Book 5, Chapter 5 and Book 5, Chapter 6, the various forms of *temporary flood storage* available in different parts of the catchment play a key role in determining how the runoff inputs from different parts of the catchment are transformed into the flood hydrograph at the catchment outlet. The effect of catchment storage on the routing of hydrographs is twofold and involves:

i. *translation* of the hydrograph peak and other ordinates forward in time and

ii. *attenuation* of the peak as the hydrograph moves along the stream network.

The different catchment representations in flood hydrograph estimation models can therefore be classified on the basis of how the different forms of temporary flood storage are conceptualised and in how much detail they are represented in the model. Other factors such as losses, which determine the flood volume (covered in Book 5, Chapter 3) and baseflow, which may modify the flood hydrograph shape and volume (covered in Book 5, Chapter 4), must also be a part of the modelling of flood hydrographs.

In situations where the interest is only on the combined hydrograph at the catchment outlet and where good flood records for current conditions are available at that point, modelling of the catchment as a single 'lumped' response unit may be sufficient. This is the approach adopted by a number of traditional flood hydrograph estimation methods such as the unit hydrograph approach, the time-area approach and the Clark and Nash models of runoff-routing. In these modelling approaches, discussed further in Book 5, Chapter 2, Section 2, the rainfall excess input is 'lumped' for the whole catchment and then transformed by some routing method to a hydrograph at the catchment outlet.

For catchments with more complex runoff production and flood hydrograph formation characteristics it is more usual to adopt a semi-distributed rather than a lumped modelling approach. Adoption of a semi-distributed approach allows the key factors that influence flood response to be represented in a spatially explicit fashion. Common factors represented in a semi-distributed manner include rainfall, losses and routing parameters, though such models also facilitate the representation of influential catchment features that control variation in the timing and/or magnitude of flood runoff from different parts of the catchment. The category of semi-distributed models (or node-link type models), dealt with in Book 5, Chapter 2, Section 3, allows for an appropriate matching of spatial model resolution with the degree of spatial variation of catchment characteristics and inputs.

Finally, developments in computing power and the availability of digital terrain information now allow a fully distributed representation of catchments in grid-based models. The emerging rainfall-on-grid modelling approach is discussed in Book 5, Chapter 2, Section 4.

Many of the following considerations on how to represent catchments in hydrologic flood estimation models apply to both rural and urban catchments. However, modelling of urban catchments is treated in more detail in Book 9.

2.2. Lumped Models

In relatively small catchments (or within each sub-catchment of a larger catchment) there is often only limited spatial variation in rainfall and loss characteristics, and it is thus acceptable to treat the catchment (or sub-catchment) as a homogeneous unit. Models that do not allow for spatial variation in runoff or routing characteristics within a catchment are referred to as 'lumped' models. It is possible to link the outputs of several lumped models to form a quasi-distributed catchment model.

The peak flow estimation methods described in Book 3 can be regarded as "lumped" type models, since the flood peaks are calculated at a single point only, without the internal characteristics of the catchment being considered.

In the runoff generation phase of lumped models, the conceptual loss and baseflow models described in Book 5, Chapter 3 and Book 5, Chapter 4 can be applied using the assumption that the rainfall inputs, losses and baseflow contributions are the same over the whole catchment. Such a simplifying assumption may be appropriate when rainfall and streamflow data are only available from a single gauge and it is thus difficult to infer any internal variation of the rainfall and runoff generation characteristics.

In the hydrograph formation or routing phase of lumped models, a range of conceptualisations may be used to represent the hydrograph translation and attenuation effects of the catchment on the runoff hydrograph input. These conceptualisations may aim to represent physical catchment processes, but they are essentially 'black box' mathematical representations. The different lumped flood modelling approaches include:

1. *The Time-Area Approach* (Book 5, Chapter 6, Section 2) - in which the different degree of translation (time lag) experienced by runoff from different parts of the catchment is modelled by dividing the catchment into a number of areas with the same delay time to the catchment outlet ('isochronal areas'). The runoff inputs to the different sub-areas are then lagged accordingly to represent the translation effects of the total catchment, but this routing method does not provide for any attenuation of peak flows on their way to the catchment outlet.

2. *The Unit Hydrograph Approach* (Book 5, Chapter 6, Section 3) - which converts rainfall excess inputs to a flood hydrograph by applying a transfer function (the unit hydrograph). The transfer function is generally inferred from the analysis of observed rainfall inputs and streamflow outputs, and there is limited potential to relate its parameterisation to

measurable catchment characteristics, though the parameters for the method may be developed from recorded data.

3. *Other lumped flood hydrograph estimation methods* - that involve use of a single (concentrated) linear storage (e.g. Clarke) or a distributed form of storage represented by a cascade of several linear storages (e.g. Nash). Variations of these methods with non-linear storages are also used. The fundamental flood routing concepts applied in these methods are further explained in Book 5, Chapter 5, Section 2 to Book 5, Chapter 5, Section 4.

While lumped flood hydrograph estimation models have the advantage of simplicity, they are limited in their application to the following situations:

- Catchments with relatively uniform spatial rainfall, loss and baseflow characteristics or where the variation of these characteristics between events is relatively minor, so that the derived unit hydrograph or other model parameters are applicable to a range of design events;

- Catchments with no significant artificial storages (reservoirs or flood detention basins);

- Applications where a flood hydrograph is only required at the catchment outlet, as for the design of drainage structures on roads and railway lines; and

- Applications that do not require extrapolation to the range of Very Rare to Extreme floods.

To the extent that they adopt a ‘black box’ approach (ie. the functions used to transform rainfalls to streamflows do not have direct links to physical catchment characteristics), lumped models depend on the availability of observed flood hydrographs for their calibration. The scope for application to ungauged catchments is thus more limited but the Clark-Johnstone synthetic unit hydrograph method (Book 5, Chapter 6, Section 3) has in the past been widely used for catchments on the east coast of Australia (Cordery et al., 1981).

The semi-distributed (node-link type) models described in the next section offer a broader range of application but are also more demanding in terms of model development, data requirements and understanding/skill of the practitioner. The lumped flood hydrograph estimation approach can be seen as a simplified version of the semi-distributed flood hydrograph estimation approach, and it is worth noting that the most widely used runoff-routing models (described in Book 5, Chapter 6, Section 4) can be configured to represent catchment response in a lumped fashion.

2.3. Semi-Distributed (Node-Link Type) Models

Semi-distributed models allow the spatial variation of inputs and key processes to be modelled explicitly. This is particularly important in large catchments and in catchments where the natural flooding characteristics have been significantly modified by various forms of development, including the construction of reservoirs, flood mitigation works and transport and drainage infrastructure.

Because of their flexibility and ability to calculate flood hydrographs throughout the catchment and to model land use and catchment changes, as well as being relatively straightforward to establish and run, node-link type models are currently the most widely used modelling approach for flood hydrograph estimation in Australia. A range of ready-to-use modelling systems (Book 5, Chapter 6, Section 4) are available to set up models for catchments of different size and complexity. These modelling systems allow the influential features and characteristics of a catchment to be represented in the model but in a highly conceptualised form. All conceptualisations involve some degree of lumping in terms of the processes modelled and spatial averaging of inputs, but the different modelling systems differ in the way they divide the catchment into various conceptual elements and in the methods they use to model the processes represented by these elements.

Figure 5.2.1 shows a simple conceptual representation of the runoff-routing process in a node-link type model. Each subarea receives a rainfall excess input which is converted to a runoff hydrograph at the node representing this subarea. The hydrographs are then routed successively through the links representing the drainage network to form the hydrograph at the catchment outlet.

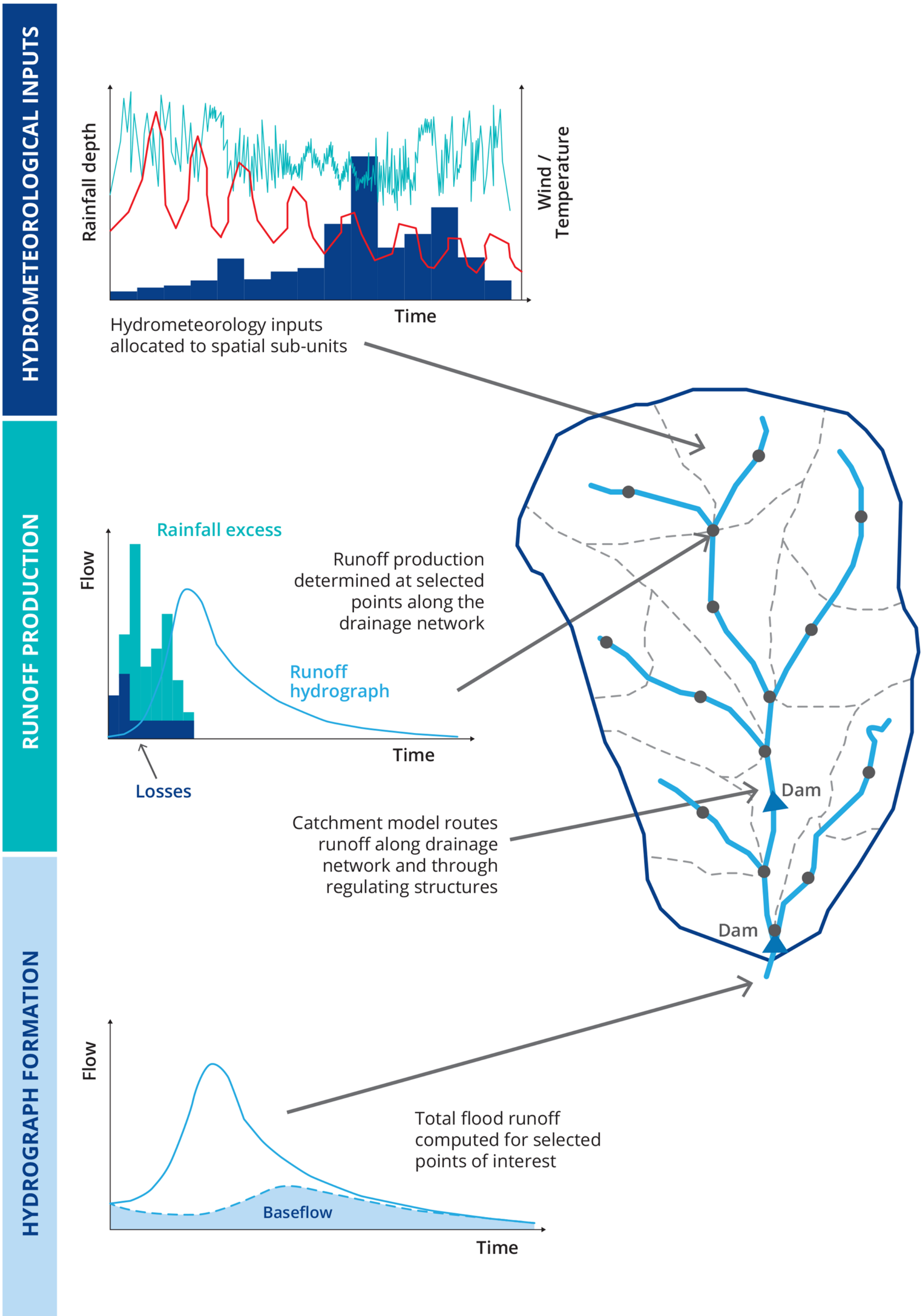


Figure 5.2.1. Conceptual Representation of the Runoff Routing Process in a Node-Link Type Model

2.3.1. Conceptual Model Elements

To build a semi-distributed model, the real catchment has to be conceptually represented as a system of nodes and links, each representing a different element of the actual catchment system. The following set of basic nodes and links can be used in different catchment models, but different models vary in how these basic elements are applied.

Nodes:

- Input nodes for hyetographs of rainfall excess or hydrographs of direct runoff (and possibly baseflow) from model sub-areas;
- Input nodes for inflow hydrographs from separately modelled catchments;
- Junction nodes – where different branches of the drainage network join (or where diversion flows re-enter);
- Diversion nodes – points where some of the flow is diverted or abstracted from the network;
- Reservoir routing nodes;
- Flood detention routing nodes; and
- Nodes for hydrograph outputs (including at gauging locations for comparison of modelled and observed hydrographs).

Links:

- Stream or channel routing links;
- Bypass links; and
- Floodplain storage links.

2.3.2. Catchment Sub-Division

The basic principle applied in dividing the catchment into a number of sub-areas for semi-distributed modelling is to provide a simplified but physically-based representation of the spatial features of the catchment, using a relatively small number of sub-areas (typically 10 to 100). The delineation of these sub-areas should generally follow topographic features that control the movement and storage of flood waters. Relevant land use features, such as the urban sections of the catchment, or areas inundated by large water bodies, need to be specifically delineated. The features that control flowpaths in relatively flat catchments may not be evident, and careful analysis of available topographic and flood data is required to define the sub-area boundaries. Detailed survey as well as aerial or satellite images may assist the catchment delineation in areas where the drainage characteristics are unclear.

The catchment sub-division should also have regard to the prevailing land uses in different parts of the catchment and should aim at sub-areas that are essentially homogeneous in terms of their runoff characteristics. This is particularly relevant in urban or urbanising catchments. Large areas with immediate runoff response, such as natural lakes and reservoirs also require special consideration. Book 7, Chapter 4 provides more detailed guidance.

2.3.3. Modelling of Runoff from 'Hill Slopes' (Overland Flow)

The term “contributing areas” is used to describe those areas of catchments where surface runoff occurs in the form of overland flow, sheet flow or flow in small channels that are not significant enough to be modelled separately (Book 4, Chapter 2, Section 2). There are two distinctly different methods used to model this runoff component:

i. The input hyetograph (areal average rainfall excess) for the sub-area is directly converted into a runoff hydrograph at a representative point within the sub-area (usually at or close to the centroid). This assumes that all runoff from the sub-area reaches this input node without any delay and there is no flow attenuation within the sub-area. Any translation and attenuation effects occurring within the hill slope elements thus need to be represented in the routing through the drainage network from the subarea input node to downstream points of interest; and

ii. The input hyetograph to the sub-area is transformed to an output hydrograph using one of the lumped catchment models introduced in Book 5, Chapter 2, Section 2. Different models use time-area, unit hydrograph or different forms of storage routing or kinematic wave routing concepts for this transformation.

Method (i) has the advantage of simplicity in that it avoids having to determine additional model parameters for runoff from contributing areas. In catchments with relatively uniform land use, when the interest is mainly on flood hydrographs at the catchment outlet for a limited range of flood magnitudes, this method can be expected to provide satisfactory results. However, it may provide conservatively high estimates of hydrograph peaks at internal points in the upper parts of the catchment due to the neglect of routing effects in the contributing areas of the catchment. Book 5, Chapter 6, Section 4 provides more detailed discussion of this method.

Method (ii) allows for better representation of processes that contribute to hydrographs in the upper parts of the catchment but it requires additional parameters. It is also better able to deal with the effects of significant land use changes in parts of the catchment and with changed runoff behaviour in Very Rare to Extreme flood events (Book 8, Chapter 5). A more detailed discussion of this approach is provided in Book 5, Chapter 6, Section 4.

2.3.4. Routing Through Network of Stream/Channel/Floodplain Elements

Except in small rural catchments, most of the translation and attenuation effects in the transformation of rainfall inputs to hydrograph outputs occur in the routing of hydrographs through the network of streams/channels, floodplains and major storages. The capability of a modelling system to adequately reflect the translation and attenuation involved in this transformation is therefore an important prerequisite for accurate flood hydrograph estimation.

Two distinctly different groups of flood routing approaches are used in node-link type flood hydrograph estimation models:

i. *Hydrologic Routing Approaches*- these flood routing methods are based on the storage routing principles described in Book 5, Chapter 5, Section 4 (linear storage routing) and Book 5, Chapter 5, Section 5 (non-linear storage routing). An important characteristic of hydrologic flood routing models is that their parameters are generally inferred from observed flood hydrographs, but it is possible to infer their parameters through close links with hydraulic methods (e.g. Muskingum-Cunge Method, Book 5, Chapter 5, Section 4) or from the results of hydraulic modelling; and

ii. *Hydraulic Routing Approaches* - these flood routing methods are based directly on the full unsteady flow equations or various simplified forms of these equations, as described in Book 5, Chapter 5, Section 5. Their parameters are inferred from the cross-sectional characteristics of streams, channels and floodplains, and the hydraulic characteristics of controlling features. The kinematic wave and diffusion wave approaches described in Book 5, Chapter 5, Section 5 are the most widely used hydraulic routing approaches incorporated into flood hydrograph estimation models.

The hydrologic routing approaches have the advantage that, by deriving their parameters from observed hydrographs, they represent an integrated routing response from the complex stream and floodplain system that is often too complex to be represented in detail. However, application of such calibrated parameters to conditions outside the ones reflected in the observed hydrographs (ie. for changed catchment conditions or significantly different flood magnitudes) involves assumptions that may not be justifiable. The hydraulic routing methods have closer links to the physical characteristics of the routing reaches, but their application still involves a significant degree of conceptualisation and some form of calibration.

Modelling of the flood routing effects over a range of flow conditions requires a clear understanding of the flood dynamics so that the simulated response can be appropriately matched to the actual flooding behaviour. Specifically this means that the adopted network should represent any breakout flows and bypass flows occurring during larger floods, as well as the effects of significant floodplain storage areas activated during large events. The additional storage availability in large flood events may be counter-balanced by increased flow efficiency as the flow depth increases. Where backwater effects are likely to have a significant impact not only on flood levels but also on the routing of flood flows through the drainage network, hydraulic routing methods based on the full unsteady flow equations may be required.

The model should also represent the varying impact of flow restrictions for different flow magnitudes. In some cases the results of detailed hydraulic modelling may be required to develop a clear understanding of the changes in flood flow behaviour with flood magnitude, so that they can be adequately reflected in the hydrologic catchment model.

One important limitation of node-link type models is that the different routing elements are conceptualised as one dimensional flow links. This means that a dominant flow direction needs to be assumed when the routing elements are defined. In cases where the flow direction changes for floods of different magnitudes, it may be necessary to introduce more complexity into the channel network so that different flowpaths are activated at different flow magnitudes. The two dimensional rainfall-on-grid approaches discussed in Book 5, Chapter 2, Section 4 are in principle better equipped to deal with changes in flow direction during a flood event and between flood events of different magnitude, but this advantage may be off-set by the difficulty of using such models to adequately represent loss processes and roughness characteristics at the scale of individual grid cells.

Whatever routing method is used, it is important to ensure that any application of the model outside its range of calibration is guided by consideration of changes in hydraulic characteristics and then reflected in the adopted network conceptualisation and parameter values. This is further discussed in Book 7, Chapter 4 and Book 7, Chapter 5.

2.3.5. Routing Trough Special Storages

When flood storage occurs in a concentrated form, such as in lakes, reservoirs, detention basins and large natural or artificial flood storageage areas, it is appropriate to model the flood modifying effects of such storages by a ’special storage’ routine. Where the relevant survey and hydrographic data are available, the storage-discharge characteristics of such special storages may be defined by storage rating curves and discharge rating curves in terms of depth or elevation. In other situations, simplified storage-discharge relationships need to be derived from observed inflow and outflow hydrographs or by trial and error during model calibration. Methods for deriving storage-discharge relationships for different conceptual storage elements are discussed in Book 5, Chapter 6, Section 4.

2.4. Grid-based (Distributed) Models

In the fully distributed or grid-based flood hydrograph estimation models (also referred to as ‘rainfall-on-grid’ models) the catchment is represented by a large number of grid cells, based on topographic data from a Digital Elevation Model (DEM). More detailed survey information on the drainage network and flow controlling features of the catchment may be superimposed on the DEM data.

Different models vary in the degree of detail adopted in modelling the runoff generated from rainfall falling on a grid element. In principle the method allows more physically-based representations of runoff processes; however, this is only likely to be valid at larger depths of overland flow. Such models currently represent saturation and ponding processes in a simplistic fashion, and similar simplifications are adopted in modelling the baseflow contribution at the scale of individual cells.

The routing of runoff from individual cells through the catchment and the stream network is then based on the principles of two dimensional dynamic wave modelling introduced in Book 5, Chapter 5, Section 5 and described in more detail in Book 6, Chapter 4, Section 7. Particular issues to be dealt with in these models are the significantly larger data requirements than for node-link type models to give a realistic representation of the catchment, characterisation of hydraulic roughness for different catchment elements, and how to deal with computational stability problems that arise when runoff is generated from initially dry cells.

As the direction of the flow between cells is determined as part of the solution process at each time step, drainage paths do not need to be pre-defined as in traditional one dimensional runoff-routing approaches. The application of hydraulic methods in the runoff-routing process also means that there is no need for linking the hydrologic model with a hydraulic model of the floodplain area.

In most catchments the catchment boundaries and the drainage network are quite well defined in the upper part of the catchment, and traditional runoff-routing models can thus adequately describe the flood hydrograph formation for these parts of the catchment. A ‘hybrid’ approach, where a two dimensional model is only used for runoff-routing in the flatter or urbanised parts of the catchment that are influenced by complex hydraulic controls, may be the most efficient approach in these situations.

The theoretical advantage of grid-based models is that a lesser degree of conceptualisation in the catchment representation is required, thus requiring less hydrologic expertise of the practitioner. However, the modelling of the overland flow phase at the scale of individual grid cells still poses challenges and requires further research. The principles applied in rainfall-on-grid models and their advantages and limitations are discussed in more detail in Book 5, Chapter 6, Section 5. While grid-based methods are apparently more directly based on physical catchment data than alternative methods, they still need to be calibrated with recorded data, and there is still uncertainty in the results from their application. This is discussed further in Book 7.

At the current stage of development of these models and with the limited level of experience gained with their practical application, it is considered premature to recommend their general use in these Guidelines. However, it is expected that further development and testing will allow rainfall-on-grid models to be more widely applied.

2.5. References

Cordery, I, Pilgrim, D.H. and Baron, B.C. (1981), Validity of use of small catchment research results for large basins. Instn. Engrs. Australia, Civil Engg. Trans., CE23: 131-137.

Chapter 3. Losses

Peter Hill, Rhys Thomson

Chapter Status	Final
Date last updated	14/5/2019

3.1. Introduction

This chapter provides advice on loss models and values for design flood estimation. It deals with the fundamental hydrologic question – how much rainfall becomes runoff? Design floods are typically derived either using flood frequency analysis or rainfall-based flood event models. Continuous simulation is covered in Book 4 and the focus of this chapter is losses for event based design flood estimation.

The loss is just one of the number of inputs to the design process (such as the critical storm duration, areal reduction factor, spatial pattern, temporal pattern, runoff routing model, model parameters and treatment of baseflow) that can affect the magnitude of the calculated design flood. These other inputs are discussed in other books.

This chapter is structured as follows:

- Book 5, Chapter 3, Section 2 – discusses how loss processes are represented in different conceptual loss models, ranging from empirical models through to more complex process models
- Book 5, Chapter 3, Section 3 – discusses the selection of conceptual loss models and different approaches to estimating loss values
- Book 5, Chapter 3, Section 4 – describes the estimation of effective impervious areas for urban catchments
- Book 5, Chapter 3, Section 5 – summarise different sources of information on loss values for rural and urban catchments that can be used to help select values for design
- Book 5, Chapter 3, Section 6 – discusses different approaches to characterising the distribution of loss values
- Book 5, Chapter 3, Section 7 – discusses a range of other considerations for selecting loss values for design flood estimation

3.2. Conceptual Loss Models

3.2.1. Loss Processes

3.2.1.1. Physical Processes

Loss is defined as the precipitation that does not appear as direct runoff, and is attributed to the following key processes (refer to Figure 5.3.1):

- Interception by vegetation;

- Infiltration into the soil;
- Retention on the surface (depression storage); and
- Transmission loss through the stream bed and banks.

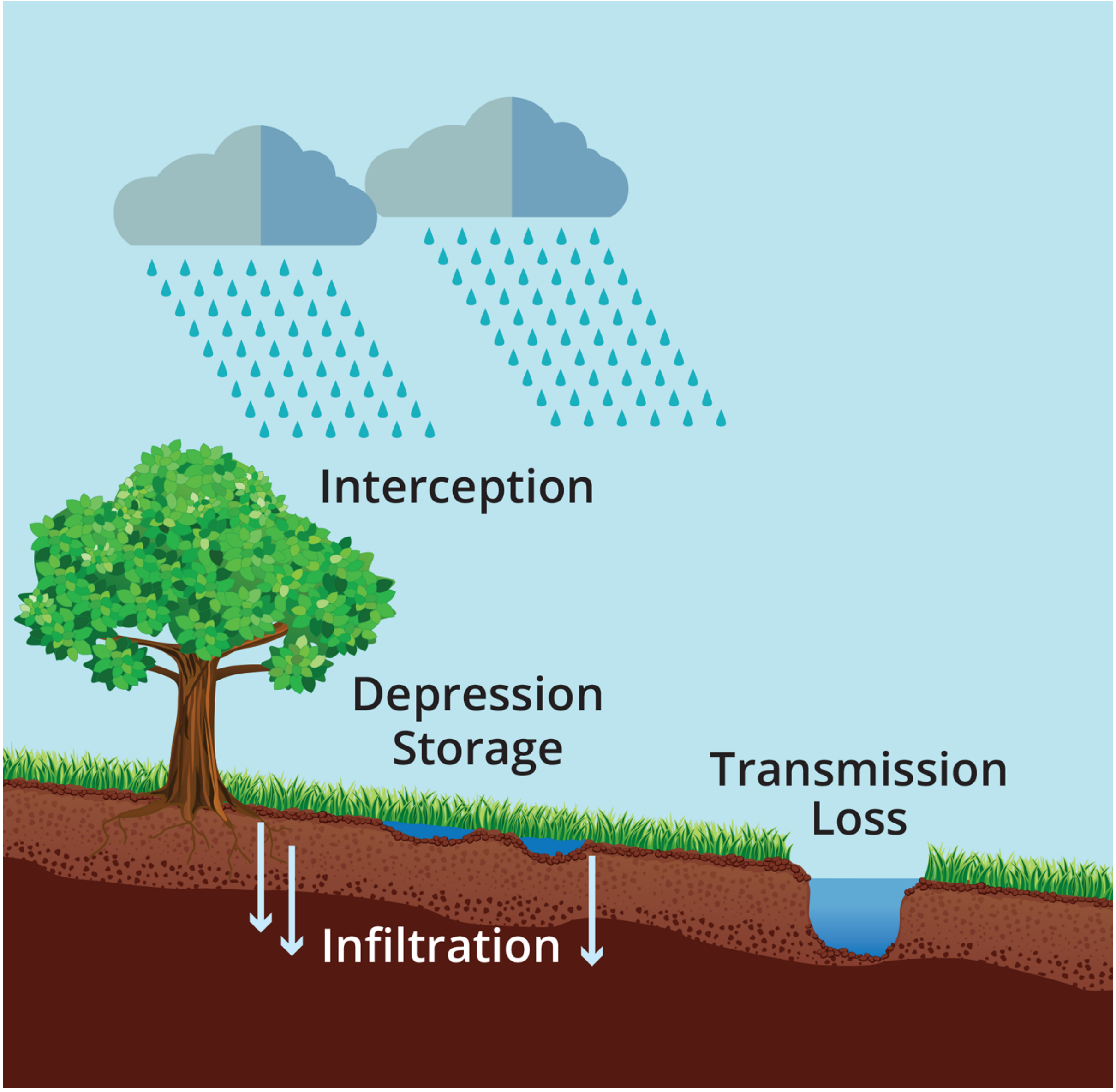


Figure 5.3.1. Physical Processes which Contribute to Rainfall Loss

More details on the runoff process are described in Book 4, while this section focuses on specific processes associated with estimation of losses.

Runoff has generally been considered to consist of surface runoff produced by rainfall excess which occurs at the ground surface when the rainfall intensity exceeds the infiltration capacity. This is known as Horton-type runoff and Fleming and Smiles (1975) provide a review of infiltration theory and its application to practical hydrology.

Over the last twenty years the classical concept of storm runoff has been challenged as a result of observations on natural catchments during storm periods and many detailed studies of instrumented plots and small areas. Two alternative types of storm runoff mechanism have been proposed:

- Saturated overland flow occurs when, on part of the catchment, the surface horizon of the soil becomes saturated as a result of either the build-up of a saturated zone above a soil horizon of lower hydraulic conductivity, or due to the rise of a shallow water table to the surface; and

- The other type of storm runoff is throughflow, which is water that infiltrates into the soil and percolates rapidly, largely through macropores such as cracks, root holes and worm and animal holes, and then moves laterally in a temporarily saturated zone above a layer of low hydraulic conductivity. It reaches the stream channel quickly and differs from other subsurface flow by the rapidity of its response and possibly by its relatively large magnitude.

Associated with the recognition of these two alternative types of storm runoff, is the concept that storm runoff may be generated from only a small part of many catchments. Additionally, this source area may vary in its extent from time to time, in different seasons and during the progress of a storm.

There are no practical methods for estimating storm losses and runoff that would take explicit account of different runoff processes, partial and variable source areas and small-scale variations in characteristics. The various existing methods assume uniform or average conditions, not accounting non-homogeneity of the catchment. Though each model attempts to simplify the overall process by different degrees, however, it is important to understand the complexity of these physical processes when reviewing the different loss models that are available (refer Book 5, Chapter 3, Section 2).

3.2.1.2. Urban Runoff

Urban runoff, even at the allotment level, is complex, involving contributions from roofs, yards, adjacent road and pavement areas. The effective rainfall excess from the various areas is subject to significantly different infiltration regimes (Goyen and O'Loughlin, 1999a; Goyen and O'Loughlin, 1999b) and these units are interconnected by complex and often transitory pathways (Riley and Fanning, 1997). Further discussion on these complex processes is discussed in Book 5, Chapter 3, Section 4.

3.2.1.3. Transmission Loss

The estimation of transmission (or channel) losses may be required in systems where the volume along a reach is required. Stewart and Boughton (1983) identified that the processes contributing to transmission losses were water hole storage, infiltration, evaporation and bank storage.

Majority of research on transmission losses for rural catchments was focused on long term losses relevant for water resource modelling and planning, rather than flood estimation. For example, there has been work done on gaining and losing river reaches within the Murray Darling Basin. Boughton (2015) explored transmission losses for 100 catchments from the east-coast of Australia.

The research on transmission losses tend to focus on specific reaches and hence the results are site specific. For very large arid catchments, transmission losses can be substantial, for example Knighton and Nanson (1994) found transmission losses of 75% for a reach of the Cooper Creek. However, for most design flood applications the channel losses will not be significant and can be combined with other processes that are implicitly covered by lumped conceptual models.

Urban catchments may also be subject to transmission losses. While the losses identified for rural catchments are generally less pronounced in urban catchments, transmission losses can occur along the drainage system. This may include leakages and ageing infrastructure, particularly when the soil around the drainage system is highly permeable.

3.2.2. Types of Loss Model

For the purposes of this chapter, loss models are broken into three broad classes:

- Empirical Models - designed to ensure depths of direct runoff and rainfall excess are in equilibrium. These types of models have minimal factors that would influence the values for an individual catchment.
- Simple Models - attempt to quantify a portion of the processes in a simplified manner. These include, for example, Hortonian Infiltration models where all losses are assumed to relate to infiltration.
- Process Models - attempt to represent the complex behaviour of losses within the catchment, and consider flow through the soil layers and over the catchment surface.

Given their complexities, process models have a large number of parameters that makes them difficult to apply to estimate design floods. In Australia, there is limited experience in applying process models for design flood estimation and therefore they are not covered in this section.

3.2.3. Empirical Models

Empirical loss models focus less on the loss processes themselves, rather more on representing their effects in producing flows. Rainfall excess models typically fall within this category.

In many of these models, the initial loss occurs in the beginning of the storm, prior to the commencement of surface runoff. It is assumed to be composed of interception losses, depression storage and infiltration before the soil surface is saturated; a continuing loss rate is then applied for the remainder of the storm. This model is consistent with the concept of runoff produced by infiltration excess, ie runoff occurs when the rainfall intensity exceeds the infiltration capacity of the soil.

These models apply the losses directly to the rainfall, subtracting from the rainfall itself, to produce a rainfall excess that is subsequently applied to the hydrological model. This concept of rainfall excess is important, as it does not consider the changing catchment characteristics during the period of rainfall (compared with Simple and Process models).

There is typically a wide range of initial loss values observed for a catchment (Rahman et al., 2002; Phillips et al, 2014; Hill et al., 2014a). This variability reflects the importance of antecedent conditions but uncertainties in the estimation of the timing and distribution of the catchment average rainfall also contribute to the range of values. This potential variability in the initial loss value is an important consideration, particularly in application of historical storms to hydrological models.

A number of these models are described further below.

3.2.3.1. Initial Loss - Continuing Loss

The continuing loss is the average loss rate throughout the remainder of the rainfall event after the initial losses are satisfied. Previous research and guidance suggests that constant loss rates are most applicable to large storm events, where a significant proportion of rainfall becomes runoff. Figure 5.3.2 provides an example of the application of a typical Initial Loss – Continuing Loss model.

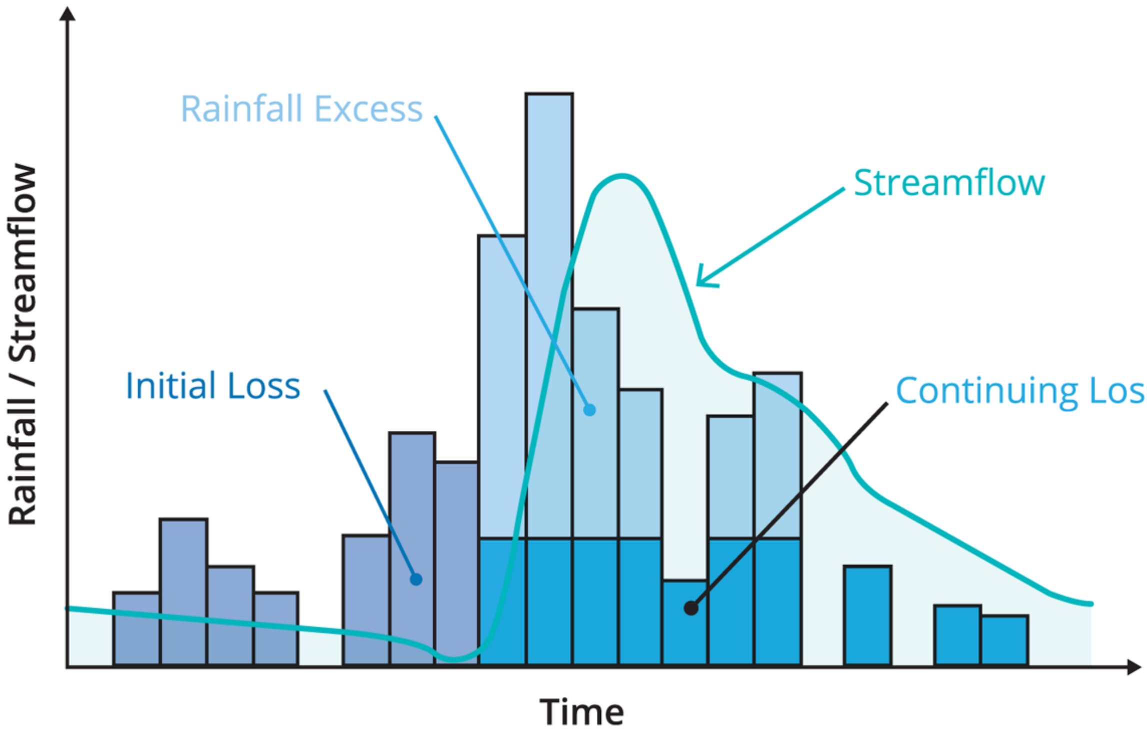


Figure 5.3.2. Initial Loss – Constant Continuing Loss Model

Despite the simple conceptual nature of the IL/CL model, there are a number of challenges in estimating continuing loss directly from recorded streamflow and rainfall.

The continuing loss rate should not be based simply on a water balance of runoff volume less initial loss divided by the duration of the event. This will underestimate the loss rate; as illustrated in Figure 5.3.2 there will likely be timesteps in which the rainfall is less than the continuing loss rate (or even zero) and hence the full value is not taken up.

Although not immediately apparent, the definition of CL also means that its magnitude is dependent on the timestep used in the analysis. This is because as the timestep reduces, there is an increased likelihood that there will be some timesteps in which the rainfall depth is less than the CL rate. Thus, to achieve the same volume of rainfall excess, the CL will typically need to be increased for shorter timesteps. This is discussed further in Book 5, Chapter 3, Section 7.

3.2.3.2. Initial Loss – Proportional Loss

The proportional loss models assume that a fixed proportion or percentage of the rainfall is lost at each time step, after the initial loss has been satisfied, which means that losses throughout the event may vary depending on the temporal patterns of rainfall. For simplicity, the proportional loss coefficient for a storm is usually taken as a constant.

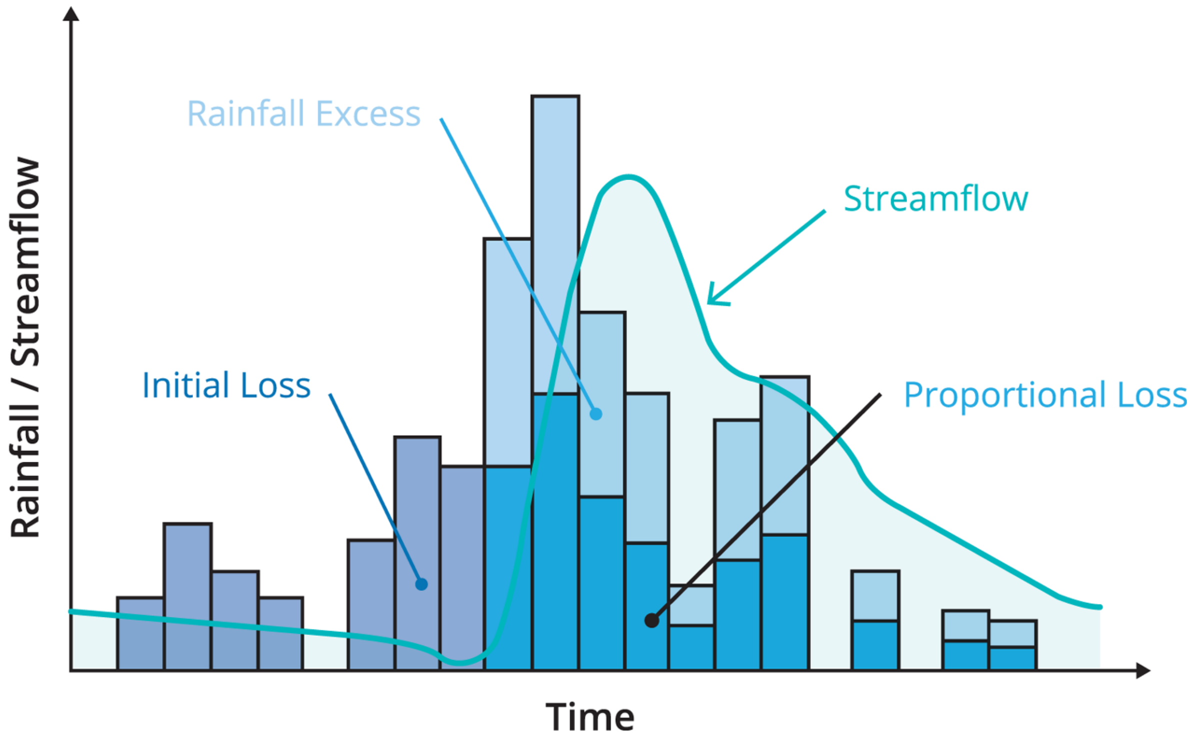


Figure 5.3.3. Initial Loss - Proportional Loss Model

Proportional loss models are consistent with runoff being generated by saturated overland flow. This assumes that runoff is generated from saturated portions of the catchment; this contributing area is expected to increase the duration and severity of the storm (Mein and O'Loughlin, 1991; Mag and Mein, 1994).

3.2.3.3. Variable Continuing Losses

In the application of the constant continuing loss and proportional loss models, it has typically been assumed that the loss rates are a constant, after the initial loss is satisfied. However, based upon consideration of physical processes it might be expected that the loss rates should decrease throughout the event as the catchment becomes wetter and infiltration reduces and/or the size of source areas enlarges.

For the IL/CL model this would suggest that the continuing loss should decrease as the event progresses and such a reduction with duration (as a surrogate for volume of infiltration) is observed from the empirical analysis of data by Ishak and Rahman (2006) and Ilahee and Imteaz (2009). The variation of continuing loss with event duration is discussed further in Book 5, Chapter 3, Section 7.

3.2.3.4. SCS Curve Number

The SCS runoff curve number (CN) is widely used in the US as well as some countries in South-East Asia. Soils in the US are classified in four hydrologic groups (A, B, C, D) according to their infiltration rate. A CN is estimated from the hydrologic soil group, the treatment of the soil (effect of cultivated agricultural lands) and the hydrologic condition (the effect of vegetation density) to add more definition to the groupings (Boughton, 1989; Woodward et al, 2002; Van Mullem et al., 2002; Ward et al., 2009).

A number of studies such as Eastgate et al. (1979) and Rajendran et al. (1982) have applied the SCS to Australian soils. There is however a lack of information on how Australian soils are classified in the SCS hydrologic groups, which limits its application in Australia.

3.2.3.5. Probability Distributed Storage Capacity Models

Most conceptual loss models are lumped so that a similar parameter value is assumed over a catchment or sub-catchment. Moore (1985) introduced the concept of probability distributed models, which can be used to account for the spatial variability in runoff generation across a catchment. This variability can arise either from:

- Differences in overall water storage capacity between sub-catchments (topography, soils, vegetation);
- Spatial variation of water storage capacity within sub-catchments (potential loss distribution);
- Stochastic variation of initial water storage status between events (different antecedent conditions); or
- Gradual variation in water storage status during an event (progressive wetting).

These models are run in a continuous or semi-continuous fashion (updated during an event) and therefore can explicitly account for antecedent conditions, as well as for variation within an event.

In general, the runoff mechanism in drier catchments is more likely to be controlled by infiltration rate whereas saturated excess is more likely to generate runoff for wetter catchments (Hill et al., 2013). The dominant mode of runoff production will depend on a range of factors including climate, soil, vegetation and topography. Those based on variable storage capacity reflect the subsurface saturation excess mechanism and include Xinanjiang (Ren-Jun et al., 1992), SWMOD (Stokes, 1989; Water and Rivers Commission, 2003) and the Revitalised Flood Hydrograph (ReFH) model in the UK (Kjeldsen et al., 2005).

These models are based on the assumption that the catchment consists of many individual storage elements with a soil moisture capacity. The depth of water in each element increases with rainfall and decreases with evaporation. When rainfall exceeds the storage capacity, direct runoff is produced. The model assumes that the soil moisture is redistributed between the elements between rainfall events.

The simplest form assumes a linear distribution of soil moisture in the catchment, from zero to its maximum capacity. This form of probability distributed model is incorporated in ReFH model in the UK. However, the above approach assumes that a portion of the catchment has zero storage capacity and hence there is no initial loss. Many catchments in arid and semi-arid areas exhibit a significant initial loss and therefore the conceptual model is extended such that the capacity varies between a minimum and maximum of the catchment. The simpler models assume that the capacities vary linearly while other models have introduced a shape parameter to describe the form of variation with capacity

3.2.3.6. SWMOD

SWMOD (Soil Water balance MODel) is developed by Stokes (1989) for the Northern Jarrah forest of Western Australia, where saturation excess overland flow is held to be the dominant runoff mechanism for storm events (Water and Rivers Commission, 2003). The model incorporates the ability of different landforms in the catchment to store water during the storm event. When the accumulated rainfall is greater than its infiltration capacity, the sub-catchment will generate saturation-excess overland flow. Infiltration capacity is assumed to vary within an area due only to soil depth.

$$C_f = C_{\max} - (C_{\max} - C_{\min}) \times (1 - F)^{1/b} \tag{5.3.1}$$

Where:

C_f is the infiltration capacity at fraction F of the sub-catchment

F is the fraction of the subcatchment

b is the shape parameter

Cmax is the maximum infiltration capacity

Cmin is the minimum infiltration capacity

The infiltration capacity is taken to mean the maximum depth of water that can be stored in the soil column. Where the accumulated rainfall is greater than the infiltration capacity that fraction of the sub-catchment will have saturation-excess overland flow.

Large infiltrations ponds (10 to 15 m^2) were used in conjunction with a ring infiltrometer and a well permeameter to determine the infiltration characteristics of a complex lateritic soil profile in the Jarrah forest of Western Australia (Ruprecht and Schofield, 1993). The logs from the construction of observation bores were able to characterise the shape of b parameter. Hill et al. (2014b) outlines the application of SWMOD to 38 catchments across Australia.

3.2.4. Simple Models

In many loss models, the interception, depression storage and transmission losses are not directly accounted for, while the loss is treated as infiltration into the soil. The main factors affecting the soil infiltration process are the soil properties, antecedent moisture conditions, layered soils, rainfall intensity and surface sealing, vegetation cover and entrapment of air, and the soil slope and land use (Siriwardene et al., 2003). Simple models attempt to incorporate this infiltration into the soil through various models.

Various representations to the complex equations for the water movement in the soil (such as Philip and Green-Ampt, Horton etc) are used to express the reduction of infiltration capacity with time (Maidment, 1992).

Skukla et al. (2003) analysed ten infiltration models including Green-Ampt and Horton's models, using double-ring infiltrometer tests and reported that Horton's model gave the best results for most land use conditions.

Siriwardene et al. (2003) undertook field infiltrometer tests at 21 sites in eight Victorian urban catchments in order to estimate the infiltration parameters related to Horton's infiltration model. They acknowledge the difficulty in selecting representative values for the infiltration parameters because of the 'significant variability with respect to soil type and land use in the catchment'.

Mein and Goyen (1988) note that despite the obvious attraction of using Simple Models, 'the problem is to specify parameters (which relate to soil type) and initial conditions which are satisfactory for design use on a given catchment. In practice, the uncertainties of soil behaviour and the areal variability of soil properties do not justify the use of anything more than the simplest model'.

3.2.4.1. Horton Model

Horton's equation has been used and modified over the years to provide an estimate of losses due to infiltration into pervious surfaces. It is based on a diminishing continual loss, as described in Equation (5.3.2) below.

$$f_t = f_c + (f_0 - f_c)e^{-kt} \tag{5.3.2}$$

where:

f_t is the infiltration capacity (mm/h)

f_c is the minimum or ultimate value of f_t(mm/h)

f_0 is the maximum or initial value of f_t (mm/h)

k is a decay coefficient (per hour)

t is the time from the beginning of the storm (h)

3.2.4.2. Green-Ampt

The most commonly used approximate theory based infiltration model is the one developed by Green and Ampt (1911), which is an approximate model utilising Darcy's law and is further discussed in Mein and Larson (1973), Chu (1978), Lee and Lim (1995) and King (2000).

William (1994) describes a pilot study for nine Victorian catchments to determine if application of the Green-Ampt equation provides a superior results to simplified models, when applied at catchment scale. Although the Green-Ampt equation was successfully applied to each catchment, the results produced were not on average superior to those produced using the simple empirical models.

3.2.4.3. Australian Representative Basin Model (ARBM)

The Australian Representative Basin Model (ARBM) was developed with the aim to classify and select hydrologically diverse basins at a significant scale for resource development (Fleming, 1974; Mein and McMahon, 1982). Furthermore, the model sought to increase the understanding of the hydrological processes in each basin.

The ARBM is structured to represent the passage of water over and through the catchment, as illustrated in Figure 5.3.4. It is based on Chapman's work (Chapman, 1968; Chapman, 1970), which originally sought to optimise certain parameters, while measuring others. However, developers Boyd et al. (1993) began optimising all parameters as it was believed that measurements were difficult, uncertain, costly and impractical (Mein and McMahon, 1982).

The model uses a deterministic mathematical model intended to represent physical processes and relationships between rainfall and runoff for the catchment. It operates in a continuous mode, considering both rainfall events and initial estimations of soil moisture conditions for each wetting event. This is done by simulating soil moisture depletion by evaporation between rainfall events (Fleming, 1974). It is expected that the parameters used would be related to physical catchment characteristics, therefore making the model applicable to any Australian gauged or ungauged catchment.

Despite being developed, the optimised parameters have not exhibited uniqueness. Mein and McMahon (1982) however, do not believe that this particular model produces outcomes that are any different to other process models developed for the same purpose.

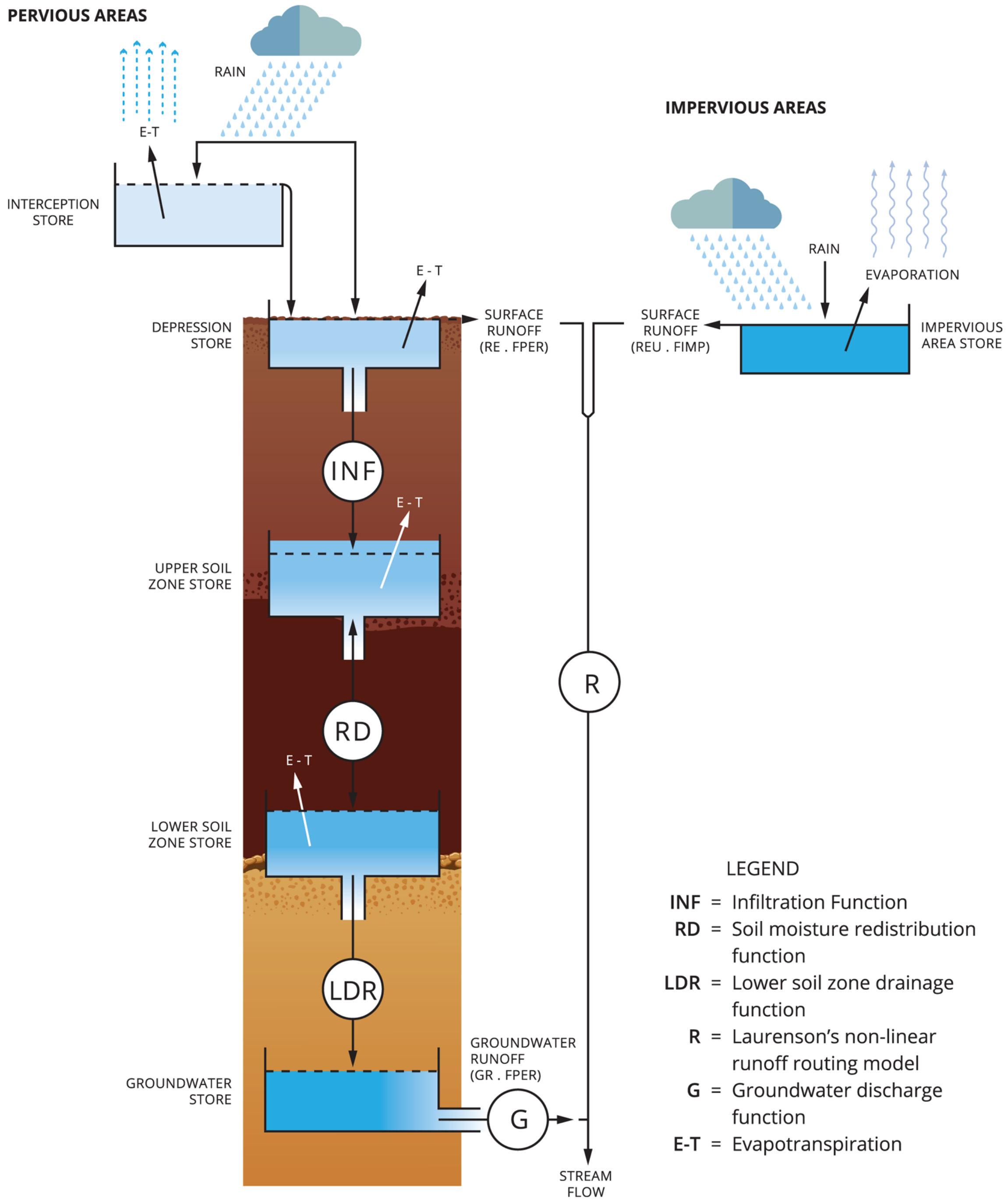


Figure 5.3.4. The model structure of the Australian Representative Basins model based on the work of Chapman (1968). Diagram obtained from Black and Aitken (1977) and Mein and McMahon (1982).

3.2.5. Process Methods

Continuous simulation is covered in Book 2, Chapter 7 and therefore the following brief overview concentrates on the loss modelling aspects.

These models typically estimate the losses from rainfall and the generation of streamflow by simulating the wetness and dryness of the catchment on a daily, hourly and occasionally sub-hourly basis. Continuous simulation eliminates the need to select representative values of loss, since the loss is explicitly included in the modelling. The focus of loss conceptualisation in continuous rainfall-runoff simulation models is less on the representation of the loss process, rather than on representing the effect on producing floods.

The majority of continuous simulation applications are for flood forecasting, rather than for design flood estimation. The development of stochastic rainfall generation techniques has encouraged their application for design flood estimation (Boughton et al., 1999; Kuczera et al., 2006).

A number of Australian studies have applied a continuous simulation approach to estimate design floods and compared the results to those from flood frequency analysis such as Boughton and Hill (1997), Muncaster et al. (1999), Boughton et al. (1999) and Heneker et al. (2003). The reported applications of continuous simulation for design flood estimation have typically involved calibration against recorded data, however,to extend the use to ungauged catchments, it will require developing regional relationships for model parameters.

3.3. Approach to Selection of Loss Model and Values

3.3.1. Selection of loss model

For real-time flood forecasting or calibration of a hydrologic model, the focus is on selection of models and parameter values that replicate the observed hydrograph. However, for design flood estimation, the objective is the derivation of unbiased estimates of specific characteristics of a design flood (typically the peak).

Thus, the key objectives of loss models and their parameterisation for design flood estimation are to:

- Close the volume balance in a probabilistic sense such that the volume of the design flood hydrograph for a given AEP should match the flood volume derived from frequency analysis of flood volumes;
- Produce a realistic time distribution of runoff to allow the modelling of peak flow and hydrograph shape;
- Reflect the effects of natural variability of runoff production for different events on the same catchment, to avoid probability bias in flood estimates; and
- Reflect the variation of runoff production with different catchment characteristics to enable application to ungauged catchments.

As discussed in Book 5, Chapter 3, Section 2, there are a range of different conceptual loss models that were derived with different conceptualisation and varying degrees of complexity from simple lumped rainfall excess models to more detailed process models.

The above objectives are important when selecting loss models for design flood estimation. It is therefore helpful to consider the following criteria when selecting a loss model for design flood estimation:

- The model produces a temporal distribution of rainfall-excess that is consistent with the effect of the processes contributing to loss;
- Suitable for extrapolation beyond calibration and is hence applicable to estimate floods over the required range of AEPs;
- Inputs are consistent with readily available data;
- Small number of parameters that need to be selected (preferably no more than 2);
- Parameters have been linked to catchment characteristics, or it is considered reasonable that such a link could be established so that the parameters can be regionalised;
- Have the potential to be easily incorporated into rainfall-runoff models; and

Considerations of such issues have typically resulted in adoption of simple rainfall excess models.

Dyer et al. (1994) compared the performance of the constant continuing loss and proportional loss models for 24 catchments using RORB and found that the proportion loss model resulted in generally improved calibrations. This finding was supported by Hill et al. (1996), who calibrated RORB models for 11 Victorian catchments. However, analyses undertaken by Phillips et al (2014) and Hill et al. (2014a), concluded that the results were inconclusive in regards to the best model. Even for catchments where one of the loss models was preferred for a majority of events, there were some events for which the alternate model was preferred. Similarly, there was no obvious relationship between the preference for a particular model and hydroclimatic or catchment characteristics that could explain the preference for a particular approach.

A number of Australian studies have demonstrated that the IL/CL model is suitable for design flood estimation where in it can be used to estimate design flood estimates over a range of AEPs. However, it is often difficult to derive unbiased estimates of floods using the IL/PL model over a range of AEPs. Specifically, the IL/PL model has the potential to underestimate peak flows for events rarer than used in the derivation of the values; this suggests that PL should vary with the AEP of the event.

Furthermore, studies that have analysed a large number of events and catchments such as Phillips et al (2014) and Hill et al. (2014a) have found that there can be a large variation in PL values, which makes it difficult to recommend a representative value for design Book 5, Chapter 3, Section 5.

Given the difficulties in characterising how PL should vary with AEP, it is considered that the IL/CL model is the most suitable of these simple rainfall excess models for design flood estimation for both rural and urban catchments. Probability distributed loss models such as SWMOD demonstrate promise and should also be considered for rural catchments where there is reliable and consistent description of hydraulic properties of soils.

If alternate loss models are to be adopted then they should be evaluated against the above criteria. An important consideration is how the loss model performs when extrapolated to events outside of the range of events used in deriving the loss values.

3.3.2. Design Rainfall Bursts v Complete Storms

In selecting loss values for design flood estimation, it is important to consider the nature of the design rainfall information. The Intensity Frequency Duraton data in Book 2 is derived from the analysis of rainfalls for standard durations, rather than complete storms. In some cases, these events represent complete storms but also include cases of bursts of rainfalls within a much longer duration storm.

The conceptual difference between the initial loss for a rainfall burst (IL_b) and for a storm (IL_s) is illustrated in Figure 5.3.5. The initial loss for the storm is assumed to be the depth of rainfall prior to the commencement of surface runoff. The initial loss for the burst however, is the portion of the storm initial loss, which occurs within the burst.

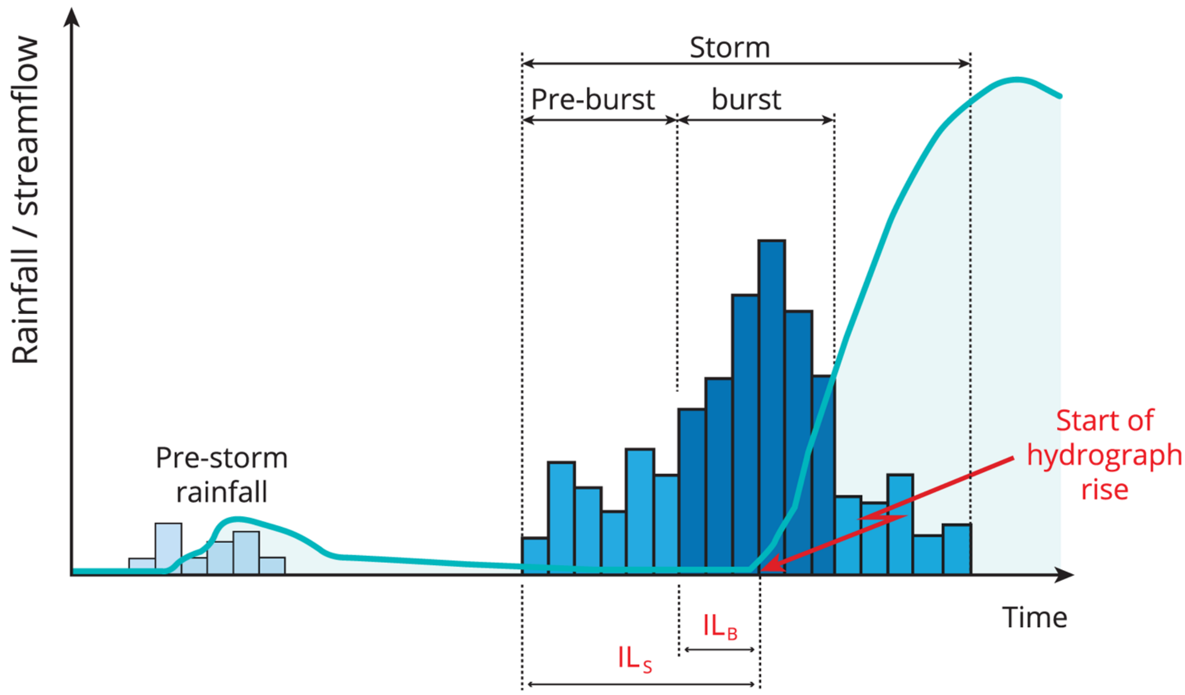


Figure 5.3.5. Distinction between Storm and Burst Initial Loss

If pre-burst rainfalls are included, then the design rainfalls will represent (near) complete design storms and therefore the storm losses can be directly applied without adjustment. The design values recommended in Book 5, Chapter 3, Section 5 are intended for application with complete storms and therefore, requires the pre-burst depths to be included.

However, if design bursts, rather than complete storms, are used in design then the burst initial loss needs to be reduced to account for the pre-burst rainfall. For the same reason, the initial moisture content for storage capacity models (such as Horton and SWMOD) need to be increased to account for this pre-burst rainfall.

This has implications for all design flood situations, but is particularly important for design situations where the outcome is sensitive to the flood volume, such as the design of retarding basins (Rigby and Bannigan, 1996). The failure to recognise the rainfall prior to design rainfall bursts has the potential to significantly underestimate the design flood.

3.3.3. Approaches to Estimating Loss Values

The most appropriate approach to estimating loss values will depend upon the objectives and required rigour of the study, and the quality and availability of at-site and regional flood data. The different approaches can be considered in the following broad classes:

1. Empirical analysis of at-site rainfall and streamflow records;
2. Information from regional analysis of data; and
3. Reconciliation of design values with independent flood frequency estimates.

3.3.3.1. At-site Event Data

If there is long-term pluviograph and streamflow data available at the site of interest it may be possible to directly estimate loss values for a number of events. In order to undertake such an analysis the streamflow should be free of significant regulation or diversion and land use within the catchment should be stationary over the period of data being analysed.

The events to be analysed should be selected carefully to ensure that the sample of events is not biased. The selection of high runoff events for loss derivation is likely to be biased towards wet antecedent conditions (ie. losses tend to be too low). Ideally, events should be selected on the basis of rainfall to remove this bias. However the selection and analysis of events by rainfall is problematic because it requires consideration of a representative duration of the rainfall and there may be little or no runoff generated from some intense bursts of rainfall if the antecedent conditions are dry.

The main limitation of deriving losses directly from the analysis of recorded data is that they may not be compatible with the other design inputs and hence suitable for design flood estimation. That is, although the loss values may reflect the loss response observed for a number of events on the catchment, this does not guarantee that their application with other design inputs results in unbiased estimates of floods. For this reason, it is also desirable to reconcile design values with independent flood frequency estimates where possible (refer Book 5, Chapter 3, Section 3).

3.3.3.2. Regional Data

Deriving loss values from an analysis from multiple catchments has the advantage that there is the opportunity to be more selective in selecting the data sets to be analysed and the larger sample allows the distribution of values to be explored. However the results from these regional analyses need to be transposed to the catchment of interest which relies on relationship being developed between the loss values and physical characteristics – a link which has generally proved to be elusive.

Loss values have been estimated for a large number of urban (Phillips et al, 2014) and rural catchments (Hill et al., 2014a). These two studies represent the most comprehensive regional studies of losses covering Australia and hence the recommended loss values summarised in Book 5, Chapter 3, Section 5 are largely based upon these studies.

As with the estimation of losses from a single site, their application with other design inputs does not guarantee unbiased estimates of floods and for this reason, it is therefore also desirable to reconcile design values with independent flood frequency estimates where possible (refer Book 5, Chapter 3, Section 3).

3.3.3.3. Reconcile Design Values with Independent Flood Frequency Estimates

Deriving loss values by comparison with flood frequency estimates has the advantage of producing design losses which are consistent with the other design parameters and the design objective of deriving peak flows of a given AEP. Indeed the use of design rainfalls to estimate design floods is in fact the sole objective of rainfall-based flood event modelling. The only difference between calibration of the model in this manner and its application is in the magnitude of the events being considered.

The fundamental limitation of this approach is that all the uncertainty in the each of the design inputs (e.g. IFD, ARF, temporal patterns, spatial patterns), modelling (model conceptualisation and parameterisation) and the flood frequency analysis (e.g. rating curve, choice and fitting of the distribution) is reflected in the resulting loss values. The loss simply becomes an error term to compensate for all of the uncertainty and biases in all other inputs. It is therefore not surprising that the values derived from such an approach (e.g. Walsh et al. (1991); Flavell and Belstead (1986)) typically display a large range and relating such values to physical catchment characteristics (that should influence infiltration and interception) has proven intractable.

A further limitation of such approach is that the resulting loss values are a function of the design flood estimation method itself and are therefore only suitable for application with the same set of inputs. For example, if new or alternate information is available on any of the inputs such as IFD, ARF then the analysis needs to be repeated. Thus, the values only work with a single combination of design inputs.

3.3.3.4. Summary of Approaches

The advantages and disadvantages of these different approaches are summarised in the table below.

Table 5.3.1. Summary of Different Approaches for Estimating Loss Values

Approach	Advantages	Disadvantages
1. Empirical analysis of at-site rainfall and streamflow records	• Data is directly relevant to the location of interest and explicitly accounts for the catchment characteristics.	• Only applicable where the catchment is free of significant regulation and diversions and the land use has been stationary • Most catchments do not have a long period of concurrent pluviograph and streamflow data • Difficulty in selecting an unbiased sample of events • Does not guarantee that values result in unbiased estimates of floods

Approach	Advantages	Disadvantages
		• Small sample of events makes it difficult to explore distribution of loss values
2. Regional information	• Can be more selective in choice of data sets for analysis • Larger sample of events allows distribution of values and relationships with characteristics to be explored	• Considerable effort required • Difficulty in selecting an unbiased sample of events • Does not guarantee that values result in unbiased estimates of floods • Difficultly in linking loss values to rainfall and catchment characteristics
3. Reconciliation of design values with independent flood frequency estimates	• Checks that, when combined with the other design inputs, the loss values produce unbiased estimates • Loss values implicitly account for the nature of the design rainfall; whether rainfall bursts or complete storms	• Unlikely to have a long-term stationary streamflow record at the location of interest • If sufficient streamflow is not available, reliance on estimates of regional flood frequency analysis introduces additional uncertainty • Different combination of loss values can result in same flood estimates but has different impact when applied outside of the magnitude used for reconciliation • All of the uncertainty in the design process is attributed to the loss which makes it difficult to infer link with rainfall and catchment characteristics • Unlikely to have sufficient information to be able to define distribution of loss values

If there is a long-term stationary streamflow record at the site, reconciliation of design values (Option 3) is preferable but if the distribution of loss values is required this will typically need to be inferred from previous studies (Option 2). In majority of cases, there will be insufficient streamflow data available at the site and therefore a combination of regional information

(Option 2) and reconciliation of design values with regional flood frequency estimates (Option 3) will typically be the most appropriate approach.

For urban catchments it is more difficult to obtain independent flood frequency estimates and therefore values will often need to be inferred from at-site data (Option 1) or values obtained from regional information (Option 2).

3.4. Estimation of Effective Impervious Area

3.4.1. Overview

3.4.1.1. Surface Types

In estimating runoff from urban catchments, four separate types of surfaces are generally recognised and are referred to in this chapter as the following:

- Directly Connected Areas, which consist of:
  - impervious areas (e.g. roofs and paved areas) which are directly connected to the drainage system – referred to as Direct Connected Impervious Areas (DCIA).
- Indirectly Connected Areas, which consist of:
  - impervious areas which are not directly connected, runoff from which flows over pervious surfaces before reaching the drainage system (e.g. a roof that discharges onto a lawn) – referred to as Indirectly Connected Impervious Areas (ICIA).
  - Pervious areas that interact with Indirectly Connected Impervious Areas, such as nature strips, garden areas next to paved patios, etc.
- Pervious areas consisting of parklands and bushland that do not interact with impervious areas.

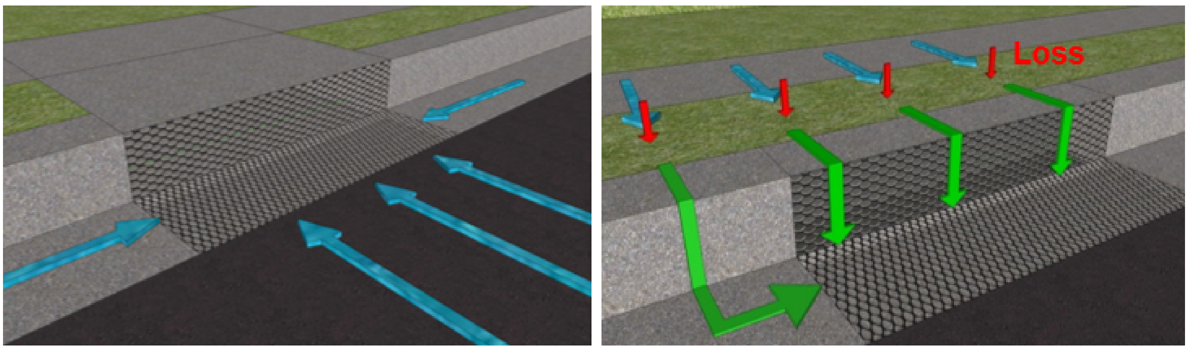


Figure 5.3.6. Example of a Directly Connected Impervious Surface (Left) and an Indirectly Connected Impervious Surface (Right)

3.4.1.2. Challenges with Total Impervious Area

Estimating the catchment imperviousness is an important step in urban rainfall runoff modelling, particularly given the sensitivity of simulated runoff to this parameter in many models (Alley and Veenhuis, 1983). Traditionally, the Total Impervious Area (TIA) is used with the assumption that, neglecting depression losses, this area contributes fully to generating runoff. This is despite the research dating back to the 1970s, identifying the importance of the Effective Impervious Area (EIA) over the TIA (refer to Cherkaver (1975); Beard and Shin (1979)).

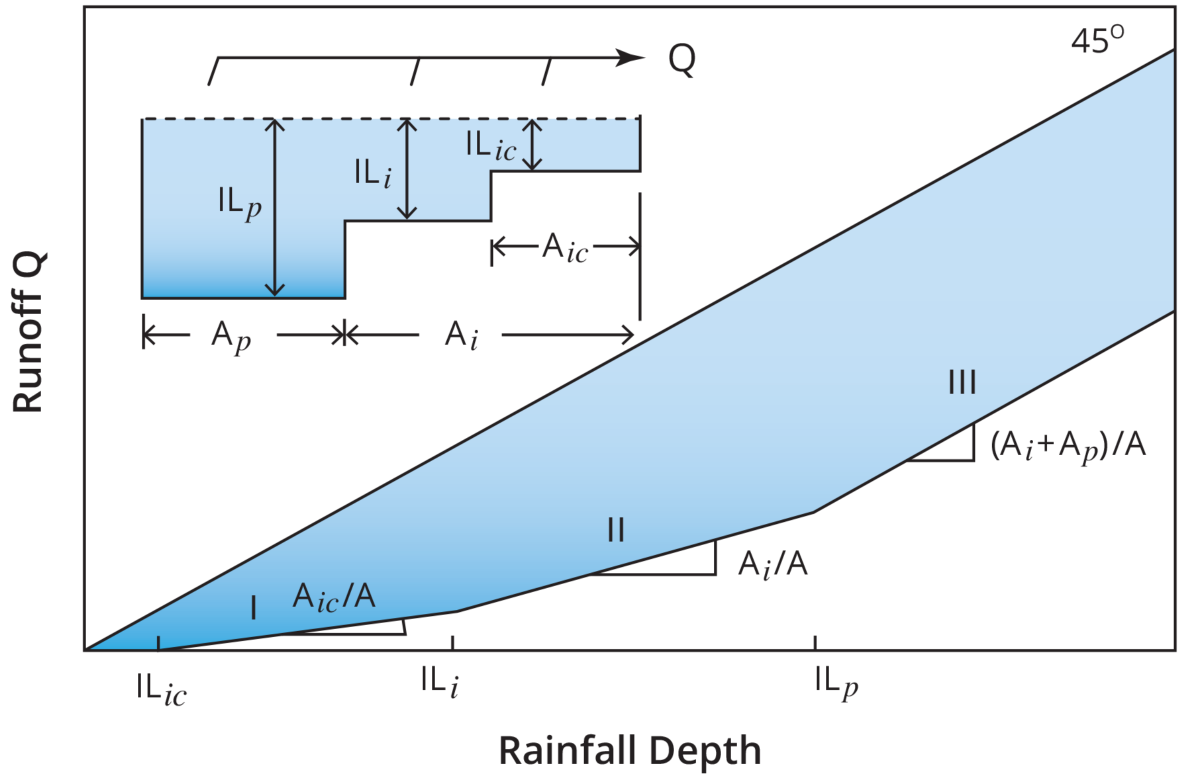
Use of the TIA, which includes impervious areas with no direct connection to the drainage network, can result in the overestimation of urban runoff volumes and peak flows. Although definitions vary, the EIA is generally considered to be representative of the area of the catchment that generates a rapid runoff response in rainfall events. It incorporates the impervious area with a hydraulic connection to the drainage network (DCIA), plus a contribution comprising discharges from an impervious area onto a pervious area (ICIA), which rapidly saturates and acts in a similar manner to an impervious area. The EIA therefore provides a more realistic measure of the impervious area that generates runoff at the catchment outlet.

3.4.1.3. Conceptualisation of Runoff Process

As rainfall continues to fall, it would be expected that additional indirectly connected impervious areas would start to contribute to runoff. A simplified representation of this is shown in Figure 5.3.7. In this schematic, when the initial loss for the Indirectly Connected Area is saturated, the Indirectly Connected Area (comprising pervious and impervious areas) will start to contribute to the runoff. Similarly, once the initial loss of the pervious area is saturated, the pervious area will start to contribute to runoff.

This conceptualisation was observed in Phillips et al (2014) by plotting cumulative runoff of the observed rainfall, the observed discharge, and the estimated runoff based on the calculated EIA estimate (Figure 5.3.8). The deviation of the cumulative observed discharge form the calculated cumulative EIA discharge would suggest the point of Indirectly Connected Area contribution.

It is noted that in Phillips et al (2014), the Indirectly Connected Area incorporated all residential components of the catchment outside of the EIA. Only areas such as large parklands, bushland areas etc were separated out of the analysis. This was following a detailed review of the behaviour, identifying only two discernible responses from within the urban components of a catchment. This approach is recommended.



A = Area
IL = Initial Loss
i = Impervious
p = Pervious
ic = Indirectly Connected Impervious

Figure 5.3.7. Schematic of Rainfall Depth v Runoff, from Boyd et al. (1993)

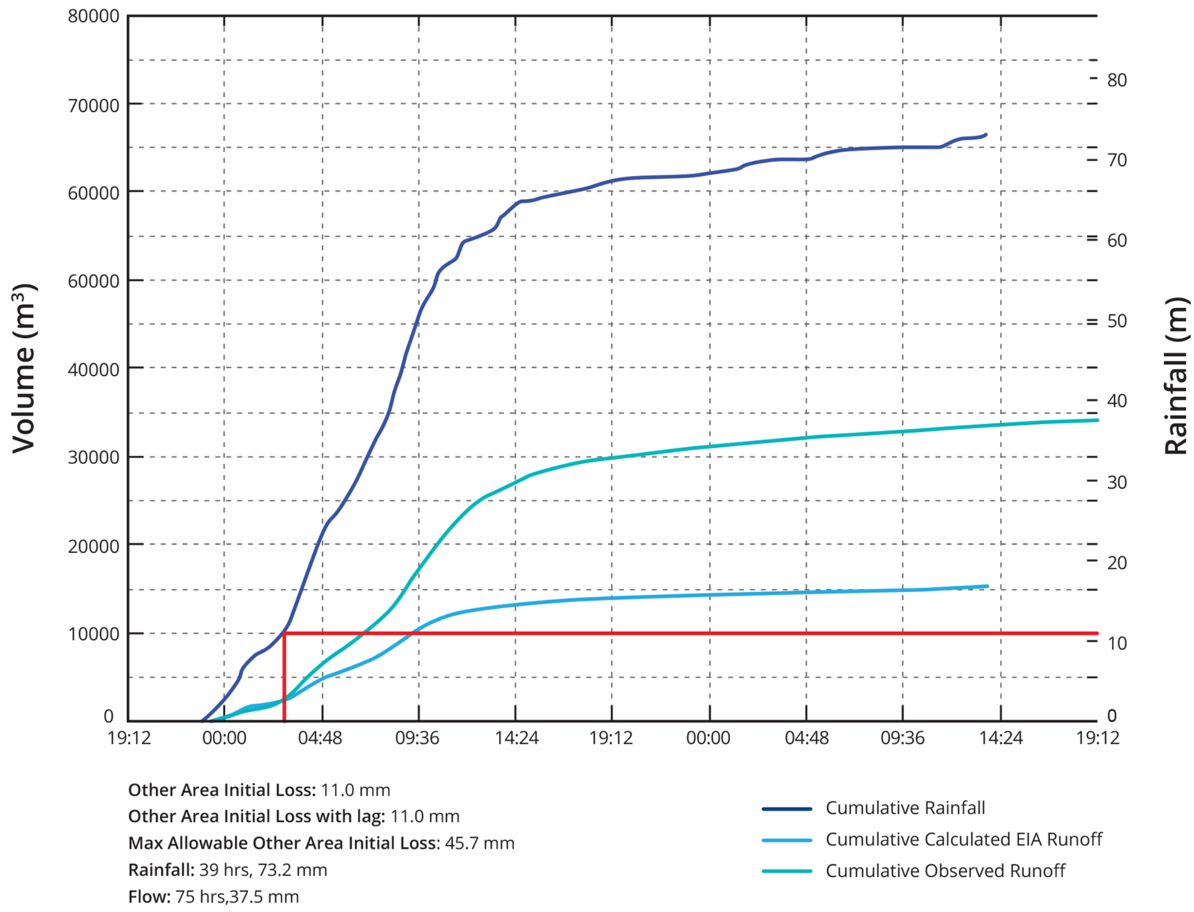


Figure 5.3.8. Cumulative Discharge Plot from Giralang (ACT), showing Cumulative Rainfall, Observed Runoff and Estimated EIA runoff (Phillips et al, 2014)

3.4.2. Estimating Effective Impervious Area

There are a number of methods for estimating EIA. These include:

- Regression analysis of streamflow and rainfall records, where sufficient data exists;
- Adoption of typical EIA/TIA ratios, based on available literature;
- GIS Mapping of TIA areas.

These are described in more detail below.

3.4.2.1. Regression Analysis

The EIA can be estimated using regression techniques on gauged urban catchments, where there are sufficient gauging records to do so. This method provides the most accurate method for estimating EIA for a specific catchment, as it does not require the extrapolation of relationships from other catchments.

This method is done by comparing flow records with a representative rain gauge that is located within or very near the catchment. The key to this method is isolating the runoff that occurs only from the EIA, and not from the other impervious and pervious areas. A method for doing this is detailed in Phillips et al (2014), with an overview of the general approach provided in Figure 5.3.9. An example of the output of this kind of analysis is provided in Figure 5.3.10.

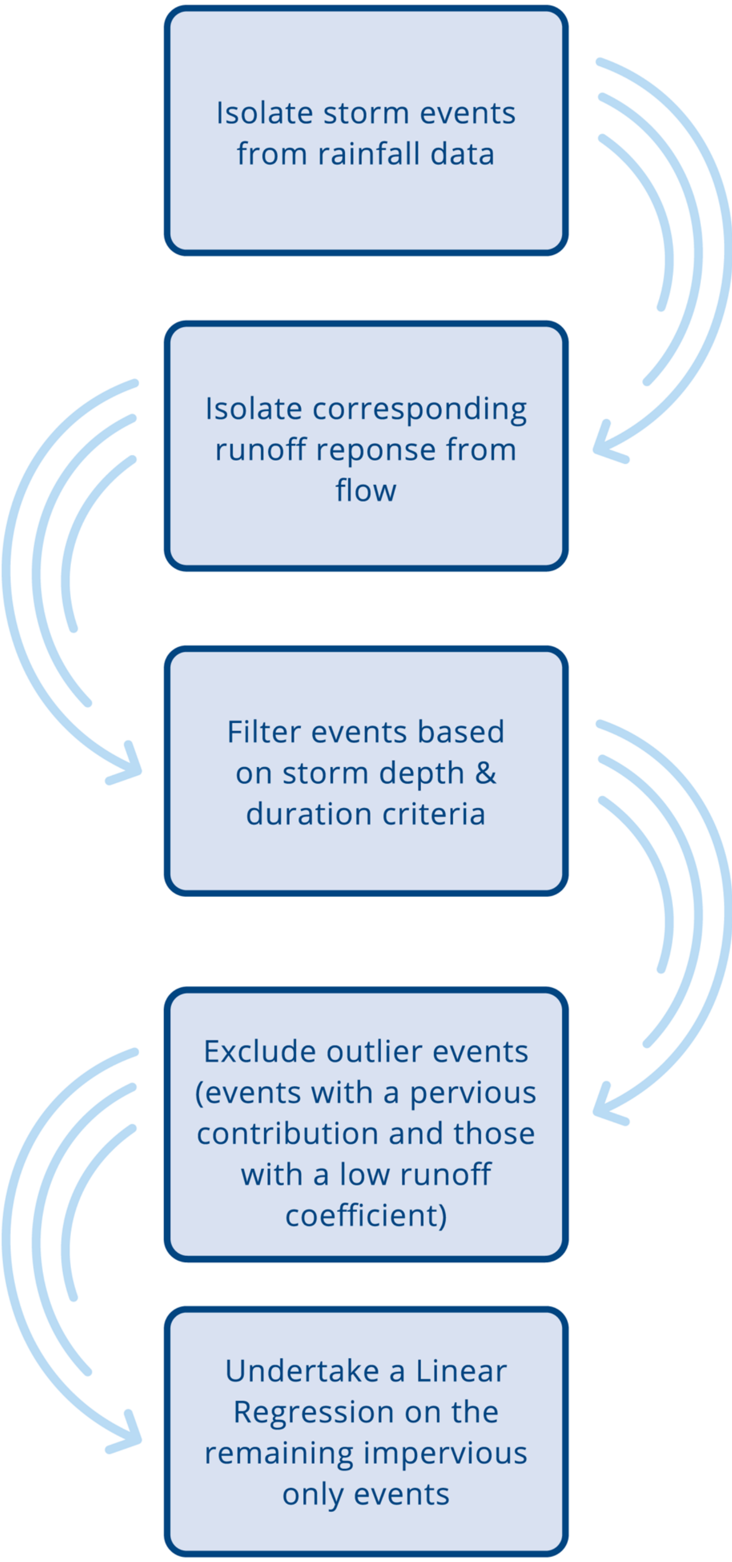


Figure 5.3.9. Overview of Regression Analysis Approach

As identified in Phillips et al (2014), the key requirements for this method are:

- A sufficient gauging (both rainfall and flow) record to undertake the analysis. Phillips et al (2014) adopted a 10 year record as a minimum, although the technique that was applied for the EIA estimation could potentially be used for much shorter records. Where shorter records are adopted, the data should also be checked to ensure that there are sufficient range of rainfall events in terms of magnitude, in order to create a reasonable regression.

- The catchment must have an acceptable gauge rating. Further details on this are discussed in Book 3, Chapter 2. It is noted that because of the technique to isolate EIA events, very large events are generally excluded due to the presence of pervious area runoff. In the absence of any detailed studies on this, a gauge that has an acceptable rating up to around an AEP of 20% may provide a reasonable representation, as long as all events above the acceptable flow level of the gauge are excluded. It is important that engineering judgement is undertaken in reviewing the suitability of the data set.

- A relatively small catchment area. A catchment area of 5 km^2 was used as a target catchment area for the Phillips et al (2014) analysis, although there is no strict guide as to what is appropriate. Larger catchment areas result in a number of potential issues:

  - Greater likelihood for influences of hydraulic controls, catchment storages etc. influencing the runoff;

  - Greater difficulty in isolating the EIA runoff due to longer lag periods from upper catchment areas;

  - More potential for baseflow which will need to be excluded from the analysis;

  - Spatial variation of rainfall becomes more important, and therefore more gauges should potentially be used in the analysis. This will require spatial averaging techniques for rainfall, as discussed in Book 2, Chapter 4.

- A relatively stationary upstream catchment during the period of record (ie. minimal changes in land-use, development intensity etc.).

This process is analytically intensive and is unlikely to be applied in simple applications.

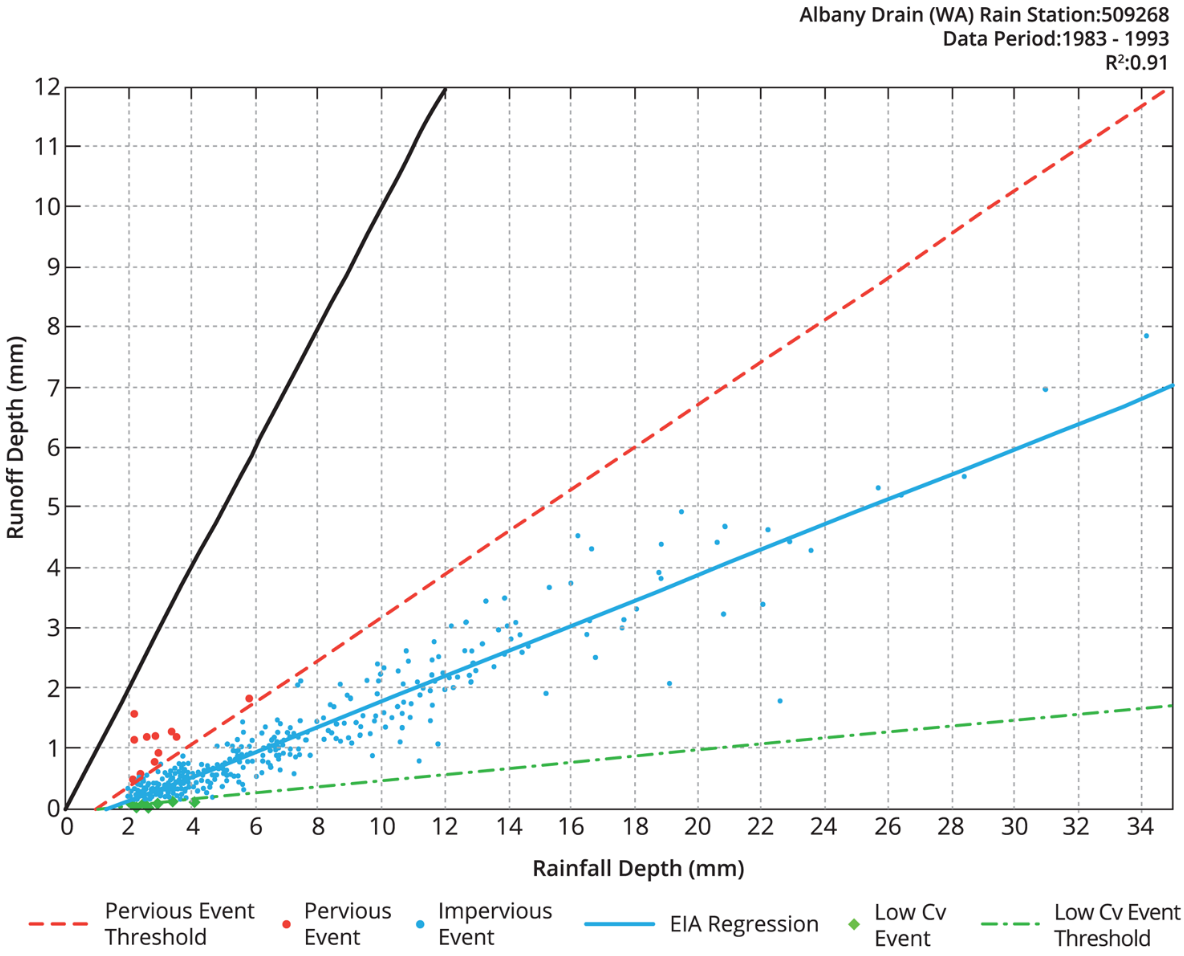


Figure 5.3.10. Example Regression Analysis for Albany Drain Catchment in Western Australia

3.4.2.2. Adoption of Typical EIA/TIA Ratios

Where appropriate flow gauging and rainfall data does not exist, an alternative method for estimating EIA is based on available research on similar catchments.

3.4.2.2.1. Relevant Research

EIA/ TIA

The EIA/ TIA ratio has been found in a number of studies (e.g. Phillips et al (2014); Ball and Powell (1998); Boyd et al. (1993)) to be a good indicator, removing the variability of total imperviousness to create a measure that can be extrapolated to other catchments.

Australian Estimates

Phillips et al (2014) analysed 8 separate catchments throughout Australia using the regression analysis discussed in Book 5, Chapter 3, Section 4. These catchments spanned across all 8 states and territories. However, it is noted that there is only limited representation from the northern part of Australia, with only one catchment from Darwin in the Northern Territory included. This is a reflection of the general urban densities throughout Australia.

This study identified that EIA is typically 55 to 65% of the TIA for most of the catchments identified. This range was recommended in the study in estimating the EIA for most Australian catchments. Based on a sensitivity analysis undertaken within Phillips et al (2014) of some of the key assumptions, the estimates of EIA are expected to fall within +/- 5% to 10% of this estimated range.

It is noted that one catchment from the ACT was identified to have a higher ratio of 74% to 80%. It was theorised that this is likely due to the higher degree of connected surfaces (as discussed in Goyen (2000)), although there were insufficient additional catchments to confirm this hypothesis.

The Phillips et al (2014) study also estimated the DCIA using GIS methods, which primarily included road, roof and driveways (where these driveways drained to the street). It is noted that the road and roof area represents the majority of this area. The analysis suggested that the EIA was roughly in the range of 70 to 80% of this area, suggesting that not all of the roof and road area contributed to runoff.

A summary of results from Phillips et al (2014) are presented in Table 5.3.2.

In order to derive the estimates of EIA/ TIA, Phillips et al (2014) used detailed mapping of different land-uses and aerial photography to estimate the TIA. This was undertaken by taking representative areas within the catchments, detailing the impervious areas, and then extrapolating this to the wider catchment based on land-use mapping which was also derived from aerial photography.

Table 5.3.2. Summary of Effective Impervious Areas Results

Catchment	Total Area (ha)	Urban Area[a] (ha)	TIA (ha)	Urban TIA Fraction[b]	EIA/TIA	DCIA (GIS)/ TIA	EIA (Reg.) / DCIA(GIS)
Albany Drain (WA)	8.2	8.2	2.9	35%	59%	83%	71%
McArthur Park (NT)	144	120	53.7	45%	66%	93%	70%
Giralang (ACT)	91	61.8	28.4	46%	74 to 80%	95%	82%
Parra Hills Drain (SA)	55.1	48.5	26.9	55%	56%	87%	64%
Kinkora Road (VIC)	202	184	122	66%	59%	87%	68%
Powells Creek (NSW)	232	223	152	68%	59 to 63%	81%	75%
Ithaca Creek (Qld)[c]	926	262	128	49%	55%	95%	58%

Catchment	Total Area (ha)	Urban Area[a] (ha)	TIA (ha)	Urban TIA Fraction[b]	EIA/TIA	DCIA (GIS)/ TIA	EIA (Reg.) / DCIA(GIS)
Argyle Street (TAS)	1900	491	292	59%	63%	93%	68%

[a]The urban area for these catchments was based on the residential developed areas, excluding parklands, bushland etc.

[b]The Urban TIA fraction is defined as the percentage of impervious area in the urban area and was based on the desktop GIS method.

[c]Note that Ithaca (QLD) and Argyle St (Tas) were noted to be limited due primarily to large pervious (bushland) areas in these catchments, which influences the results

Ball and Powell (1998) estimated the EIA for Powells Creek in NSW (also analysed by the Phillips et al (2014) research). The analysis of rainfall and runoff data was undertaken by also comparing the antecedent moisture (AMC) in the catchment, based on rainfall in the days leading up to the storm event (refer Book 5, Chapter 3, Section 4). The analysis showed an EIA percentage of the total catchment area (not the TIA) of ranging from 35% to 44% depending on the AMC. Based on the TIA estimated in Table 5.3.2 for the same catchment, this represents an EIA/ TIA of around 53% to 67%, depending on the AMC. This range is very similar to that found under Phillips et al (2014), both for Powells Creek as well as for the wider catchments analysed. Similarly, Chiew and McMahon (1999) undertook an analysis of Powells Creek and found an EIA to total catchment area of around 40%.

Zaman and Ball (1994) undertook a study on Salt Pan Creek in NSW (Southern Sydney). This study estimated the EIA using two alternative methods. The first looked at estimates using orthophoto maps and applying estimates of the EIA to different land-uses. The second analysed rainfall and runoff records to estimate the EIA. Both methods estimated an approximate EIA to Total Area of 39% to 41% of the total catchment area. However, it is noted in the description of this catchment that there are open space areas, making it difficult to directly compare this to other studies. Also, there was no clear description of the TIA to be able to provide a comparative EIA/TIA ratio. Chiew and McMahon (1999) by comparison estimated an EIA to Total Area of 27% for this catchment.

Boyd et al. (1993) analysed 26 catchments, 9 located within Australia (Sydney, Canberra and Melbourne), while the others were international (USA, Canada, UK, Japan and a number of European countries). This study undertook a similar analysis to that of Phillips et al (2014). A regression analysis was undertaken on the EIA/Total Area versus the TIA/Total Area and estimated that the EIA/ TIA ratio of around 74%. A summary of the results is provided in Book 5, Chapter 3, Section 8.

One key thing to note from this study is that there were a number of catchments where the EIA was identified as being greater than the TIA. This is unlikely to be the case in reality, unless the pervious areas have little infiltration. This was not discussed in the paper, but it assumed that the TIA estimated based on aerial photography may not have been appropriate. The data from this study was re-analysed where EIA > TIA catchments are excluded from the analysis, with the results shown in Figure 5.3.12 (the Australian catchments are circled for reference). This was also mapped against Urban Area (ie excluding large pervious areas like bushland and parks), rather than Total Area, to normalise it with the Phillips et al (2014) study and exclude the effects of bushland etc. in the catchments. The re-analysed EIA/TIA ratio is 71%, which is close to the 55% to 65% range identified in Phillips et al (2014).

Dayaratne (2000), which is also referenced in O'Loughlin and Stack (2014), obtained relationships with housing density from modelling storms on 16 gauged residential catchments in four Victorian municipalities:

$$\mathrm{DCIA}/\mathrm{TA}\ (\%) = -0.85\mathrm{hhd}^2 + 23.38\ \mathrm{hhd} - 101.19\ \left(R^2 = 0.90\right) \tag{5.3.3}$$

$$\mathrm{ICIA}/\mathrm{TA}\ (\%) = -0.04\mathrm{hhd}^2 + 1.13\ \mathrm{hhd} - 3.79\ \left(R^2 = 0.91\right) \tag{5.3.4}$$

Where hhd = number of houses per hectare.

It is important to note that this study was based on a range of 7 to 14 houses per hectare. Beyond this range, the equation has significant limitations, as demonstrated in Figure 5.3.11 (ie DCIA reduces with increasing households per hectare for households greater than around 15 per hectare. Also, DCIA reduces below 0% for less than 5 households per hectare). The Phillips et al (2014) results are also shown on this graph for reference.

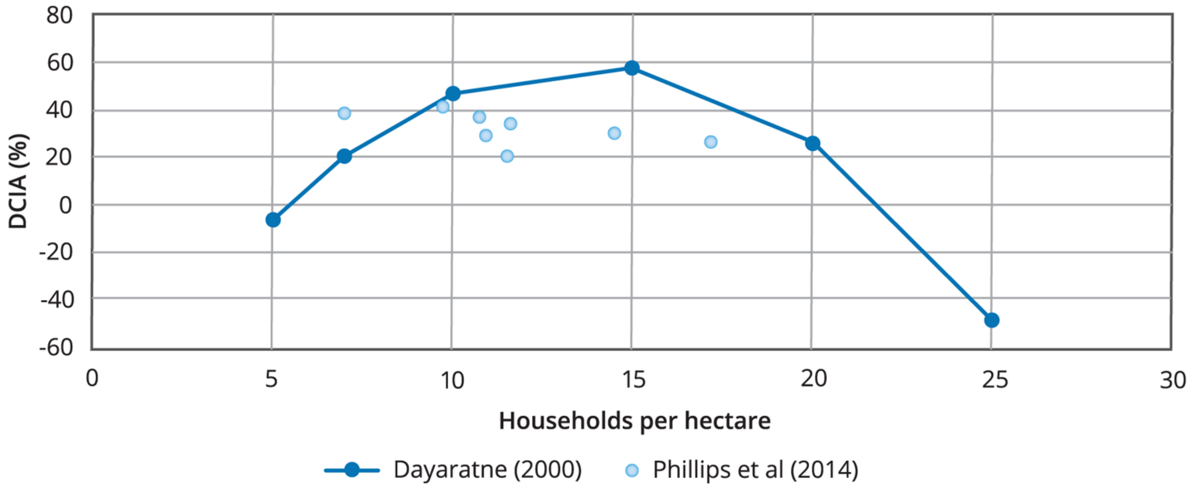


Figure 5.3.11. Representation of Dayaratne (2000) Relationship for DCIA

A further review of just the Australian data from the Boyd et al. (1993) study was compared with that of the Phillips et al (2014) study. The results of this review are shown in Figure 5.3.13 (and removing those catchments which overlap in both studies). The EIA/ Urban Area from this analysis is estimated to be around 60%.

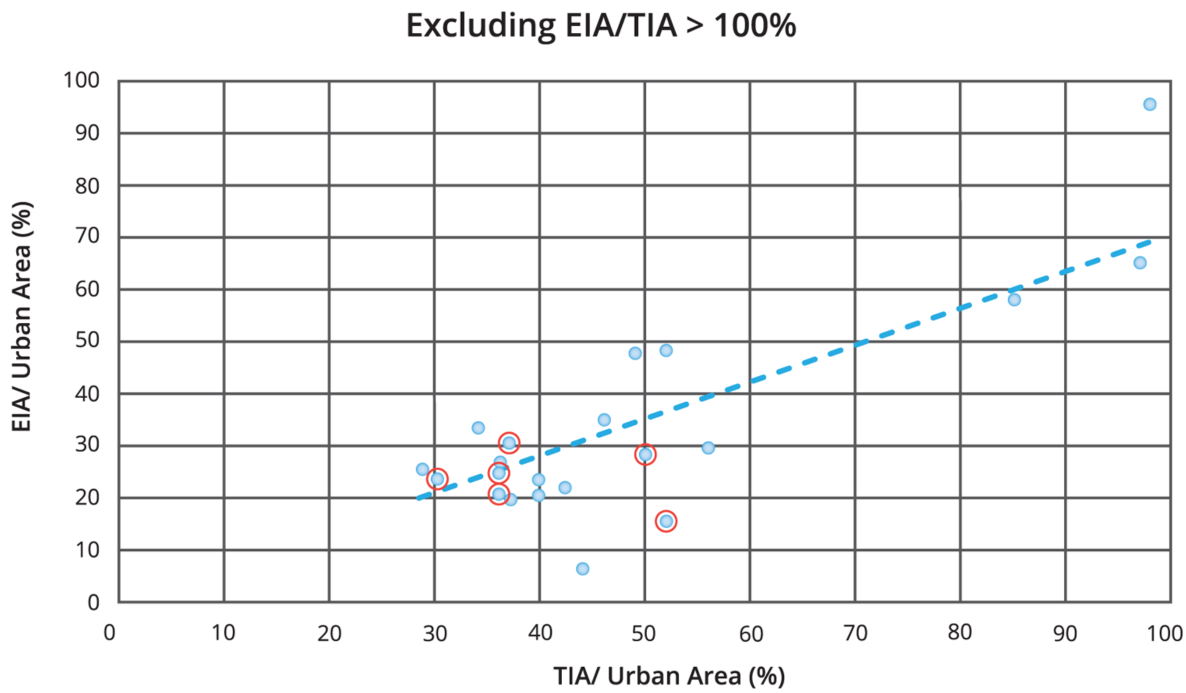


Figure 5.3.12. Re-Analysis of Boyd et al. (1993) Data[1]

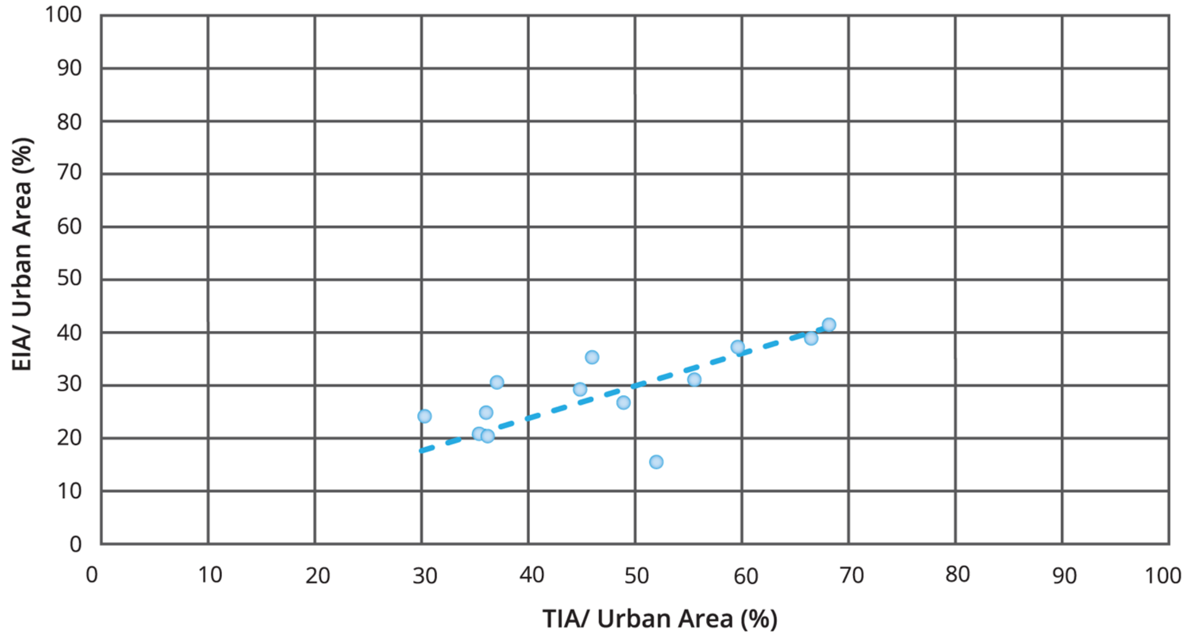


Figure 5.3.13. Australian data from Phillips et al (2014) and Boyd et al. (1993)

International Estimates

Table 5.3.4 provides an overview of international literature on the estimation of EIA/TIA.

The research by Alley and Veenhuis (1983) shows close correlation with the Australian studies for residential catchments (between 53% to 77%, refer Table 5.3.4). This study also incorporated commercial and industrial land-uses, indicating 94% and 77% respectively. As

[1]Australian catchments circled for reference

many of the studies have been undertaken in residential (or predominantly residential) areas, this provides a useful comparison for commercial and industrial land-uses.

Alley and Veenhuis (1983) also derived a relationship between the TIA/TA ratio versus the EIA/TA ratio as a combination of all 19 catchments analysed. This equation is as follows:

$$^{EIA}/_{TA} = 0.15\left(^{TIA}/_{EA}\right)^{1.41} \quad (5.3.5)$$

It is noted that when applying this relationship both the TIA/TA and EIA/TA should be expressed in terms of the percentage multiplied by 100.

In the Boyd et al. (1993) research, 23 of the 26 catchments were predominantly residential (although some had low rise apartments). Non-residential catchments included Sample Road (highway, industrial), Fort Lauderdale (large shopping mall) and Vika (city centre). Both Sample Road and Fort Lauderdale resulted in much higher EIA/TIA, with 74% and 98% respectively. With Fort Lauderdale being nearly completely impervious, this high value of EIA/ TIA would seem reasonable.

Pompano Creek (USA) in the Boyd et al. (1993) data was identified as having limited pipe infrastructure, and where water flowed through grass swales prior to reaching this infrastructure. This may provide some basis for suggesting greater infiltration through WSUD style features, although there is insufficient data to draw any detailed conclusions.

A compilation of the available data (where individual catchment data is available) is provided in Figure 5.3.14. Both the Alley and Veenhuis (1983) (the equation for which was derived from data from Denver USA) and the USA/ Canada data from Boyd et al. (1993) generally align, and are generally higher than the European and Australian data. The reason for the difference unknown, and may come down to variations in methods as well as catchment characteristics and rainfall patterns.

Table 5.3.3. Overview of International Literature on EIA/TIA

Country	Catchments	Authors	EIA/TIA	Comments
Finland	7	Melanen and Laukkanen (1981)	75%	As quoted in Boyd et al. (1993)
Denmark	6	Jensen (1990)	90%	As quoted in Boyd et al. (1993)
USA	2	Janke and Wilson (2011)	32 to 33%	Both catchments around 50% impervious.
USA	19	Alley and Veenhuis (1983)	56 to 94%	Analysis undertaken in Denver. See Table 5.3.4. Residential between 56% to 65%.
USA, Canada, UK, France, Denmark,	17	Boyd et al. (1993)	Variable	Refer to Table 5.3.15 and Figure 5.3.12

Country	Catchments	Authors	EIA/TIA	Comments
Sweden, Italy, Norway, Yukoslavia, Japan				

Table 5.3.4. EIA Results of Alley and Veenhuis (1983) for 19 Catchments in Denver, as summarised in Shuster et al. (2005)

			Total Impervious Area (TIA) (%)		**Effective Impervious Area (EIA) (%)**			**Effective impervious area/Total impervious area**	
Land Use	Lot size, in acres	Number of Basins[a]	Mean	Range	Mean	Range	Mean predicted using mean total impervious area[b]	Mean	Range
Single-family residential	< 1/4	12	39	30-49	23	18-32	26	0.66	0.52-0.66
	1/4 - 1/2	2	26	22-31	15	11-19	15	0.56	0.52-0.61
	1/2 - 1	2	15	13-16	8.5	7-10	6.8	0.58	0.54-0.62
Multifamily residential	-	3	60	53-64	42	33-52	48	0.65	0.57-0.77
Commercial	-	4	88	66-98	83	51-98	83	0.94	0.78-1.0
Industrial	-	1	60	-	46		48	0.77	-

[a]A total of 19 basins were used to derive these averages. However, certain basins had more than one land use type, so that the sum number of basins studied exceeds the sample set

[b]$\mathrm{EIA} = 0.15\ (\mathrm{TIA})^{1.41}, r^2 = 0.98$

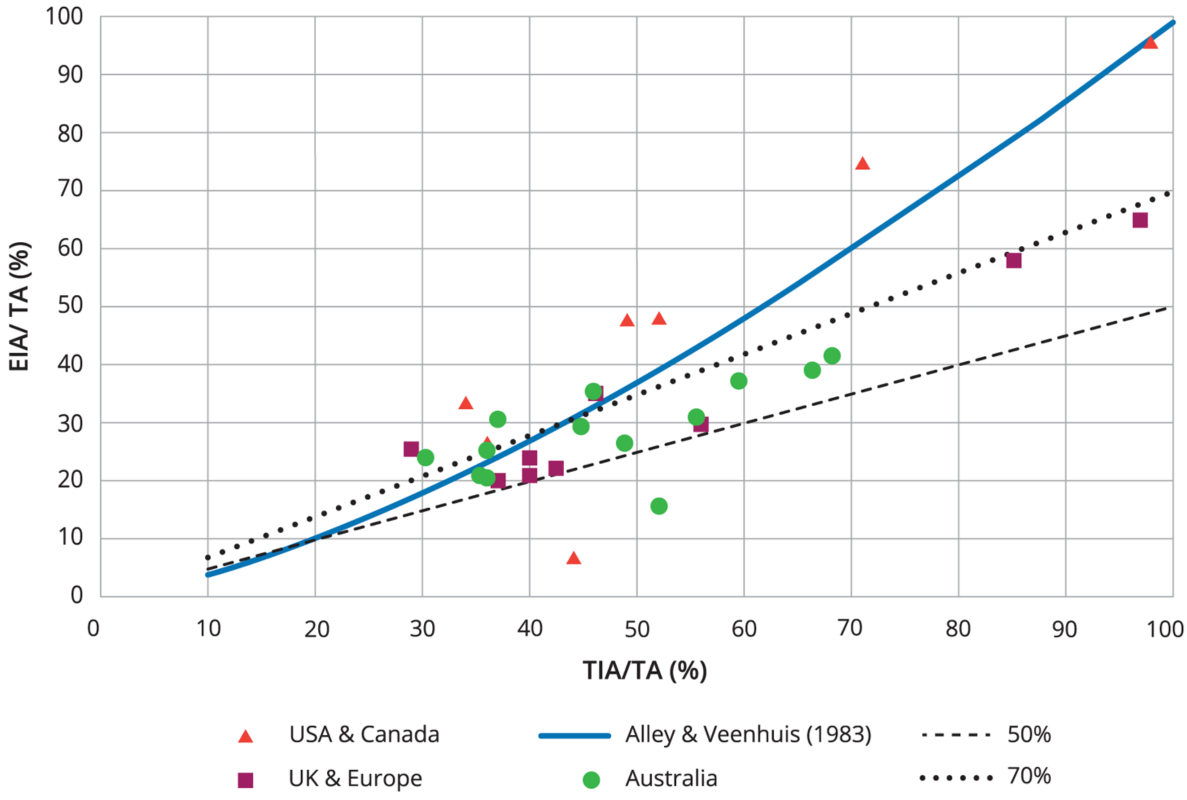


Figure 5.3.14. Compilation of Available EIA Data[2]

3.4.2.2.2. Recommended Values

EIA/TIA

Based on the international literature, and in the absence of any local streamflow and rainfall data, an EIA/ TIA ratio of 50% to 70% would appear to be appropriate for the large majority of urban catchments. Most values from the recent Phillips et al (2014) study fit within a more refined range of 55% to 65%, and this range could be used if the catchments are similar to those described in Phillips et al (2014) (primarily single lot residential).

In choosing the value of EIA/ TIA, the following should be considered:

- Whether the roof areas are connected to the stormwater infrastructure. Where this is not the case, it is likely to be on the lower end of the range; and

- If the drainage infrastructure is piped or whether WSUD features (eg swales) are adopted. Some international studies would suggest that large lengths of drainage swales (rather than pipes) results in a lowering of EIA/TIA, although there is insufficient data to adequately characterise this effect.

The following additional points should be noted in relation to the recommended range of values:

[2]Australian data derived from Boyd et al. (1993) and Phillips et al (2014). USA, Canada and European data is from Boyd et al. (1993). TA = Urban Area for Boyd et al. (1993) and Phillips et al (2014)

The 50% and 70% lines provide the recommended ranges in Book 5, Chapter 3, Section 4

- This range has been adopted primarily from residential catchments, and generally catchments with TIA/ total area of between 30% to 70%. There are no Australian catchments in the literature identified with percentage impervious greater than 70%;

- In one situation in Phillips et al (2014), a catchment in Canberra exhibited EIA/ TIA in the 74% to 80% range. It is possible that this catchment had a higher proportion of "connected" areas, although there is insufficient data to explore this further; and

- Results from US based catchments suggests that for highly impervious industrial and commercial areas, there is a higher level of connectivity, resulting in a much higher EIA/ TIA. In one catchment, nearly 100% EIA/ TIA was observed (although the total imperviousness was also around 100%). However, this was not observed in data from Europe, and the US data does not appear to correlate with European or Australian results. There are insufficient results from Australia with catchments with industrial/ commercial land-uses and high total imperviousness to compare with the international studies. However, it may be appropriate to adopt higher values for EIA/TIA for highly impervious industrial, commercial as well as metropolitan areas (ie total imperviousness greater than 80%).

Estimating the TIA

The above method estimates the EIA as a proportion of the TIA, and therefore needs a reasonable estimate of the TIA. Most of the research undertaken as summarised in Book 5, Chapter 3, Section 4 estimated the TIA based on a detailed analysis of aerial imagery. In most applications, this will be the most appropriate method for estimating TIA. Further discussion on the use of GIS and mapping methods to estimate TIA are provided in Book 5, Chapter 3, Section 4.

3.4.2.3. GIS/ Mapping Methods

GIS methods use data such as aerial photography, drainage maps, land-use maps, cadastral information and terrain to derive estimates of TIA and EIA.

3.4.2.3.1. Overview of GIS Methods

There are different approaches that can be undertaken with the use of GIS. Two different levels of analysis have been identified here:

- Level 1 – undertake mapping of land-use areas within a catchment, and use references to derive the proportion of TIA based on these land-use types; and

- Level 2 – undertake detailed mapping of representative sub-areas within the catchment, and apply the estimated imperviousness from this mapping to the land-use maps from Level 1 analysis.

The Level 2 analysis provides a higher level of certainty, but requires additional work in undertaking the mapping.

The following is a step by step process for a Level 2 analysis:

1. Undertake GIS mapping of key land-use areas in the catchment;

2. Identify small representative areas within the catchment that represent the different land-uses;

3. Undertake detailed mapping of these representative areas. Refer Figure 5.3.15 for an example. Use this mapping to estimate the TIA for the sample area;

4. Apply the estimates from Step 3 to the land-use areas from Step 1, to identify the overall TIA within the catchment; and

5. Using this TIA estimate, the EIA can be estimated from the recommended ratio of EIA/ TIA from Book 5, Chapter 3, Section 4.

Figure 5.3.15. Example Sample Area Analysis for Residential and Commercial Land Use for the Giralang Catchment (ACT)

3.4.2.3.2. Estimating EIA based on GIS Estimate of TIA

Based on the TIA estimates from either Level 1 or Level 2 analysis, the EIA can be estimated based on the EIA/ TIA recommendations as identified in Book 5, Chapter 3, Section 4.

3.4.2.3.3. Estimating EIA using GIS Estimate of DCIA

A Level 2 GIS method will allow identification of different types of impervious areas, which can be identified as being potentially DCIA or ICIA. For example, the impervious areas can be broken into rooves, roads, driveways drainage to the street, footpaths etc. The key challenge in attempting to identify DCIA areas using these methods is that it relies on an interpretation of what is directly connected. Studies such as Phillips et al (2014), Goyen (2000) and Ball and Powell (1998) have shown that DCIA from these methods are generally over-estimates.

Phillips et al (2014) showed that the large majority of what was estimated to be DCIA using GIS mapping of the catchments analysed was road and roof area. The EIA calculated from regression analysis represented approximately 70% (+/-5%) of this area. However, it is noted that this general rule did not apply to all catchments. For example, Giralang (ACT) had a higher EIA/DCIA ratio of around 82%, although this is likely due to the higher degree of connected surfaces (as discussed in Goyen (2000) and also as evidenced by the higher EIA(regression)/TIA ratio of around 78%).

The outcomes of these studies effectively suggest that areas that are traditionally thought of DCIA (e.g. roof areas), are in fact not directly connected. This is likely due to a number of factors, such as transmission loss (e.g. cracks in stormwater pipes), poorly connected roof drainage, drainage swales along roads etc.

The result is that the use of GIS methods to estimate DCIA and subsequently EIA can be problematic. In general, if GIS methods are to be used, then a reduction factor will be required in order to convert the estimated DCIA to the EIA.

In the absence of other information or data, then a range of 70% to 80% could be adopted in converting the GIS DCIA estimate to EIA.

3.4.3. Additional Considerations

3.4.3.1. Antecedent Conditions

Antecedent conditions have the potential to influence the EIA. Ball and Powell (1998), analysing Powells Creek in NSW (Sydney), showed the EIA to total area ranged from around 35% for six days or greater of no rainfall preceding the storm, to around 44% for less than six days of no rainfall preceding the storm. However, other studies have not reported on this effect. In undertaking sensitivity testing on the EIA, Phillips et al (2014) did not identify a clear influence of the antecedent rainfall on the EIA. The relative importance of antecedent conditions is discussed further in Book 5, Chapter 3, Section 7.

3.4.3.2. Changes in Urban Density

Changes in the urban development over time can alter the density of development. This has the potential to influence results of EIA estimates.

For the regression analysis, as identified in Book 5, Chapter 3, Section 4, it is important that this be taken into account. Ideally a stationary catchment (i.e. limited or not change in density) for the period of the gauging record should be used to estimate the EIA.

For GIS methods, consideration should be made on changes to the catchment since the date of the aerial photography. Where the aerial photography is older, it may no longer be representative of the existing development. Alternatively, for analysis of historical flooding, the aerial photography may not be representative of the development at that time.

3.4.3.3. Water Sensitive Urban Design

Water Sensitive Urban Design is common practice for most new development. WSUD principles would be expected to counteract, to some degree, the increase in imperviousness as a result of the development.

The literature in estimating EIA/ TIA ratios is based on catchments with reasonably long flow gauge records (typically greater than 10 years), and with stationary catchment conditions (i.e. limited new development). WSUD has increased in prevalence in design over the last 10 years or so. As a result, it is unlikely that any of these catchments in the research incorporate a large proportion of WSUD features, and even if one or two did, there would be insufficient data to establish any trends.

Therefore, while it is expected that WSUD may influence the estimate of EIA in catchments, there is insufficient data at this time to fully understand the impact.

3.5. Regional Loss Information

3.5.1. Introduction

Book 5, Chapter 3 describes the different approaches to selecting loss models and suitable parameter values for design flood estimation. In most cases there is insufficient data to undertake a detailed analysis of the data and therefore loss values should be inferred from consideration of regional information as well as reconciliation of design values with other independent flood estimates (such as at-site or regional flood frequency estimates). However, if losses are to be derived for the specific study of interest then the analysis should be cognisant of the issues discussed in Book 5, Chapter 3, Section 3.

The recommendations in this chapter have been drawn largely from regional studies of loss values undertaken by Phillips et al (2014) for urban catchments and Hill et al. (2014a), Hill et al. (2015), and Hill et al. (2016) for rural catchments.

These 2 studies concluded that the IL/CL model is typically the most suitable for design flood estimation and hence the recommendations in this chapter relate to this model. If other loss models are to be used for design then it is important that consideration is given to the requirements discussed in Book 5, Chapter 3, Section 2.

The loss values recommended in this chapter are intended for application to complete design storms. Thus the initial loss is denoted as IL_s to indicate that it is applicable to a complete storm. However, if design bursts, rather than complete storms, are used in design then the burst initial loss needs to be reduced to account for the pre-burst rainfall Book 5, Chapter 3, Section 2.

3.5.2. Rural Catchments

This section describes the recommended values of median IL_s and CL for rural catchment. Further description of the development of the prediction equations used to estimate these values is available in Hill et al. (2016).

3.5.2.1. Prediction Equations

The prediction equations used to develop the recommended loss values utilised attributes from the Australian Water Resource Assessment – Landscape (AWRA-L) model system which was developed by CSIRO and the Bureau of Meteorology (Frost et al., 2015). The AWRA-L model simulates the water balance on a continental scale with a spatial resolution of ~5km×5km and daily temporal resolution from 1911 to present (Smith et al., 2016). Model outputs include soil moisture, runoff, actual and potential evapotranspiration (ET), deep drainage and leaf area index (LAI) (Smith et al., 2016). AWRA-L was used to explain the variability of loss for its consistency, continuity and availability on the national scale.

Initial attempts to derive prediction equation using all 35 catchment across Australia resulted in considerable uncertainty in the estimated loss values and therefore prediction equations were developed for different regions which were based upon soil moisture characteristics from AWRA-L. Assessing regions on the basis of differences soil moisture characteristics provides a more logical basis for regionalisation than rainfall alone, as changes in soil moisture reflect the combined influence of climate regime and catchment storage.

The hydrologic similarity was assessed on the basis of two measures representing the seasonality and magnitude of variations in soil moisture. Regional differences in soil moisture characteristics were determined using cluster analysis, and mapping of the identified groups revealed that catchments allocated to the same group were located in largely geographically contiguous regions.

Four regions were defined (refer Figure 5.3.16). Regions 1 and 3 represent the primary summer- and winter-dominant regions, and region 4 largely represents catchments in the south-west of Western Australia. Region 2 represents a more uniform climate: while the region is very large, information is only available on catchment losses for a small eastern portion of this region. The seasonality of average gridded soil moisture in each of the 4 regions is shown in Figure 5.3.17.

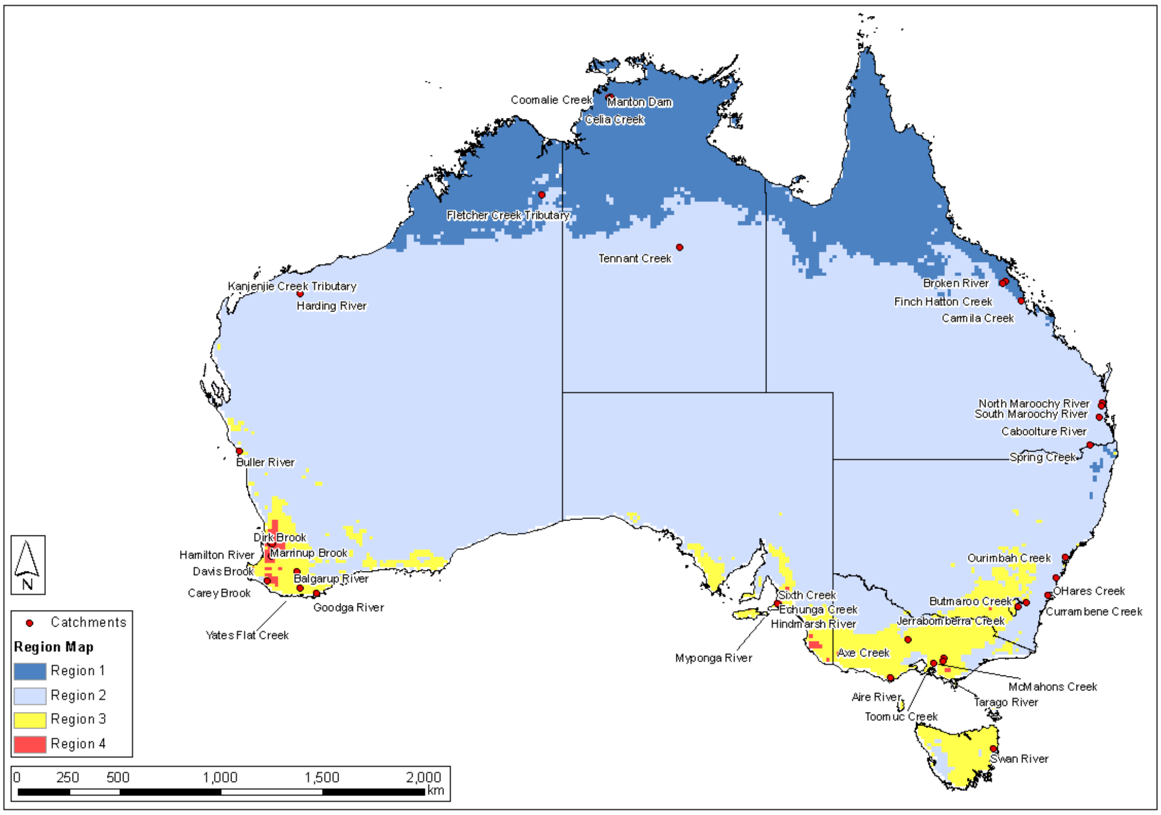


Figure 5.3.16. Regions Adopted for Loss Prediction Equations

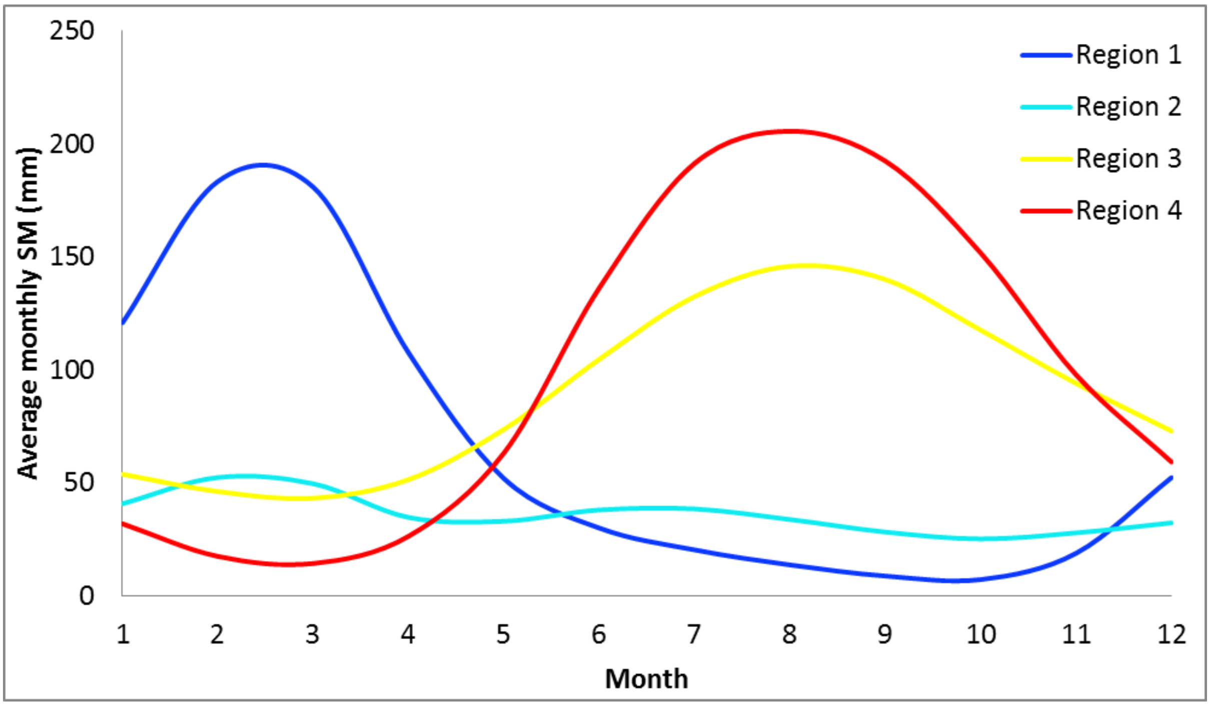


Figure 5.3.17. Seasonality of Average Gridded Soil Moisture in Each Defined Region (Using Gridded Data)

Multi-linear regression was used to develop prediction equations for IL_s and CL in each of the four regions. Given the relatively small number of catchments in each region, the number of independent variables was limited to a maximum of two. The resulting prediction equations are:

Region 1

There are 7 catchments in Region 1 and the prediction equations of loss parameters are displayed below. Initial loss is a function of maximum storage capacity of the shallow soil layer while CL is a function of mean annual PET and surface soil hydraulic conductivity.

$$IL_s = -0.37 * Ssmax + 136.0, R^2 = 0.77, SE = 23\% \quad (5.3.6)$$

$$CL = 2.20 * meanPET - 0.0015 * KO_{sat} - 3.2, R^2 = 0.67, SE = 36\% \quad (5.3.7)$$

Where:

IL_s is the storm Initial Loss (mm)

CL is the Continuing Loss (mm/h)

S_{smax} is the maximum storage of the shallow soil layer (mm)

meanPET is the mean annual potential ET (mm/d)

KO_{sat} is the saturated hydraulic conductivity of surface soil layer (mm/d)

Region 2

9 catchments are in Region 2 and most of them are located near the coast. The prediction equations for this region are:

$$IL_s = 7.89 * meanPET + 0.44 * SOLPAWHC - 47.1, R^2 = 0.68, SE = 24\% \quad (5.3.8)$$

$$CL = 0.77 * KS_{sat} + 0.29 * SO\max - 12.7 \quad (5.3.9)$$

Where:

IL_s is the storm Initial Loss (mm)

CL is the Continuing Loss (mm/h)

meanPET is the mean annual potential ET (mm/d)

SOLPAWHC is the average plant available water holding capacity across catchment (mm)

$KS_{sat}$is the saturated hydraulic conductivity of shallow soil layer (mm/d)

SO_{max} is the maximum storage of the surface soil layer (mm)

Region 3

There are 11 catchments in Region 3 and their loss parameters were estimated as follows:

$$IL_s = -1.57 * s0\_wtr + 0.14 * DES\_RAIN\_24HR + 18.8, R^2 = 0.71, SE = 18.3\% \quad (5.3.10)$$

$$CL = 0.03 * DES\_RAIN\_24HR + 0.06 * SO\max + 5.1, R^2 = 0.38, SE = 45\% \quad (5.3.11)$$

Where:

IL_s is the storm Initial Loss (mm)

CL is the Continuing Loss (mm/h)

s0_wtr is the soil moisture in the surface store in winter season (mm)

DES_RAIN_24HR is the design Rain Intensity (I24,50) (mm)

SO_{max} is the maximum storage of the surface soil layer (mm)

Region 4

There are 8 catchments in this region and the prediction equations are presented as follows:

$$IL_s = -56.2 * slope + 0.28 * SO\max + 16.4, R^2 = 0.47, SE = 21\% \quad (5.3.12)$$

$$CL = 0.088 * SOLPAWHC - 4.9, R^2 = 0.88, SE = 19\% \quad (5.3.13)$$

Where:

IL_s is the storm Initial Loss (mm)

CL is the Continuing Loss (mm/h)

slope is the average slope of catchment (radians)

SO_{max} is the maximum storage of the surface soil layer (mm)

SOLPAWHC is the average plant available water holding capacity across catchment (mm)

The above equations were applied to the relevant regions in Australia using independent variables derived for a grid size of 15 km x 15 km. Given the uncertainty in the prediction equations and the desire to have smooth variations in loss across catchment areas, the gridded values were smoothed using a window of 45 km x 45 km.

Based upon the range of values used in the derivation of the prediction equations, the median loss values were constrained so that the IL_s varied between 0 and 80 mm and the CL constrained between 0 and 10 mm/h.

The range of values for the independent variables and the loss values for the 35 catchments used to derive the prediction equations is summarised in Table 5.3.5 and Table 5.3.6.

Table 5.3.5. Range of Values Used in developing IL_s Prediction Equations

Region	N	Equation	Parameter	Min	Max	Median
Region 1	7	5.5.6	ssmax	180.6	315.4	258.6
			IL_s	22.5	70.0	41.5
Region 2	9	5.5.8	meanPET	3.26	8.61	4.09
			SOLPAWHC	88.48	147.00	118.29
			IL_s	20.0	60.0	37.5
Region 3	11	5.5.10	s0_wtr	0.9	15.9	3.0
			DES_RAIN_24HR	106.1	238.9	137.7
			IL_s	17.0	47.0	27.5
Region 4	8	5.5.12	slope_rad	0.1	0.2	0.1
			s0max	18.2	45.0	29.5
			IL_s	14.0	25.0	18.0

Table 5.3.6. Range of Values Used in Developing CL Prediction Equations

Region	N	Equation	Parameter	Min	Max	Median
Region 1	7	5.5.7	meanPET	4.8	7.7	6.2
			K0_sat	476.5	4153.5	3036.2
			CL	1.6	10.4	5.4
Region 2	9	5.5.9	KS_sat	1.55	9.27	3.37
			S0max	41.37	56.19	46.03
			CL	1.4	8.3	2.7
Region 3	11	5.5.11	DES_RAIN_24HR	1.6.1	238.9	137.7
			S0max	17.2	62.8	42.6
			CL	0.5	6.0	3.1
Region 4	8	5.5.13	SOLPAWHC	82.8	136.9	103.4
			CL	2.2	8.1	3.5

3.5.2.2. Recommended Loss Values

The recommended loss values are shown in Figure 5.3.18 and Figure 5.3.19 and were derived using the prediction equations in the preceding section. For arid areas with mean annual rainfalls less than 350 mm (shown in grey in both figures) there are no recommendations for design loss information because the prediction equations were developed using data from wetter catchments. Recommended loss values can be accessed via the ARR Data Hub (Babister et al. (2016), accessible at http://data.arr-software.org/).

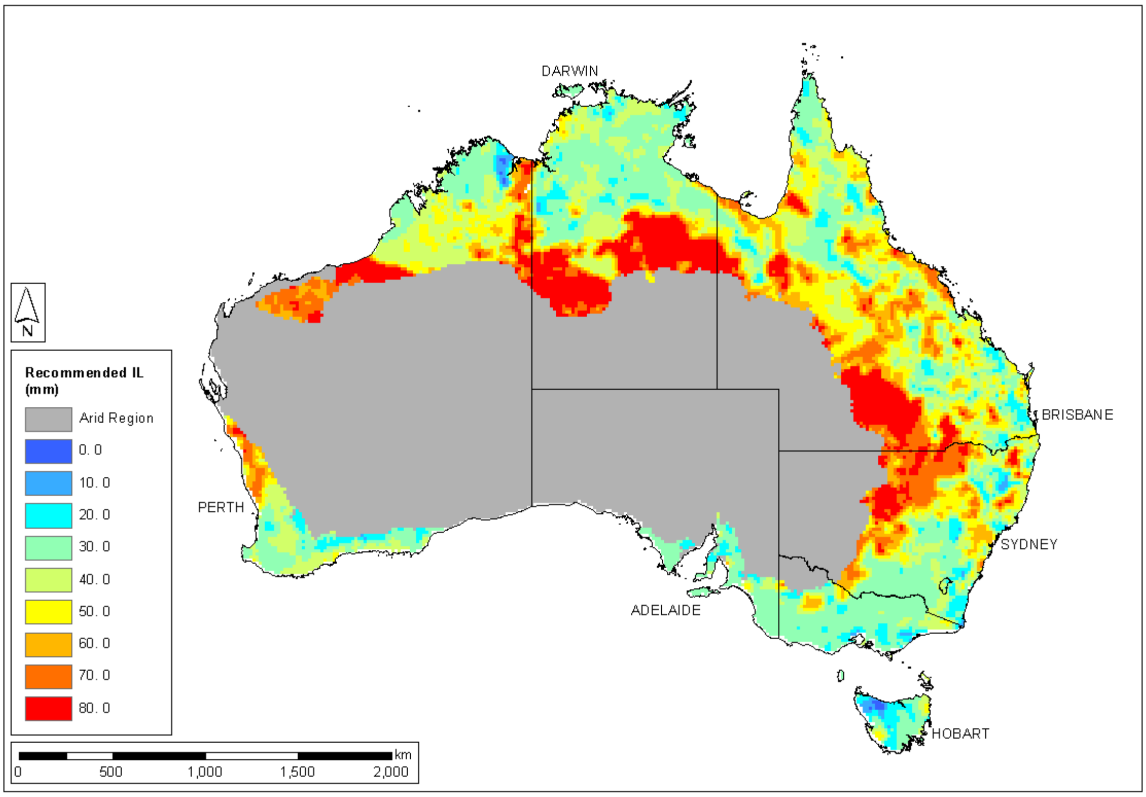


Figure 5.3.18. Recommended Median IL_s (mm)

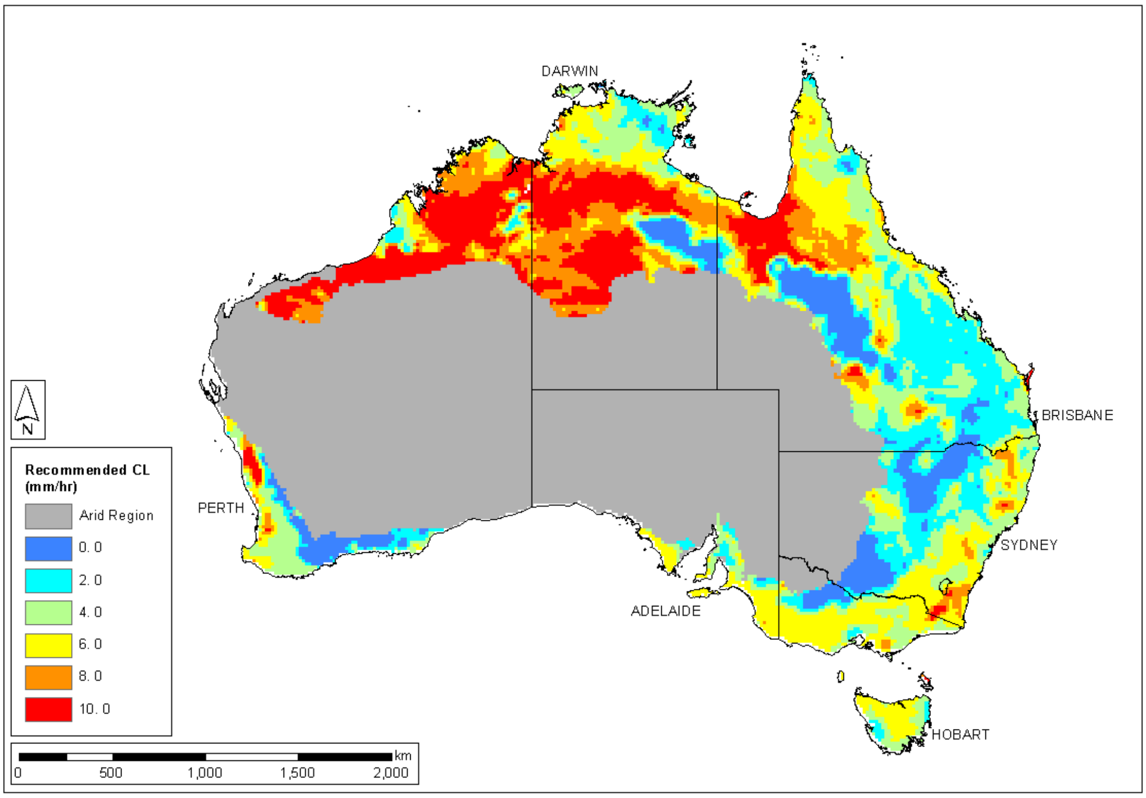


Figure 5.3.19. Recommended Median CL (mm/hr)

It should be noted that the recommended values were derived based upon only 35 catchments and the standard error of the estimates range between 20% and 50%.

Because of the limited number of catchments available, the prediction equations are based upon one or two independent variables. However, it is anticipated that a wide range of characteristics combine to influence the loss values for a particular catchment and therefore judgement is recommended when selecting suitable values for use in design. For example for catchments with very dense vegetation, it would be expected that the loss values would be higher. Similarly, steep catchments with little vegetation would be expected to have lower loss values. Any such adjustment from the regional values should be done giving consideration to the range of loss values obtained in Hill et al. (2014a) and other studies and the implications on the design flood estimates.

Lastly, it is important to note that the recommended loss values in the above figure relate to the *median* for a particular catchment. It is expected that the loss for any particular event could lie well outside of this range. For many catchments, the storm initial loss for any particular event could range from nearly zero, if the storm occurs on a wet catchment, to more than 100 mm if there is little antecedent rainfall.

3.5.2.3. Loss Values for the Arid Region

There is generally a lack of suitable catchments to estimate loss values for the arid regions of Australia.

Kemp and Wright (2014) noted high loss rates for the Gammon Ranges in mid-north South Australia. They attributed the high values to the runoff processes being dominated by the large amounts of storage within the gravel bed of the tributaries and main streams which absorbs a significant amount of runoff from the hillsides. Thus the initial loss represents loss in the tributaries and main stream in addition to that occurring within the catchment. This explanation is supported by observations in arid western New South Wales (Cordery et al, 1983).

Board et al. (1989) estimated losses for the Emily Creek and Todd River catchments in central Australia by calibrating a RORB model to 3 floods. The IL_s varied from 10 to 60 mm and the CL varied from 1.5 to 4.5 mm. It should be noted that the events were selected based upon the largest floods and hence the sample is likely to be biased to wet antecedent conditions which would indicate that the IL_s values are likely to underestimate the median value.

The median loss values in the Pilbara are some of the highest in Australia. There was only 1 catchment from the Pilbara included in the Hill et al. (2014a) study but information is also available from Pearcey et al. (2014) which documents the calibration of RORB models to 19 catchments in the Pilbara region. Although the events in Pearcey et al. (2014) were selected on the basis of streamflow rather than rainfall (and hence potentially biased towards wet antecedent conditions), they support high values of loss.

For the Pilbara, Flavell and Belstead (1986) recommended IL values of approximately 40 to 50 mm and a CL of 5 mm/h. It should be noted that the loss values were derived from reconciling rainfall based estimates with flood frequency analysis and thus the IL reflects a burst initial loss and a higher initial loss would be expected if complete storms are adopted so the range of IL reported by Flavell and Belstead (1986) should be considered a lower limit of expected IL_s values.

3.5.2.4. South-Western WA

The runoff characteristics of much of south-west Western Australia are different from that found in many other parts of Australia. The highly permeable soils and large soil water storages of the south-west landforms means that the continuing loss rates tend to be high.

The dominate contribution to runoff is believed to be saturated areas in the broad valley floors which represent a relatively small proportion of the total catchment. For catchments where runoff is predominately generated via this mechanism, then a storage capacity loss model such as SWMOD should be applied to estimate the rainfall excess (Refer Book 5, Chapter 3, Section 2). For other catchments in south-west WA the IL/CL model is recommended.

For Southwest WA the rainfall and losses are markedly seasonal in nature and it should be noted that the 160 events analysed by Hill et al. (2014a) were biased towards the winter months with 70% of them occurring in May, June or July. Considerations for the selection of seasonal loss values is discussed in Book 5, Chapter 3, Section 7.

3.5.2.5. Collation of Loss Values

To support the recommended loss values, values from a range of studies have been collated and summarised in Appendix A. The loss values have been drawn from the following studies: Waugh (1991), Hill et al. (1996), Ilahee (2005), Rahman et al. (2002), El-Kafagee and Rahman (2011), Hill et al. (2014a) and Loveridge (Unpublished). These studies have been selected on the basis that the loss values were derived directly from the analysis of rainfall and streamflow (rather than reconciliation of design flood estimates with flood frequency quantiles) to ensure that the values are independent of the design inputs used in their derivation. Furthermore, in each study the sample of events was selected based upon rainfall rather than streamflow (to avoid any bias towards wet antecedent conditions).

For some studies the CL was estimated as the volume of loss (less the IL_s) divided by the duration of the event post the commencement of surface runoff. As discussed in Book 5, Chapter 3, Section 2 this will underestimate the CL as there are likely to be timesteps where rainfall is less than the CL rate. Values of CL from such studies were removed from the dataset and hence there are less values of CL than IL_s provided in Book 5, Chapter 3, Section 8.

3.5.3. Urban Catchments

As identified, in Book 5, Chapter 3, Section 4, the urban catchments have been conceptualised as EIA, Indirectly Connected Areas (a combination of Indirectly Connected Impervious and Pervious Areas) and Pervious Areas. The following provide guidance on the losses to apply to these areas.

3.5.3.1. Losses for Effective Impervious Areas

3.5.3.1.1. Research

Phillips et al (2014) estimated initial loss based on the EIA analysis undertaken (refer to Book 5, Chapter 3, Section 4), plotting the runoff volume against the rainfall volume. The intersection of the best fit line with the x-axis represents the initial loss on the EIA. Should data be available for a catchment, then this provides a way in which the initial loss can be estimated.

The analysis in Phillips et al (2014) had initial losses ranging from 1 to 3 mm across the country, with no real identifiable trend between the different regions (although the data was limited). A similar approach was undertaken by Kemp and Lipp (1999), Ball and Zaman (1994) and Chiew and McMahon (1999), and these studies identified typical values in the order of 0 to 1mm

Bufill and Boyd (1992) analysed 16 catchments, 10 within Australia and 6 international catchments. Their analysis identified initial loss by estimating the mean initial loss across all events for these catchments. It is important to note that this is a slightly different way of undertaking the analysis from Phillips et al (2014), and therefore can result in slightly different results. However, this study found that typical initial loss rates for the 10 catchments within Australia are around 0 to 1 mm. These catchments were subsequently re-analysed using the EIA technique in Boyd et al. (1993).

Bufill and Boyd (1992) also analysed 6 international catchments, using the method noted above, and found an initial loss ranging from around 0.5 to 1 mm. Boyd et al. (1993) had 17 international catchments in their analysis, with initial loss values typically ranging from 0 to 1.3 mm, although two catchments in the US had 3.7 mm and 6.1 mm.

A summary of the different initial loss estimates available for the different studies are provided in Book 5, Chapter 3, Section 8. It is important to note that there is only one catchment from the northern parts of Australia (i.e. Monsoonal North, Wet Tropics, Pilbara) or the central portion of Australia (Rangelands). However, given the consistency of the estimates across the regions, and with international estimates, it is unlikely that the initial loss for EIA in these areas will be significantly different.

All studies assumed that the ongoing losses on the EIA were effectively zero.

3.5.3.1.2. Recommended Loss Value for EIA

It is recommended to adopt a storm initial loss of between 1 to 2 mm for EIA. Continuing losses for EIA can be assumed to be zero.

3.5.3.2. Losses for Indirectly Connected Areas

3.5.3.2.1. Initial Losses

Literature

One of the key challenges in urban catchments is the lack of gauged urban catchments with reasonable records and relatively stable development. In addition to this, many rainfall events do not produce any Indirectly Connected Area runoff, which can make it difficult to obtain sufficient data to determine appropriate losses.

Phillips et al (2014) used the same catchments as those identified in Book 5, Chapter 3, Section 4 for EIA. In order to isolate events with flow generated from the Indirectly Connected Area, they selected events where the flow generated was 10% higher than the flow estimated by calculating the runoff from the EIA area alone. A number of other criteria were adopted. Further details on the storm event selection are identified in Phillips et al (2014). A summary of the number of events identified for each of the catchments is summarised in Table 5.3.7.

Two catchments from the EIA analysis in Phillips et al (2014) were excluded:

- Parra Hills (SA) – only a limited number of storms were identified. The number of storms was too small to provide any meaningful analysis of the losses.
- Kinkora Road (VIC) – further analysis of the data suggested some unusual behaviour, with periods of runoff with no rainfall and vice versa for the selected events. This catchment was therefore not included in any further analysis.

Table 5.3.7. Total Storms identified for Analysis Phillips et al (2014)

	Giralang (ACT)	Powells Creek (NSW)	Albany Drain (WA)	McArthur Park (NT)	Argyle St (TAS)
Total Identified Storms	41	14	30	20	49

One of the key challenges in the use of the urban data in Phillips et al (2014), and with many other studies, is that the length of record is relatively small. This, together with the filtering method, reduces the number of large storm events to estimate losses. Figure 5.3.20 shows the storm magnitude from the selected events from Phillips et al (2014), based on the ARR 1987 AEPs (storms not shown are less than 63% AEP or 1 EY).

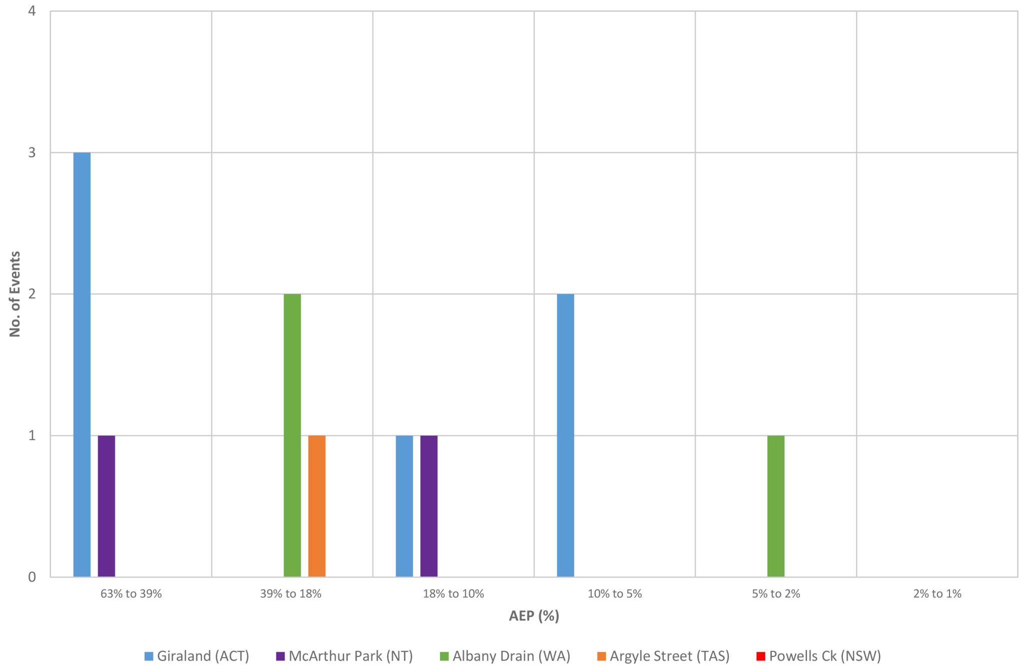


Figure 5.3.20. Storm Magnitude from Phillips et al (2014)

Based on the selected storms, Phillips et al (2014) identified that mean storm initial losses across the range of storms for the catchments analysed were generally in the range of 20 to 30 mm, with the majority between 10 and 40 mm, and would appear fairly consistent across most of the catchments. A summary of the results is provided in Figure 5.3.20, providing an indication of the ranges. Phillips et al (2014) noted that the Argyle Street (TAS) was not well suited to the analysis that was undertaken, due to large pervious areas and bushland. As with the rural catchments, there is significant variation in the results, which is driven by numerous factors such as antecedent conditions and temporal pattern of the storm.

The storms in Phillips et al (2014) were real storms, and may differ to the design storm temporal patterns. Furthermore, the higher loss values may be skewed by rainfall events that occurred where low depths continued for a prolonged period of time at the start of the event. On this basis, the storm initial loss for design storms may be lower than the range estimated in Phillips et al (2014).

Boyd et al. (1994) fitted a model for urban catchment runoff to 3 catchments in Australia (all in ACT). This model conceptualised that the urban catchment was comprised of EIA, pervious area that contributed for small rainfall events (<40 mm) and pervious areas that contributed for larger rainfall events (>40mm). For the “small pervious” area, the study indicated initial losses of 0 to 4 mm, while for the “large pervious” area, the study indicated 30 to 50 mm. It is noted that in other studies the “small pervious” area is effectively lumped into the EIA (such as Phillips et al (2014)), so that the 30 to 50 mm initial loss would be generally consistent with the initial loss concept for the Indirectly Connected Area in this chapter. However, key challenges with the Boyd et al. (1994) catchments is that they are low density (5 to 25% total impervious fraction), and the model applied effectively incorporates a proportional loss into the fraction of “small pervious” and “large pervious” areas.

Kemp and Lipp (1999) analysed three catchments in South Australia. For the Indirectly Connected Areas, they were not able to identify any clear runoff events from these areas for these catchments. Based on the available research, this paper recommended 45 mm of initial loss be adopted for Adelaide, although this could go as high as 60 mm. However, these estimates were identified as preliminary, and there was no runoff records to verify this. Interestingly, Phillips et al (2014) were also unable to identify sufficient events with runoff from the Indirectly Connected Area for South Australia.

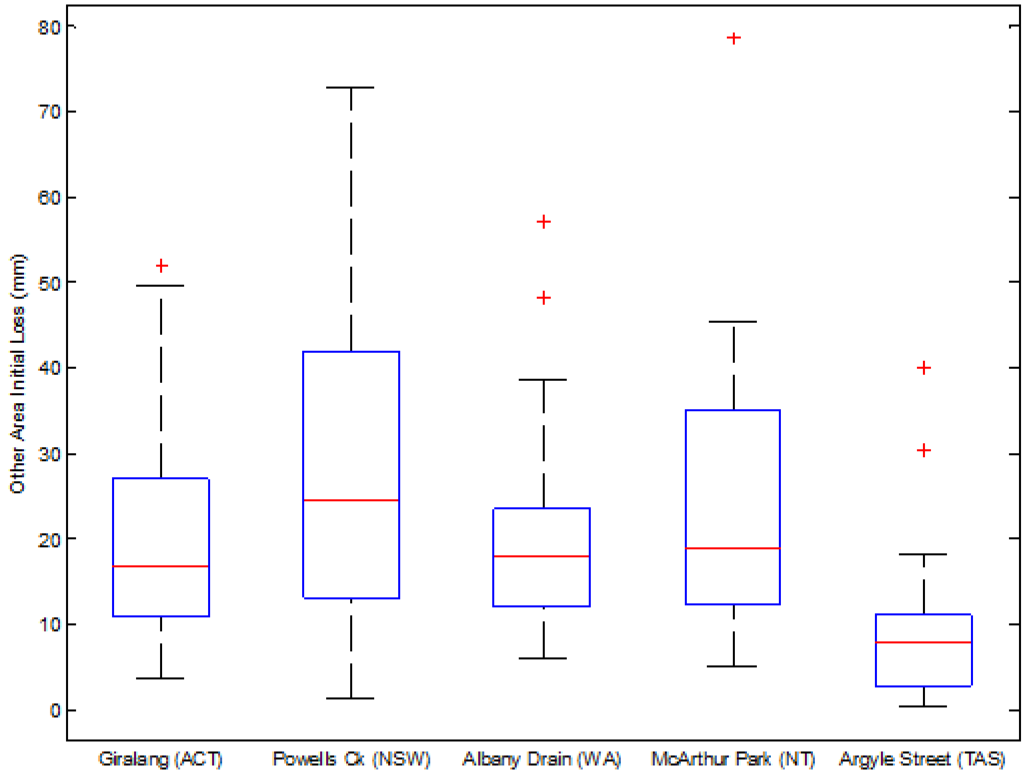


Figure 5.3.21. Summary of Initial Losses for Urban Catchments (from Phillips et al (2014))

Conceptualisation

As per the conceptualisation in this chapter, the Indirectly Connected Area is composed of impervious and pervious areas interacting with each other. The pervious area would be expected to tend towards the rural losses, although will be modified due to urban pervious area modification (such as import of top-soil, differing vegetation etc). However, the impact of the impervious area would be expected to lessen the initial loss across the combined area.

A comparison of the initial loss derived in the literature with the recommended IL_s for Rural catchments is provided in Table 5.3.8. This table also provides the ratio of the Indirectly Connected Impervious Area over Indirectly Connected Area. Key points:

- The initial loss values are similar to the recommended rural loss values although obviously there is significant scatter in both data sets;
- The initial loss values are similar to the recommended rural loss values. Generally, the values are in the order of 60 to 80% of the recommended median Rural IL_s, although obviously there is significant scatter in both data sets;
- There is insufficient data to determine any appropriate relationships with ICIA/ICA with initial loss and its relation to the recommended median rural loss.

- The catchments from Boyd et al. (1994) have almost no ICIA, and the median values for IL_S are in the range of the recommended median rural loss.
- There is insufficient data covering all regions as identified in the rural section.

Table 5.3.8. Comparison of Initial Loss Literature Values with Rural Recommended Values

Catchment	Region	Median IL_S (mm)	Reference	Recommended Rural IL_S (mm)	ICIA/ ICA
Giralang (ACT)	Murray Darling	17	Phillips et al (2014)	23	9%
Powells Creek (NSW)	East Coast	24.5	Phillips et al (2014)	33	43%
Albany Drain (WA)	South-West WA	18	Phillips et al (2014)	31	18%
McArthur Park (NT)	Monsoonal North	18.9	Phillips et al (2014)	25	17%
Argyle Street (TAS)	South-East Coast	7.9	Phillips et al (2014)	27	6%
Long Gully Creek (ACT)	Murray Darling	34	Boyd et al. (1994)	23	0%
Mawson (ACT)	Murray Darling	49	Boyd et al. (1994)	23	5%
Curtin (ACT)	Murray Darling	31	Boyd et al. (1994)	18	0%
South Australia (3 catchments)	South-Central SA	45	Kemp and Lipp (1999)	23	

Recommendation

Based on the limited available information, it is recommended that a median IL_S of 60 to 80% of the recommended rural catchment IL_S be adopted.

It is noted that this may trend towards 100% as the proportion of impervious area in the Indirectly Connected Area reduces. Based on the data that is available, this might occur when the impervious area drops below 5% of the total Indirectly Connected Area.

3.5.3.2.2. Continuing Loss

Literature

The constant continuing losses estimated in Phillips et al (2014) ranged generally from 0 to 4 mm/h across the catchments. However, this excludes the catchment in Northern Territory which was influenced by the presence of a large detention basin that affected results. However, more recent analysis of this catchment with regards to timestep (see Book 5, Chapter 3, Section 7), would suggest that for a 6 minute interval the CL estimate is 2.8 mm/h, which is within the range of the other catchments. Phillips et al (2014) also cautioned with the results of the Tasmanian catchment (Argyle Street), which is influenced by a large bushland area component, significantly larger than the urban area of the catchment.

Both NSW and ACT had median constant continuing loss values in the order of 2.5 mm/h. WA exhibited higher continuing losses, with a median value close to 4mm/h.

A comparison of the median continuing loss values with those of the recommended values for the rural catchments is provided in Table 5.3.9. It is difficult to draw any real conclusions based on the limited data set, other than to note that they are generally in the same range.

Table 5.3.9. Urban Continuing Loss Values Compared with Rural Continuing Loss Values

Catchment	Median CL(mm/h)	
	Phillips et al (2014)	Regional rural estimate from Book 5, Chapter 3, Section 5
Giralang (ACT)	2.5	3.6
Powells Creek (NSW)	2.6	1.8
Albany Drain (WA)	3.8	3.3
McArthur Park (NT)	5.1	4.1
Argyle Street (TAS)	1.4	3.8

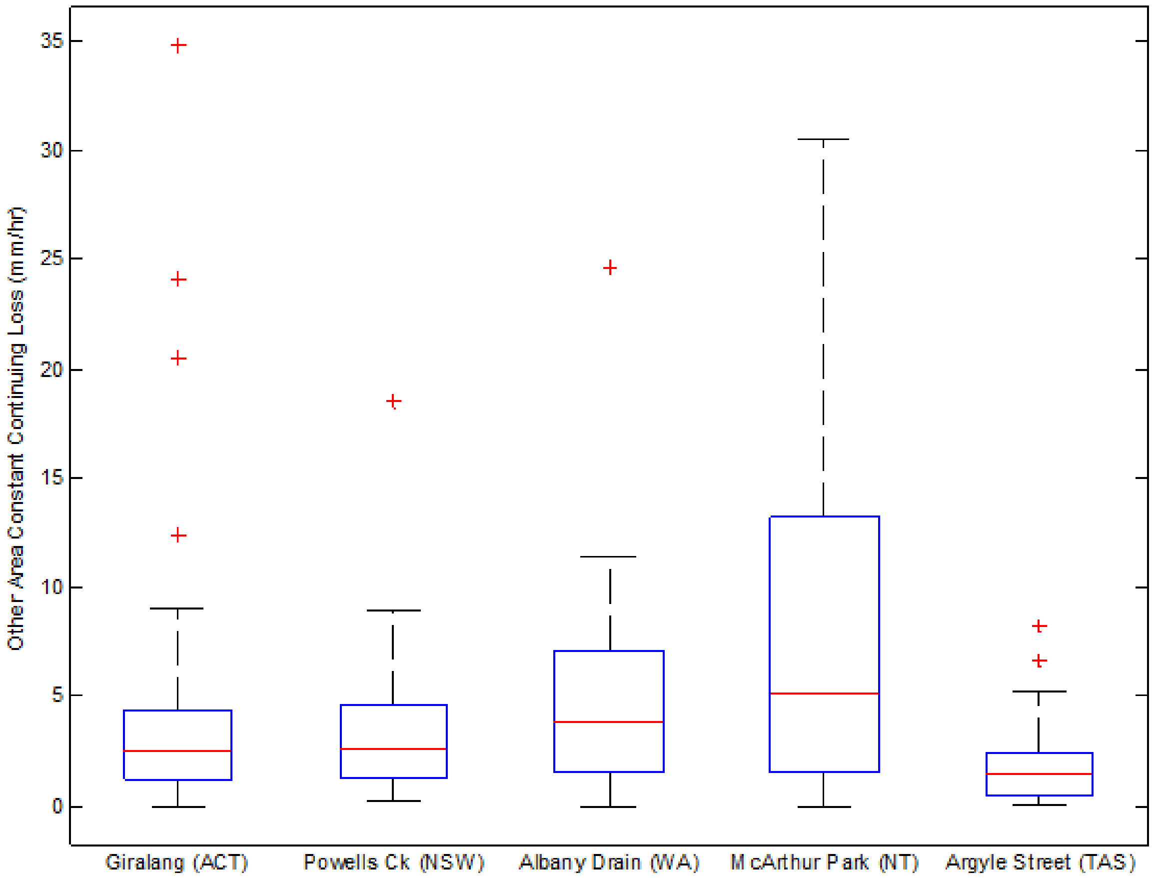


Figure 5.3.22. Indirectly Connected Area Continuing Loss Estimates (from Phillips et al (2014))

Recommendations

In the absence of other data, the following is recommended where appropriate gauging data is not available:

- For southeastern Australia, a typical value of 2.5mm/h, with a range of 1 to 3 mm/h, would be appropriate. The value should be adjusted based on engineering judgement and reviewing the catchment characteristics such as soil types, interaction of indirectly connected impervious areas with pervious areas etc.
- For other areas, adopt a range of 1 to 4 mm/h.
- Similar to initial losses, where the impervious proportion of the indirectly connected area is very low, it may be appropriate to adopt the rural continuing losses. However, there is insufficient data to confirm this.

3.5.3.3. Losses for Urban Pervious Areas

Urban pervious areas represent areas that do not interact directly with impervious areas (i.e. those not within the Indirectly Connected Area), such as pockets of bushland, parks, recreational ovals etc. Traditionally, practitioners have adopted similar loss values for these areas as for those they would adopt in rural areas.

The challenge in the research for these areas is identifying the runoff component from these portions, which are typically dominated by runoff from impervious areas. However, there is nothing to say that these areas will behave the same as rural areas. Areas like parks and sporting fields are highly disturbed from their natural state, and therefore may exhibit very different characteristics. However, with little research and information available, the losses for the rural catchment provide the best estimate that is available at this time.

Therefore, in the absence of better information, it is recommended to adopt the loss values for rural catchments from Book 5, Chapter 3, Section 5.

3.5.3.4. Alternative Loss Models for Indirectly Connected Areas

As noted in Book 5, Chapter 3, Section 5, the recommended loss model for urban catchments is the IL/CL model, based on the results of Phillips et al (2014). However, it is also recognised that a number of other loss models have been and are in use in Australia. Furthermore, there is insufficient data to categorically identify one loss model over another as being preferred, and there are circumstances where specific loss models may suit a particular catchment well.

Two alternative models that are commonly applied in urban environments are the proportional loss models and the Horton loss models. These two loss models are described in the following sections.

Initial Loss - Proportional Loss Model

The initial loss proportional loss model is described in Book 5, Chapter 3, Section 2.

The initial loss for the model should be adopted as per Book 5, Chapter 3, Section 5.

In addition to testing the IL/CL loss model, Phillips et al (2014) also tested the proportional loss model. A summary of the results from this assessment are provided in Figure 5.3.23. As identified in Book 5, Chapter 3, Section 5, some care should be taken with the interpretation of the results from Tasmania (Argle Street) and Northern Territory (McArthur Park).

The key challenge with the results of this analysis is that the results range from median proportional losses of around 45% through to 80%. This makes it difficult to provide a general guidance for catchments.

As discussed in Book 5, Chapter 3, Section 3, the greatest challenge in applying the IL/PL model for design flood estimation is understanding how the proportional loss varies with AEP. Great care should therefore be exercised if the IL/PL is to be applied to events outside of the range of events used in the derivation of the values. For this reason it is generally not considered appropriate for estimating rare or extreme events (ie AEP < 1%).

Therefore, it is recommended that this method only be used where suitable data is available to calibrate the loss model, either for a specific catchment or for a similar catchment nearby. Alternatively, should more research become available this could assist in informing appropriate parameters for design.

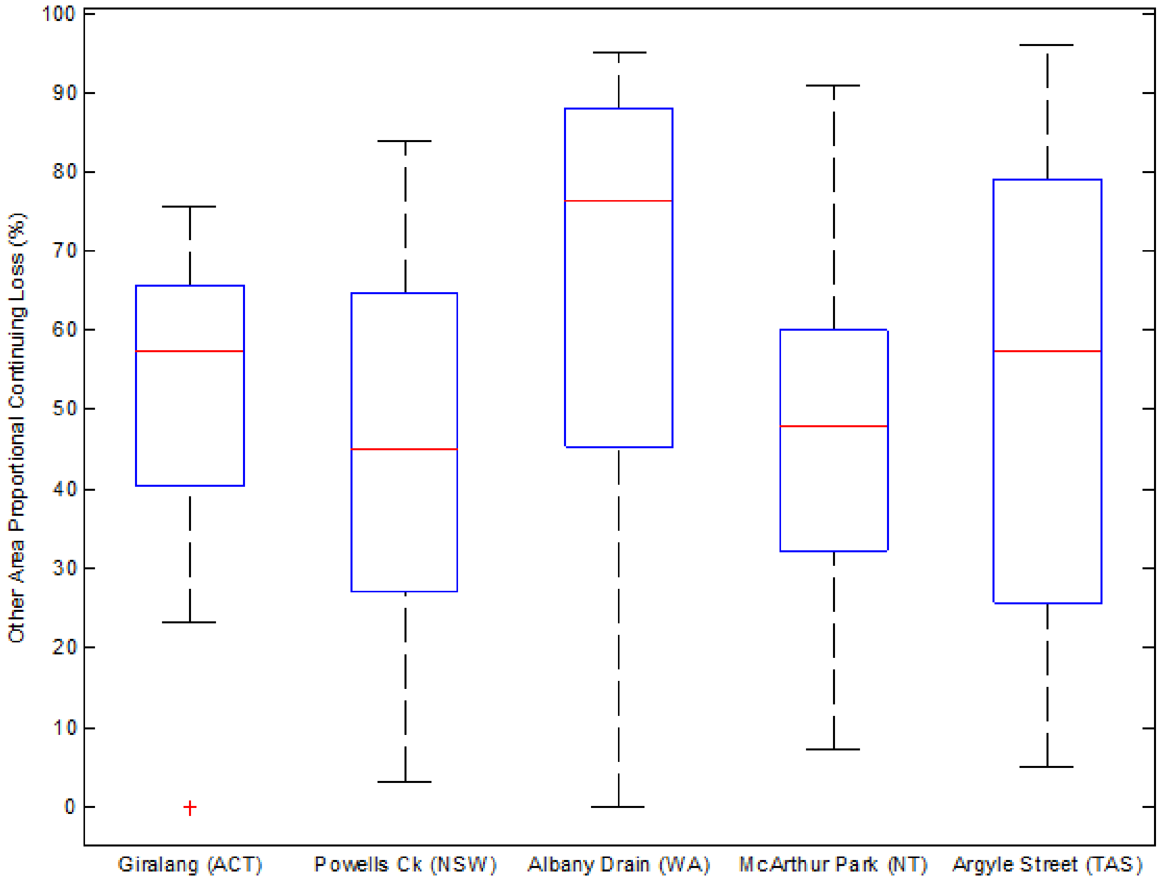


Figure 5.3.23. Indirectly Connected Area Proportional Loss Estimates (from Phillips et al (2014))

Horton Loss Model

The Horton loss model is described in Book 5, Chapter 3, Section 2, with the equation below repeated for reference.

$$f_t = f_c + (f_0 - f_c)e^{-kt} \tag{5.3.14}$$

where:

f_t is the infiltration capacity (mm/h)

f_c is the minimum or ultimate value of f_t

f_0 is the maximum or initial value of f_t (mm/h)

k is a decay coefficient (per hour)

t is the time from the beginning of the storm (h)

This model is for pervious areas only. The hydrological models that use this model separate out the Indirectly Connected Impervious Areas from the Indirectly Connected Pervious Areas, treating the losses separately. An examples of this is ILSAX, as detailed in O'Loughlin and Stack (2014). O'Loughlin and Stack (2014) has been used as the key reference for this section of the chapter, and the parameters reported here are based on this reference.

The Horton model assumes that the losses (or infiltration) of runoff decreases over the duration of the storm. The shape of this decay function is described by the k value, which is typically assumed to be 2 (h-1). The remaining parameters to describe the decay curve are the initial infiltration rate (f_0) and the final infiltration rate (f_c). These are defined by the soil characteristics. Soil classifications that are used are described in Table 5.3.10, which are based on numerous reference and reproduced from O'Loughlin and Stack (2014).

Table 5.3.10. Soil Classifications in Horton Model

Soil Classification	Description
A	low runoff potential, high infiltration rates (consists of sand and gravel)
B	moderate infiltration rates and moderately well-drained
C	slow infiltration rates (may have layers that impede downward movement of water);
D	high runoff potential, very slow infiltration rates (consists of clays with a permanent high water table and a high swelling potential).

In applying the model, a “starting point” is required for the analysis. This represents the infiltration rate at the start of the storm, which is based on the Antecedent Moisture Condition (AMC). The AMC can be categorised from Table 5.3.11 (based on O'Loughlin and Stack (2014)).

Using the soil classification and the AMC number, the Horton Loss Model parameters can be defined based on Table 5.3.12. The resulting loss models for the different classifications, together with the AMC numbers, are shown in Figure 5.3.24.

Table 5.3.11. Antecedent Moisture Condition Number

Number	Description	Total rainfall in 5 days preceding the storm (mm)
1	Completely Dry	0
2	Rather Dry	0 to 12.5
3	Rather wet	12.5 to 25
4	Over Saturated	> 25

Table 5.3.12. Horton Loss Model Parameters

	Soil Type			
	A	B	C	D
Initial Rate (f_0) (mm/hr)	250	200	125	75
Final rate (f_c) (mm/hr)	25	13	6	3
Shape Factor (k) (h^{-1})	2	2	2	2
Initial Infiltration Rates (mm/h) for AMCs				
1	250	200	125	75
2	162.3	130.1	78	40.9
3	83.6	66.3	33.7	7.4
4	33.1	30.7	6.6	3.0

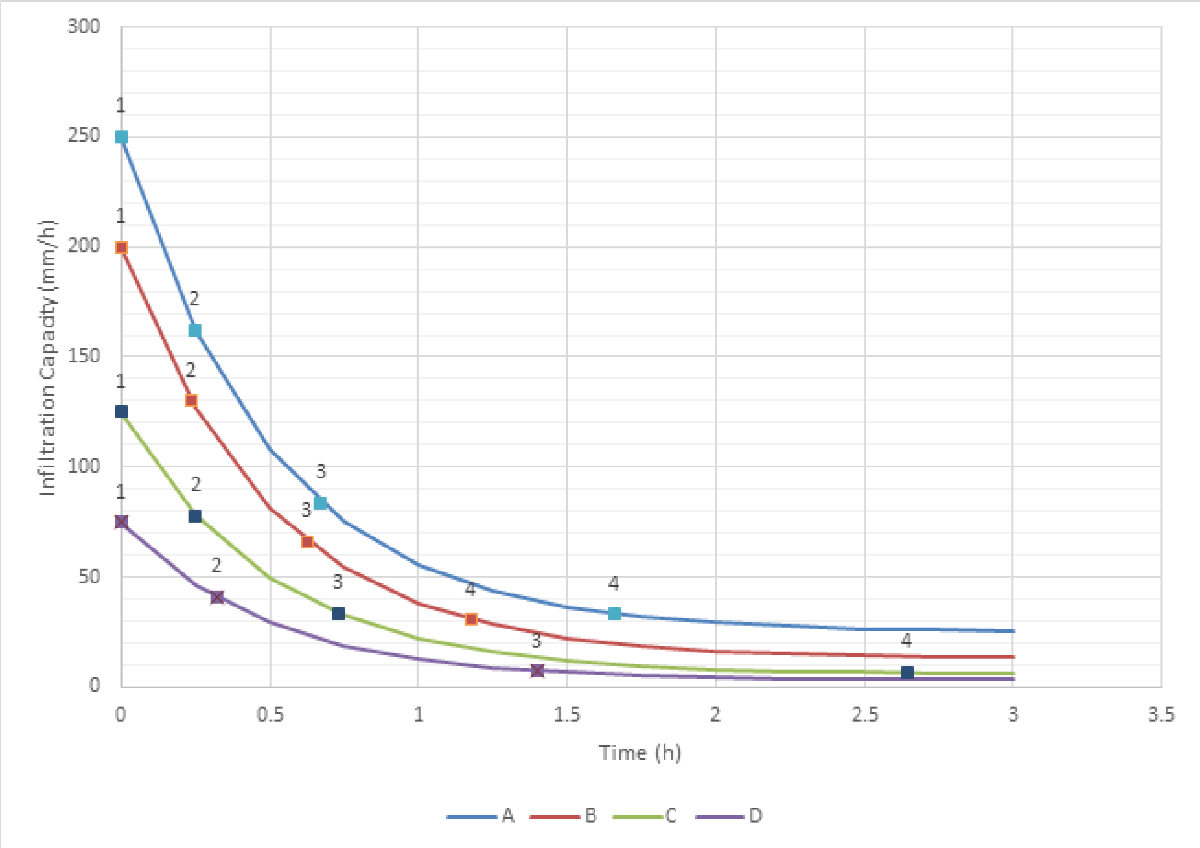


Figure 5.3.24. Horton Loss Model with Different Soil Classifications & AMC Numbers

O'Loughlin and Stack (2014) report that this method has been used in the calibration of a number of ILSAX models to gauged catchments. While no references are provided, it is anticipated that this calibration is for whole hydrological and hydraulic models, which includes a number of parameters not just isolated to losses.

O'Loughlin and Stack (2014) report that Siriwardene et al. (2003) compared the infiltration rates from with those measured with infiltrometers at eight urban gauged catchments in

Victoria. These rates suggested slightly higher rates than those reported in Table 5.3.12. However, the research only focused on the soils, and did not look at the other components that loss models are trying to represent (such as those identified in Book 5, Chapter 3, Section 2).

More research is required to compare the effectiveness of this loss model in comparison to constant continuing loss model.

3.6. Distribution of Loss Values

The discussion in the previous sections concentrates on a single representative (median) value of loss. However, joint probability approaches to design flood estimation allow a distribution of loss values rather than simply some measure of central tendency (eg (Goyen, 1983; Rahman et al., 2002; Nathan et al., 2003; Kuczera et al., 2006)).

The degree of variability in the loss values reflects both natural variability in the factors contributing to loss (initial state of catchment wetness, seasonal effects on vegetation) and impacts of error in rainfall and streamflow data. As long as these errors are of a random rather than systematic nature, they should not bias the estimated loss distribution.

These approaches can be grouped into parametric and non-parametric and are discussed in the following sections.

3.6.1. Non Parametric Approaches

Nathan et al. (2003) describes the derivation of a standardised loss distribution by standardising the values by the median for each catchment. The concept of how the location of the loss distribution changes but not its shape is discussed in Nathan et al. (2003) and is illustrated in Figure 5.3.25. Thus, if the median loss value can be determined, then these standardised distributions can be applied to estimate the distribution of losses for any given catchment.

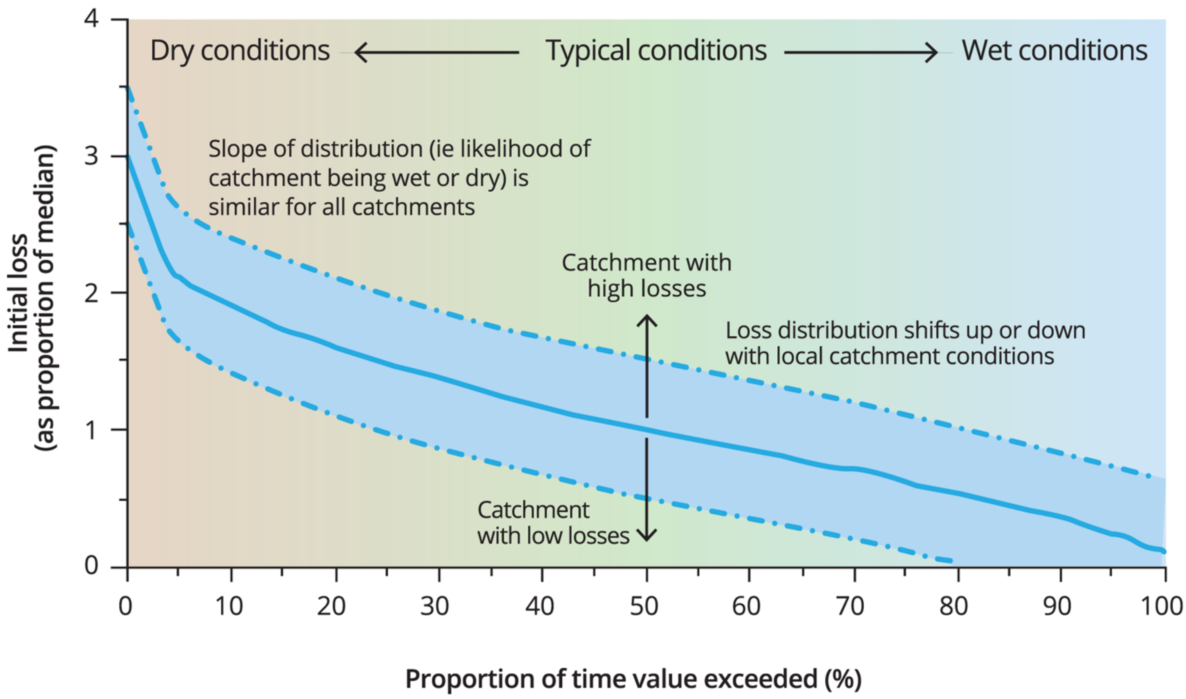


Figure 5.3.25. Variation in Location but Not Shape of Initial Loss Distribution Nathan et al. (2003)

The standardised distributions of storm initial loss and continuing loss from Hill et al. (2014a) are shown in Figure 5.3.26 and the values presented in Table 5.3.13. These standardised loss distributions are remarkably consistent for the different regions across Australia, which demonstrates that while the magnitude of losses may vary between different regions, the shape of the distribution does not.

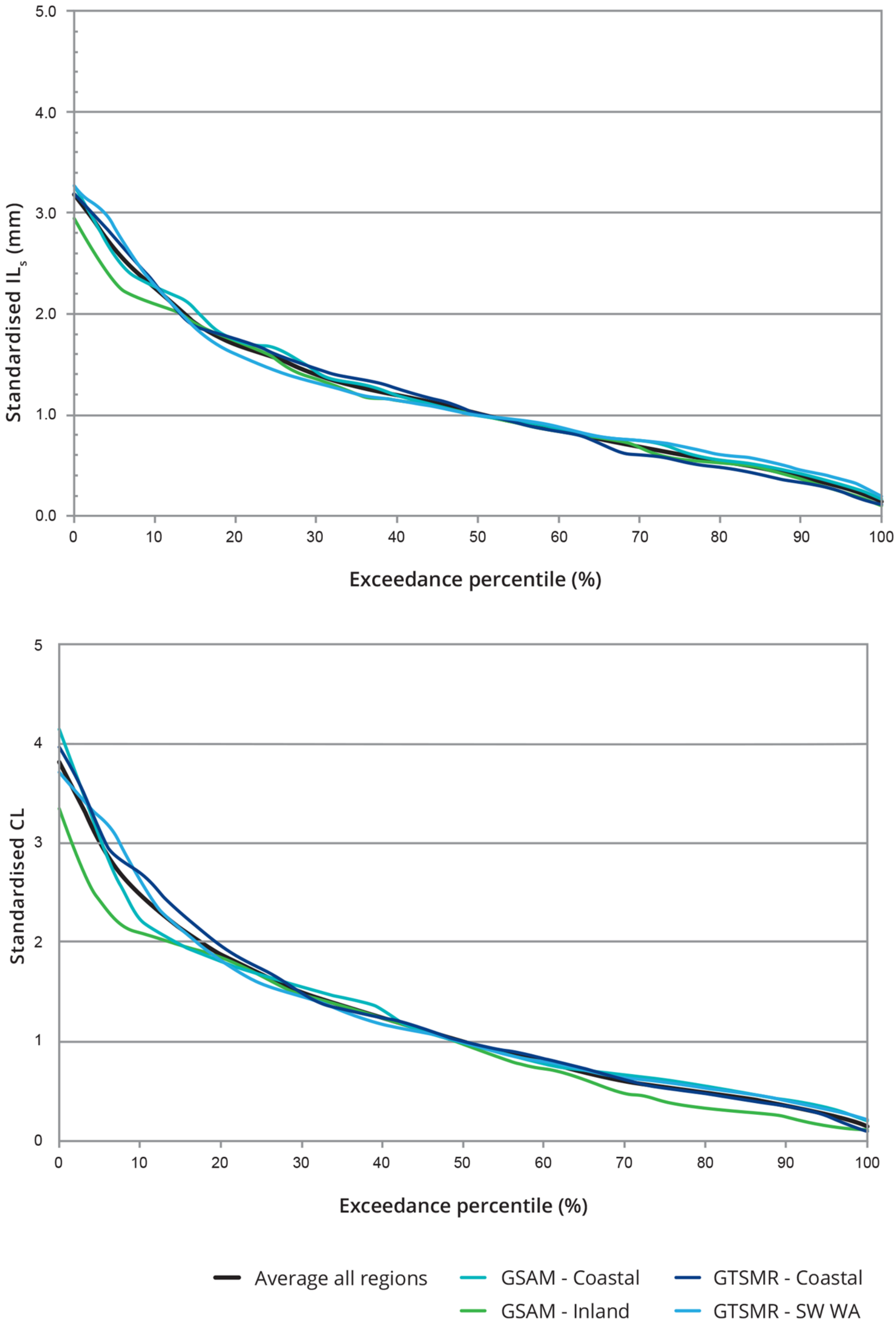


Figure 5.3.26. Regional average standardised loss distributions (Hill et al., 2014a)

Table 5.3.13. Standardised Loss Factors (Hill et al., 2014a)

Percentile	Standardised IL_s	Standardised CL
0	3.19	3.85
10	2.26	2.48
20	1.71	1.88
30	1.40	1.50
40	1.20	1.24
50	1.00	1.00
60	0.85	0.79
70	0.68	0.61
80	0.53	0.48
90	0.39	0.35
100	0.14	0.15

3.6.2. Parametric Approaches

Typical parametric distributions can also be fitted to the sample of loss values derived for a catchment. Parametric distributions provide an efficient means of describing the distribution and help facilitate characterising uncertainty.

A number of studies have investigated different candidate distributions for storm initial loss from samples of catchments from Victoria (Rahman et al., 2002), Queensland (Tularam and Ilahee, 2007) and from around Australia (Hill et al., 2013). Based on these studies the four-parameter Beta distribution is recommended for its flexibility and because its parameters lend themselves readily to physical interpretation. The lower limit can be set to zero, thus reducing the number of parameters to 3.

There has been comparatively less attention paid to the distribution of continuing loss. Based upon a case study of three catchments in Queensland, Ilahee and Rahman (2003) found that the continuing loss values could be approximated by an exponential function. Ishak and Rahman (2006) investigated the probabilistic nature of continuing loss for four Victorian catchments and none of the distributions fitted the observed the distribution satisfactory, however the four-parameter Beta distribution providing the best approximation. Hill et al. (2013) investigated different distributions for 10 catchments from around Australia and concluded that the Gamma (two parameter) distribution was best. Given these different outcomes, further work is required before a preferred distribution for the continuing loss is recommended.

3.6.3. Correlation between Initial and Continuing Loss

The limited number of studies that have explored the correlation between initial and continuing loss values have concluded that there is little systematic dependence between the two. This apparent lack of dependence may reflect reality (for example it might be supposed that variation in continuing loss rates may be more dependent upon rainfall intensity rather than antecedent conditions) or else it may reflect the difficulties of parameter estimation given the limitations of the conceptual model adopted.

It is likely, however, that the observed variation in continuing loss between one event and the next is more due to the propagation of data errors in the analysis rather than differences in event processes. In applying joint probability approaches to design flood estimation, a number of authors have stochastically modelled the storm initial loss while keeping the continuing loss at a constant value.

The correlation between loss parameter values requires further investigation. In addition to the correlation of loss values for individual events it would be useful to analyse the distribution of total loss.

3.7. Other Considerations for Selecting Loss Values

3.7.1. Variation of Loss with Event AEP

The majority of Australian studies of losses at catchment scale have concluded that both IL_s and CL do not vary systematically with the severity of the event; that is loss is independent of AEP.

This conclusion is not surprising because any potential variation of loss with rainfall severity is difficult to infer from the empirical analysis of data due to the lack of severe rainfall events in the recorded data. This is compounded where the storm severity is characterised as the AEP of the rainfall burst, whereas the loss values relate to the complete storm, and this discrepancy further hinders the identification of any trend with storm severity.

The Australian studies that present loss values that vary with AEP tend to be those where the loss values are derived by verification against flood frequency quantiles. In such studies it is difficult to ascertain whether any variation in loss is meaningful or simply a reflection of the uncertainty in the flood frequency quantiles and the link to the adopted design inputs.

The conclusions that there is no evidence to vary loss with magnitude is supported by the analysis of rainfall antecedent to extreme storms recorded over Southeast Australia which showed that the antecedent rainfall was not significantly greater than normal for the location and time of the year (Minty and Meighen, 1999). The implication of this is that the storm initial losses for large and extreme storms should be similar to those of smaller, more frequent storms".

The recommendation is therefore to keep the IL_s and CL values the same for AEPs unless there is specific evidence to suggest that there is a systematic variation of loss with AEP.

It should be noted that the stores in a storage capacity loss models such as SWMOD fill up during event and hence the proportion of the catchment contributing to the loss increases throughout the event. The net effect of this is an initial loss (which represents the initial filling of the smallest store) following by a variable proportional loss. This proportional loss decreases throughout the event and also decreases for larger rainfall events.

In considering how loss varies with event magnitude it is worth considering that extreme rainfalls may be associated with changed runoff behaviour from that observed for more frequent events with the stripping of vegetation cover.

Book 8 discusses how continuing loss and proportional loss would be expected to vary with event magnitude. The interpretation of proportional loss as the unsaturated proportion of the catchment implies that with larger storm events the unsaturated proportion of the catchment is reducing and the proportional loss reduces. It is however difficult to extrapolate the rate of this reduction to extreme events and hence how proportional loss varies with event magnitude. However, continuing loss is expected to approach a limiting value for saturated catchment conditions which makes it more suitable for application in extreme flood estimation.

For urban catchments, Phillips et al (2014) undertook an analysis of the correlation between the peak 1 hour intensity after the commencement of indirectly connected area runoff with the continuing loss that was estimated. That study found that for almost all catchments, there was no clear relationship between the two. The one exception to this was the Giralang (ACT) catchment, which showed a strong correlation with the 1 hour peak rainfall intensity after ICA runoff and the continuing loss estimate. The reasoning for this exception was not clear, although it was thought it could be due to the types of storms that fell on the Giralang catchment, which tended to be high intensity in the first hour after the Indirectly Connected Area runoff occurred.

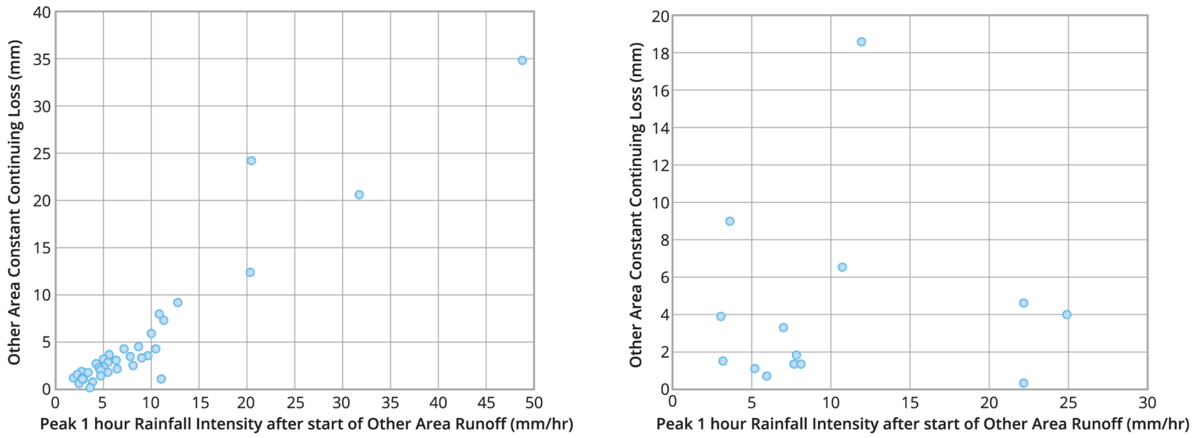


Figure 5.3.27. Correlation between 1 hr Peak Rainfall Intensity and Continuing Loss (left Giralang (ACT) and right Powells Creek (NSW)) (from Phillips et al (2014))

3.7.2. Reduction of Continuing Loss for Long Events

In the application of the IL-CL model it is typically assumed that the loss rates are a constant after the initial loss is satisfied. However for some very large rural catchments where the critical duration is multiple days, it has been noted that the CL reduces throughout the event. This is consistent with the expectation that the loss rates should decrease throughout the event as the catchment becomes wetter and infiltration reduces (as characterised in the Horton model – Book 5, Chapter 3, Section 2) and/or the size of source areas enlarge.

This reduction of continuing loss with duration has also been noted by studies which have analysed a large number of events for rural catchments (e.g. (Ilahee and Imteaz, 2009), and (Ishak and Rahman, 2006)).

Developing a relationship which explains the reduction in CL with duration or infiltrated volume is confounded by the uncertainty in the estimation of CL for specific events. There tends to be very large variation of CL for a particular catchment and a large proportion of this variability is simply likely to be due to uncertainties in the catchment average rainfall depths.

Although potentially important for real-time applications, the potential decrease of CL with duration is not significant for most design applications because the critical duration is typically shorter than a day. For very large rural catchments where the critical duration can be multiple days then it would be reasonable to reduce the CL. Ideally this relationship with duration should be based upon analysis of at site data but can also be informed by theoretical infiltrations relationships such as Manley-Phillips Loss Model (Manley, 1974). Such an approach in included in the URBS rainfall-runoff model (Carroll, 2012).

For storage capacity loss models such as SWMOD, the moisture content is continuously updated throughout the event which results in a variable proportional loss. This reduction in proportional loss throughout the event may have advantages for modelling of large rural catchments.

3.7.3. Influence of Timestep on the Estimation of Continuing Loss

The definition of Storm Initial Loss as the rainfall depth before surface runoff is generated suggests that its estimation should not be sensitive to the timestep used in the analysis. Similarly, the Proportional Loss and storage capacity loss models such as SWMOD should also not be affected by the timestep.

However, the definition of Continuing Loss as the threshold rate above which rainfall excess is generated, means that it is dependent upon the timestep. This is because as the timestep reduces there is an increased likelihood that there will be some timesteps in which the rainfall depth is less than the Continuing Loss rate threshold. Thus to achieve the same volume of rainfall excess the Continuing Loss will typically need to be increased for shorter timesteps.

This is demonstrated in Figure 5.3.28 for an event at Currambene Creek at Falls Creek in NSW. If the modelling timestep is reduced from 1 hour to 5 minutes the continuing loss rate needs to increase from 4.5 to 7.2 mm/h to maintain the same volume of rainfall excess. This is because for 5 minutes there is a higher proportion of timesteps for which the rainfall depth is less than the threshold value.

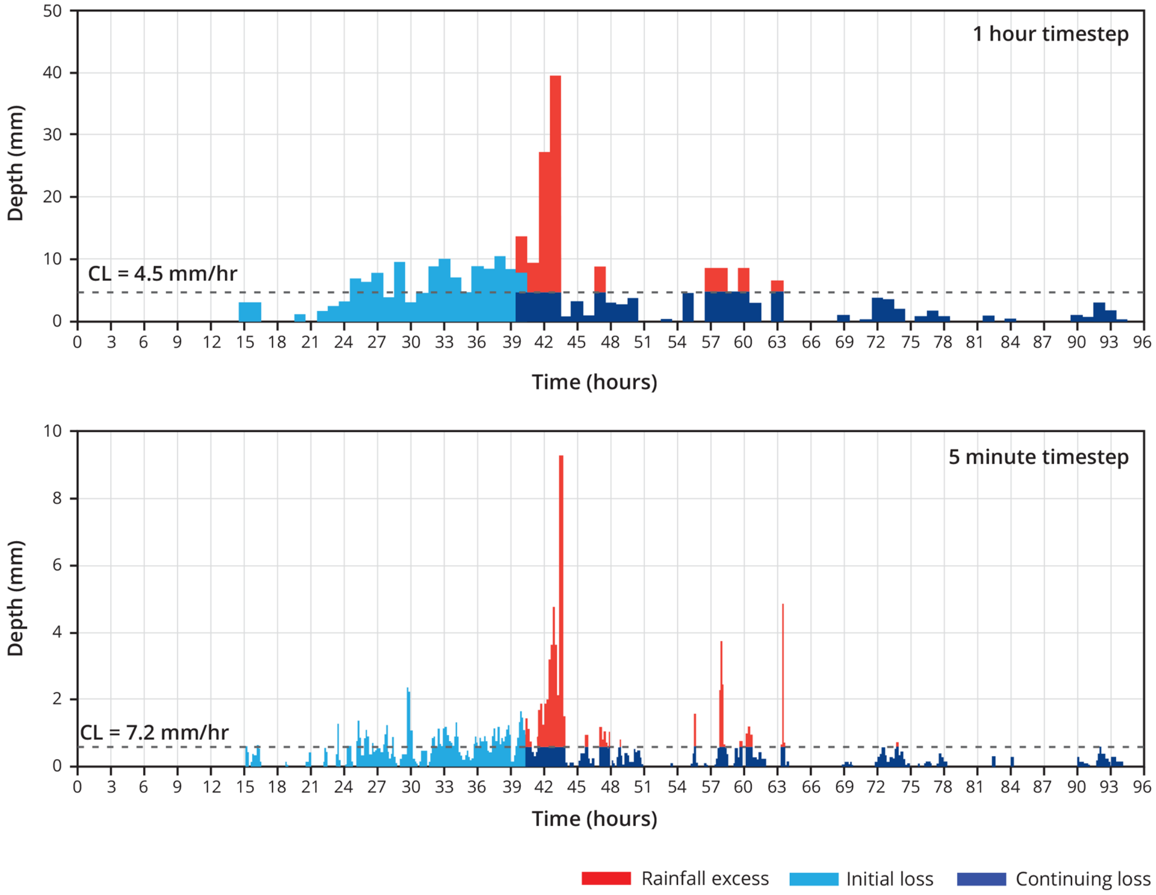


Figure 5.3.28. Example of Continuing Loss Varying with Modelling Timestep (February 1977 event at Currambene Creek at Falls Creek in NSW)

This adjustment of CL is important if the timestep used in the derivation of the loss values is different from that used in design. It should therefore be noted that the timestep used in the derivation of the regional loss information presented in Book 5, Chapter 3, Section 5 was 1 hour for the rural catchments and up to 5 minutes for the urban catchments. If a different timestep is to be adopted in design then the continuing loss should be adjusted accordingly.

3.7.3.1. Rural Catchments

The relationship between CL and the timestep used in the analysis for rural catchments is shown in Figure 5.3.29. The factor relates the CL derived for a timestep less than 1 hour to that for a timestep of 1 hour. The information was based upon the analysis of a number of storms at different timesteps for 18 catchments across Australia. Further details are contained in Lang et al. (2015).

The factor is a function of the rainfall depth with the adjustment factor increasing for smaller rainfall depth. For larger depths, it is more likely that the full CL value can be satisfied in each timestep which reduces the adjustment factor.

This line of best fit can be used to relate continuing losses modelled at sub-hourly time steps to hourly values and vice versa. For example, if the average storm depth is approximately 200 mm and the timestep is reduced from 1 hour to 15 minutes then the continuing loss needs to be increased by 30%.

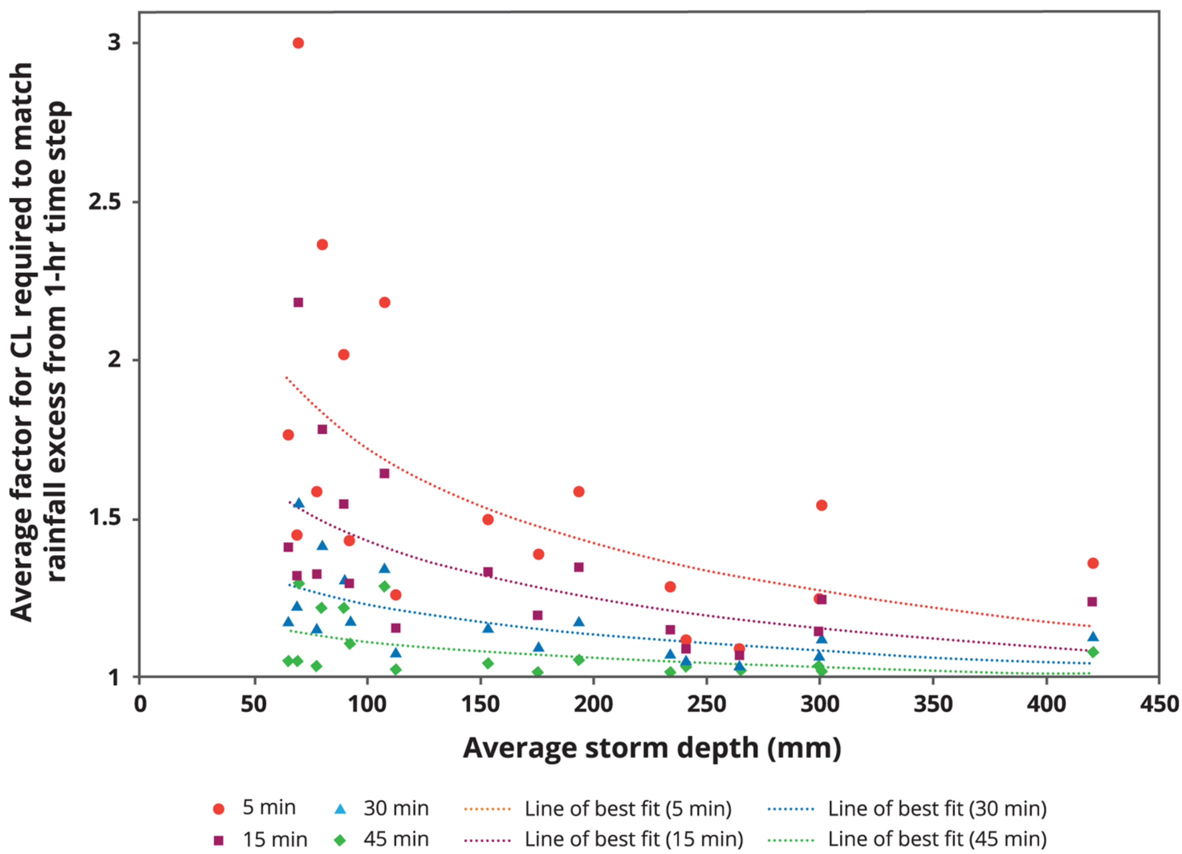


Figure 5.3.29. Variation of Continuing Loss with Modelling Timestep for Rural Catchments

3.7.3.2. Urban Catchments

Urban catchments differ from rural catchments in that generally shorter timestep rainfall data is used. This is in order to represent the fast response that typically occurs in urban catchments. The shorter timestep is required to represent the peak flow appropriately (although the runoff volume may be appropriate).

The urban catchment data derived in Phillips et al (2014) was based on shorter timestep pluviometer data than the rural catchments.

This data was re-analysed comparing with 6 minute time intervals, which is similar to a large majority of the pluviometer data that is available in Australia, and typical of the duration that would be analysed in urban catchments. This re-analysis suggests that there are minimal changes to the CL loss estimates, and well within the error margins of the estimates.

There are insufficient storms for the loss estimates to develop a similar graph to that for the rural catchment (i.e. Figure 5.3.29). Instead, the different catchments were plotted against timestep, and are shown in Figure 5.3.30. This graph shows the percentage change relative to the median CL estimate for 6 minute timestep. A recommended curve is also provided, based on the average of the values provided. The equation of this can be broadly approximated by:

$$\frac{\mathrm{CL}_x}{\mathrm{CL}_{6\mathrm{min}}} = 0.2 \ln t + 1.35 \tag{5.3.15}$$

Where t=time in minutes

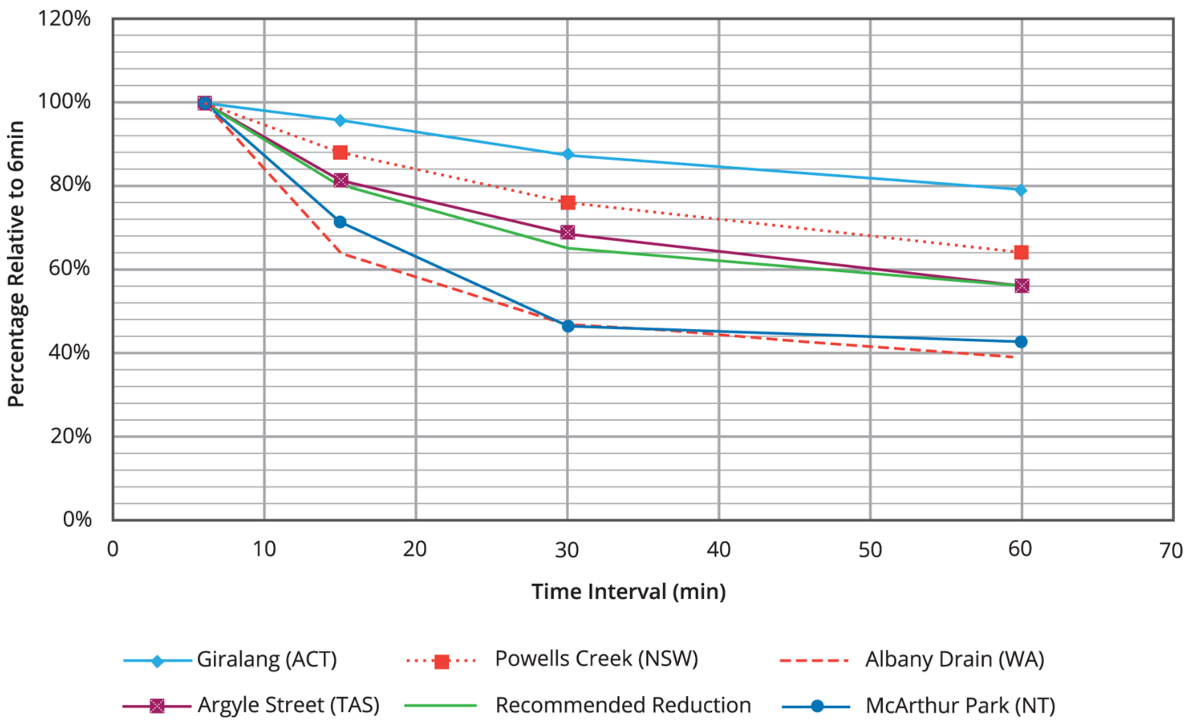


Figure 5.3.30. Relationships of Urban Continuing Loss with Timestep

3.7.4. Antecedent Rainfall and Soil Moisture

3.7.4.1. Antecedent Precipitation

It would be expected that the initial loss, and potentially the CL, is negatively correlated with the antecedent rainfall, as it is a surrogate for the soil moisture.

To avoid the arbitrary selection of the period over which to define the antecedent rainfall, the Antecedent Precipitation Index (API) can be used as a measure of the initial wetness of a catchment. API is calculated by discounting the time series of daily rainfall prior to the event using an empirical decay factor and the basic equation is (Cordery, 1970):

$$\mathrm{API}_d = P_d + \mathrm{k}\, P_{d-1} + k^2 P_{d-2} + \ldots \qquad (5.3.16)$$

Where *k* is an empirical decay factor less than unity and *Pd* is rainfall for day *d*. The value of *k* varies typically in the range of 0.85 to 0.98. Linsley et al. (1982) and Cordery (1970) found that the average value for Australian catchments was 0.92. The value of *k* is considered to vary seasonally and has been linked to the variation in potential evapotranspiration (Mein et al., 1995).

Cordery (1970) then related the $\mathrm{IL_s}$ to the API using a relationship of the form:

$$\mathrm{IL}_s = \mathrm{IL}_{\max}(N)^{\mathrm{API}} \qquad (5.3.17)$$

3.7.4.2. Soil Moisture

There are a number of products that have recently become available that estimate soil moisture over the whole of Australia either from remote sensing, conceptual water balance modelling or a combination of both. One estimate of soil moisture that shows promise for explaining the variability of loss is AWRA-L (refer Book 5, Chapter 3, Section 5 and Frost et al. (2015)).

There are only a limited number of studies that have investigated the relationship between loss values and these estimates of soil moisture but it is expected that soil moisture will be provide a more useful estimate of loss than indices based only upon rainfall.

3.7.4.3. Rural Catchments

Preliminary results showed that soil moisture conditions in the combined layer over the upper 1m explains the most variation in loss values in most catchments. Accordingly, soil moisture conditions over this depth were adopted for all subsequent analyses.

The results for IL are shown in Figure 5.3.31 and demonstrate that soil moisture has a higher correlation with IL than API for the majority of the catchments. There are still, however, some catchments for which the initial loss is relatively independent of both the API and AWRA-L soil moisture.

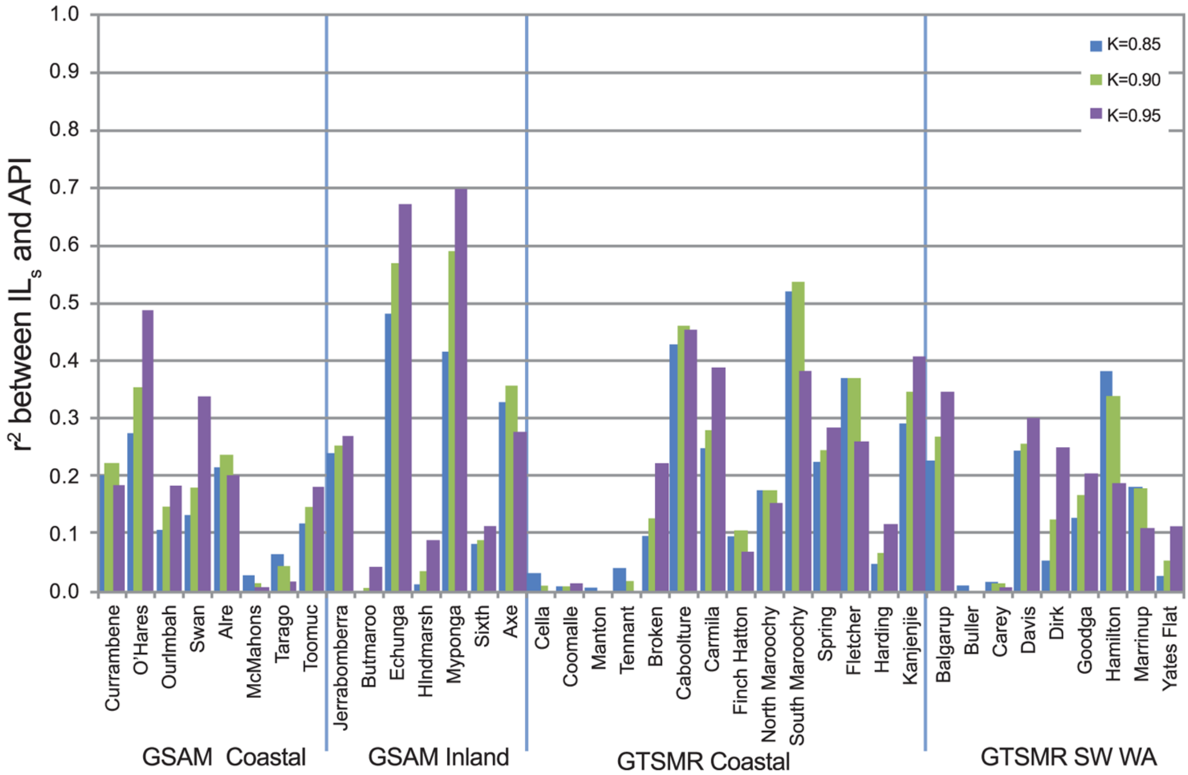


Figure 5.3.31. Proportion of Variance Explained (r^2) between Storm Initial Loss and API (Hill et al., 2014a)

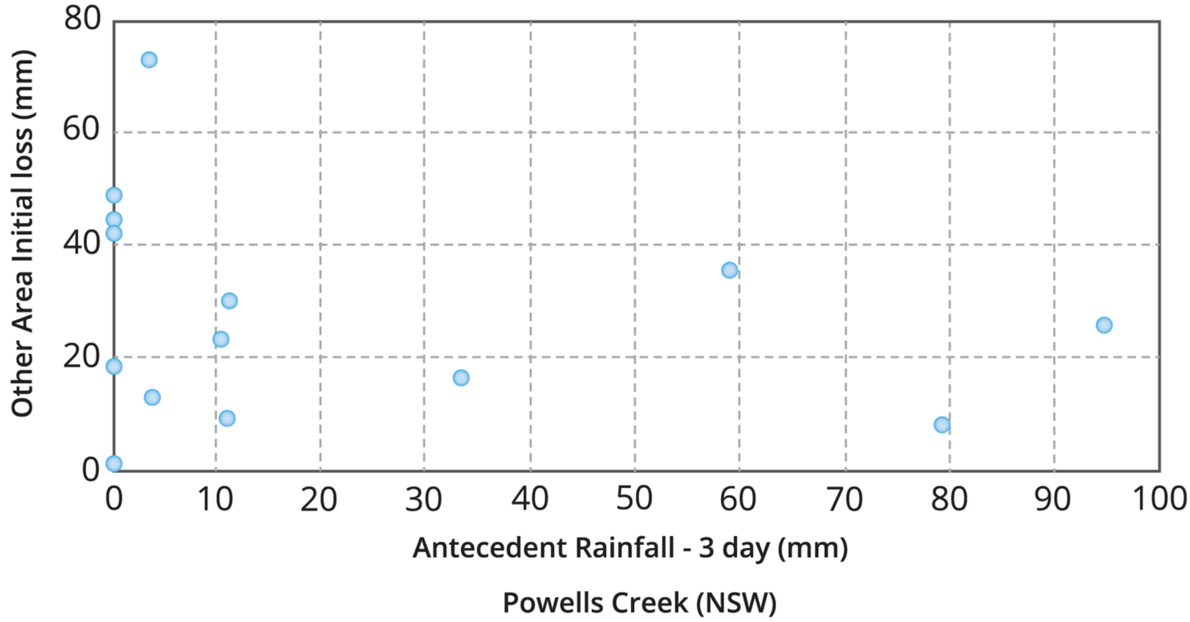
Studies by the Bureau of Meteorology (T. Pagano pers comm.) and Seqwater (D. Pokarier per. comm.) have also found that the storm initial loss is also more highly correlated with soil moisture estimated by AWRA-L than API.

Where such relationships can be established, they can help inform the absolute and seasonal distribution of IL_s.

3.7.4.4. Urban Catchments

In residential areas garden watering may influence antecedent wetness particularly if several dry days occur (Woolmington and Burgess, 1983; Boyd et al., 1994) and therefore measures of the preceding days rainfall (such as API) may not reflect the true antecedent conditions of the catchment.

Phillips et al (2014) undertook an analysis of the antecedent rainfall in the days leading up to the storms used to estimate the initial loss. They compared the rainfall in 1, 3 and 7 days prior to the event with the initial loss and continuing loss estimates, and found no clear correlation between the two. Samples of this analysis are provided in Figure 5.3.32.



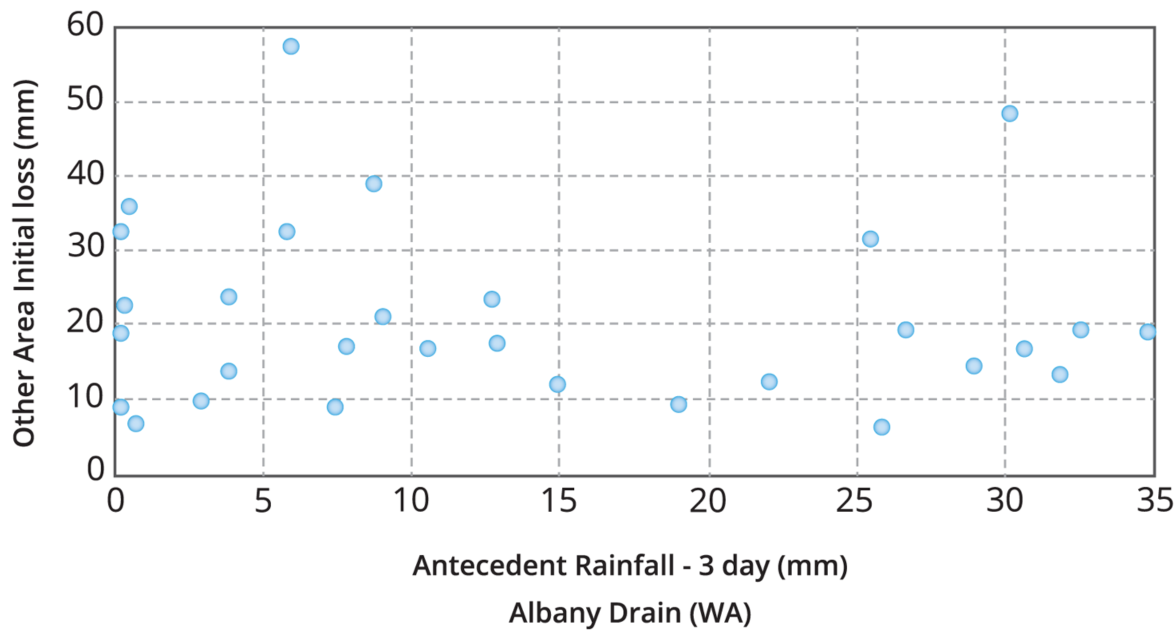


Figure 5.3.32. Plot of Antecedent Rainfall Versus Initial Loss for Indirectly Connected Area (Phillips et al, 2014)

3.7.5. Seasonality

The discussion of losses in the preceding sections has concentrated on the median annual values. However, due to the seasonal variation in rainfall, evapotranspiration (and to a lesser extent vegetation) many regions in Australia are characterised by distinct seasonality in hydrology. The estimation of seasonal design inputs, including loss values, is required in cases where:

- there is a strong seasonal variation in the flood producing mechanisms which need to be accounted for in order to estimate the annual risk; or

- the risk is required to be assessed for a particular period within the year such as the flood risk during construction or upgrade of major infrastructure.

The loss parameters (both initial and continuing loss) can be influenced by antecedent moisture and therefore may display significant seasonal variation. This is likely to primarily reflect changes in antecedent moisture but vegetation change may also contribute in some locations.

The different seasonality across Australia is demonstrated in Figure 5.3.33. This shows the seasonal distribution of the 803 events analysed by Hill et al. (2014a) which were selected on the basis of rainfall. It is clear that in south-eastern Australia the events are reasonably distributed throughout the year, whereas the majority of events in Northern Australia occur in summer and south-west WA is dominated by the winter months.

It is important to consider this seasonal variation when selecting losses for design flood estimation. In some cases the loss values may need to be adjusted to account for a bias in the sample and for some locations it may be necessary to explicitly incorporate the seasonality in the adopted losses and design flood estimation framework.

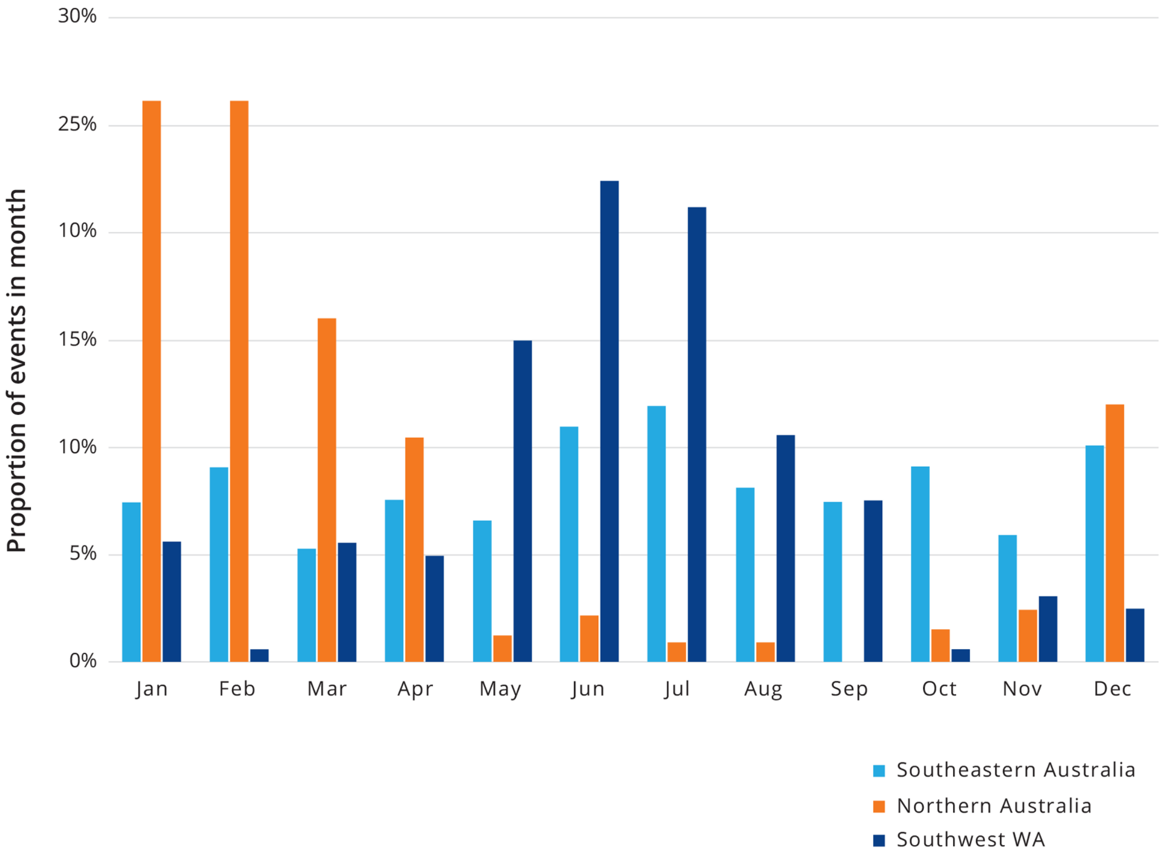


Figure 5.3.33. Seasonal Distribution of Events Analysed by Hill et al. (2014a)

The seasonality of the loss values is demonstrated in Figure 5.3.34. In south-eastern Australia the median losses are lowest in July and steadily increase until summer. For northern Australia the highest losses are at the beginning of the wet season and the losses are slightly lower for late summer and autumn. For south-west WA the highest median losses occur in Summer.

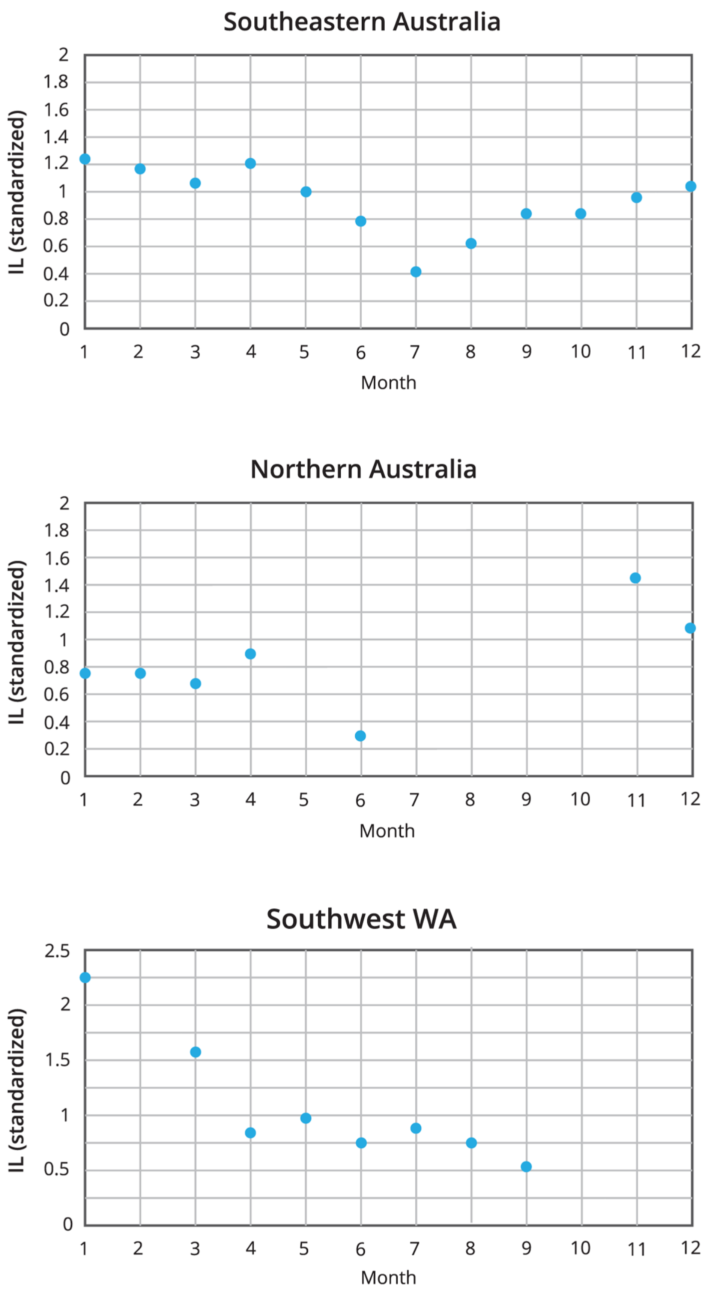


Figure 5.3.34. Seasonality of Standardised Storm Initial Loss Values for Different Regions in Australia

In south-west WA, two different rainfall mechanisms have been identified which result in distinctly seasonal nature of rainfall and losses. The majority of events have been recorded in the winter seasons which are typically associated with lower losses, however the rarer events are more likely to occur in summer when there is typically higher losses. It is therefore important to recognise this in the selection of losses.

This is highlighted by Pearce (2011) which developed seasonal runoff coefficients for south-west WA by generalising the results of the application of SWMOD to a number of catchments (refer to Figure 5.3.35). The results show the important influence of the season and the proportion of the catchment cleared of vegetation (refer to Book 5, Chapter 3, Section 7).

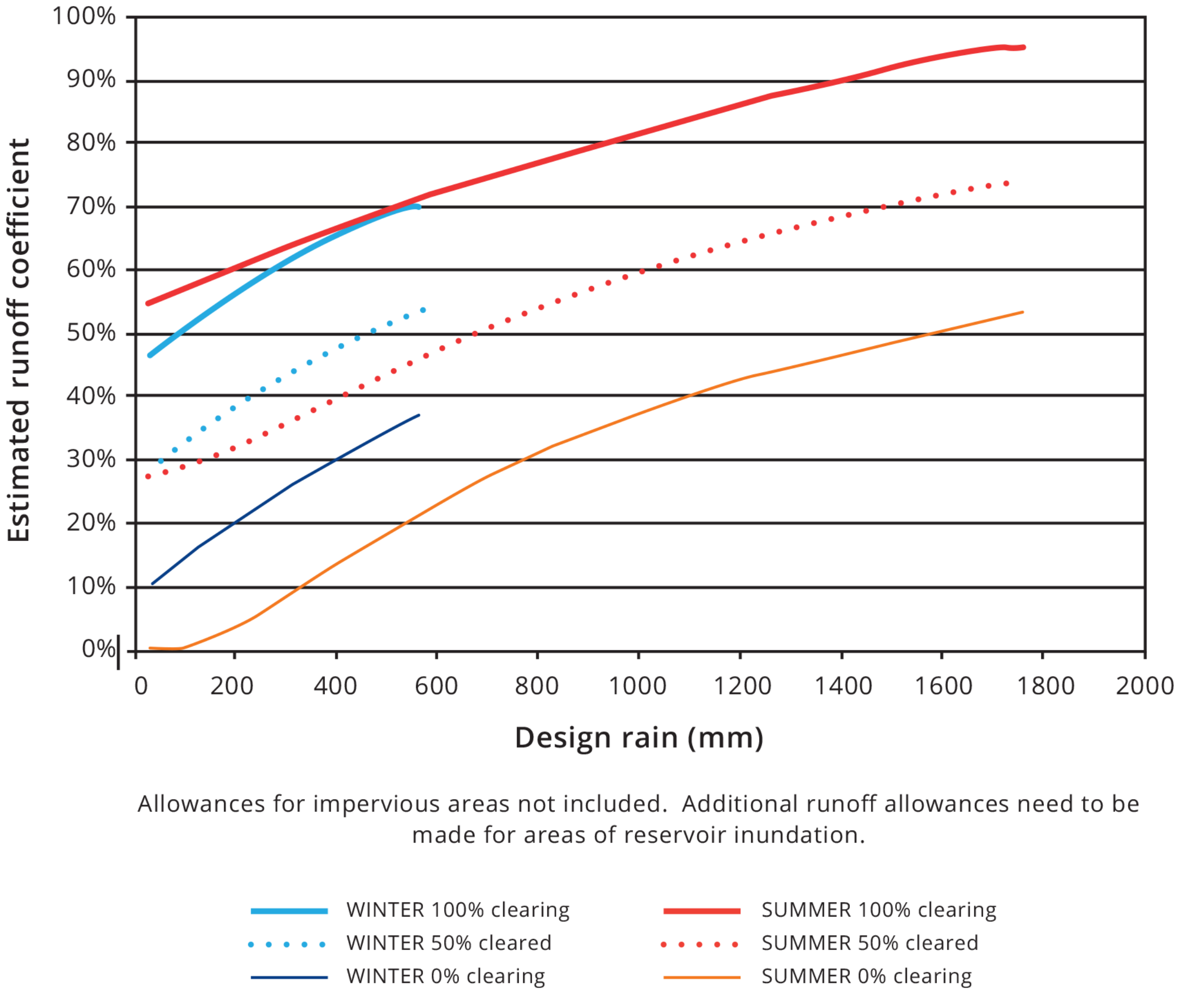


Figure 5.3.35. Regional Runoff Coefficient Curves for South-West Western Australia (Pearce, 2011)

Another approach to deriving seasonal loss values is to verify the rainfall-based estimates from a rainfall runoff model to seasonal flood frequency quantiles. That is, adopt a similar approach as outlined in Book 5, Chapter 3, Section 3 but on a seasonal, rather than annual, basis.

3.7.6. Influence of Vegetation

The presence of vegetation is expected to increase interception and hence loss. However, a number of studies have failed to find a link between the proportion of the catchment vegetated and the loss values (e.g. Cordery and Pilgrim (1983)). This is likely to be due to the uncertainties in estimating both loss values and representatives measures of interception from vegetation at the catchment scale.

One exception to this is south-west WA where the losses have been directly linked to the proportion of the catchment cleared of native vegetation (Pearce, 2011).

3.7.7. Interaction with Routing Parameters

Although it may not be readily apparent, there is an interdependency between the adopted conceptual loss model and the inferences regarding the routing characteristics of the catchment. This is because the different conceptual loss model result in different temporal distributions of loss and hence rainfall excess.

The rainfall excess obtained from loss models where the excess is a proportion of the total rainfall in the timestep (such as IL/PL or SWMOD) will tend to result in a more temporally uniform (less peaky) rainfall excess when compared to IL/CL loss model in which a constant rate of loss is applied. This is because a greater volume of loss is extracted in the timesteps of greatest rainfall.

This means that more attenuation is required for the rainfall excess resulting from the IL/CL loss models than those for the IL/PL or SWMOD.

For example, Australian Rainfall and Runoff Revision Project 6 (Hill et al., 2014a) considered the routing parameters for IL/CL and SWMOD for 38 catchments from across Australia and demonstrated that the adopted loss model affected the selection of the C0.8 value (non-dimensional routing parameter in RORB). With the C0.8 value for SWMOD being typically 75% of that for the IL/CL model (refer Figure 5.3.36).

Therefore, the routing parameters derived using one conceptual loss model are not necessarily applicable for an alternate loss model. This is important if different loss models are to be applied to flood models than those used in calibration.

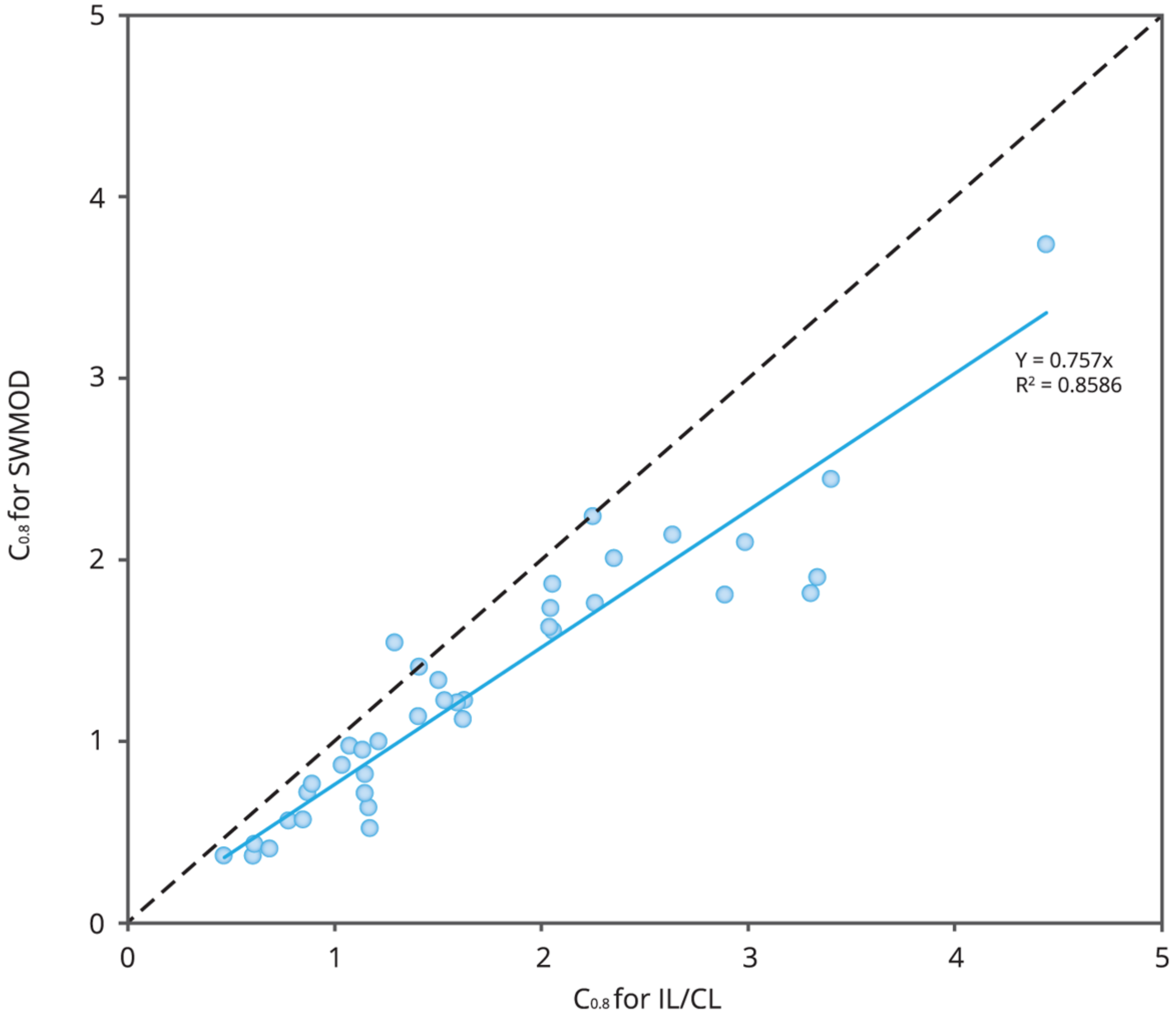


Figure 5.3.36. Comparison of Adopted Routing Parameters for IL/CL and SWMOD

3.7.8. Influence of Snowpack

There is only a relatively small proportion of Australia that experiences substantial snow cover, and even for these catchments this only occurs for a portion of the year. These catchments in parts of south-eastern Australia are typically above an elevation of approximately 1,500 mAHD. The presence of a snowpack influences the losses, runoff generation and routing characteristics of a catchment. With respect to losses, there is typically low losses for rain on the snowpack and therefore it can be assumed that there are low losses for the proportion of the catchment covered by snowpack. For example USACE (1998) adopts a continuing loss of 1 mm/h for rain on snowpack.

3.7.9. Link to Climate Drivers and Change

There has been little research on the potential role that large scale climate drivers such as El Niño/Southern Oscillation (ENSO), Interdecadal Pacific Oscillation (IPO), Indian Ocean Dipole (IOD) and Southern Annular Mode (SAM) have on influencing antecedent rainfalls and hence loss rates.

A number of studies have shown significant dependence of annual maxima floods in Eastern Australia on the Interdecadal Pacific Oscillation (IPO). However, the annual maxima precipitation does not exhibit a similar level of dependency on the IPO. Pui et al (2009) hypothesize that the difference in flood characteristics as a function of the IPO is a result of catchment antecedent conditions prior to the rainfall event. From the analysis of 88 daily rainfall stations in Eastern Australia they found that the antecedent conditions prior to storm events varied significantly across the two IPO phases.

The influence of these key climate drives on loss rates warrants further research.

3.8. Appendix

Table 5.3.14. Median Loss Values for Rural Catchments

Region	Method	Gauge	River	Name	Area	N	Median IL_s (mm)	Median CL (mm/hr)	Study
East Coast	QLD	141001	South Maroochy	Kiamba	33	22	38	2.7	Hill et al. (2014a)
East Coast	QLD	141009	North Maroochy	Eumundi	41	23	20	2.2	Hill et al. (2014a)
East Coast	NSW	201001	Oxley River	Eungella	218	53	50	2.6	Loveridge (Unpublished)
East Coast	NSW	203010	Leycester River	Rock Valley	179	48	65	0.3	Loveridge (Unpublished)
East Coast	NSW	204017	Bielsdown Creek	Dorrigo No. 2 & No.3	82	57	50	1.4	Loveridge (Unpublished)
East Coast	NSW	204025	Orara River	Karangi	135	37	71	4	Loveridge (Unpublished)
East Coast	NSW	208007	Nowendoc River	Nowendoc	218	37	50	2.3	Loveridge (Unpublished)
East Coast	NSW	210068	Pokolbin Creek	Pokolbin Site 3	25	36	40	2.0	Loveridge (Unpublished)
East Coast	NSW	211013	Ourimbah Creek	Upstream Weir	83	25	40	3.7	Hill et al. (2014a)
East Coast	NSW	213200	O'Hares Creek	Wedderburn	73	22	60	1.6	Hill et al. (2014a)
East Coast	QLD	136108A	Monal Creek	Upper Monal	92	12	13		Ilahee (2005)
East Coast	QLD	141009A	N. Maroochy River	Eumundi	38	22	42		Ilahee (2005)
East Coast	QLD	142001A	Caboolture	Upper Caboolture	94	20	50	1.4	Hill et al. (2014a)
East Coast	QLD	143110A	Bremer River	Adams Bridge	125	37	39		Ilahee (2005)
East Coast	QLD	145003B	Logan River	Forest Home	175	42	31		Ilahee (2005)

Region	Method	Gauge	River	Name	Area	N	Median IL_s (mm)	Median CL (mm/hr)	Study
East Coast	QLD	145010A	Running Creek	5.8 km Deickmans Bridge	128	20	32		Ilahee (2005)
East Coast	QLD	145011A	Teviot Brook	Croftby	83	37	30		Ilahee (2005)
East Coast	QLD	145101D	Albert River	Lumeah Number 2	169	35	44		Ilahee (2005)
Monsoonal North	WA	809312	Fletcher	Frog Hollow	30.6	19	30	10.4	Hill et al. (2014a)
Monsoonal North	QLD	118003A	Bohle River	Hervey Range Road	143	24	29		Ilahee (2005)
Monsoonal North	QLD	120014A	Broughton River	Oak Meadows	182	19	18		Ilahee (2005)
Monsoonal North	QLD	120216A	Broken	Old Racecourse	78	34	68	6.2	Hill et al. (2014a)
Monsoonal North	QLD	916003A	Moonlight Creek	Alehvale	127	7	29		Ilahee (2005)
Monsoonal North	QLD	917114A	Routh Creek	Beef Road	81	7	30		Ilahee (2005)
Monsoonal North	NT	G8150151	Celia	U/S Darwin R Dam	52	15	25	5.4	Hill et al. (2014a)
Monsoonal North	NT	G8170066	Coomalie	Stuart HWY	82	30	50	8.1	Hill et al. (2014a)
Monsoonal North	NT	G8170075	Manton	upstream Manton Dam	29	32	42	1.6	Hill et al. (2014a)
Murray Darling	VIC	403226	Boggy Creek	Angleside	108	33	15	3.7	Hill et al. (1996)
Murray Darling	VIC	404208	Moonee Creek	Lima	91	28	19	6.5	Hill et al. (1996)
Murray Darling	VIC	405229	Wanalta Creek	Wanalta	108	24	31	1.4	Hill et al. (1996)
Murray Darling	VIC	405257	Snobs Creek	Snobs Creek Hatchery	51	12	8	11	Hill et al. (1996)
Murray Darling	VIC	405261	Spring Creek	Fawcett	60	17	27	4.2	Hill et al. (1996)
Murray Darling	VIC	406208	Campaspe River	Ashborne	33	7	48	6.2	Hill et al. (1996)
Murray Darling	VIC	406216	Axe Creek	Sedgewick	34	12	28	6	Hill et al. (2014a)

Region	Method	Gauge	River	Name	Area	N	Median IL_s (mm)	Median CL (mm/hr)	Study
Murray Darling	VIC	407258	Myers Creek	Myers Flat	55	9	36	2.7	Hill et al. (1996)
Murray Darling	ACT	410736	Orroral River	Crossing	90	36	18	7.1	Hill et al. (1996)
Murray Darling	ACT	410739	Tidbinbilla Creek	Mountain Creek	25	31	10	8.8	Hill et al. (1996)
Murray Darling	ACT	410743	Jerrabomberra Creek	Four Mile Creek	52	20	22	2.1	Hill et al. (2014a)
Murray Darling	ACT	410751	Ginninderra Creek	u/s Barton Highway	48	20	38	6.5	Hill et al. (1996)
Murray Darling	NSW	411003	Butmaroo Creek	Butmaroo	65	21	40	2.6	Hill et al. (2014a)
Murray Darling	QLD	422321	Spring	Killarney	32	27	30	5.1	Hill et al. (2014a)
Pilbara	WA	709007	Harding	Marmurrina Pool U-South	49.4	17	60	8.3	Hill et al. (2014a)
Rangelands	NT	G0290240	Tennant	Old Telegraph Stn	72.3	24	0	5.2	Hill et al. (2014a)
South-west WA	WA	602199	Goodga River	Black Cat	49.2	27	30	4.8	Hill et al. (2014a)
South-west WA	WA	603190	Yates Flat Creek	Woonanup	53	17	27	0.8	Hill et al. (2014a)
South-west WA	WA	608002	Carey Brook	Staircase Rd	30.3	19	20	3.8	Hill et al. (2014a)
South-west WA	WA	609005	Balgarup River	Mandelup Pool	82.4	13	25	2.5	Hill et al. (2014a)
South-west WA	WA	612004	Hamilton River	Worsley	32.3	13	47	3.3	Hill et al. (2014a)
South-west WA	WA	613003	Harvey	Paganini Farm	148		16		Waugh (1991)
South-west WA	WA	613013	Bancell Creek	Wagerup	13		11		Waugh (1991)
South-west WA	WA	614003	Marrinup Brook	Brookdale Siding	45.6	19	16	7.3	Hill et al. (2014a)
South-west WA	WA	614003	Marrinup Brook	Brookdale Siding	46		12		Waugh (1991)
South-west WA	WA	614005	Dirk Brook	Kentish Farm	36	20	14	6.7	Hill et al. (2014a)

Region	Method	Gauge	River	Name	Area	N	Median IL_s (mm)	Median CL (mm/hr)	Study
South-west WA	WA	614005	Dirk Brook	Kentish Farm	36		15		Waugh (1991)
South-west WA	WA	614016	North Dandalup River	Nth Dandalup Dam	153		20		Waugh (1991)
South-west WA	WA	614047	Davis Brook	Murray Valley Plntn	65.7	18	25	8.1	Hill et al. (2014a)
South-west WA	WA	701006	Buller River	Buller	33.9	14	32	3.8	Hill et al. (2014a)
South-Central SA	SA	A5040523	Sixth Creek	Castambul	44	24	15	3.3	Hill et al. (2014a)
South-Central SA	SA	AW501500	Hindmarsh River	Hindmarsh Vy Res Offtake	56	33	15	3.2	Hill et al. (2014a)
South-Central SA	SA	AW502502	Myponga River	upstream Dam and Rd Br	77	15	23	2.6	Hill et al. (2014a)
South-Central SA	SA	AW503506	Echunga Creek	upstream Mt Bold Res.	34	13	25	2.2	Hill et al. (2014a)
South-east Coast	TAS	2219	Swan River	upstream Hardings Falls	38	19	40	0.5	Hill et al. (2014a)
South-east Coast	NSW	214003	Macquarie Rivulet	Albion Park	35	26	69	2.9	Loveridge (Unpublished)
South-east Coast	NSW	214003	Macquarie Rivulet	Albion Park	35	26	69		Loveridge (Unpublished)
South-east Coast	NSW	216004	Currambene Creek	Falls Creek	95	17	35	3.9	Hill et al. (2014a)
South-east Coast	NSW	216004	Currambene Creek	Falls Creek	95	37	37		Loveridge (Unpublished)
South-east Coast	VIC	224209	Cobbannah Creek	Bairnsdale	106	13	52	1.7	Hill et al. (1996)
South-east Coast	VIC	226222	La Trobe River	Near Noojee	62	7	19	3.4	Hill et al. (1996)
South-east Coast	VIC	227226	Tarwin River East Branch	Dumbalk Nth	127	5	41	1.7	Hill et al. (1996)
South-east Coast	VIC	227228	Tarwin River East Branch	Mirboo	43	5	21	3.6	Hill et al. (1996)
South-east Coast	VIC	228217	Toomuc Creek	Pakenham	42	25	24	2.5	Hill et al. (2014a)

Region	Method	Gauge	River	Name	Area	N	Median IL_s (mm)	Median CL (mm/hr)	Study
South-east Coast	VIC	229106	McMahons Creek	Upstream Weir	40	21	20	3.7	Hill et al. (2014a)
South-east Coast	VIC	231213	Lerderderg River	Sardine Creek	153	9	25	1.1	Hill et al. (1996)
South-east Coast	VIC	231219	Goodman Creek	above Lerderderg Tunnel	32	19	35	2.4	Hill et al. (1996)
South-east Coast	VIC	233223	Warrambine Creek	Warrabine	57	17	26	1.5	Hill et al. (1996)
South-east Coast	VIC	235212	Chapple Creek	Chapple Value	28	23	28	2.6	Hill et al. (1996)
South-east Coast	VIC	235219	Aire River	Wyelangta	90	17	19	3	Hill et al. (1996)
South-east Coast	VIC	235219	Aire River	Wyelangta	90	30	17	3	Hill et al. (2014a)
South-east Coast	VIC	235229	Ford River	Glenaire	56	23	21	2.6	Hill et al. (1996)
South-east Coast	VIC	238231	Glenelg River	Big Cord	57	17	24	5.1	Hill et al. (1996)
South-east Coast	VIC	228206B	Tarago River	Neerim	78	22	24	3.9	Hill et al. (2014a)
Wet Tropics	QLD	125006	Finch Hatton	Dam Site	36	30	23	5.2	Hill et al. (2014a)
Wet Tropics	QLD	112003A	N. Johnston River	Glen Allyn	173	15	34		Ilahee (2005)
Wet Tropics	QLD	114001A	Murray River	Upper Murray	155	23	66		Ilahee (2005)
Wet Tropics	QLD	116008B	Gowrie Creek	Abergowrie	124	61	22		Ilahee (2005)
Wet Tropics	QLD	116015A	Blunder Creek	Wooroora	127	48	71		Ilahee (2005)
Wet Tropics	QLD	116017A	Stone River	Running Creek	157	55	33		Ilahee (2005)
Wet Tropics	QLD	124002A	St. Helens Creek	Calen	129	11	54		Ilahee (2005)
Wet Tropics	QLD	126003A	Carmila	Carmila	82	19	70	3.1	Hill et al. (2014a)
Wet Tropics	QLD	922101B	Coen River	Racecourse	166	59	25		Ilahee (2005)
Wet Tropics	QLD	926003A	Bertie Creek	Swordgrass Swamp	130	8	1		Ilahee (2005)

Table 5.3.15. Summary of EIA Results from (Boyd et al., 1993)

Catchment	Country	Area	Impervious Fraction	EIA/TA	EIA/TIA	Urban Area (%)
Maroubra	NSW, Australia	57.3	52%	16%	30%	100%
Strathfield	NSW, Australia	234	50%	29%	58%	100%
Jamison Park	NSW, Australia	22.1	36%	21%	58%	100%
Fishers Ghost	NSW, Australia	226	36%	25%	70%	100%
Giralang	ACT, Australia	94	25%	35%	140%	85%
Long Gully	ACT, Australia	490	5%	6%	118%	16%
Mawson	ACT, Australia	445	26%	21%	80%	86%
Curtin	ACT, Australia	2690	17%	17%	102%	57%
Vine Street	Vic, Australia	70	37%	31%	83%	100%
Pompano Beach	USA	15.4	44%	7%	16%	100%
Sample Road	USA	23.5	36%	27%	74%	100%
Fort Lauderdale	USA	7.7	98%	96%	98%	100%
Kings Creek	USA	5.26	71%	75%	106%	100%
Gray Haven	USA	9.4	52%	48%	93%	100%
Malvern	Canada	23.3	34%	34%	99%	100%
East York	Canada	155	49%	48%	98%	100%
Clifton Grove	UK	10.6	40%	24%	60%	100%
St Marks Road	UK	10.3	56%	30%	53%	100%
Porsoberg	Sweden	13	40%	21%	52%	100%
Munkerisparken	Denmark	6.44	46%	35%	76%	100%
Livry Gargan	France	235.5	33%	17%	53%	78%
Miskole	Hungary	25.4	15%	13%	89%	52%
Luzzi	Italy	1.73	85%	58%	68%	100%

Catchment	Country	Area	Impervious Fraction	EIA/TA	EIA/TIA	Urban Area (%)
Vika	Norway	10.1	97%	65%	67%	100%
Miljakovic	Yukoslavia	25.5	37%	20%	54%	100%
Kotta	Japan	1281	23%	32%	137%	84%

Table 5.3.16. EIA Initial Loss Estimates from Various Studies

Catchment	Initial Loss Estimate (mm)	Reference
Dee Why Creek (NSW)	1	Chiew and McMahon (1999)
Fishers Ghost Ck (NSW)	0.9	Bufill and Boyd (1992)
	0	Boyd et al. (1993)
Ithaca Creek (QLD)	2.8	Phillips et al (2014)
Jamison Park (NSW)	0.8	Bufill and Boyd (1992)
	0	Boyd et al. (1993)
Maroubra (NSW)	0.3	Bufill and Boyd (1992)
	0	Boyd et al. (1993)
Powells Creek (NSW)	2.6 to 2.9	Phillips et al (2014)
	0.7	Bufill and Boyd (1992)
	0	Boyd et al. (1993)
Upper Salt Pan Creek (NSW)	0.8	Ball and Zaman (1994)
McArthur Park (NT)	5.0[a]	Phillips et al (2014)
Curtin (ACT)	1	Bufill and Boyd (1992)
	0	Boyd et al. (1993)
Giralang (ACT)	1.3 to 1.6	Phillips et al (2014)
	0.9	Bufill and Boyd (1992)
	3.26	Boyd et al. (1993)
Long Gully Creek (ACT)	1	Bufill and Boyd (1992)
	0	Boyd et al. (1993)
Mawson (ACT)	0	Boyd et al. (1993)
Paddocks Catchment (SA)	1.3	Kemp and Lipp (1999)
Parra Hills Drain (SA)	1	Phillips et al (2014)
Argyle Street (TAS)	0.9	Phillips et al (2014)
Blackburn Lake (VIC)	1	Chiew and McMahon (1999)
Elster Creek (VIC)	1	Bufill and Boyd (1992)
Kinkora Road (VIC)	2.5	Phillips et al (2014)

Catchment	Initial Loss Estimate (mm)	Reference
Vine Street (VIC)	1	Bufill and Boyd (1992)
	0	Boyd et al. (1993)
Albany Drain (WA)	1.4	Phillips et al (2014)

[a]Phillips et al (2014) notes that this catchment had a large detention basin that may have influenced results

3.9. References

Alley, W.M. and Veenhuis, J. E. (1983), 'Effective impervious area in urban runoff modeling.'J. Hydraul. Eng., 109(2), 313-319.

Babister, M., Trim, A., Testoni, I. and Retallick, M. 2016. The Australian Rainfall and Runoff Datahub, 37th Hydrology and Water Resources Symposium Queenstown NZ

Ball, J.E. and Powell, M. (1998), Inference of catchment modelling system control parameters, Proc. UDM '98: Developments in Urban Drainage Modelling, London, UK, 1, 313-320.

Ball, J.E. and Zaman, S. (1994) 'Simulation of small events in an urban catchment', National Conference Publication - Institution of Engineers, Australia, pp: 353-358.

Beard, L.R. and Shin Chang. (1979), Urbanization impact on streamflow, Journal of the Hydraulics Division ASCE, 105: 647-59.

Black, D.C. and Aitken, A.P. (1977), Simulation of the Urban Runoff Process, Australian Government Publishing Service, 75/82.

Board, R.J., Barlow, F.T.H. and Lawrence, J.R. (1989), Flood and Yield Estimation in the Arid Zone of the Northern Territory. Hydrology and Water Resources Symposium 1989. Christchurch, New Zealand, pp: 367-371

Boughton, W.C. (2015), Master recession analysis of transmission loss in some Australian streams [online]. Australian Journal of Water Resources, 19(1), 43-51. Available at http://search.informit.com.au/

Boughton, W.C. (1989), A Review of the USDA SCS Curve Number Method. Australian Journal of Soil Research, 27: 511-523

Boughton, W.C. and Hill, P.I. (1997), A Design Flood Estimation Procedure using Data Generation and a Daily Water Balance Model. CRC for Catchment Hydrology. Report 97/8.

Boughton, W.C., Muncaster, S.H., Srikanthan, R., Weinmann, P.E., Mein, R.G. (1999) Continuous Simulation for Design Flood Estimation - a Workable Approach. Water 99: Joint Congress; 25th Hydrology & Water Resources Symposium, 2nd International Conference on Water Resources & Environment Research; Handbook and Proceedings; pp: 178-183.

Boyd, M.J., Bufill, M.C. and Knee, R.M. (1993), Pervious and Impervious Runoff in urban catchments, Hydrological Sciences Journal, 38(6), 463-478.

Boyd, M.J., Bufill, M.C. and Knee, R.M. (1994), Predicting Pervious and Impervious Storm Runoff from Urban Drainage Basins, Hydrological Sciences Journal, 39(4), 321-332.

Bufill, M.C. and Boyd M.J. (1992), A sample flood hydrograph model for urban catchment, Proceedings of the International Symposium on Urban Management, 98: 98-103, Sydney.

Carroll, D.G. (2012), URBS (Unified River Basin Simulator) A Rainfall Runoff Routing Model for Flood Forecasting and Design. Version 5.00 Dec, 2012.

Chapman, T.G. (1968), Catchment Parameters for a Deterministic Rainfall-Runoff Model in 'Land Evaluation' (ed G.A. Stewart), Macmillan, Melbourne, pp: 312-323.

Chapman, T.G. (1970), Optimisation of a rainfall-runoff model for an arid zone catchment, in Symposium on the results of research on representative and experimental basins, pp. 126-144, IASH Publ. No. 96, Wellington, New Zealand.

Cherkaver, D.S. (1975), The hydrologic response of small watersheds to suburban development: Observations and modeling in urbanization and water quality control, in Whiple Jr. (Eds), Proceedings of the American Water Resources Association (pp: 110-119), Herndon,VA.

Chiew, F.H.S. and McMahon, T.A. (1999), Modelling Runoff and diffuse pollution loads in urban areas. Water Science Technology, 39(12), 241-248.

Chu, S.T. (1978), Infiltration during an Unsteady Rain. Water Resources Research. 14(3), 461-466.

Cordery, I. (1970)' Antecedent Wetness for Design Flood Estimation, Civil Engineering Transaction, Institution of Engineers, Australia, CE12(2), 181-184.

Cordery, I. and Pilgrim, D.H. (1983), On the Lack of Dependence of Losses from Flood Runoff on Soil and Cover Characteristics, IAHS Pub., (140), 187-195.

Cordery, I., Pilgrim, D.H. and Doran, D.G., (1983), Some Hydrological Characteristics of Arid Western New South Wales. Hydrology and Resources Symposium. I.E.Aust. Nat. Conf. Pub., (83/13), 287-292.

Dayaratne, S.T. (2000), Modelling of Urban Stormwater Drainage Systems using ILSAX, PhD Thesis, Victoria University of Technology, August.

Dyer, B.G., Nathan, R.J., McMahon, T.A. and O'Neill, I.C. (1994), Development of Regional Prediction Equations for the RORB Runoff Routing Model. CRC for Catchment Hydrology Report 94/1. March 1994.

Eastgate, W.I., Swartz, G.L. and Briggs, H.S. (1979), Estimation of Runoff Rates for Small Queensland Catchments, Department of Primary Industries, Queensland Technical Bulletin No.15.

El-Kafagee, M. and Rahman, A. (2011), A study on initial and continuing losses for design flood estimation in New South Wales. In Chan, F., Marinova, D. and Anderssen, R.S. (eds) MODSIM2011, 19th International Congress on Modelling and Simulation, (pp: 3782-3787), Perth, Australia.

Flavell, D.J. and Belstead, B.S. (1986), Losses for Design Flood Estimation in Western Australia, Hydrology and Water Resources Symposium, Griffith University, pp: 203-208.

Fleming, P.M. (1974), The Australian Representatives Basins Programme, Journal of Hydrology, 13(1), 21-31.

Fleming, P.M. and Smiles, D.E. (1975), Infiltration of Water into Soil. In Chapman, T.G. and Dunin, F.X. eds. Prediction in Catchment Hydrology. Canberra. Australian Academy of Science. Pp: 83-110.

Frost, A., Ramchurn, A., Hafeez, M., Zhao, F., Haverd, V., Beringer, J., and Briggs, P. (2015), Evaluation of AWRA-L: the Australian Water Resource Assessment model. Modsim 2015 - 21st International Congress on Modelling and Simulation, 29 Nov - 4 Dec 2015, Gold Coast, 0, 2047–2053.

Goyen, A.G. (2000), Spatial and Temporal Effects on Urban Rainfall / Runoff Modelling. PhD Thesis. University of Technology, Sydney. Faculty of Engineering.

Goyen, A.G. (1983), A Model to Statistically Derive Design Rainfall Losses Hydrology and Water Resources Symposium 1983: Preprints of Papers; pages: 220-225. Institution of Engineers, Australia, National conference publication, No.83/13.

Goyen, A.G., O'Loughlin, G.G. (1999a), The Effects of Infiltration Spatial and Temporal Patterns on Urban Runoff. Water 99: Joint Congress; 25th Hydrology & Water Resources Symposium, 2nd International Conference on Water Resources & Environment Research; Handbook and Proceedings, pp: 819-822.

Goyen, A.G., O'Loughlin, G.G. (1999b), Examining the basic building blocks of urban runoff. Urban Storm Drainage - 8th International Proceedings. Sydney, 30 Aug.-3 Sept. 1999, 3: 1382-1390.

Green, R.E. and Ampt, G.A. (1911), Studies on Soil Physics: 1. Flow of Air and Water through Soils. Journal of Agricultural Science, 4: 1-24

Heneker, T.M, Lambert, M.F. and Kuczera, G. (2003), Overcoming the Joint Probability Problem Associated with Initial Loss Estimation in Design Flood Estimation. Australian Journal of Water Resources, 7(2), 101-109.

Hill, P.I., Graszkiewicz, Z., Sih, K. and Rahman, A. (2013), Revision project 6: Loss models for catchment simulation. Stage 2 Report P6/S2/016A.

Hill, P.I., Maheepala, U. and Mein, R.G. (1996), Empirical Analysis of Data to Derive Losses: Methodology, Programs and Results. CRC for Catchment Hydrology Working Document 96/5.

Hill, P.I., Graszkiewicz, Z., Loveridge, M., Nathan, R.J. and Scorah, M. (2015), Analysis of loss values for Australian rural catchments to underpin ARR guidance. Hydrology and Water Resources Symposium 2015, Hobart, 9-10 December 2015.

Hill et al

Hill, P.I., Graszkiewicz, Z., Taylor, M. and Nathan, R.J. (2014a), ARR Revision Project 6 Loss models for catchment simulation. Stage 4 Analysis of rural catchments. May 2014.

Hill, P.I., Graszkiewicz, Z., Nathan, R.J., Stephens, D.A. and Pearce, L. (2014b), Testing the Suitability of a probability distributed storage capacity loss model for design flood estimation. 2014 Hydrology and Water Resources Symposium, Perth.

Ilahee, M. (2005), Modelling Losses in Flood Estimation, A thesis submitted to the School of Urban Development Queensland University of Technology in partial fulfilment of the requirements for the Degree of Doctor of Philosophy, March 2005.

Ilahee, M. and Imteaz, M.A. (2009), Improved Continuing Losses Estimation Using Initial Loss-Continuing Loss Model for Medium Sized Rural Catchments. American J. of Engineering and Applied Sciences, 2(4), 796-803.

Ilahee, M. and Rahman, A. (2003), Investigation of Continuing Losses for Design Flood Estimation: A Case Study in Queensland. 28th International Hydrology and Water Resources Symposium, 10-14 November 2003,Wollongong, NSW, 1: 191-197.

Ishak, E., and Rahman, A. (2006) Investigation into Probabilistic Nature of Continuing Loss in Four Catchments in Victoria. In: 30th Hydrology & Water Resources Symposium: Past, Present & Future; pages: 432-437.

Janke, B. and Wilson, N. (2011), Development of techniques to quantify effective impervious cover.

Jensen, M. (1990), Rain-runoff parameters for six small gauged urban catchments. Nordic and drol., 21: 165-184.

Kemp, D.J. and Lipp, W.R. (1999), Predicting Stormwater Runoff in Adelaid - What do we Know?, Living with Water Seminar, The Hydrological Society of South Australia, Adelaide.

Kemp, D.J. and Wright, C. (2014), Flood Hydrology in an Arid Area - Findings from the Gammon Ranges Project. 2014 Hydrology and Water Resources Symposium, Perth. 24-27 February 2014, pp: 109-116.

King, K.W. (2000), Response of Green-Ampt Mein-Larsen Simulated Runoff Volumes to Temporally Aggregated Precipitation. Journal of the American Water Resources Association, 36(4), 791-797

Kjeldsen, T.R., Stewart, E.J., Packman, J.C., Folwell, S.S. and Bayliss, A.C. (2005), Revitalisation of the FSR/FEH rainfall-runoff method. Joint Defra/EA Flood and Coastal Erosion Risk Management R&D Programme. R&D Technical Report FD1913/TR.

Knighton, A.D. and Nanson, G.C. (1994), Flow transmission along an arid zone anastomosing river, Cooper Creek, Australia. Hydrological Processes, 8: 137-154.

Kuczera, G., Lambert, M., Heneker, T., Jennings, S., Frost, A. and Coombes, P. (2006), Joint probability and design storms at the crossroads, Australian Journal of Water Resources, 10(1), 63-79.

Lang, S.M., Hill, P.I. Scorah, M. and Stephens, D.A. (2015), Defining and calculating continuing loss for flood estimation, Proceedings of the 2015 Hydrology and Water Resources Symposium, Hobart.

Lee, J. and Lim, W.H. (1995), Green-Ampt Equation and the Soil Water Storage Depth in a Watershed Model, Proceedings of the Second International Symposium on Urban Stormwater Management, pp: 157 -161, 11-13 July, Melbourne.

Linsley, R.K., Kohler, M.A. and Paulhus, J.L. (1982), Hydrology for Engineers, ed 3, NewYork: McGraw-Hill.

Loveridge, M. (unpublished), Evaluation of Monte Carlo simulation for flood frequency curve derivation using an event-based rainfall-runoff model, Doctoral Thesis, University of Western Sydney.

Mag, V.S. and Mein, R.G. (1994), A flood forecasting procedure which combines the RORB and TOPOG models, Proceedings of the Hydrology and Water Resources Symposium, I.E. Nat. Conf. 94/15: 217-222.

Maidment, D.R. (1992), Handbook of Hydrology, New York: McGraw-Hill.

Manley, R.E. (1974), Catchment models for river management, MSc Thesis, University of Birmingham.

Mein, R.G. and Goyen, A.G. (1988), Urban Runoff. Transactions of the Institution of Engineers, Australia: Civil Engineering, CE30(4), 225-238.

Mein, R.G. And Larson, C.L. (1973), Modeling infiltration during a steady rain, Water Resources Research, 9: 384-394.

Mein, R. G. and McMahon, T. A. (1982), Review of the role of process modelling in the Australian Representative Basins Program, In Review of the Australian Representative Basin Program. Rep. Basin Ser. Rep.4.

Mein, R.G. and O'Loughlin, E.M. (1991), A new approach to flood forecasting, Proceedings of the Hydrology and Water Resources Symposium, I.E.Aust. Nat. Conf. Pub. No. 91/12, pp: 219-224, Perth.

Mein, R.G., Nandakumar, N. and Siriwardena, L. (1995), Estimation of initial loss from soil moisture indices (Pilot Study), Working Document 95/1, Cooperative Research Centre for Catchment Hydrology.

Melanen, M. and Laukkanen, R. (1981), Dependence of runoff coefficient on area type and hydrological factors, Proceedings of the 2nd International Conference on Urban Storm Drainage, Water Resources Publications, pp: 404-410, Littleton, Colorado, USA.

Minty, L.J. and Meighen, J. (1999), Development of temporal distributions of rainfall antecedent to large and extreme design bursts over southeast Australia, Hydrology Report Series HRS Report No. 6, Hydrometeorological Advisory Service, Melbourne: Bureau of Meteorology.

Moore, R.J. (1985), The probability-distributed principle and runoff production at point and basin scales, Hydrological Sciences Journal, 30(2), 273-297.

Muncaster, S.H., Weinmann, P.E. and Boughton, W.C. (1999), The representation of loss in continuous simulation models for design flood estimation, Proceedings of Water 99 Joint Congress, Institution of Engineers, Australia, pp: 184-189, 6-8 July, Brisbane.

Nathan, R.J., Weinmann, P.E. and Hill, P.I. (2003), Use of a Monte-Carlo Simulation to estimate the Expected Probability of large to extreme floods, Proceedings of the 28th International Hydrology and Water Resources Symposium, pp: 1105-1112, 10-14 November, Wollongong.

O'Loughlin, G. and Stack, R. (2014), DRAINS User Manual - a manual on the DRAINS program for urban stormwater drainage system design and analysis, Available at: <http://www.watercom.com.au/DRAINS%20Manual.pdf>.

Pearce, L.J. (2011), Regional runoff coefficients for summer and winter design flood events in south-west Western Australia, Proceedings of the 24th IAHR World Congress - Balance and Uncertainty, 26 June - 1 July, Brisbane, Australia.

Pearcey, M., Pettett, S., Cheng, S., and Knoesen, D. (2014), Estimation of RORB Kc parameter for ungauged catchments in the Pilbara Region of Western Australia, Proceedings of the 2014 Hydrology and Water Resources Symposium, 24 - 27 February, Perth.

Phillips, B., Goyen, A., Thomson, R., Pathiraja, S. and Pomeroy, L. (2014), Australian Rainfall and Runoff Revision Project 6: Loss models for catchment simulation - Urban Losses Stage 2 Report, February.

Pui, A., Lall, A. and Sharma, A. (2009), How does the interdecadal pacific oscillation affect design floods in Eastern Australia, Proceedings of H2009 - 32nd Hydrology and Water Resources Symposium, November, pp: 266-274.

Rahman, A., Weinmann, P.E. and Mein, R.G. (2002), The use of probability-distributed initial losses in design flood estimation, Australian Journal of Water Resources, 6(1), 17-29.

Rajendran, R., Mein, R.G. and Laurenson, E.M. (1982), A spatial model for the prediction of losses on small rural catchments, Department of Civil Engineering, Monash University, Australian Water Resources Council Technical Paper, No.75, Canberra.

Ren-Jun., Z (1992) The Xinanjiang model applied in China. Journal of Hydrology. 135: 371-381

Rigby, E.H. and Bannigan, D.J. (1996), The embedded design storm concept - a Critical Review, Proceedings of the Hydrology and Water Resources Symposium 1996: Water and the Environment, Preprints of Papers, National Conference Publication, (96/05), 453-459.

Riley, S.J. and Fanning, P.C. (1997), Spatial patterns of infiltration in suburban developments in the Hawkesbury-Nepean catchment,Science and Technology in the Environmental Management of the Hawkesbury-Nepean Catchment, Institution of Engineers, Australia, NCP 97/01, 10-11 July, Nepean.

Ruprecht, J.K. and Schofield, N.J. (1993), Infiltration characteristics of a complex lateritic soil profile, Hydrological Processes, 7(1), 87-97.

Shuster, W.D., Bonta, J., Thurston, H., Warnemuende, E. and Smith, D.R. (2005), Impacts of impervious surface on watershed hydrology: A review, Urban Water Journal, 2(4), 263-275.

Siriwardene, N.R., Cheung, B.P.M. and Perera, B.J.C. (2003), Estimation of soil infiltration rates of urban catchments, Proceedings of the 28th International Hydrology and Water Resources Symposium, Institution of Engineers, Australia, Wollongong.

Skukla, M.K., Lal, R. and Unkefer, P. (2003), Experimental validation of infiltration models for different land use and soil management systems, Journal of Soil Science, 168(3), 178-191.

Smith, A., Rahman, J., Baron-hay, S., and Shipman, D. (2016), A new web based water information service leveraging the Australian Water Resources Assessment Modelling System Australian Water Resources Assessment Modelling System, (January).

Stewart, B.J. and Boughton, W.C. (1983), Transmission losses in natural streambeds - a Review, Proceedings of the Hydrology and Water Resources Symposium 1983, Institution of Engineers, Australia, preprints of papers, pp: 226-230.

Stokes, R.A. (1989), Calculation file for Soil Water Model - Concept and theoretical basis of soil water model for the south west of Western Australia, Unpublished Report, Water Authority of W.A. Water Resources Directorate.

Tularam, G.A. and Ilahee, M. (2007), Initial loss estimates for tropical catchments of Australia, Environmental Impact Assessment Review, 27(6), 493-504.

United States Army Corps of Engineers (USACE) (1998), Runoff from snowmelt, engineering manual, 31 March, Washington: USACE.

Van Mullem, J.A., Woodward, D.E., Hawkins, R.H., and Hjelmfelt, A.T. Jr. (2002), Runoff curve number method: beyond the handbook. In: Hydrologic Modeling for the 21st Century, Proceedings of the Second Federal Interagency Hydrologic Modeling Conference, July 28 - August 1, Las Vegas.

Walsh, M.A., Pilgrim, D.H. and Cordery, I. (1991), Initial losses for design flood estimation in New South Wales, Proceedings of the International Hydrology and Water Resources Symposium, I.E.Aust. Nat. Conf. Pub. (91/19), 283-288.

Ward, T.J., Hawkins, R.H., Van Mullem, J.A., Woodward. D.E. (2009), Curve number hydrology: state of the practice, College Park: American Society of Civil Engineers.

Water and Rivers Commission (2003), SWMOD A rainfall loss model for calculating rainfall excess - user manual (version 2.11), Prepared by Hydrology and Water Resources Branch Resource Science Division, September.

Waugh, A.S. (1991), Design losses in flood estimation, Proceedings of the International Hydrology and Water Resources Symposium, I.E.Aust. Nat. Conf. Pub. No.91/19, pp: 629-630, Perth.

William, J. (1994), Preliminary report on the use of the Green Ampt Infiltration Model for measuring surface runoff on rural catchments,unpublished report 18 February 1994, CRC for Catchment Hydrology.

Woodward, D.E., Hawkins, R.H. and Quan, Q.D. (2002), Curve number method: origins, applications and limitations, Proceedings of Hydrologic Modeling for the 21st Century, Second Federal Interagency Hydrologic Modeling Conference, July 28 - August 1, Las Vegas.

Woolmington, E. and Burgess, J.S. (1983), Hedonistic water use and low-flow runoff in Australia's national capital, Urban Ecology, 7(3), 215-227.

Zaman, S. and Ball, J.E. (1994), Simulation of small events in an urban catchment, Proceedings of the 1994 Hydrology and Water Resources Conference - Water Down Under '94, I.E.Aust. Nat. Conf. Pub. No.94/15, pp: 353 - 358, Adelaide.

Chapter 4. Baseflow Models

Peter Hill, Rachel Brown, Rory Nathan, Zuzanna Graszkiewicz

Chapter Status	Final
Date last updated	14/5/2019

4.1. Introduction

Streamflow consists of two components based on response timing, following a rainfall event. Water that enters a stream rapidly is termed as quickflow and is sourced from rainfall excess, after the loss has been satisfied (representing a range of processes such as interception, infiltration and depression storage). On the contrary, water that takes longer to reach a river is termed as baseflow and is sourced primarily from groundwater discharge into the river. Also, different locations have varying degrees of baseflow contribution to streamflow, based on regional hydrogeological conditions.

According to Nathan and McMahon (1990) and Brodie and Hostetler (2005), the baseflow hydrograph has the following features:

- The low flow before the start of a flood event is assumed to consist entirely of baseflow;
- The rapid rise of river during a rainfall event increases the volume of water held as bank storage, which returns to the main streamflow after a delay and creates a baseflow peak after the main flood peak;
- The recession of the baseflow peak continues after the recession of the streamflow peak;
- The baseflow recession generally follows an exponential decay function; and
- The baseflow hydrograph rejoins the total hydrograph as the quickflow ceases.

The majority of design flood estimation in Australia utilises flood event models that focus on surface runoff processes. In such models the baseflow component is either ignored or incorporated after the surface runoff has been estimated. The conceptualisation of some continuous models or fully integrated surface water - groundwater models explicitly incorporate the estimation of the baseflow component. However, given the prevalence of flood event models for design flood estimation, the following guide concentrates on estimating a baseflow hydrographs to combine with an existing estimate of surface runoff.

ARR Revision Project 7 - Baseflow for Catchment Simulation, developed a method to calculate and incorporate the baseflow contribution to design flood estimates. Stage 1 of the project (Murphy et al., 2009; Murphy et al., 2011a; Graszkiewicz et al., 2011), focussed on the physical processes of groundwater-surface water interaction and theoretical approaches to baseflow separation. The identified methods were applied to eight case study catchments across Australia in order to develop a suitable approach for wide scale application. Stage 2 of Project 7 (Murphy et al., 2011b) covered the analysis of 236 catchments across Australia, the development of prediction equations to estimate baseflow parameters and the development of a method for their application to design estimates for catchments across Australia.

This document utilises the method developed in ARR Revision Project 7 to provide guidance on how to estimate baseflow for design flood estimation.

The remainder of this chapter is structured as follows:

- Book 5, Chapter 4, Section 2 describes those characteristics of the baseflow that need to be estimated;
- Book 5, Chapter 4, Section 3 discusses considerations when selecting the approach to estimating baseflow;
- Book 5, Chapter 4, Section 4 outlines the different approaches to estimating the baseflow contribution to design hydrographs; and
- Book 5, Chapter 4, Section 5 provides 2 worked examples.

4.2. Guiding Principles

This guide on baseflow draws upon a significant body of work undertaken through ARR Revision Project 7, and provides advice on how to estimate baseflow for design flood estimation.

Users should consider the characteristics of the particular catchment with respect to the underlying assumptions that form the basis of the method outlined in this chapter. The following approach is applicable across the vast majority of Australian catchments. However, users should draw upon their understanding of the particular catchment of interest to make an informed decision regarding the relevance of each step, considering the following issues:

- **Snow melt**, which is not considered in this approach.
- Significant farm dam development or other flow regulations in the catchment, which can mask the contribution of baseflow.
- **Design flood estimation for Rare to Extreme events.** The method outlined below is only relevant to events up to approximately the 1% AEP and guidance for baseflow contribution to very rare and extreme events is provided in Book 8.
- **Seasonality of events,** Seasonality is not explicitly considered in this approach. Region-specific analysis should be undertaken where seasonality of flood producing factors is important. Kinkela and Pearce (2014) describe such a study for the south-west of Western Australia.
- **Urbanised catchments.** The approach and catchments considered in developed of the method were selected to represent rural conditions, therefore the approach is not applicable to urban catchments (flood estimation for urban catchments is covered in Book 9). Baseflow is typically a small contribution to the flows.
- **Small catchments away from the main stem of the river network.** The regional estimates relate to a location on the main stem of the river and reflect the characteristics of all the contributing catchments. The baseflow characteristics of individual tributaries may be different from those in the larger contributing catchment.
- **Estimation of historic events.** The approach in this chapter has been developed for application in design flood estimation, but not for the estimation of the baseflow component of streamflow for individual historic events.

- **Estimation of baseflow for extended periods.** This guide is relevant for design events only. Users should refer to the technical documents supporting ARR Revision Project 7 for more general information on baseflow estimation for longer sequences.

If any of the above factors are deemed to be important for the catchment of interest, it is recommended that the user considers the suitability of this approach to their catchment of interest. It is also relevant to interpret the outcomes in the light of underlying assumptions, and draw on local data to supplement the approach or to consider alternative methods, where these assumptions are not fully met.

Users should refer to the technical reports and analyses undertaken for ARR Revision Project 7 (Murphy et al., 2009; Murphy et al., 2011b) for full details of the data analysis and assumptions that form the basis of the method outlined in this chapter. Users may also like to refer to these supporting documents and data to draw further local conclusions from the significant body of work undertaken through the study.

4.3. Baseflow Characteristics

For design flood estimation in Australia, baseflow has traditionally been considered to be a relatively minor contributor to the flood hydrograph, but baseflow can potentially be significant in more frequent events. This is particularly the case where the catchment geology consists of high yielding aquifers.

For instance, for a 10% AEP, about two-thirds of Australian unregulated rural catchments have baseflow contributions that are estimated to be between approximately 5% and 30% of the peak flow. There are only about 5% of catchments that have a higher proportion of baseflow, which these tend to be located in south-west WA, south-east SA and some areas in the tropics. In just less than a third of unregulated rural catchments, the baseflow is estimated to be less than 5% of the peak (for a 10% AEP). Baseflow is typically insignificant in urban catchments due to the degree of channel modifications and extent of impervious areas.

The variation of baseflow with exceedance probability is discussed in Book 5, Chapter 4, Section 2. For events more frequent than a 10% AEP, the baseflow can represent a significant proportion of the peak flow, particularly volume. For rarer events, baseflow makes up a smaller relative contribution to the surface runoff. For the majority of catchments, it is likely that the contribution of baseflow for extreme events will be less important; although, for volume dependent systems, the baseflow volume may still be significant. Guidance for baseflow contribution to very rare and extreme events is provided in Book 8.

This chapter concentrates on the estimation of baseflow contribution to be included with surface runoff estimated from a flood event model, rather than separating baseflow from recorded streamflow. Approaches for separating baseflow are described in the ARR Revision Project 7 Stage 1 report (Murphy et al., 2009).

In the context of design flood estimation, a surface runoff hydrograph will typically be generated using a flood event model that excludes baseflow. It is therefore necessary to estimate the baseflow contribution in order to represent the total event peak and volume, and to generate a total streamflow hydrograph. This concept is represented in Figure 5.4.1, which depicts the following features of an event hydrograph:

- *Surface runoff peak* - the maximum flow associated with the surface runoff event.
- *Time to the surface runoff peak* - measured from the start of the event to the surface runoff peak.

- *Volume of surface runoff for the event* - event volume, represented by the area under the hydrograph.
- *Baseflow peak* - maximum baseflow associated with the event.
- *Time to the baseflow peak* - measured from the start of the event to the baseflow peak.
- *Baseflow under the surface runoff peak* - baseflow that occurs at the time of the surface runoff peak.

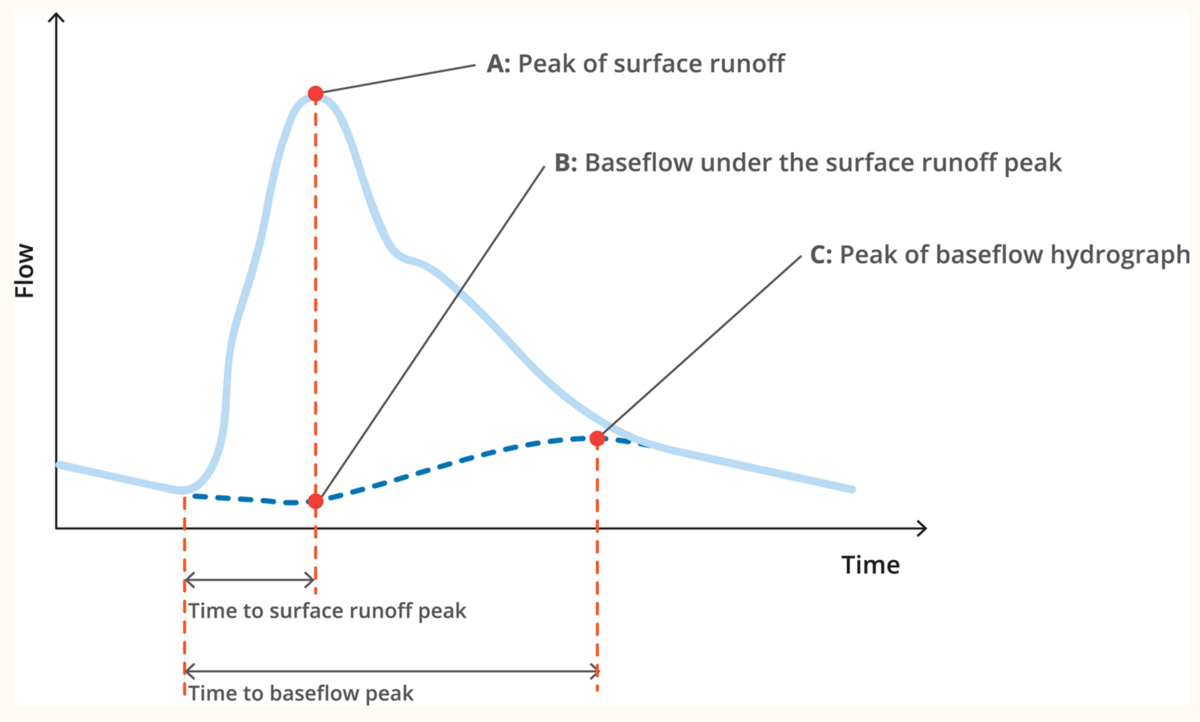


Figure 5.4.1. Key Characteristics for Calculation in a Flood Hydrograph

4.4. Selection of Approach

The recommended approach to quantifying baseflow is dependent on the catchment characteristics, data availability and baseflow characteristics of the catchment. Figure 5.4.2 provides a decision pathway to determine the most suitable approach to estimate baseflow contribution to design flood events, based on site specific criteria.

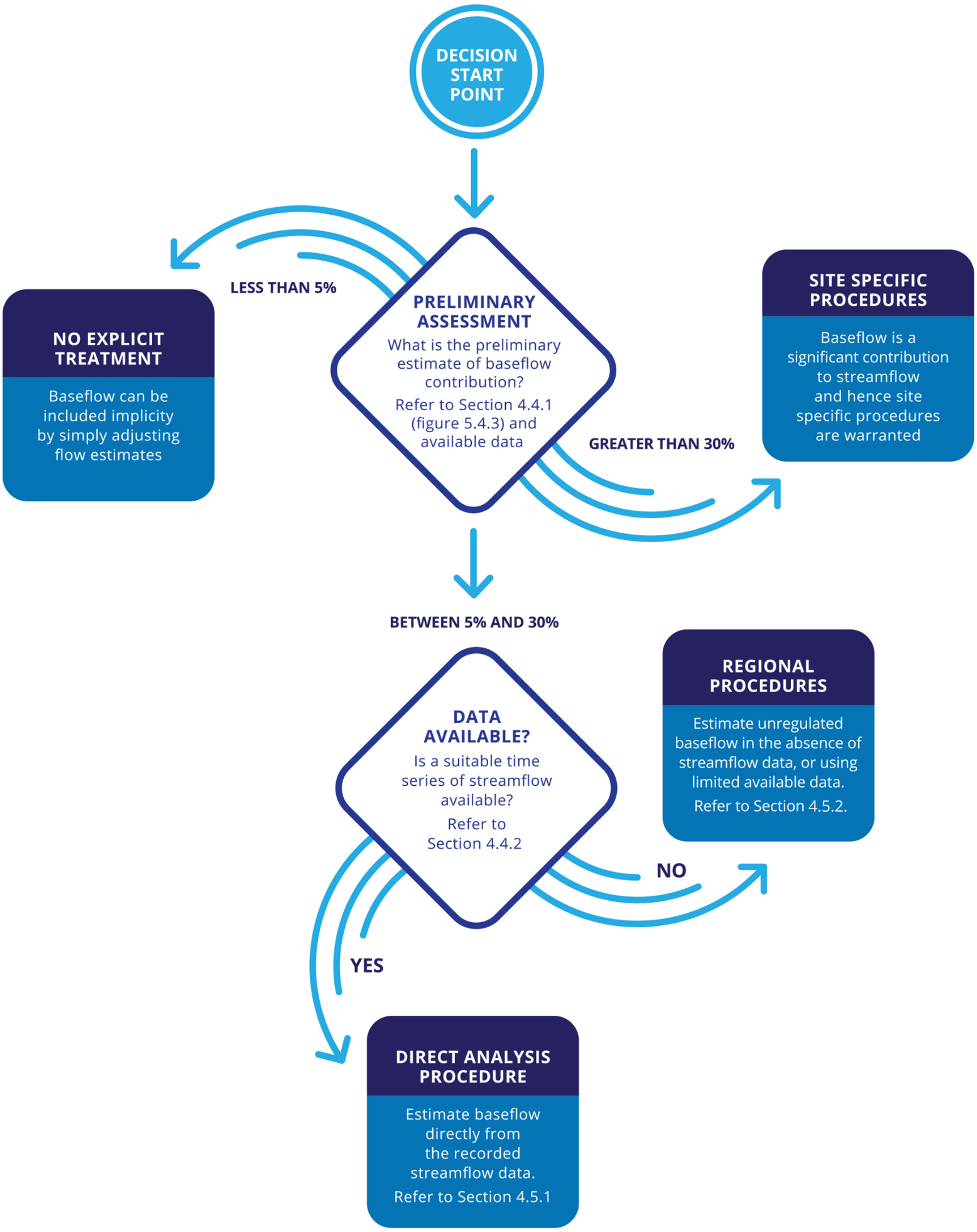


Figure 5.4.2. Decision Tree for Method to Estimate Baseflow Contribution to Design Flood

4.4.1. Preliminary Assessment of Baseflow

A preliminary assessment should be undertaken, in order to consider whether baseflow is likely to be a significant component of the design flood hydrograph. However, a detailed analysis is not suggested at this point,instead, this assessment is intended to be a coarse screening test to help determine the most appropriate approach in estimating baseflow for the catchment of interest based on the expected baseflow characteristics. A number of tools are available to support this assessment:

- Figure 5.4.3 is provided to readily identify the relative magnitude of baseflow compared to surface runoff for catchments across Australia for a 10% AEP event. Practitioners can identify their catchment of interest within this data set and note the value of Baseflow Peak Factor. This map shows catchments to match with the screening criteria identified below, while Figure 5.4.5 provides a more detailed estimate of the baseflow. The data used to generate these figures is available in Geographic Information System (GIS) format, to assist locating catchments/points of interest.

- If available, streamflow data for the catchment should be reviewed. The magnitude of flows between flood events relative to peaks can be used to determine whether baseflow is likely to be an important component of the design flood hydrograph.

Where baseflow is expected to be a small component compared to the surface runoff, the design flood peak can be adjusted up to approximately 5% to make an allowance for baseflow. Catchments with a Baseflow Peak Factor less than 0.05 are considered suitable for this approach. This reflects approximately 30% of the catchments mapped in Figure 5.4.3.

It is suggested that catchments that have a baseflow contribution greater than approximately 5% of the surface runoff should explicitly incorporate the baseflow component into the design flood, using a more rigorous approach.

While baseflow is a very large component of the event (a contribution of more than approximately 30% is suggested, reflected by a Baseflow Peak Factor greater than 0.3), the baseflow contributions should be estimated using techniques that are suited to the nature and availability of local data (e.g. Brodie and Hostetler (2005), Chapman and Maxwell (1996), Nathan and McMahon (1990), and Ladson et al. (2013)). Approximately 5% of Australian catchments, generally the areas of tropical north Australia, south-west Western Australia and the south-east coastline of South Australia, fall into this category. The specific approach of relevance will depend on local conditions and the user is guided to the above references to determine the most appropriate baseflow estimation technique.

Where baseflow is between approximately 5% and 30% of the surface runoff (Baseflow Peak Factor between 0.05 and 0.3), the approach outlined below is recommended. This relates to approximately 65% of the catchments mapped in Figure 5.4.3.

4.4.2. Suitability of Stream Flow Data

Where possible, recorded streamflow data should be used directly to quantify baseflow. However, ideally more than 10 years of continuous streamflow data would be required to undertake detailed site-specific analyses. Appropriate data quality checks should be undertaken prior to ascertaining the period of record available for analysis. Preferably, the streamflow data should extend over a period of record that enables the identification of an event of similar magnitude to the design flood of interest. Some subjectivity may be required to determine the suitability of available streamflow for this approach, depending on the period of record and the events represented within this data, with reference to event magnitude and duration of interest.

Additionally, there are various activities that can impact upon the flow characteristics associated with the baseflow. These activities include:

- *Flow regulation from upstream reservoirs* – reservoirs that release outflows that are different to inflows will produce a low flow response that can be misinterpreted as baseflow at downstream flow gauges.

- *Catchment farm dams* – high concentration of catchment farm dams could influence baseflow but only where the dams are located in a manner where they intercept and store flows arising from long-term depletion of catchment storage.
- *Major diversions* – diversions for consumptive use such as irrigation channels, urban diversions, etc. These diversions decrease low flows and hence appear to reduce estimates of baseflow. Allowances can be made for those diversions where they are metered.
- *Urbanisation* – in urban areas, features such as excess garden or sports field watering can increase low flows during summer that appear similar to baseflow in streamflow data.
- *Return flows* – water can be returned to rivers from sewage treatment plants or from industry,increasing low flows and appearing similar to baseflow.

Where present, these activities will influence the observed flow characteristics making it difficult to identify and quantify baseflow. While it is possible to estimate baseflow in these locations using the regional approach, the baseflow estimate will reflect the unregulated baseflow conditions.

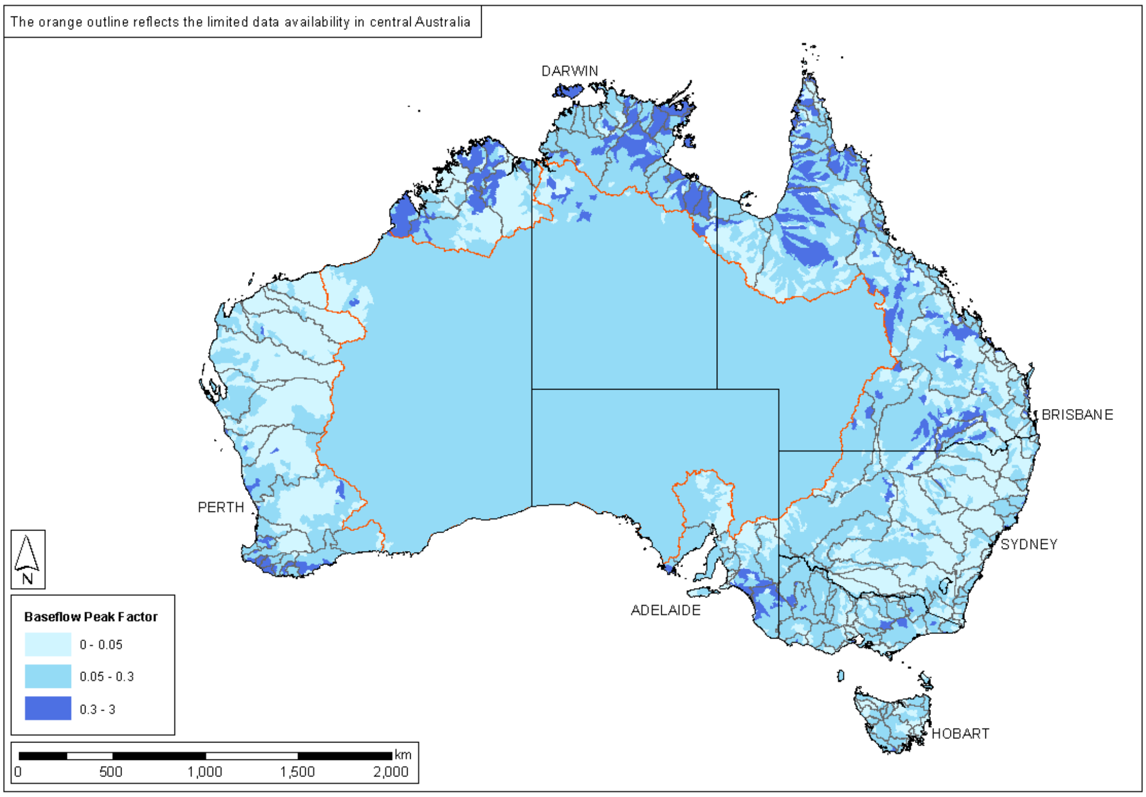


Figure 5.4.3. Preliminary Assessment of Baseflow Peak Factor for a 10% AEP Event

4.5. Quantifying Baseflow Contribution to Design Flood Estimates

4.5.1. Estimating Baseflow Using Streamflow Data

As outlined in Book 5, Chapter 4, Section 4, available streamflow data should meet a number of conditions, to be considered suitable for application to assess baseflow. If streamflow data does not meet these criteria, this approach is not relevant and practitioners are directed to the regional approach outlined in Book 5, Chapter 4, Section 5.

To estimate baseflow directly from available streamflow data, the following steps should be followed:

a. **Review Data Quality**

   Review the streamflow data and eliminate any poor quality data, as determined using the quality codes for each time step.

b. **Check Record Length**

   Determine the resulting period of record available for analysis. If less than 10 years of data is available, the regional approach in Book 5, Chapter 4, Section 5 may be more appropriate for application. If more than approximately 10 years of data is available, the following steps can be applied.

c. **Flood Frequency Analysis**

   Extract a series of peak flows from the recorded streamflow data and undertake a flood frequency analysis as described in Book 3, Chapter 2. It is recommended that as a minimum, the 10% AEP event should be identified. If the streamflow data record is suitable, identify events of similar magnitude to the design flood of interest.

d. **Estimate Baseflow from Recorded Floods**

   Estimate the baseflow for flood events identified above. Literature, such as Nathan and McMahon (1990), Chapman and Maxwell (1996) and Brodie and Hostetler (2005), provides guidance on key features of the baseflow hydrograph. If the streamflow data is suitable, the baseflow should be estimated for events of similar magnitude to the design flood of interest.

e. **Adjust for Different AEPs**

   If the above step has been applied to events of similar magnitude to the design flood of interest, the estimated baseflow magnitude and volume can be used directly with the design flood surface runoff hydrograph to generate the total streamflow estimate. Refer to Book 5, Chapter 4, Section 5 for a description of how to generate the total streamflow hydrograph. In this case, the key baseflow features, including timing, should be taken from baseflow estimated above .

   If the design events are outside of the range of recorded events it is necessary to scale the baseflow contribution to reflect the AEP of interest. The method outlined in Book 5, Chapter 4, Section 5 should be applied, with the key baseflow characteristics determined from the recorded streamflow rather than the regional approach, as outlined in the following section.

4.5.2. Estimating Baseflow in the Absence of Streamflow Data

A regional method to estimate baseflow contribution to design flood events has been developed so that it is applicable for unregulated catchments across Australia. This method was developed using catchments across Australia with catchment areas between 7 and 7800 km^2, and as such the approach is considered most suitable for application for catchments within this range. Practitioners should be mindful of these constraints when applying and interpreting the outcomes from this method.

The following three parameters are defined to characterise the contribution of baseflow to design flood hydrographs:

1. **Baseflow Peak Factor:** This factor is applied to the estimated surface runoff peak flow to give the value of peak baseflow for a 10% AEP event.
2. **Baseflow Volume Factor:** This factor is applied to the estimated surface runoff volume to give the volume of the baseflow for a 10% AEP event.
3. **Baseflow Under Peak Factor:** This factor is applied to the estimated surface runoff peak flow to give the baseflow at the time of the peak surface runoff and can be determined from the Baseflow Peak Factor, such that the Baseflow Under Peak Factor = 0.7 x Baseflow Peak Factor.

The Baseflow Peak Factor and Baseflow Volume Factor are presented in Figure 5.4.5 and Figure 5.4.6, which covers the whole of Australia. It should be noted that the maps represent the values for the total area upstream of the main stem of the river, rather than any smaller sub-catchments. As baseflow characteristics may vary from the main stream, the estimation of baseflow in subcatchments may require the approach to be supplemented with additional local data or through an alternative approach, such as transposition from another location.

ARR Revision Project 7 developed a series of regression relationships to estimate the Baseflow Peak Factor and Baseflow Volume, based on catchment characteristics (Murphy et al., 2011a). The resulting values are presented in Figure 5.4.5, Figure 5.4.6 and supporting spatial data for use with GIS, and can be used to determine the Baseflow Peak Factor and the Baseflow Volume Factor for a 10% AEP event for the catchment of interest. This data is available on the ARR Data Hub (Babister et al. (2016), accessible at http://data.arr-software.org/).

These factors provide information on baseflow contribution to design flood events for a 10% AEP event. Table 5.4.1 shows the AEP scaling factors that should be applied to the 10% AEP Baseflow Peak Factor and Baseflow Volume Factor to scale relevant factors to reflect events of other AEPs.

Table 5.4.1. AEP Scaling Factors, F_{AEP}, to be applied to the 10% AEP Baseflow Peak Factor and the Baseflow Volume Factor to determine the Baseflow Peak Factor for events of various AEPs

EY	AEP (%)	Baseflow Peak Factor	Baseflow Volume Factor
2	86.47	3.0	2.6
1	63.21	2.2	2.0
0.5	50	1.7	1.6
0.2	18.13	1.2	1.2
0.11	10	1.0	1.0
0.05	5	0.8	0.8
0.02	2	0.7	0.7
0.01	1	0.6	0.6

For events of AEPs not shown in Table 5.4.1, Figure 5.4.4 can be used to determine an appropriate AEP factor. This is to be multiplied by the 10% AEP Baseflow Peak Factor or the

Baseflow Volume Factor as relevant, to determine the factor for other event magnitudes. Guidance for baseflow contribution to Rare and Extreme Events is provided in Book 8.

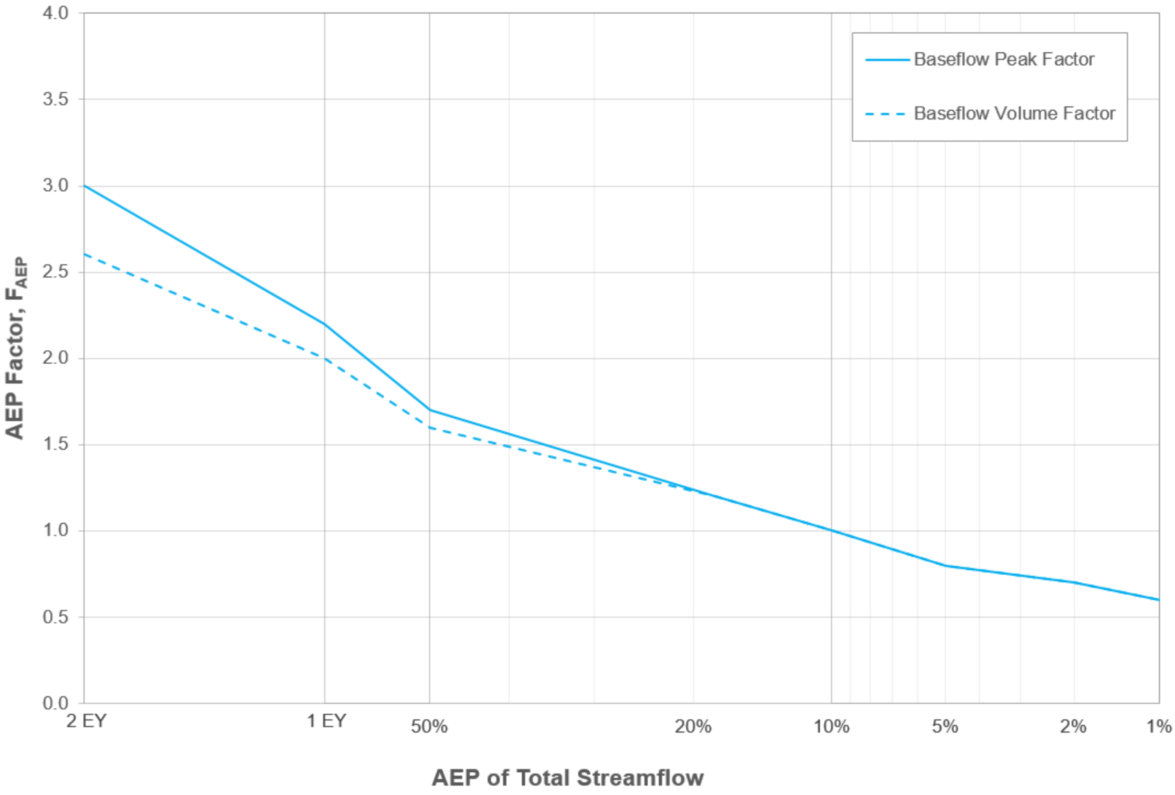


Figure 5.4.4. AEP Factors, F_{AEP} to be applied to the 10% AEP Baseflow Volume Factor to determined the Baseflow Volume Factor for events of various AEPs

This information is applied to the design flood event using the procedure outlined in the relationships below that relate to the typical flood hydrograph in Figure 5.4.1.

To Calculate the Peak Baseflow (Point C in Figure 5.4.1):

1. Determine the Baseflow Peak Factor for a 10% AEP ($R_{\mathrm{BPF,10\%AEP}}$) from Figure 5.4.5.

2. Determine the AEP factor, corresponding to the event AEP using Table 5.4.1 or Figure 5.4.4. Scale the 10% AEP Baseflow Peak Factor appropriately to determine the Baseflow Peak Factor for the event severity of interest.

3. Apply the Baseflow Peak Factor to the calculated peak surface runoff as in Equation (5.4.1).

$$Q_{\text{Peak baseflow}} = R_{\text{BPF}} Q_{\text{Peak surface runoff}} \tag{5.4.1}$$

4. Calculate the timing of the baseflow peak using Equation (5.4.2). The time to the peak surface runoff should be applied in units of hours from the start of the event.

$$T_{\text{Peak baseflow}} = 0.92\ \mathrm{T}_{\text{Peak surface runoff}} + 33.4 \tag{5.4.2}$$

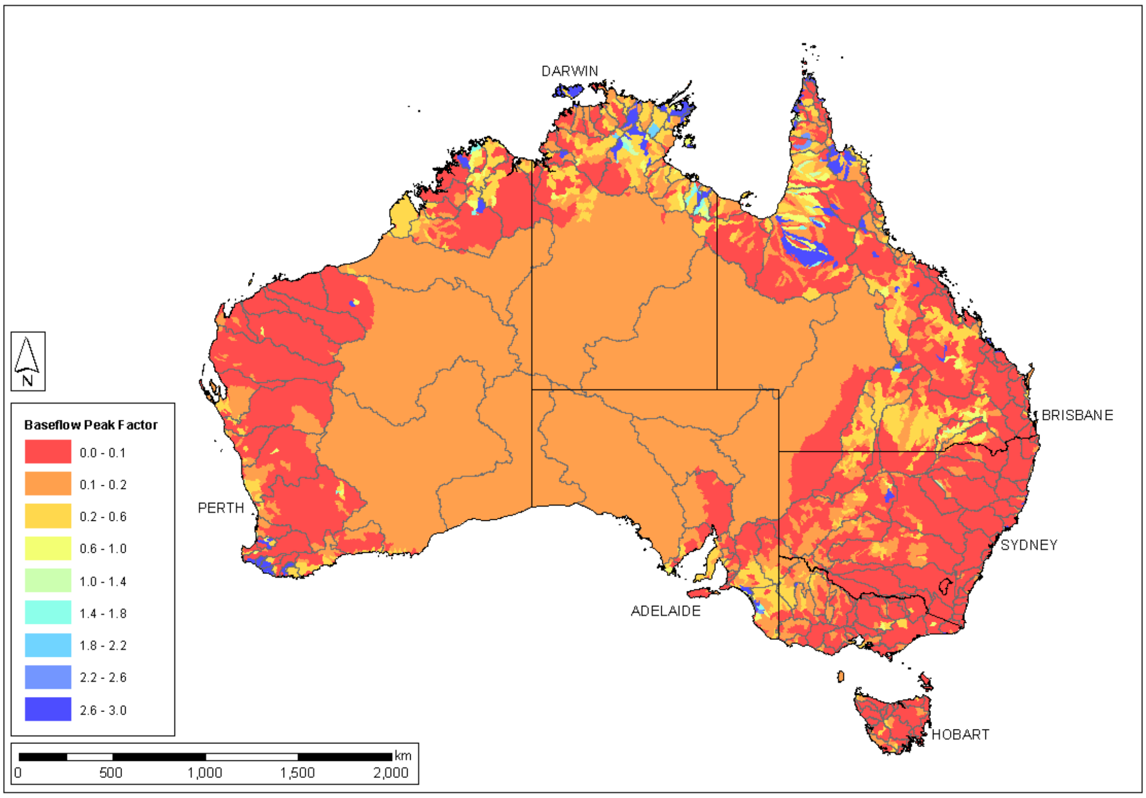


Figure 5.4.5. Map of Baseflow Peak Factor for a 10% AEP

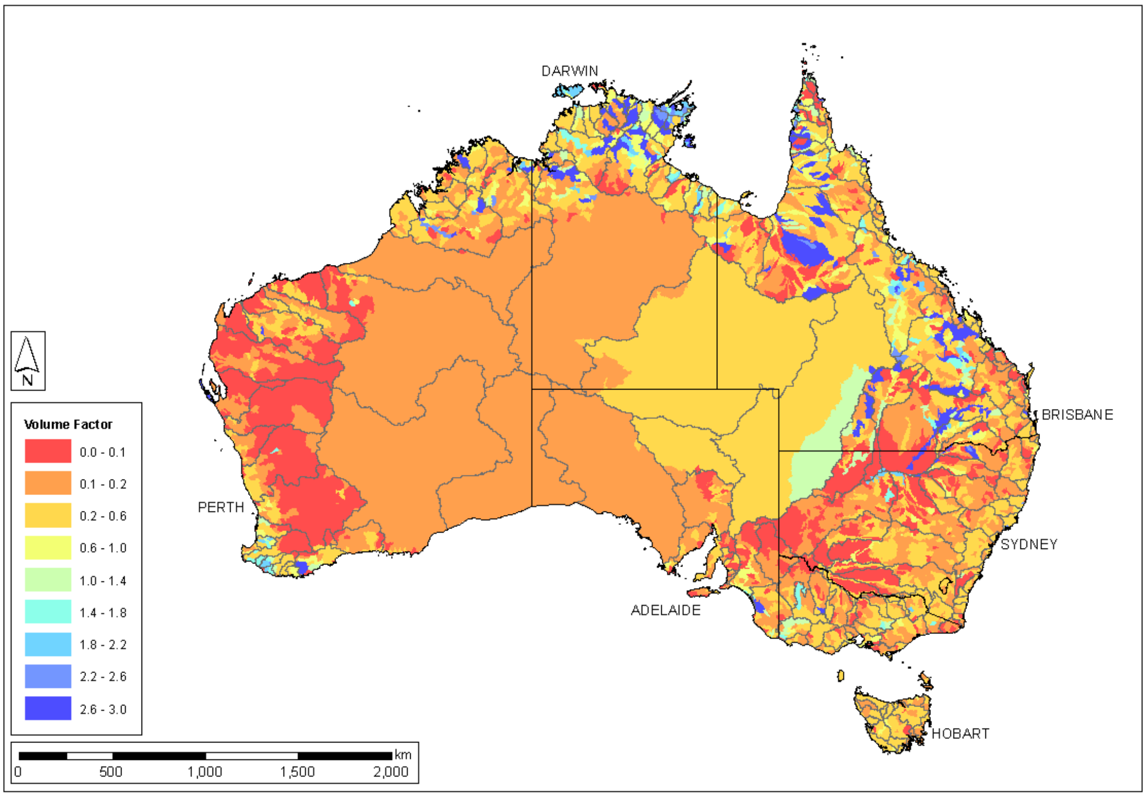


Figure 5.4.6. Map of Baseflow Volume Factor for a 10% AEP

To calculate the baseflow under the peak streamflow (Point B in Figure 5.4.1):

1. The Baseflow Peak Factor (R_{BPF}) calculated for the appropriate AEP event as above should be used in Equation (5.4.3) to calculate the Baseflow Under Peak Factor (R_{BUPF}).

$$R_{\text{BUPF}} = 0.7 \times R_{\text{BPF}} \tag{5.4.3}$$

2. R_{BUPF} should be used as in Equation (5.4.4) to calculate the baseflow under the peak streamflow.

$$Q_{\text{Baseflow under peak streamflow}} = R_{\text{BUPF}} Q_{\text{Peak surface runoff}} \tag{5.4.4}$$

To Calculate the Total Streamflow Peak (Point A in Figure 5.4.1):

1. Calculate the baseflow under the streamflow peak for the appropriate AEP as above.

2. Add the baseflow under the streamflow peak calculated using Equation (5.4.4), to the calculated peak surface runoff as in Equation (5.4.5).

$$Q_{\text{Peak streamflow}} = Q_{\text{Peak surface runoff}} + Q_{\text{Baseflow under peak streamflow}} \tag{5.4.5}$$

To Calculate the Total Baseflow Volume for an Event:

1. Determine the Baseflow Volume Factor for a 10% AEP ($R_{BVF,10yrARI}$) from Figure 5.4.5.

2. Determine the AEP factor corresponding to the AEP event using Table 5.4.1 or Figure 5.4.4. Scale the 10% AEP Baseflow Volume Factor appropriately to determine the Baseflow Volume Factor (R_{BVF}) for the event.

3. Apply the Baseflow Volume Factor to the calculated surface runoff volume as in Equation (5.4.6).

$$V_{\text{Baseflow}} = R_{\text{BVF}} V_{\text{Surface Runoff}} \tag{5.4.6}$$

To Calculate the Total Streamflow Volume for an Event:

1. Calculate the baseflow volume for the event using the appropriate AEP factors.

2. The baseflow volume calculated using Equation (5.4.6) should be added to the calculated surface runoff as in Equation (5.4.7).

$$V_{\text{Total streamflow}} = V_{\text{Surface runoff}} + \text{V}_{\text{Baseflow}} \tag{5.4.7}$$

This approach can be directly applied to estimate the baseflow contribution to any event between a 2 EY and a 1% AEP.

4.5.3. Generating the Total Streamflow Hydrograph

The characteristics of surface runoff, baseflow and total streamflow can be used to estimate the hydrograph for the event. For simplicity, a linear approach can be used to estimate the baseflow at each time step, by fitting between the data values estimated through the process described above and matching the baseflow volume. This time series can be manually added to the surface runoff time series data to generate a time series for the total streamflow, which is generated through this process and should be reviewed. It may require smoothing to produce a more realistic temporal distribution of baseflow. This process is presented in Figure 5.4.7.

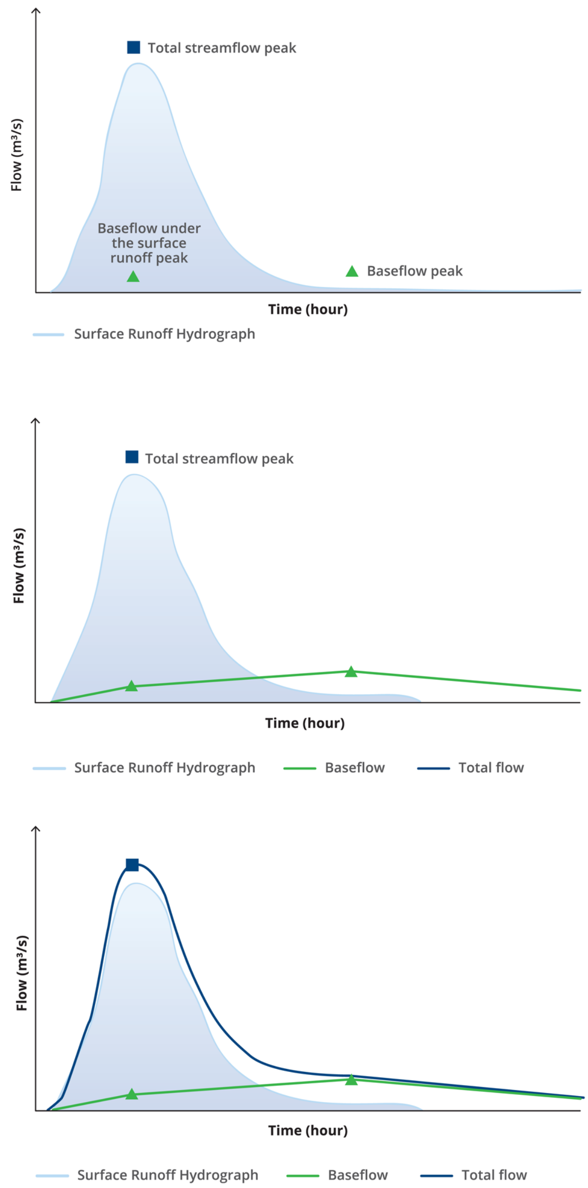


Figure 5.4.7. Total flow hydrograph generation approach, where (a) the data values calculated through the baseflow estimation process are plotted; (b) linear interpolation between the baseflow data points and matching the area under the curve to the baseflow event volume is used to estimate the baseflow time series, which is plotted on the hydrograph in green; and (c) the total streamflow time series is generated by summing the surface runoff and baseflow time series values, with the streamflow hydrograph plotted in dark blue.

4.6. Example

The process described in Book 5, Chapter 4, Section 5 is worked through in a number of different case study examples.

4.6.1. North Maroochy River at Eumundi, Queensland

Catchment 1 is located in south-east Queensland and has a catchment area of 40 km^2. Hourly flow data has been collected at this location since 1982, providing approximately 30 years of data. Very little data was missing or of poor quality, during this period.

The 10% AEP event is of interest for this case study. The reviewed flow data was used to identify flood peaks, in particular the 10% AEP event. A comparable event was identified in the record in February 1999. The event hydrograph was plotted and key characteristics of the baseflow were identified manually (Figure 5.4.8; manually identified baseflow features shown by green points). Straight lines were used to join the key baseflow features, to estimate the baseflow time series. In this instance, the baseflow peak occurs 18 hours after the peak of the streamflow.

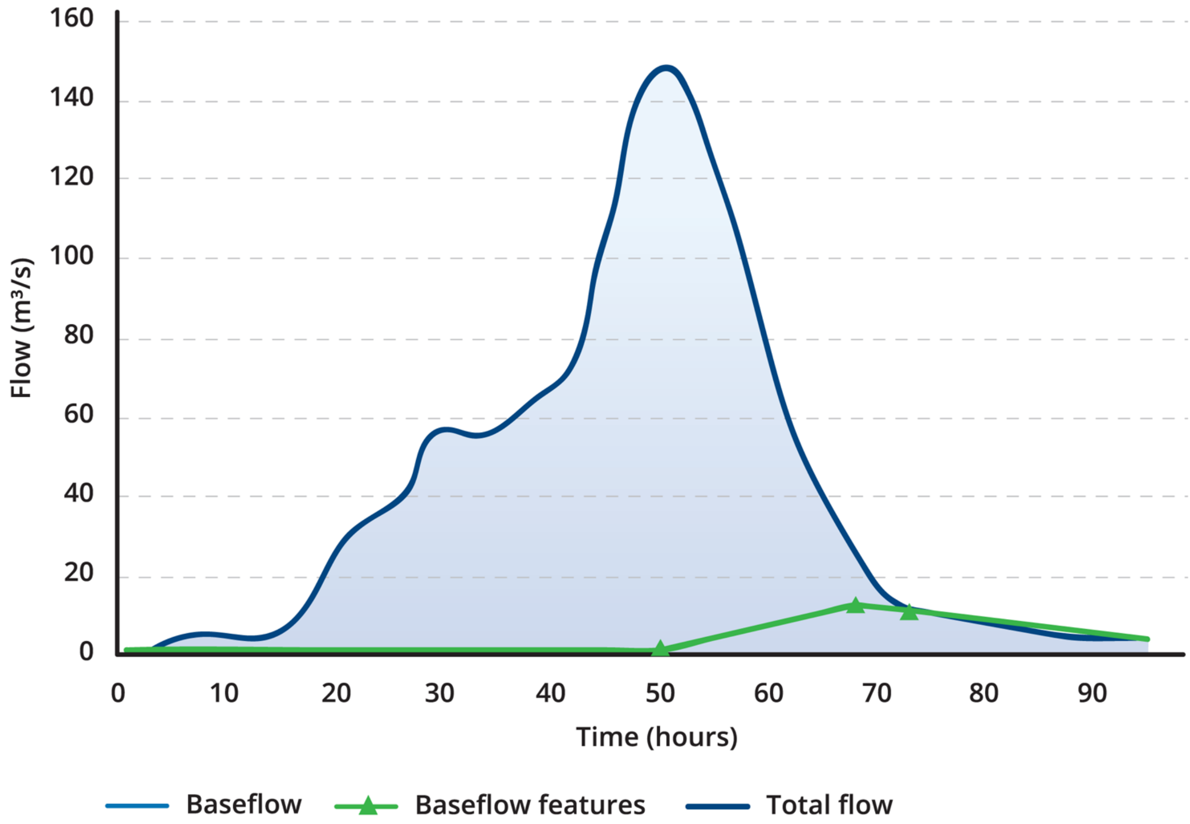


Figure 5.4.8. Streamflow Hydrograph Approximating the 10% AEP Event for the North Maroochy River at Eumundi

The surface runoff hydrograph for the design flood event was generated using a flood event model with a critical duration of 18 hours (Figure 5.4.11, and details in Table 5.4.3).

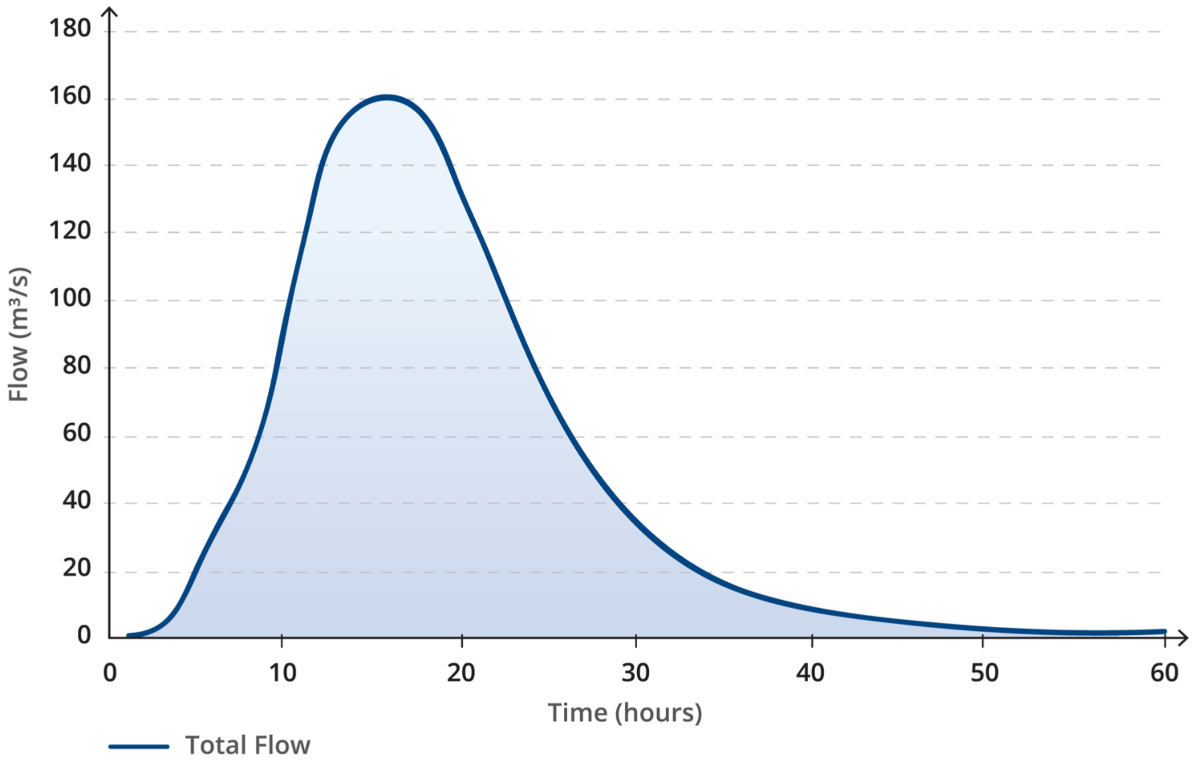


Figure 5.4.9. Surface Runoff Hydrograph for the 10% AEP Design Flood event at Eumundi

Table 5.4.2. Key Surface Runoff Characteristics for the 10% AEP Design Flood Event at Eumundi

Characteristic	Data from design flood
Surface runoff peak flow (m^3/s)	160.6
Time to the surface runoff peak (hours, from the start of the event)	16
Volume of surface runoff for the event (m^3)	9.9×10^6

The baseflow series estimated above was used directly to approximate the baseflow for the 10% AEP design flood event. The baseflow at the time of the streamflow peak (from Figure 5.4.8) was aligned with the Surface Runoff Peak in the design hydrograph (Figure 5.4.11), with the rest of the baseflow hydrograph used to guide the behaviour through the duration of the design event.

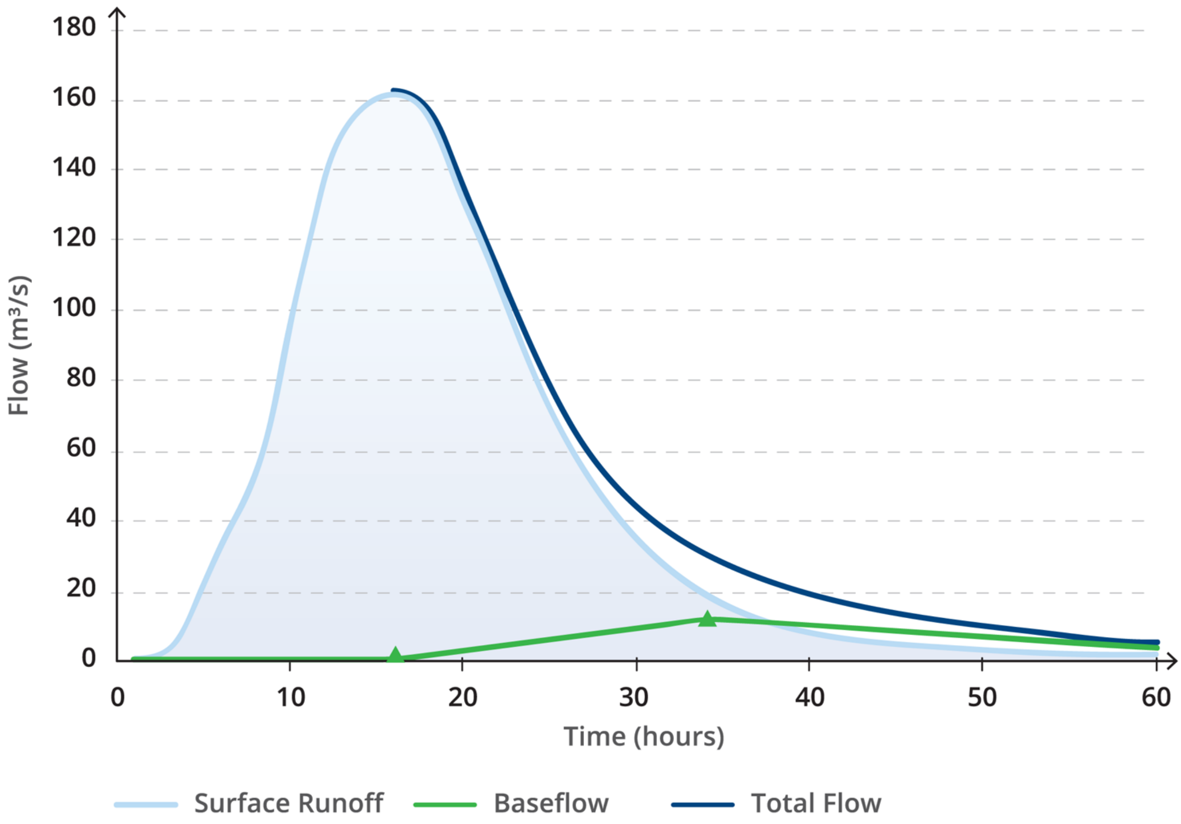


Figure 5.4.10. Surface Runoff, Baseflow and Total Streamflow Hydrographs for the 10% AEP Event at Eumundi

4.6.2. Dirk Brook, Western Australia

Catchment 2 is located in south-west Western Australia. The 1% AEP event is of interest. The Surface Runoff Hydrograph for this event was generated using a flood event model with a critical duration of 18 hours (Figure 5.4.11, and details in Table 5.4.3). This case study assumes that no streamflow data is available for use. The process described in Book 5, Chapter 4, Section 5 has been used to estimate baseflow.

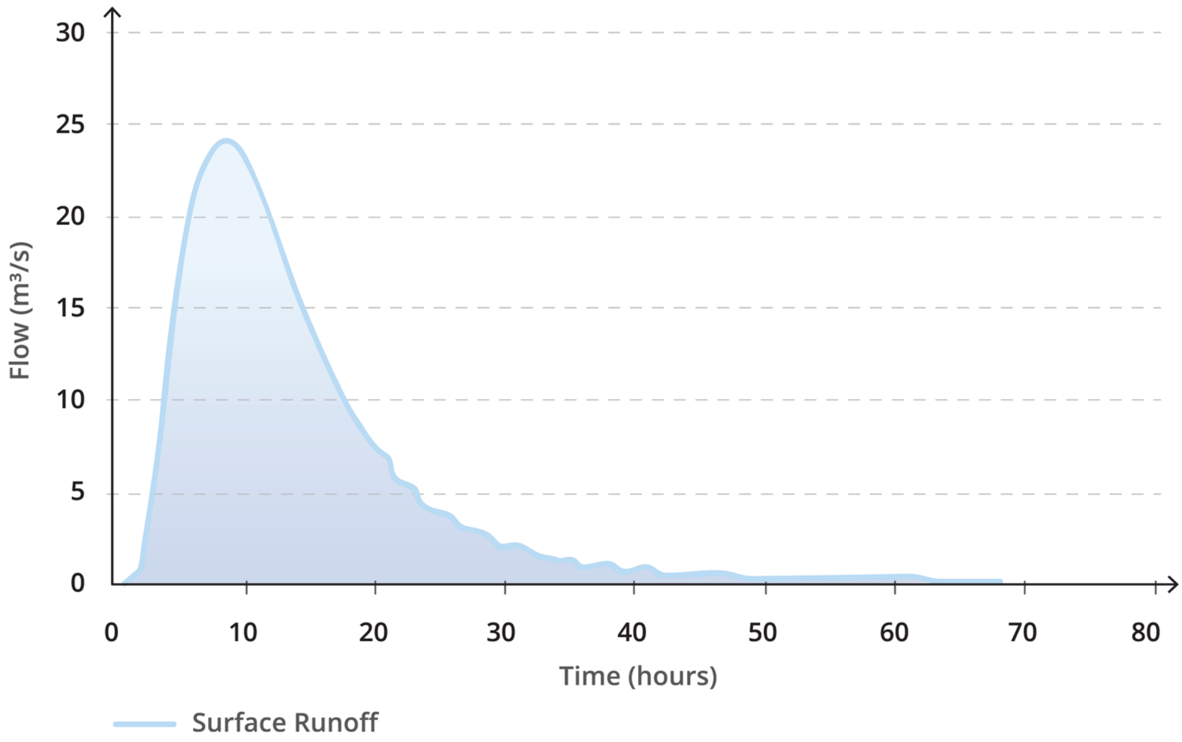


Figure 5.4.11. Surface Runoff Hydrograph for the 1% AEP Design Flood Event at Dirk Brook

Table 5.4.3. Key Surface Runoff Characteristics for the 1% AEP Design Flood Event at Dirk Brook

Characteristic	Data from design flood event
Surface runoff peak flow (m^3/s)	23.9
Time to the surface runoff peak (hours, from the start of the event)	8
Volume of surface runoff for the event (m^3)	1.25×10^6

Aligning the catchment boundary shape file with the spatial data from Figure 5.4.5 allows the Baseflow Peak Factor and Baseflow Volume Factor for the 10% event to be extracted for the catchment area directly:

$R_{BPF\ 10\ \%\ AEP} = 0.186$

$R_{BVF\ 10\ \%\ AEP} = 1.099$

The scaling factor for the 1% AEP event is sourced from Table 5.4.1, with a value of 0.6 for both the peak and volume calculations. Using the relationships described earlier, the final Baseflow Peak Factor, Baseflow Volume Factor and Baseflow Under Peak Factor for application are outlined in Table 5.4.4. These values are applied to calculate the baseflow and total streamflow characteristics in Table 5.4.5, and plotted in Figure 5.4.12.

Table 5.4.4. Calculation of Baseflow Factors for the 1% AEP Design Event for the Dirk Brook

Factors for application	Factor values for 1% AEP design event
Final Baseflow Peak Factor	= 0.6 x 0.186

Factors for application	Factor values for 1% AEP design event
$R_{BPF} = F_{AEP}R_{BPF,\ 10\ \%\ AEP}$	= 0.11
Final Baseflow Volume Factor $R_{BVF} = F_{AEP}R_{BVF,\ 10\ \%\ AEP}$	= 0.6 x 1.099 = 0.66
Final Baseflow Under Peak Factor $R_{BUBF} = 0.7 \times R_{BPF}$	= 0.7 x 0.112 = 0.08

Table 5.4.5. Calculation of Baseflow and Total Streamflow Characteristics for the 1% AEP Event for the Dirk Brook Catchment

Baseflow and total streamflow characteristics	Factor values for 1% AEP
Peak Baseflow	
Peak Baseflow Equation (5.4.1) $Q_{Peak\ baseflow} = R_{BPF}\ Q_{Peak\ surface\ runoff}$	= 0.11 x 23.9 = 2.6 m^3/s
Time to Peak Baseflow Equation (5.4.2) $T_{Peak\ baseflow} = 0.92T_{Peak\ surface\ runoff} + 33.4$	= (0.92 x 8) +33.4 = 41 hours
Baseflow Under the Peak	
Baseflow Under the Streamflow Peak Equation (5.4.4) $Q_{Baseflow\ under\ peak\ streamflow} = R_{BUPF}\ Q_{Peak\ surface\ runoff}$	= 0.08 x 23.9 = 1.9 m^3/s
Total Streamflow Peak	
Total Streamflow Peak Flow Equation (5.4.5) $Q_{Peak\ streamflow} = Q_{Peak\ surface\ runoff} + Q_{Baseflow\ under\ peak\ streamflow}$	= 23.9 + 1.9 = 25.8 m^3/s
Baseflow Volume	
Baseflow Volume Equation (5.4.6) $V_{Baseflow} = R_{BVF}\ V_{Surface\ runoff}$	= 0.66 x 1.25 x 10^6 = 0.83 x 10^6
Total Streamflow Volume	
Total Streamflow Volume Equation (5.4.7) $V_{Total\ streamflow} = V_{Surface\ runoff} + V_{Baseflow}$	= 1.25 x 10^6 + 0.83 x 10^6 = 2.08 x 10^6

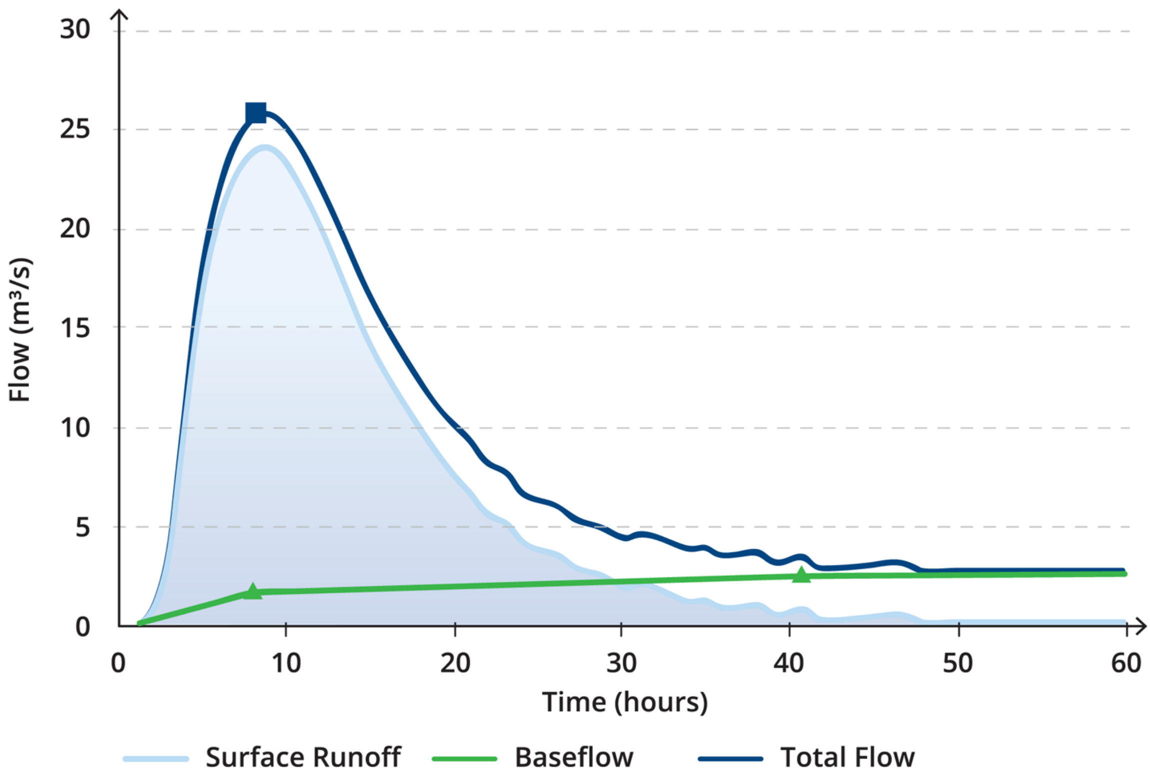


Figure 5.4.12. Surface Runoff, Baseflow and Total Streamflow Hydrographs for the 1% AEP Event at Dirk Brook

4.7. Appendix - Calculation of the Timing of the Baseflow Peak

Analyses undertaken to develop most of the method outlined in Book 5, Chapter 4, Section 5 is described in detail in separate technical documents, available online at the ARR website (Murphy et al., 2010). However, the background behind the calculation of the timing of the baseflow peak is not captured in those documents. A full description of the development of Equation (5.4.2) is provided below. The description below assumes that the reader has an understanding of the work presented in the separate technical documents, and a full background of the broader study concepts is not provided here.

More than 230 suitable catchments across Australia were identified for analysis for ARR Revision Project 7, and hourly streamflow data was collated for each of these locations. A baseflow time series was generated from each flow record using the Lyne and Hollick digital filter, modified to suit hourly streamflow data. The top 4N flood events were identified in the hourly time series data for each catchment, generating a data set of more than 30,000 flood events across the 236 catchments. For each of these events, the magnitude and timing of the total streamflow peak and baseflow peak were identified. The time to these peaks was calculated from the start of the event. At each location, the average time to the streamflow and baseflow peaks were then calculated based on the 4N events.

For the purposes of this assessment, it was assumed that the total streamflow and surface runoff peaks would coincide. This is considered a reasonable assumption since surface runoff is generally the main component of the total streamflow hydrograph.

Analysis of the average time to peak data identified a strong relationship between the time to the surface runoff (and streamflow) and baseflow peaks, as presented in Figure 5.4.13. This relationship provides a direct calculation from which to estimate the timing of the baseflow peak, based upon knowledge of the surface runoff event generated using a flood event model. That is, the time to the baseflow peak (in hours from the start of the event) can be calculated as:

$$T_{\text{Peak baseflow}} = 0.92\ \mathrm{T}_{\text{Peak surface runoff}} + 33.4 \qquad (5.4.8)$$

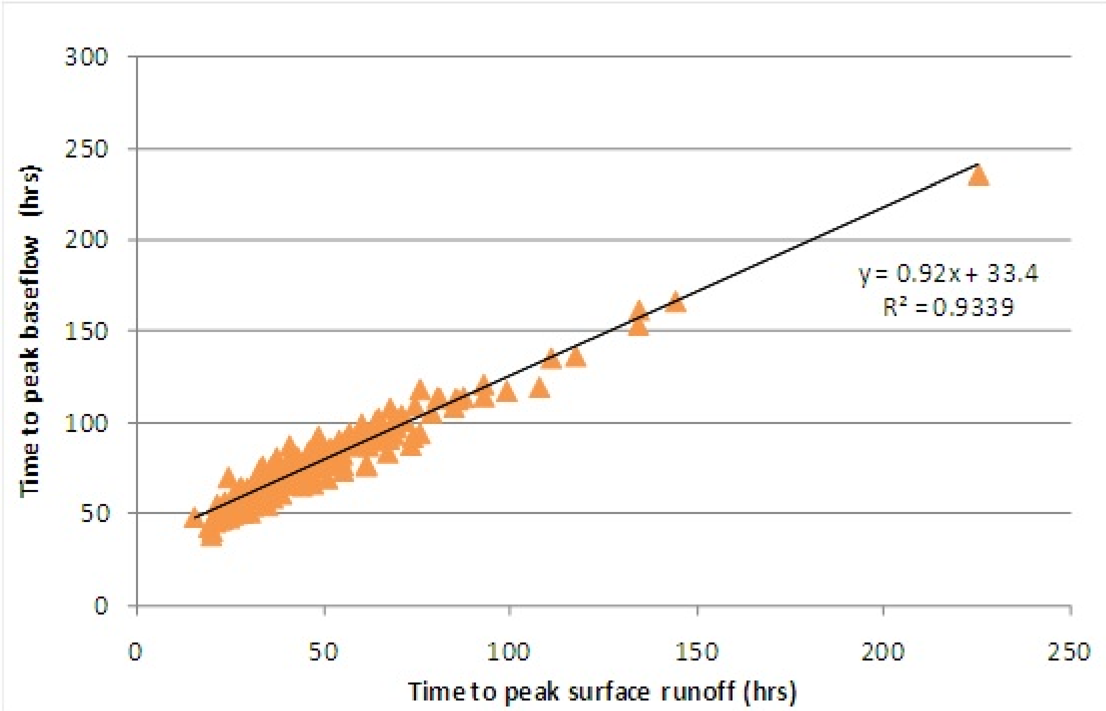


Figure 5.4.13. Comparison of Average Time to Surface Runoff Peak and Time to Baseflow Peak, Based on Analysis of more than 30,000 Flood Events from Catchments across Australia.

4.8. References

Babister, M., Trim, A., Testoni, I. and Retallick, M. 2016. The Australian Rainfall and Runoff Datahub, 37th Hydrology and Water Resources Symposium Queenstown NZ

Brodie, R.S. and Hostetler, S. (2005), A review of techniques for analysing baseflow from stream hydrographs, Proceedings of the NZHS-IAH-NZSSS 2005 conference, 28 November - 2 December, Auckland, New Zealand.

Chapman, T.G. and Maxwell, A.I. (1996), Baseflow separation - Comparison of numerical methods with tracer experiments, Proceedings of the 23rd Hydrology and Water Resources Symposium, 21-24 May, Hobart, Australia.

Graszkiewicz, Z., Murphy, R.E., Hill, P.I., Nathan, R.J. (2011), Review of techniques for estimating the contribution of baseflow to flood hydrographs, Proceedings of the 34th IAHR World Congress, 26 June - 1 July Brisbane, Australia.

Kinkela, K., Pearce, L.J. (2014), Assessment of baseflow seasonality and application to design flood events in southwest Western Australia, Australian Journal of Water Resources, 18(1), 27-38.

Ladson, A.R., Brown, R., Neal, B. and Nathan, R. (2013), A standard approach to baseflow separation using the Lyne and Hollick filter. Aus J Water Resour., 17(1), 25-33.

Murphy, R., Graszkiewicz, Z., Hill, P., Neal, B., Nathan, R., and Ladson, T. (2009), Project 7: Baseflow for catchment simulation, Phase 1 - selection of baseflow separation approach. Report prepared for the Australian Rainfall and Runoff Technical Committee by Sinclair Knight Merz, AR&R Report Number P7/S1/004, ISBN 978-085825-9218.

Murphy, R., Graszkiewicz, Z., Hill, P. and Nathan R. (2010), Project 7: Baseflow for catchment simulation, Data collection and catchment characteristics, Report prepared for Engineers Australia. Avaliable at http://arr.ga.gov.au/downloads-and-software/revision-project-reports

Murphy, R.E., Graszkiewicz, Z., Hill, P.I., Neal, B.P., Nathan, R.J. (2011a), Predicting baseflow contributions to design flood events in Australia, Proceedings of the 34th IAHR World Congress, 26 June - 1 July , Brisbane, Australia.

Murphy, R., Graszkiewicz, Z., Hill, P., Neal, B., and Nathan, R. (2011b), Project 7: Baseflow for catchment simulation, Phase 2 - development of baseflow estimation approach. Report prepared for the Australian Rainfall and Runoff Technical Committee by Sinclair Knight Merz, ARR Report Number P7/S2/017, ISBN 978-0-85825-916-4.

Nathan, R.J. and McMahon, T.A. (1990), Evaluation of automated techniques for base flow and recession analyses, Water Resources Research, 26(7), 1465-1473.

Chapter 5. Flood Routing Principles

James Ball, Erwin Weinmann, Michael Boyd

Chapter Status	Final
Date last updated	14/5/2019

5.1. Introduction

This chapter deals with the modelling of how the direct runoff and baseflow contributions from different parts of the catchment (derived from the models discussed in Book 5, Chapter 3 and Book 5, Chapter 4) are combined and modified on their movement through the catchment to form a hydrograph at points of interest, both at the catchment outlet and inside the catchment. In Book 5, Chapter 5, Section 2 a number of fundamental concepts relevant to flood routing are introduced. Book 5, Chapter 5, Section 3 then deals with the hydrologic principles and methods of storage routing applied in the most widely used flood hydrograph estimation models. In Book 5, Chapter 5, Section 4 the storage routing principles are expanded from linear to non-linear models. Finally, Book 5, Chapter 5, Section 5 introduces a range of hydraulic flood routing approaches that are based on various forms of the unsteady flow equations.

The focus of the descriptions of flood routing approaches and methods in this chapter is on explaining the background, merits and limitations of the different methods employed in flood hydrograph estimation models. The details of how these flood routing principles and methods are applied in flood hydrograph estimation models are covered in Book 5, Chapter 6. For more details on hydraulic analysis and modelling approaches refer to Book 7.

This chapter focuses on rural catchments. Similar routing approaches also apply for urban catchments, but specific issues in urban hydrology are described in detail in Book 9.

5.2. Fundamental Concepts

The runoff inputs generated by various processes in different subareas or sub-catchments are gradually transformed into a combined flood hydrograph at a downstream location. This process is determined principally by various forms of *temporary flood storage* available in the catchment as well as by transmission losses along the flow route. The different elements of a catchment where temporary flood storage occurs include:

- Catchment surfaces (overland flow segments);
- Stream channels;
- Stream banks;
- Floodplains; and
- Drainage channels (or pipes).

These forms of storage are *distributed* in nature – the storage is spread along these catchment elements. In flood hydrograph estimation modelling the different forms of storage do not need to be represented separately but can be modelled as *combined (conceptual) storage elements*.

In addition to the distributed forms of storage, a catchment may also contain lakes, reservoirs or flood detention basins where the storage occurs in a more concentrated form and is represented in models by *concentrated storage elements*. For these concentrated storage elements a more direct relationship exists between inflow and outflow than for distributed forms of storage, as is explained further in Book 5, Chapter 5, Section 3. It is also possible to use concentrated storage elements as a simplified representation of distributed forms of storage (Book 5, Chapter 5, Section 4).

The effects of the different forms of catchment storage on the transformation of flow inputs are twofold (Figure 5.5.1):

i. *Translation* of the hydrograph peak and other ordinates forward in time or, expressed differently, delaying the arrival of the hydrograph peak at a downstream location; and

ii. *Attenuation* or flattening of the hydrograph as it moves along the stream network; this results in a reduction of the peak flow but also in diffusion (spreading out) of the hydrograph, thus extending its duration.

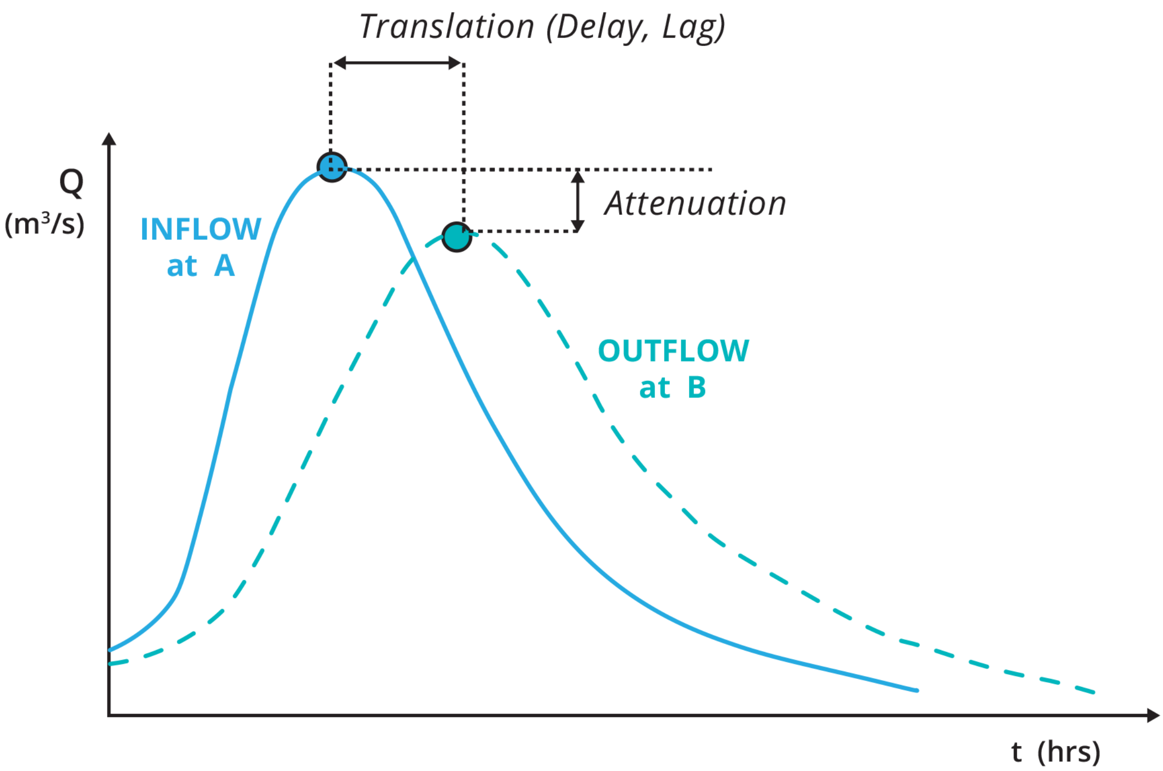


Figure 5.5.1. Effects of Storage on Transforming Inflow Hydrograph

The effects of storage can be modelled through the formulation of the continuity equation for a specific catchment element and over a time interval Δt:

$$I_v = O_v + \Delta S \tag{5.5.1}$$

where I_v is the volume of inflow to the catchment element, O_v is the volume of outflow from the element, and ΔS is the change in the storage during the time interval. The inflow volume (I_v) may represent runoff and baseflow inputs or outflow from an upstream element. While ΔS is positive, the inflow volume to the element is greater than the outflow volume and therefore the storage within the element will increase. Conversely, when ΔS is negative, the outflow volume is greater than the inflow volume and the storage in the element will decrease.

Due to the principle of mass conservation, the total volumes of inflow to and outflow from the catchment element must be equal. In many situations and particularly in the arid and semi-arid regions of the country, flow in a channel may infiltrate into the banks or bed of the channel; in other words, transmission losses will occur. In these situations, the principle of mass conservation remains, with the volume of the inflow being equal to the volume of the outflow hydrograph plus the volume of the transmission loss.

The application of the continuity equation in the form above refers to *temporary storage* or *detention storage* in different catchment elements. In contrast to this form of storage, where all water is released again during the flood event, there may also be catchment elements with *retention storage* (e.g. reservoirs with a flood storage compartment), where water is retained more permanently and released from the storage mostly after the flood event by controlled releases or, more gradually, through evapotranspiration and seepage losses.

These fundamental flood routing concepts form the basis of the runoff-routing approaches to flood hydrograph estimation discussed in Book 5, Chapter 6, Section 4. Different flood hydrograph estimation modelling systems use flood routing approaches of different complexity, with correspondingly different data requirements. The following Book 5, Chapter 5, Section 3 to Book 5, Chapter 5, Section 5 explain in more detail the theoretical basis and practical application of these different flood routing approaches.

5.3. Hydrograph Translation (Lag)

The simplest method for routing a hydrograph through a channel, pipe, stream or floodplain element is to simply translate all ordinates by a fixed travel time or lag. This method of routing produces pure translation without any attenuation of the hydrograph peak. It is useful for flood routing in systems with little storage (e.g. piped drainage systems) or in situations where the timing of the hydrograph peak is of principal interest (e.g. flood warning systems).

In piped drainage systems the travel time through a pipe segment can be directly determined from the flow velocity through the pipe.

In channels or natural streams the travel time T of a flood hydrograph through a routing reach of length Δx is related to the kinematic wave speed c_k:

$$T = \frac{\Delta x}{c_k} \tag{5.5.2}$$

The travel time or lag is thus directly proportional to the length of the channel reach. For a wide rectangular channel and constant Manning's n, the kinematic wave speed can be approximated as 1.67 times the average flow velocity through the routing reach.

For practical flood routing applications, estimates of the lag time are generally based on systematic analysis of observed flood peak travel times and their variation with flood magnitude. Wong and Laurenson (1983) have examined the variation of the wave speed (reach length divided by travel time of flood peak) with flow magnitude in a number of Australian river reaches. They found that for in-bank flow the wave speed typically increases with flow magnitude but reaches a maximum before bank-full flow and then reduces rapidly, most likely because the effects of bank vegetation become more pronounced. With fully developed floodplain flow the wave speed increases again. This means that travel time estimates from smaller floods cannot be directly applied to estimate travel times for larger floods and vice versa.

A variation of the simple hydrograph translation approach that also takes into account attenuation effects is the 'Lag and Route' method in which the hydrograph is first translated by the appropriate lag time and then routed through a concentrated linear storage (Fread, 1985).

5.4. Storage Routing

Storage routing methods have been developed as a convenient form of hydrologic routing, to track the movement of a flood wave on its way through a catchment system and to assess the effects of storage on the transformation of an inflow hydrograph to an outflow hydrograph. Storage routing is a lumped approach – it considers only the inputs (inflows) and outputs (outflows) of the system without considering what is happening within the system. Different applications of storage routing principles focus on different types of systems with different forms of storage, e.g. level pool routing methods (concentrated storage as in reservoirs) and river routing methods, including different forms of the Muskingum Method (distributed storage).

Following some basic background on storage routing principles introduced in Section 5.4.1, the main methods in practical use are described in Section 5.4.2 to Section 5.4.4. All the storage routing methods described in these sections are based on a linear relationship between storage and discharge. Some important limitations of the storage routing methods are discussed in Book 5, Chapter 5, Section 4. The non-linear storage routing methods described in Book 5, Chapter 5, Section 5 can overcome some of these limitations.

5.4.1. Basic Equation

The storage routing methods are based on the Conservation of Mass principle which is reflected in the *continuity equation*, expressed as:

$$I - O = \frac{dS}{dt} \quad (5.5.3)$$

Where I and O respectively are the average rates of inflow and outflow and dS is the change in storage during the time interval dt. Multiplication of Equation (5.5.3) by the time interval dt yields the continuity equation expressed in terms of volumes:

$$\text{INFLOW - OUTFLOW} = \text{CHANGE OF STORAGE} \quad (5.5.4)$$

It is important to note that only the change in storage is considered, rather than the total storage volume; this means that the datum used for the determination of storage volumes is not important as it does not influence the routing calculations.

Storage routing methods do not use a momentum equation (see Book 6, Chapter 2, Section 8) but can reflect the conservation of momentum (dynamic effects) through appropriate selection of their parameters (Koussis, 2009).

5.4.2. Reservoir (Level Pool) Routing

5.4.2.1. Traditional Methods

The category of storage routing approaches commonly referred to as reservoir routing or level pool routing is suitable for systems where storage and outflow are related by a unique invariant function (ie. a function not subject to hysteresis). These relationships imply that for a given stage (water surface elevation) the outflow is unique and independent of how that stage is developed. Reservoirs or systems with horizontal water surfaces have relationships of this type. For these concentrated storage systems, the peak outflow from the reservoir occurs when the outflow hydrograph intersects the recession limb of the inflow hydrograph, as illustrated in Figure 5.5.2.

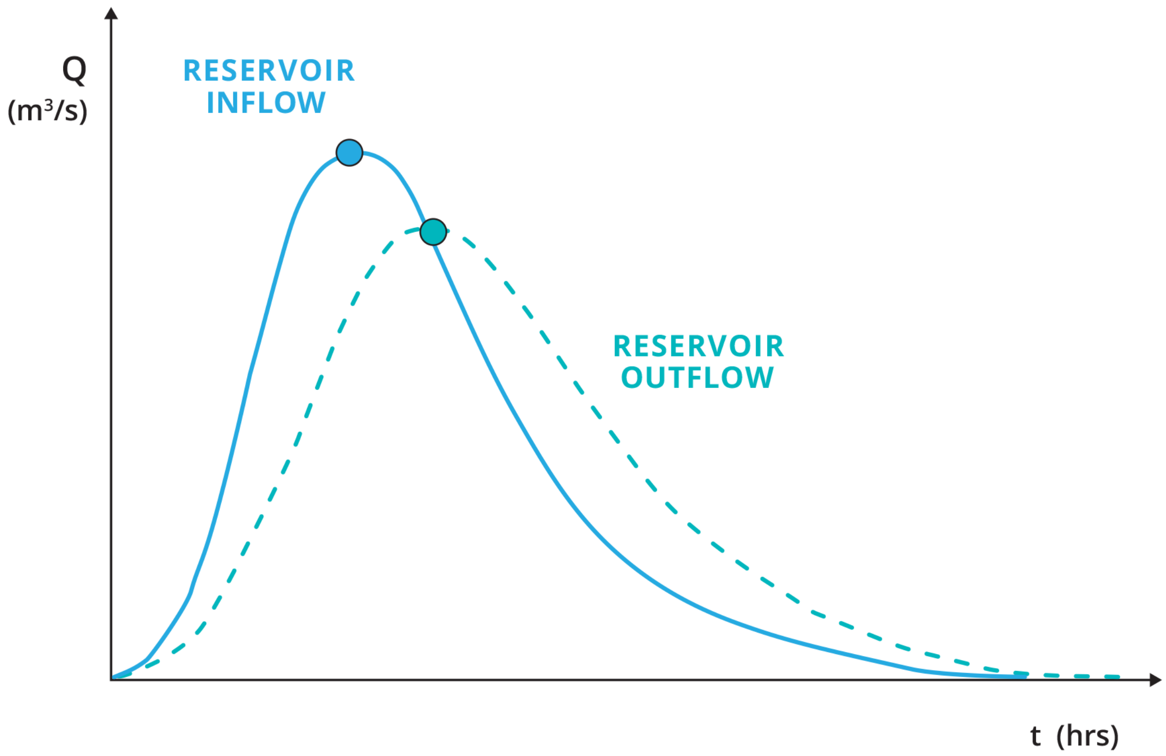


Figure 5.5.2. Effects of Reservoir on Transforming Inflow Hydrograph

The suitability of the assumption of a horizontal water surface during a flood event should be considered when level pool storage routing techniques are applied. If backwater effects create a 'wedge storage' effect (similar to the wedge storage discussed in Book 5, Chapter 5, Section 4) then it might be necessary to develop a storage-discharge relationship for the reservoir where storage depends not only on outflow but also on inflow.

Using a finite difference approximation, Equation (5.5.3) can be written as:

$$\frac{1}{2}(I_1 + I_2)\Delta t - \frac{1}{2}(O_1 + O_2)\Delta t = S_2 - S_1 \qquad (5.5.5)$$

where Δt, is the time increment used for the calculations, and subscripts 1 and 2 refer to the start and end, respectively, of the time period being considered. All variables with the subscript 1 are known either from the initial conditions or from previous calculations. In addition, the inflow at the end of the time period (I_2) is known. Hence, only S_2 and O_2 (ie. the storage and the outflow at the end of the time period) are unknown.

The relationship between storage in a reservoir or detention pond and discharge from it through spillways and outlets is generally highly nonlinear. This means that Equation (5.5.5) cannot be solved analytically but requires a numerical solution method (or traditionally a graphical solution technique).

There are a large number of alternative numerical and graphical techniques for solving Equation (5.5.5); some of these alternatives are presented by Henderson (1966) and Bedient et al. (2008). Possibly, the most commonly used method is the Modified Puls method. The basis of this method is Equation (5.5.5) and the *storage indication curve* given by:

$$\frac{2S}{\Delta t} + Q \text{ vs. } Q \tag{5.5.6}$$

To use the *storage indication curve*, Equation (5.5.5) is rearranged to give:

$$\frac{2S_2}{\Delta t} + O_2 = (I_1 + I_2) + \left(\frac{2S_1}{\Delta t} - O_1\right) \tag{5.5.7}$$

In this equation, all of the known parameters are on the right hand side of the equation while all of the unknown parameters are on the left hand side of the equation. As the value of the right hand side of Equation (5.5.7) is known, Equation (5.5.6) can be used to determine values for $S_2$and O_2. Calculations then proceed to the next time step.

An alternative approach was presented by Henderson (1966) which has the advantage of being self-correcting; in other words, an error in estimating flows at one time period will not flow into subsequent time periods. The approach is based on using a variable N defined by:

$$N = \frac{S}{\Delta t} + \frac{O}{2} \tag{5.5.8}$$

Substituting this variable into Equation (5.5.5), after rearranging, results in:

$$N_2 = N_1 + \frac{1}{2}(I_1 - I_2) - O_1 \tag{5.5.9}$$

In this form, all the unknown variables in Equation (5.5.9) are located on the left-hand side of the equation and all the known variables are found on the right-hand side. Equation (5.5.9), therefore, can be solved incrementally for values of N_2 which, with Equation (5.5.8), enables prediction of the unknown outflow rate (O_2).

5.4.2.2. Computer-based Methods

Computer-based solution techniques for the non-linear storage routing equation (Equation (5.5.5)) employ a number of different numerical solution schemes. The first-order Euler scheme produces the following simple expression for reservoir routing (Fenton, 1992):

$$S_2 = S_1 + \Delta t(I_1 - O_1) \tag{5.5.10}$$

Where the outflow O is a well-defined function of the storage content S. This explicit numerical scheme is stable and accurate if the computational time step Δt is chosen sufficiently small (significantly smaller than the time steps used to define the inflow hydrograph).

If the storage-discharge relationship can be expressed as a power function (Equation (5.5.32)) or other function for which the first derivative can be readily determined, then the Newton-Raphson numerical method can be applied. Other numerical methods such as the Regula Falsi (False Position) method or Runge-Kutta methods are more widely applicable (e.g. Chapra and Canale (2010), Bedient et al. (2008)) give an example of the application of Runge-Kutta numerical solution approaches for detention basin routing.

The general purpose runoff routing modelling systems described in Book 5, Chapter 6, Section 4 incorporate options for routing through reservoirs and detention basins as special cases. Generally the routing routines applied allow for a range of different non-linear formulations of the storage-discharge relationship.

An alternative form of the governing equation for storage routing is given by Fenton (1992), based on expressing both storage content and outflow as a function of *h*, the water surface level in the reservoir (or the head above the spillway crest). The storage increment ΔS can be expressed as the product of the reservoir surface area *A* and level increment Δh. The outflow *O* is then also defined as a function of *h*, and in cases where the outflow depends on operational decisions (e.g. for gated spillways), also as a function of time. Numerical solution methods discussed by Fenton (1992) range from a first order approximation by Euler's method to second order and higher order Runge-Kutta methods.

5.4.2.3. Reverse Routing

At many smaller reservoirs there is no gauging of reservoir inflows but records of reservoir levels (and corresponding storage volumes) and outflows are available from the reservoir operation. In these situations inflow hydrographs can be derived by the process of reverse routing. Reverse reservoir routing is also based on the solution of Equation (5.5.5) but in this case for the unknown inflow I_2. However, most numerical solution schemes exhibit instabilities in the form of severe oscillations in the calculated inflow hydrograph (Boyd et al., 1989). These oscillations arise from relatively small variations in the measured reservoir level, which include random fluctuations due to the effects of wind, waves and measurement inaccuracies.

Boyd et al. (1989) and Zoppou (1999) showed that the following centred explicit finite difference scheme produces stable estimates of the inflow hydrograph without the need for any filtering or smoothing of the calculated hydrograph:

$$I(t) = Q(t) + \frac{S(t+\Delta t) - S(t-\Delta t)}{2\Delta t} \tag{5.5.11}$$

However, as demonstrated by (Boyd et al., 1989), some oscillations may still be introduced if the time step selected is too small, requiring the application of a simple smoothing algorithm to the calculated hydrograph ordinates.

5.4.3. Muskingum Hydrologic Routing

The Muskingum Method of routing flood waves along channels was developed by the US Army Corps of Engineers during a study of the Muskingum River Basin in Ohio, USA (McCarthy, 1938). The basis of the technique is the application of the continuity equation to a control volume and relating the storage within the control volume to the discharge from the control volume. Due to its simplicity, the Muskingum Method is a widely used flood routing technique and also forms the basis of the procedures used in many flood hydrograph estimation models for routing direct runoff to the catchment outlet.

Despite this apparent simplicity, the Muskingum Method, with appropriate selection of its parameters, can be shown to be equivalent to the solution of the convective diffusion equation, the simplest physically-grounded flood routing model (Koussis, 2009). There are many papers in the technical literature discussing its strengths and limitations, as well as proposed enhancements to the classical Muskingum Method. Among these is the classical paper by Cunge (1969) which led to the development of the now widely used Muskingum-Cunge flood routing procedure (Book 5, Chapter 5, Section 4).

5.4.3.1. Muskingum Storage-Discharge Relationship

For level pool flood routing, it is assumed that there is a unique relationship between the storage (*S*) at a given pool level and the discharge or outflow (*O*) from the pool. In contrast, for Muskingum routing, this is replaced by an assumption that the outflow from the control volume will depend on the water level at both the upstream and downstream ends of the control volume. It then follows that the storage within the control volume will depend on the inflow as well as the outflow, and there will be no unique relationship between storage and outflow.

Using this concept, it is common to subdivide the storage within the control volume into prism and wedge storage. These two conceptual storages are schematically illustrated in Figure 5.5.3. The prism storage is formed by a volume of constant cross-section along the length of the prismatic section which is dependent only on the outflow. Wedge storage is dependent on the difference between the inflow and the outflow from the control volume. During the rising limb of the hydrograph, inflow will exceed the outflow and the wedge storage will be positive. Similarly, during the falling limb of the hydrograph outflow will exceed the inflow and the wedge storage will be negative.

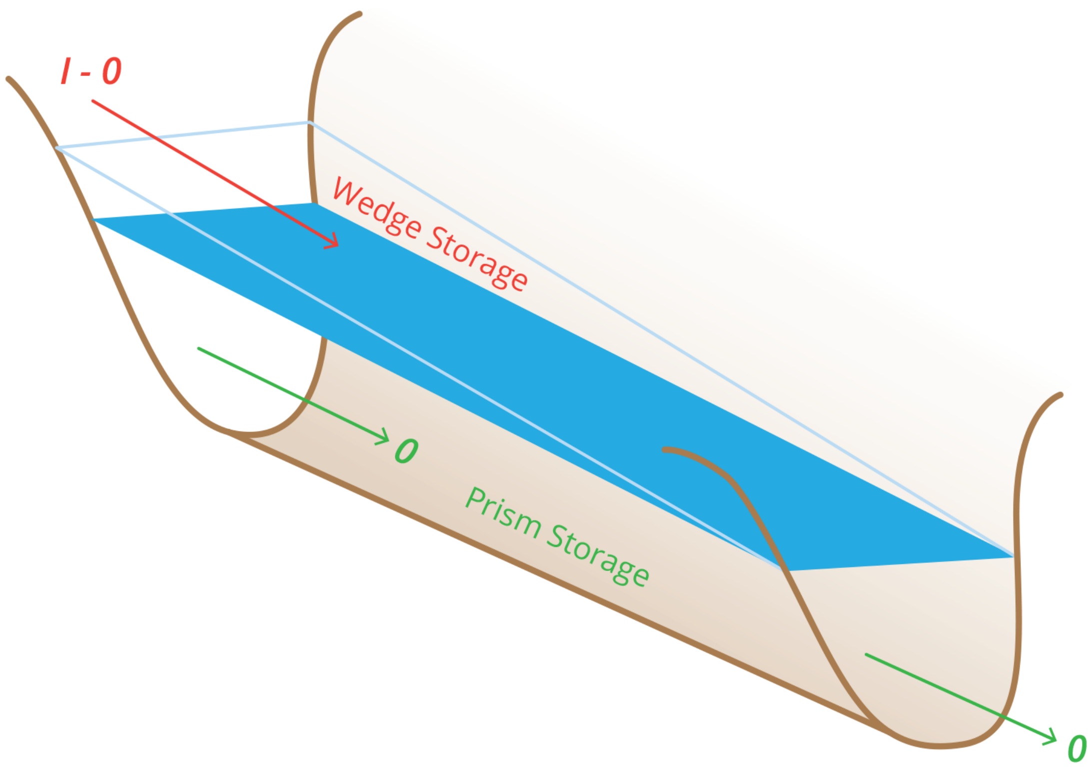


Figure 5.5.3. Prism Storage and Wedge Storage in a River Reach on the Rising Limb of the Hydrograph

The concepts of prism storage and wedge storage can also be applied to non-prismatic natural channels (rivers, streams and floodplains) with prism storage representing uniform flow conditions in the irregular channel.

Assuming a linear relationship between the storage and outflow from the control volume, the prism storage can be shown to be equal to *KO* while the wedge storage will be *KX(I-O)* where K is a proportionality coefficient and X is a weighting factor in the range $0 \leq X \leq 0.5$. The total storage, which is the sum of the two components, is then given by:

$$S = K[O + X(I - O)] \tag{5.5.12}$$

which can be rearranged to give:

$$S = K[XI + (1 - X)O] \tag{5.5.13}$$

This is the standard form of the storage-discharge relationship used with the Muskingum Method.

The two coefficients, X and K, can be related to physical characteristics of the routing element. The 'weighting coefficient' X depends on the shape of the wedge storage and K reflects the travel time of the flood wave through the routing element (given by the time lag between the centroids of the inflow and the outflow hydrographs). The value of X varies from 0 for a reservoir type storage to 0.5 for a full wedge (or fully distributed storage). When X is equal to 0, there is no wedge and, hence, the inflow has no influence on the storage volume; this being the implicit assumption made with level pool routing. In this case the Muskingum equation reduces to $S = KQ$, the storage-discharge (S-Q) relationship for a fully concentrated storage. In most natural streams, X is approximately 0.2 but can vary from 0 to 0.3. A value of X equal to 0.5 corresponds to fully distributed storage, where the hydrograph is translated with little attenuation. Great accuracy in determining the value of X is not necessary as the predicted hydrograph is relatively insensitive to the value of this parameter.

The storage coefficient (K) has dimensions of time and represents the average travel time of the flood wave through the reach; this time can be estimated by considering the centroids of the inflow and outflow hydrographs. The relationship of K with the physical characteristics of the routing element is further discussed in Book 5, Chapter 5, Section 4.

5.4.3.2. Muskingum Equation – Classical Coefficients

The development of the Muskingum Method is based on a finite difference approximation to the continuity equation (Equation (5.5.3)), ie:

$$\frac{I_n + I_{n+1}}{2} - \frac{O_n + O_{n+1}}{2} = \frac{S_{n+1} - S_n}{\Delta t} \tag{5.5.14}$$

together with Equation (5.5.11) expressed for time t_n and time t_{n+1} respectively as:

$$S_n = K\left[XI_n + (1 - X)O_n\right] \tag{5.5.15}$$

$$S_{n+1} = K\left[XI_{n+1} + (1 - X)O_{n+1}\right] \tag{5.5.16}$$

where Δt is the time increment between times t and n t_{n+1}. Subtracting Equation (5.5.13) from Equation (5.5.14) gives the change in storage over time Δt as:

$$S_{n+1} - S_n = K\left\{\left[XI_{n+1} + (1 - X)O_{n+1}\right] - \left[XI_n + (1 - X)O_n\right]\right\} \tag{5.5.17}$$

Combining Equation (5.5.15) with Equation (5.5.12) results in the routing equation for the Muskingum Method which usually is expressed as:

$$O_{n+1} = C_1 I_{n+1} + C_2 I_n + C_3 O_n \tag{5.5.18}$$

where the Muskingum coefficients (C_1, C_2 and C_3) are given by:

$$C_1 = \frac{\Delta t - 2KX}{2K(1-X) + \Delta t}$$
$$C_2 = \frac{\Delta t + 2KX}{2K(1-X) + \Delta t} \quad (5.5.19)$$
$$C_3 = \frac{2K(1-X) - \Delta t}{2K(1-X) + \Delta t}$$

It should be noted that summation of the Muskingum coefficients should give a value of unity ($C_1 + C_2 + C_3 = 1$). This provides an easy and a quick check that the coefficient values have been calculated correctly.

Example: Muskingum Flood Routing – Werribee River (after Laurenson (1998))

The outflow hydrograph from Melton Reservoir is to be routed through a 20 km reach of the Werribee River to Werribee Weir. As described in Book 5, Chapter 5, Section 4, analysis of an observed flood event has produced the following routing parameter estimates: K = 4.64 hours, X = 0.25. The routing calculations use a time step Δt = 2 hours. The Muskingum coefficients C_1, C_2 and C_3 are calculated from Equation (5.5.19) and are shown at the top of Table 5.5.1.

Table 5.5.1. Calculations for the Muskingum Routing Example

Time	Inflow	C_1	C_2	C_3	Outflow (m^3/s)	
(hours)	(m^3/s)	-0.036	0.482	0.554	Actual	Calculated
14:00	0				**0**	**0**
16:00	66	-2	0	0	**8**	**-2**
18:00	150	-5	32	-1	**34**	**25**
20:00	253	-9	72	14	**64**	**77**
22:00	325	-12	122	43	**147**	**153**
0:00	391	-14	157	85	**245**	**227**
2:00	420	-15	189	126	**310**	**299**
4:00	309	-11	203	166	**356**	**357**
6:00	247	-9	149	198	**330**	**338**
8:00	211	-8	119	187	**290**	**299**
10:00	166	-6	102	165	**245**	**261**
12:00	139	-5	80	145	**216**	**220**
14:00	88	-3	67	122	**185**	**185**
16:00	86	-3	42	103	**150**	**142**
18:00	82	-3	41	79	**122**	**117**
20:00	63	-2	40	65	**104**	**102**
22:00	55	-2	30	57	**96**	**85**
0:00	54	-2	27	47	**90**	**72**
2:00	52	-2	26	40	**76**	**64**
4:00	50	-2	25	35	**68**	**59**
6:00	49	-2	24	32	**62**	**55**

Time (hours)	Inflow (m^3/s)	C_1 -0.036	C_2 0.482	C_3 0.554	Outflow (m^3/s) Actual	Calculated
8:00	48	-2	24	30	**59**	**52**
10:00	47	-2	23	29	**57**	**50**
12:00	37	-1	23	28	**55**	**49**
14:00	36	-1	18	27	**53**	**44**
16:00	36	-1	17	24	**50**	**40**
18:00	36	-1	17	22	**42**	**38**
20:00	36	-1	17	21	**36**	**37**

Figure 5.5.4 shows the inflow hydrograph and the observed and calculated outflow hydrographs. The inflow hydrograph at Melton Reservoir (Column 1) has a peak flow of 420 m^3/s at 2.00 am of Day 2, while the calculated outflow peak at the end of the 20 km reach (Column 7) is 357 m^3/s occurring at 4.00 am on the same day. This is close to the actual (observed) peak flow of 356 m^3/s (Column 6).

The calculated hydrograph has a small ‘initial dip’ (negative flow values) at time 16.00 hour on Day 1. Such a dip results for values of X > 0 if the time step is shorter than the travel time through the reach (in other words, the outflow is calculated before the change in inflow has travelled through the reach).

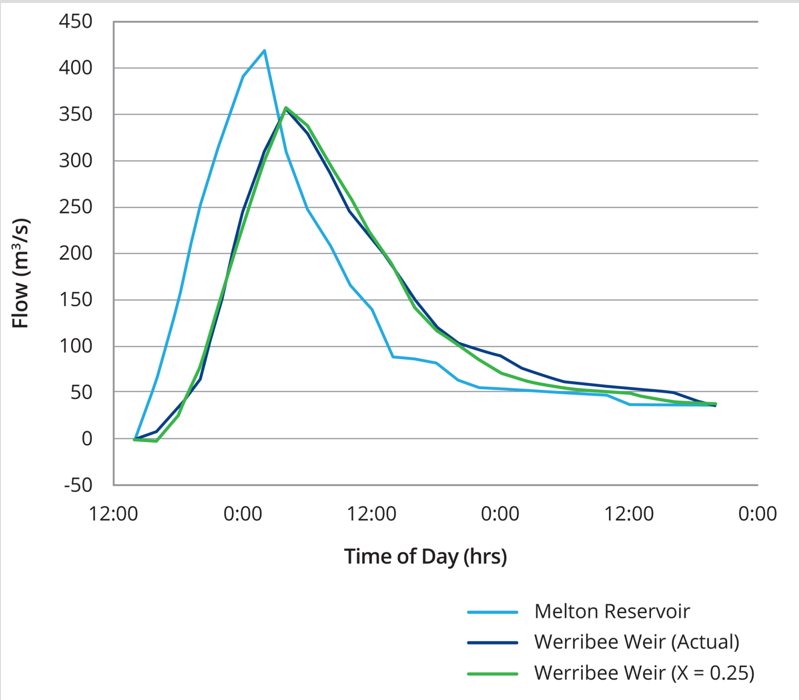


Figure 5.5.4. Werribee River Example – Inflow Hydrograph and Observed and Calculated Outflow Hydrographs

Figure 5.5.5 illustrates the impact of changing the routing parameter *X* from the optimum value of *X* = 0.25 to *X* = 0 (concentrated or reservoir-type storage) and *X* = 0.5 (fully distributed storage). It can be seen that a concentrated storage results in greater attenuation, with the peak of the outflow hydrograph on the falling limb of the inflow hydrograph, while fully distributed storage results in almost pure translation of the inflow hydrograph. With *X* = 0.5, there is a very noticeable initial dip in the outflow hydrograph.

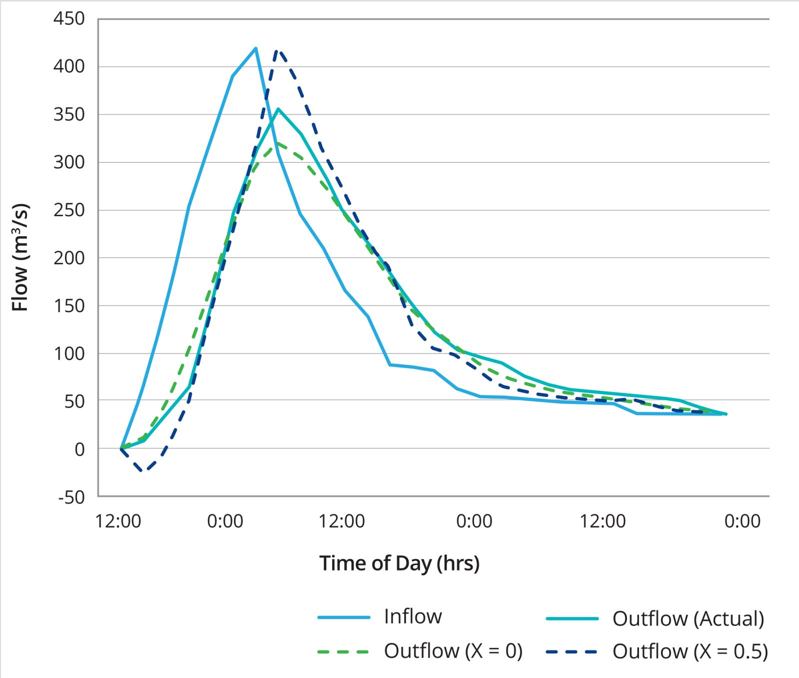


Figure 5.5.5. Werribee River Example – Impact of Routing Through Concentrated Storage (X = 0) or Fully Distributed Storage (X = 0.5)

5.4.3.3. Muskingum Equation – Nash Coefficients

An alternative development of the coefficients in the Muskingum routing equation was presented by Nash (1959). These alternative coefficients are:

$$\begin{aligned} C_1 &= 1 - \frac{K(1-c)}{\Delta t} \\ C_2 &= \frac{K(1-c)}{\Delta t} - c \\ C_3 &= c \\ c &= e^{\frac{-\Delta t}{K(1-X)}} \end{aligned} \tag{5.5.20}$$

Pilgrim (1987) suggests that these coefficients are more accurate than the classical coefficients. The basis of this suggestion is that the coefficients proposed by Nash (1959) do not require the ratio *Δt/K* to be small and, furthermore, the coefficients are not based on the finite difference approximation to the continuity equation (Equation (5.5.12)) but rather on the differential form of the continuity equation.

When *Δt/K* is small, the two alternative estimates of the routing coefficients should converge.

Given that current approaches to implementation of any flood routing approach are based on computer applications, the historical need for large *Δt* and hence large ratios of *Δt/K* due to the use of hand calculations is no longer relevant. Therefore, those applying Muskingum techniques within computerised applications should not notice any difference between use of the classical and Nash formulations of the coefficients, so long as an appropriately short time step is adopted for the simulations.

For the Werribee River example, replacing the standard Muskingum coefficients with Nash coefficients results in an outflow peak of 353 m^3/s, a difference of only about 1% from the original result.

5.4.3.4. Estimation of Muskingum Parameters X and K

When the Muskingum flood routing method is used, it is necessary to determine the values of two parameters; these are the parameters *K* and *X*. In general, estimation of the values for these coefficients requires recorded flood hydrograph information.

There are a number of methods by which the recorded flood information can be used to derive values for *K* and *X*. These vary from graphical approaches as outlined below to optimisation approaches as presented by Stephenson (1979) and Chang et al. (1983).

A classical graphical method is based on combining Equation (5.5.14) and Equation (5.5.17) which, after rearrangement, results in:

$$K = \frac{0.5\Delta t\left[\left(I_{n+1} + I_n\right) - \left(O_{n+1} + O_n\right)\right]}{X\left(I_{n+1} - I_n\right) + (1 - X)\left(O_{n+1} - O_n\right)} \qquad (5.5.21)$$

The numerator represents the change in storage during the time interval *Δt* and the denominator is the weighted discharge for a selected value of *X*. The computed values of the cumulative storage values are plotted against the weighted discharge for each time interval, with the usual result being a graph in the form of a loop. The value of *X* that produces a loop closest to a straight line is adopted as the value for *X*. The value for *K* is given by the slope of the line.

Figure 5.5.4 illustrates typical results obtained with this technique for the Werribee River example introduced in Book 5, Chapter 5, Section 4 (Laurenson, 1998). In this example a value of *X* = 0.25 produced the narrowest loop. The *K* value is computed as the slope of the fitted line:

$$K = \frac{1}{3600}\frac{5750000 - (-100000)}{350 - 0} = 4.64 hours \qquad (5.5.22)$$

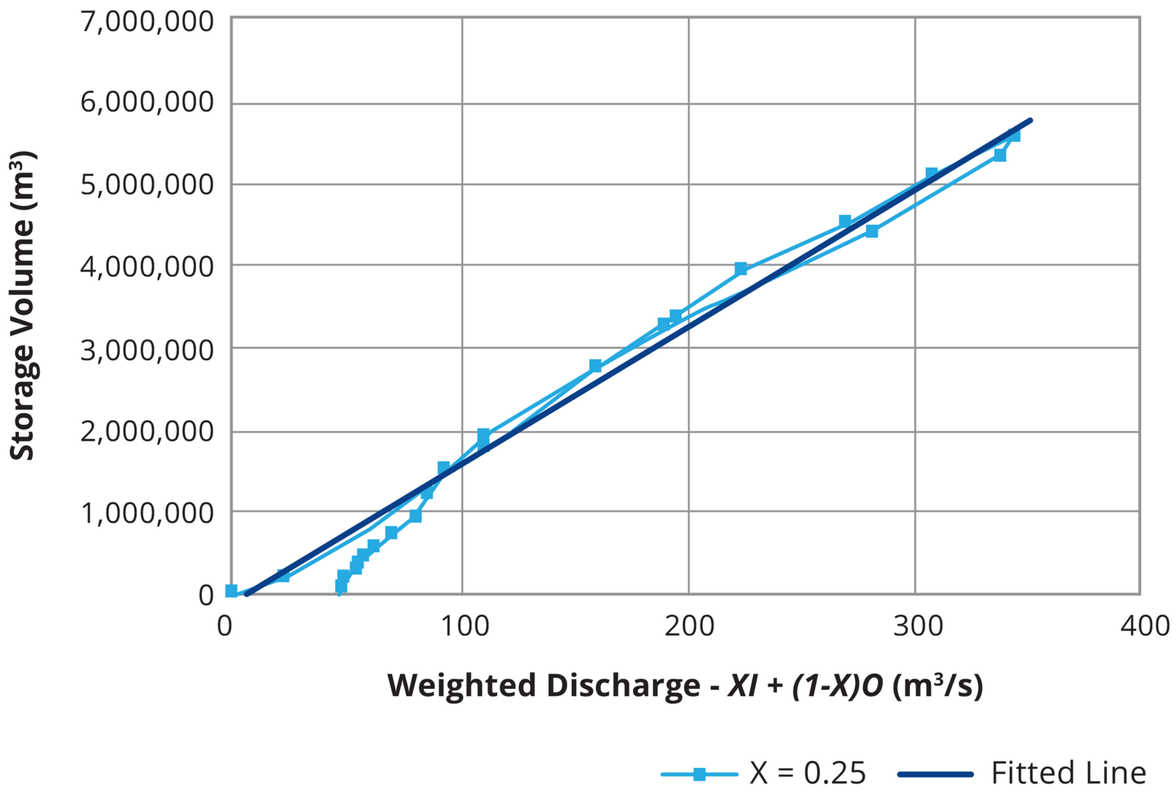


Figure 5.5.6. Graphical Estimation of X and K (after Laurenson (1998))

Some points to note with respect to the estimation of *X* and *K* are:

- Failure to collapse to a straight line indicates that the length of the reach being considered is too long;
- Inflow and outflow hydrograph peaks of similar magnitude indicates that *X* will be close to 0.5;
- A peak of the outflow hydrograph much smaller than the peak inflow indicates that *X* will be close to zero;
- An inconsistent slope of the line after evaluation of *X* indicates a change in the storage characteristics. This change may be due to, for example, inundation of the floodplains adjacent to the river channel. In these circumstances, the practitioner needs to use the slope of the line most appropriate for the problem being investigated to select the value of *K*; and
- As discussed in Book 5, Chapter 5, Section 3, flood travel times vary substantially with flood magnitude. If floods of different magnitudes are to be routed, the storage analysis needs to be carried out for a range of floods and the flood routing parameters varied accordingly.

In a discussion of determining the Muskingum coefficients, Chang et al. (1983) suggested the use of classical approaches based on the best fit between weighted storage and discharge need not result in optimal values of the routing coefficients; in other words, the classical approaches for determining the Muskingum coefficients may not result in values that minimise the error between observed and predicted hydrographs. The alternative to the classical approaches for determining the values of the two routing coefficients in the Muskingum method is the application of optimisation techniques.

Stephenson (1979) presents one such application where linear programming was used to minimise the difference, or error, between a predicted and recorded hydrograph for a recorded inflow hydrograph. The error function used in this application was the sum of the absolute values of the differences between the recorded and predicted hydrographs; values of the routing coefficients that minimised this error function were assumed to be the appropriate values for the coefficients. It was noted, however, that use of alternative error functions would result in different values for the routing coefficients.

5.4.3.5. Reverse Routing in River Reach

In some situations it may be desirable to determine a hydrograph at an upstream river location from an observed hydrograph at a downstream location. As shown by (Boyd et al., 1989) this can also be achieved by the solution of Equation (5.5.14) by a Muskingum-type numerical scheme, but to avoid numerical oscillations, the reverse routing calculations need to be carried out backward in time, starting at the end of the hydrograph. An equivalent to Equation (5.5.18) can then be written as:

$$I_{n-1} = \frac{1}{C_2}Q_n - \frac{C_s}{C_2}Q_{n-1} - \frac{C_1}{C_2}I_n \qquad (5.5.23)$$

5.4.4. Muskingum-Cunge Storage Routing

5.4.4.1. Introduction

The Muskingum-Cunge technique was developed from a discussion of the Muskingum Method by Cunge (1969). The basis of this discussion was an attempt to explain the apparent attenuation of a flood wave when the Muskingum technique is used to route a hydrograph through a river reach. The Muskingum technique assumes that there is a singular relationship between the storage and the discharge. This assumption leads to a differential equation whose analytical solution does not allow for wave damping (attenuation of the flood wave).

However, application of the Muskingum technique results in attenuation of the flood wave as it moves downstream. This contradiction between the analytical and the numerical applications required investigation. It is worthwhile noting that other methods, such as numerical solutions of the kinematic wave equation, also demonstrate similar characteristics, ie. application of the method results in attenuation of flood waves despite theoretical considerations indicating that no flood wave attenuation should occur.

Since its proposal by Cunge (1969), the Muskingum-Cunge technique has achieved widespread usage; for example, it is an option available in several flood hydrograph modelling systems for routing of flows along channels. One advantage of the Muskingum-Cunge technique is that its application does not require the use of historical flood events for estimation of the lag parameter or the weighting coefficient.

5.4.4.2. Derivation of Muskingum-Cunge Routing Scheme

As explained in Koussis (2009), Cunge showed in his seminal 1969 paper that the Muskingum flood routing scheme can be derived either from a second order approximation of the *convective diffusion* (*Equation (5.5.21)*) or by a particular discretisation of the *kinematic wave equation* – equation Equation (5.5.21) with the right hand side (the diffusion term) set to zero (the derivation of equation Equation (5.5.21) is further explained in Book 5, Chapter 6, Section 5):

$$\frac{\partial Q}{\partial t} + C_k \frac{\partial Q}{\partial x} = D \frac{\partial^2 Q}{\partial x^2} \quad (5.5.24)$$

with

$$C_k = \frac{\partial Q}{\partial A} \quad (5.5.25)$$

representing the kinematic wave celerity, and

$$D = \frac{Q}{2BS_o} \quad (5.5.26)$$

the diffusion coefficient.

where Q is the discharge, x the distance along the channel, A the cross-sectional area, B the water surface width and S_o the channel slope.

Through the diffusion term, the convective diffusion equation allows for the diffusive effects of the flood wave movement (attenuation of the flood peak) on its movement downstream. It should be noted that, through inclusion of the 'pressure term' from the complete momentum equation, the diffusion wave equation allows for backwater effects to be reflected in the flood routing. However, this feature is lost through the second order approximation in the Muskingum-Cunge method.

5.4.4.3. Muskingum-Cunge Coefficients

The Muskingum-Cunge technique uses the same coefficient equation as the classical Muskingum Method:

$$O_{n+1} = C_1 I_{n+1} + C_2 I_n + C_3 O_n \quad (5.5.27)$$

However, the direct link to the hydraulically-based convective diffusion or kinematic wave equation now allows the coefficients given by Equation (5.5.17) or Equation (5.5.18) to be determined using the hydraulic characteristics of the channel reach.

The Muskingum lag parameter K (which has the dimensions of time) is directly linked to the kinematic wave celerity C_k:

$$K = \frac{\Delta x}{C_k} \quad (5.5.28)$$

where Δx is the length of the routing reach and C_k is as defined by Equation (5.5.25). When Q is calculated from the Manning Equation, and the cross-sectional area, wetted perimeter and the roughness parameter are known functions of depth or stage, the derivative dQ/dA in Equation (5.5.25) can be evaluated. For a wide rectangular channel and Manning's n constant with changing flow depth, the kinematic wave celerity can be approximated as 1.67 times the average flow velocity through the routing reach.

To avoid confusion with the distance (x), the Muskingum weighting coefficient (X) is now labelled θ and evaluated as:

$$\theta = \frac{1}{2}\left(1 - \frac{Q}{C_k B S_o \Delta x}\right) \quad (5.5.29)$$

where Q is a representative discharge for the hydrograph being routed and the other terms are as defined before (corresponding to the same representative discharge).

The direct links of the Muskingum-Cunge coefficients to physical routing reach characteristics allows the application of the method in ungauged catchments and in situations where the routing characteristics are modified from those experienced during model calibration.

For the Werribee River example from Example: Muskingum Flood Routing – Werribee River (after Laurenson (1998)) the relevant routing reach characteristics are as follows:

Δx = 20 km

C_k = 1.2 m/s

Q = 210 m^3/s

B = 35 m and

S_0 = 0.0005.

Application of Equation (5.5.28) and Equation (5.5.29), respectively, gives values of K = 4.63 hours and

X = 0.25. As these values are almost identical to the values used in the original calculations, there is little difference in the results obtained with the Muskingum-Cunge method.

5.4.4.4. Representing Distributed Storage by a Series of Concentrated Storages

It has been shown by Kalinin and Miljukov (1958) and Laurenson (1962) that a similar representation of the translation and attenuation effects of distributed storage as in the Muskingum Method can be achieved by routing through a series of concentrated storages (ie. with the Muskingum weighting coefficient X = 0). However, for this method to be essentially equivalent to the Muskingum-Cunge Method, the length of the routing reaches represented by a concentrated storage (Δx) has to be selected in accordance with the following criterion (Weinmann, 1977; Wong , 1985):

$$\Delta x^* = \frac{Q}{S_o B C_k} \tag{5.5.30}$$

where Δx^* is the characteristic reach length proposed by Kalinin and Miljukov (1958). The optimum number of sub-reaches represented by a concentrated storage(N^*) can then be calculated as $L/\Delta x^*$, where L is the total length of channel to be routed through.

This means that the steeper the channel and the faster the flood wave travels for a given discharge per unit width, the shorter the routing reaches and thus the larger the number of routing reaches required. For very flat channels and slow moving flood waves, the number of sub-reaches required approaches one; the whole river reach can thus be expected to act like a concentrated storage. Using a number of storages less than N^* will tend to overestimate the degree of attenuation compared to translation, while using a larger number of storages will have the opposite effect.

Wong (1985) confirmed that using too few concentrated storages resulted in underestimation of both the peak flow and the travel time. Conversely, using a greater number of concentrated storages enhances the translation effects and increases the peak flow. However, beyond a certain number of storages the lag time does not increase any further but the peak flow will still increase.

These findings have implications for the application of runoff-routing models using a series of linear or nonlinear storages, as discussed in Book 5, Chapter 6, Section 4.

Werribee River Flood Routing Example – Kalinin-Miljukov Method

For this example the total routing reach of 20 km is divided into two sub-reaches of 10 km length, each being represented by a concentrated storage (X = 0). If the same wave speed of c_k = 1.2 m/s is used as for the Muskingum-Cunge Method, the routing parameter is calculated as $K = \Delta x / c_k$ = 2.31 hours.

Figure 5.5.7 shows the outflow hydrographs for the two sub reaches; it indicates that routing through a cascade of two linear storages (ie. the application of the Kalinin-Miljukov Method) results in slightly greater attenuation of the peak flow than obtained with the single reach Muskingum Method. The calculated peak flow is 338 m^3/s, which is 5% less than the actual observed flow at Werribee Weir.

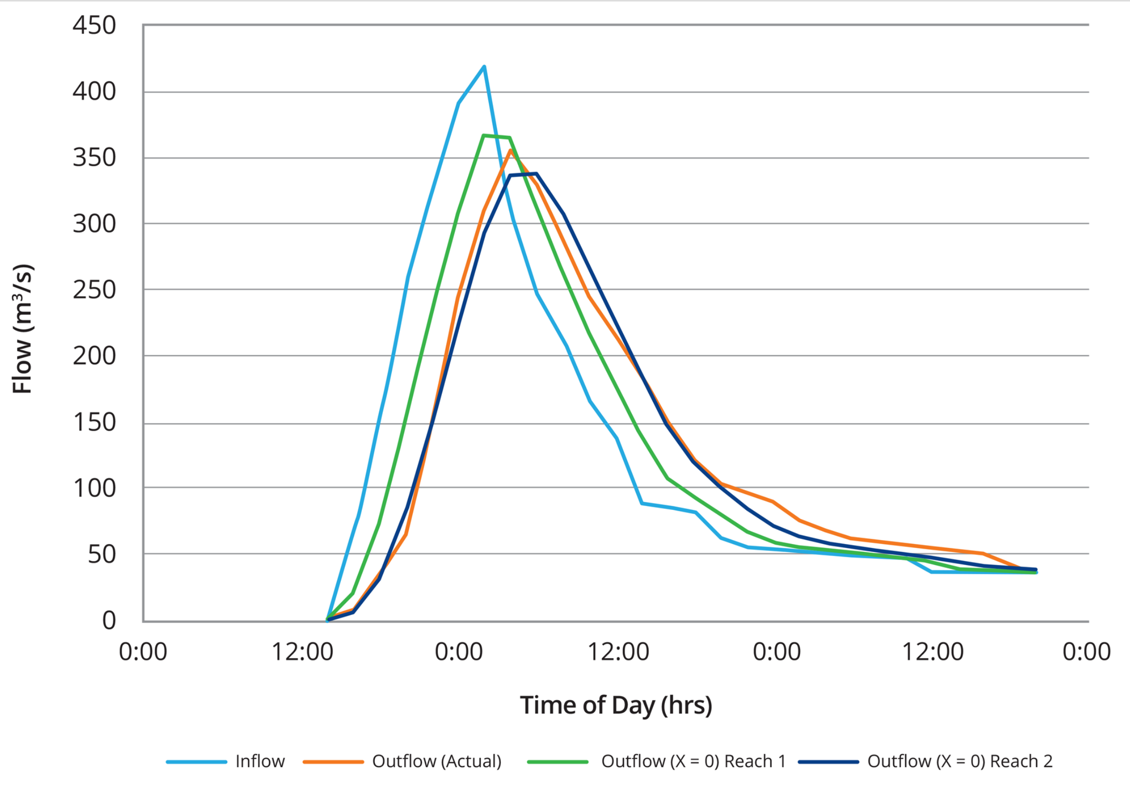


Figure 5.5.7. Werribee River Example – Application of the Kalinin-Miljukov Method (Two Routing Reaches of 10 km Length)

5.4.5. Limitations of the Muskingum Method

While the Muskingum Method of flood routing in its various forms has become very popular, due to its relative simplicity when compared to the more complete flood routing techniques, there are some limitations to its usage. Among these limitations are:

i. The assumption of a linear relationship between *S*, *I* and *O*. Although, in many instances, the actual relationships approximate a straight line when suitable values of *X* and *K* are selected, there is no physical or theoretical justification for this assumption. The linear assumption limits the degree of extrapolation to flood events similar in magnitude to those used in the calibration of the routing parameters (approaches to overcome this limitation are discussed in Book 5, Chapter 5, Section 5);

ii. For the classical Muskingum Method the evaluation of *X* and *K* requires the use of historic flood data and therefore is based on the channel geometry within the limited range of that flood data. Extrapolation for higher flood levels may require modification of the values for *X* and *K* to reflect any significant changes in channel characteristics. The Muskingum-Cunge Method can at least partly overcome this limitation;

iii. The need for the volume of the inflow hydrograph to equal the volume of the outflow hydrograph, which means that lateral inflows have to be added at either end of the routing reach;

iv. The method has an inherent problem in that it may produce physically unrealistic negative outflows (an ‘initial outflow dip’) when the inflow hydrograph rises steeply. This can be overcome by specifying a minimum routing time step Δt of $2KX$;

v. The inability of the method to consider downstream disturbances that propagate upstream (backwater effects). This places limitations on the application of the method in relatively flat stream reaches; and

vi. The limited ability to deal with fast rising hydrographs, due to the neglect of the acceleration terms in the momentum equation.

Limitations (ii) to (vi) also apply to the non-linear storage routing methods described in Book 5, Chapter 5, Section 5.

5.5. Non-linear Storage Routing

5.5.1. Introduction

All the methods discussed in Book 5, Chapter 5, Section 3 involved the assumption that the storage (*S*) in a routing element is related to the characteristic discharge (*Q*) in a linear fashion, in other words doubling the discharge corresponds to a doubling of storage. For concentrated forms storage (as in the level pool routing methods discussed in Book 5, Chapter 5, Section 4) the linear *S*-*Q* relationship is based on the outflow from the storage, while for the distributed forms of storage (as in the different forms of the Muskingum Method discussed in Book 5, Chapter 5, Section 4 to Book 5, Chapter 5, Section 4) the *S*-*Q* relationship uses a weighted average between inflow to and outflow from the routing reach.

The linear storage-discharge relationship can be expressed in general form as:

$$S = KQ \tag{5.5.31}$$

The constant coefficient K represents the *lag time* between inflow and outflow (or the average travel time through the routing element).

As shown in the Book 5, Chapter 5, Section 5, hydraulic analysis of various routing elements indicates that their S-Q relations are typically non-linear and that they can be approximated by a power function relationship of the following form:

$$S = kQ^m \tag{5.5.32}$$

where, k is a dimensionless coefficient and the exponent m is a dimensionless constant. Depending on the storage and discharge characteristics of the routing element, the exponent m can be smaller or greater than the value of 1.0 (which applies to the linear form of the S-Q relationship). The formulation in Equation (5.5.32) implies also a lag time K that varies with discharge:

$$K = \frac{S}{Q} = kQ^{m-1} \tag{5.5.33}$$

Non-linear storage routing methods require an iterative numerical procedure for their solution, such as the Regula Falsi (False Position) method or the Newton-Raphson method (e.g. Chapra and Canale (2010)). A numerical method for non-linear flood routing has been developed by Laurenson (1986) and is summarised in Pilgrim (1987).

5.5.2. Different Forms of Non-linearity

To examine different form of non-linearity in the S-Q relationship it is useful to express Equation (5.5.32) in logarithmic form:

$$\log S = \log k + m \log Q \tag{5.5.34}$$

This relationship plots as a straight line on log-log paper, and the exponent m represents the slope of the line. For any two points on the line:

$$m = \frac{\Delta \log S}{\Delta \log Q} \tag{5.5.35}$$

The exponent m can thus be interpreted as indicating the relative efficiency of storage and discharge with increasing water level (or increasing flood magnitude). Furthermore, Equation (5.5.33) indicates how the lag time changes for different values of m. Three different cases can be distinguished:

i. m = 1 (equivalent to the linear S-Q relationship) means that storage and discharge increase at a similar rate – the lag time remains constant;

ii. m < 1 represents relatively efficient flow and storage increasing slowly – the lag time decreases with increasing flood magnitude; and

iii. m > 1 indicates that flow is relatively inefficient and storage increases rapidly – the lag time increases with increasing flow.

An example of case (ii) is discharge and storage in a wide rectangular channel of Length L with water depth y (Mein et al., 1975):

$$S = yBL \tag{5.5.36}$$

$$Q = \frac{S_o^{\frac{1}{2}}}{n} B y^{\frac{5}{3}} \tag{5.5.37}$$

Substitution of y from Equation (5.5.37) into Equation (5.5.36) yields:

$$S = \frac{n^{0.6} B^{0.4} L}{S_o^{0.3}} Q^{0.6} \tag{5.5.38}$$

In this case the exponent *m* is 0.6 (efficient flow compared to storage) and the coefficient *k* is represented by the fraction before Q in Equation (5.5.38). A similar analysis for a triangular cross-section will yield an exponent *m* = 0.75.

In contrast to this, the analysis of storage and discharge for a rectangular channel being blocked by an embankment with a culvert, where discharge occurs as flow through an orifice of fixed size (inefficient flow), will yield a value of the exponent *m* substantially greater than 1.0.

Examples of *S*-*Q* curves with different values of the exponent *m* are illustrated in Figure 5.5.8, where values of S are plotted against Q on logarithmic scale axes. The different curves are plotted so that they cross at a representative discharge of about 30 m^3/s (representing the middle of the range of flood magnitudes used in model calibration).

It follows from the examples plotted in Figure 5.5.8 that different combinations of *k* and *m* can give similar values of storage for a given discharge. Calibration of a runoff-routing model over a limited range of flood magnitudes can thus only give a broad indication of the appropriate degree of non-linearity when the model is applied for the flow conditions of a different flood magnitude, and application in the extrapolated range needs to be guided by consideration of the physical characteristics of the routing reach.

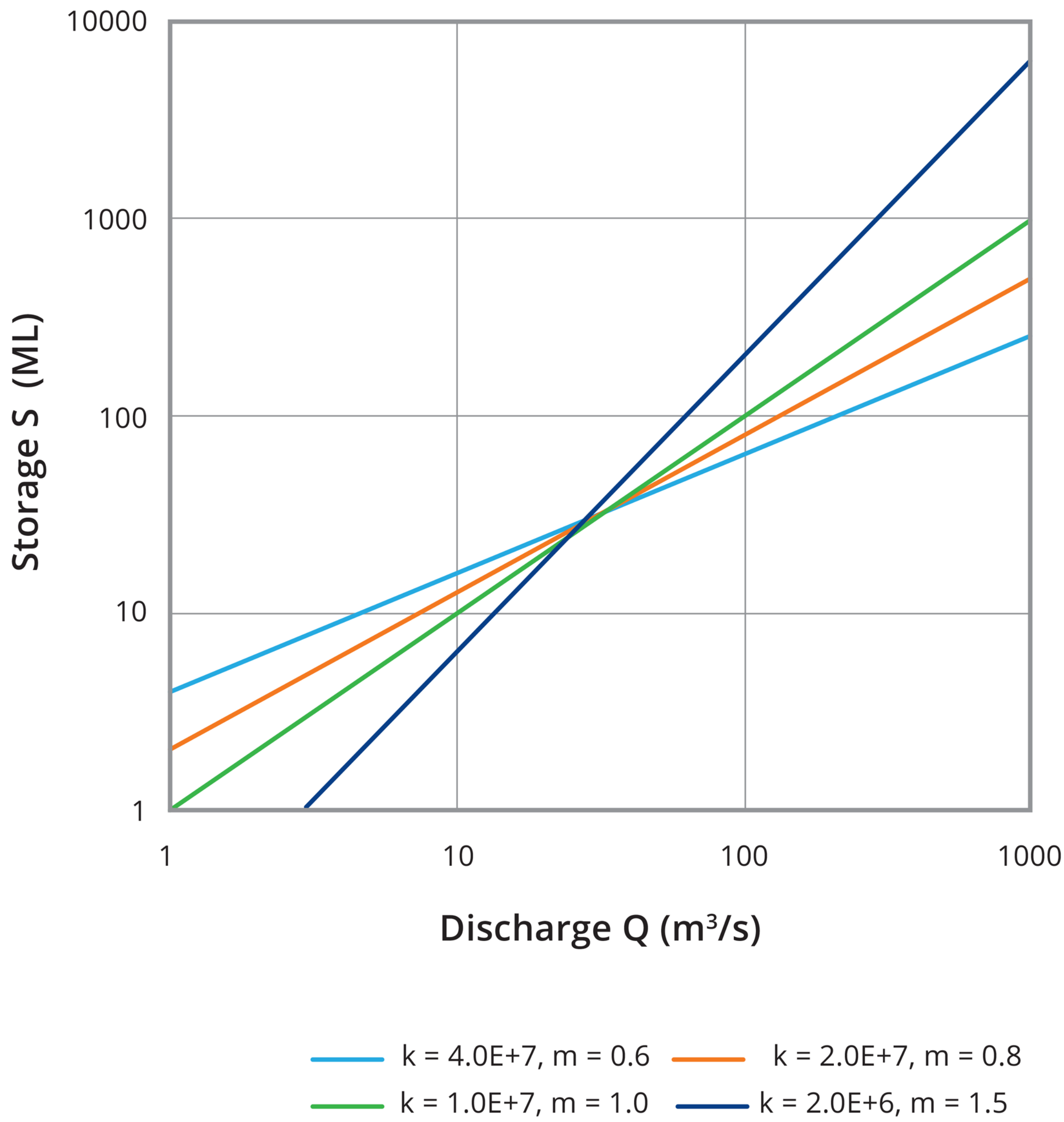


Figure 5.5.8. Storage-Discharge Relationships with Different Degrees of Non-linearity

The non-linear nature of catchment storage and values of the exponent *m* for application in runoff-routing applications are further discussed in Book 5, Chapter 6. Application of runoff-routing models for the range of Very Rare to Extreme floods is discussed in Book 8.

5.5.3. Non-linear Distributed Storage

The distributed form of the S-Q relationship used in the Muskingum equation (Equation (5.5.27)) can also be applied as a nonlinear relationship:

$$S = k[XI + (1 - X)O]^m \qquad (5.5.39)$$

or more simply:

$$S = kQ_w{}^m \tag{5.5.40}$$

where Q_w is the weighted discharge for the routing reach.

An example of the application of the non-linear Muskingum Method for routing hydrographs through river reaches is in the URBS runoff routing model (Carroll, 2012).

The translation and attenuation effects of non-linear distributed storage represented by Equation (5.5.39) can also be replicated by routing through a number of non-linear storages placed in series. This is the approach incorporated in the RORB runoff-routing model (Laurenson et al., 2010). However, as discussed for the case of linear routing methods (Book 5, Chapter 5, Section 4), for accurate representation of the attenuation characteristics of a river reach, the number of sub-reaches used for routing needs to be carefully selected.

The effect of different non-linearity assumptions used for routing hydrographs through non-linear distributed storage (or a series of concentrated non-linear storages) is to produce different degrees of attenuation when the calibrated k and m parameters are applied to routing floods of different magnitude. This is illustrated in Figure 5.5.9 for the case where the non-linear routing model is applied to a flood hydrograph twice the magnitude of the observed flood used for calibrating the model. It is shown that a lower value of m (with a correspondingly higher value of k) produces a higher peak that occurs earlier than if a k and m parameter combination for a linear model had been used.

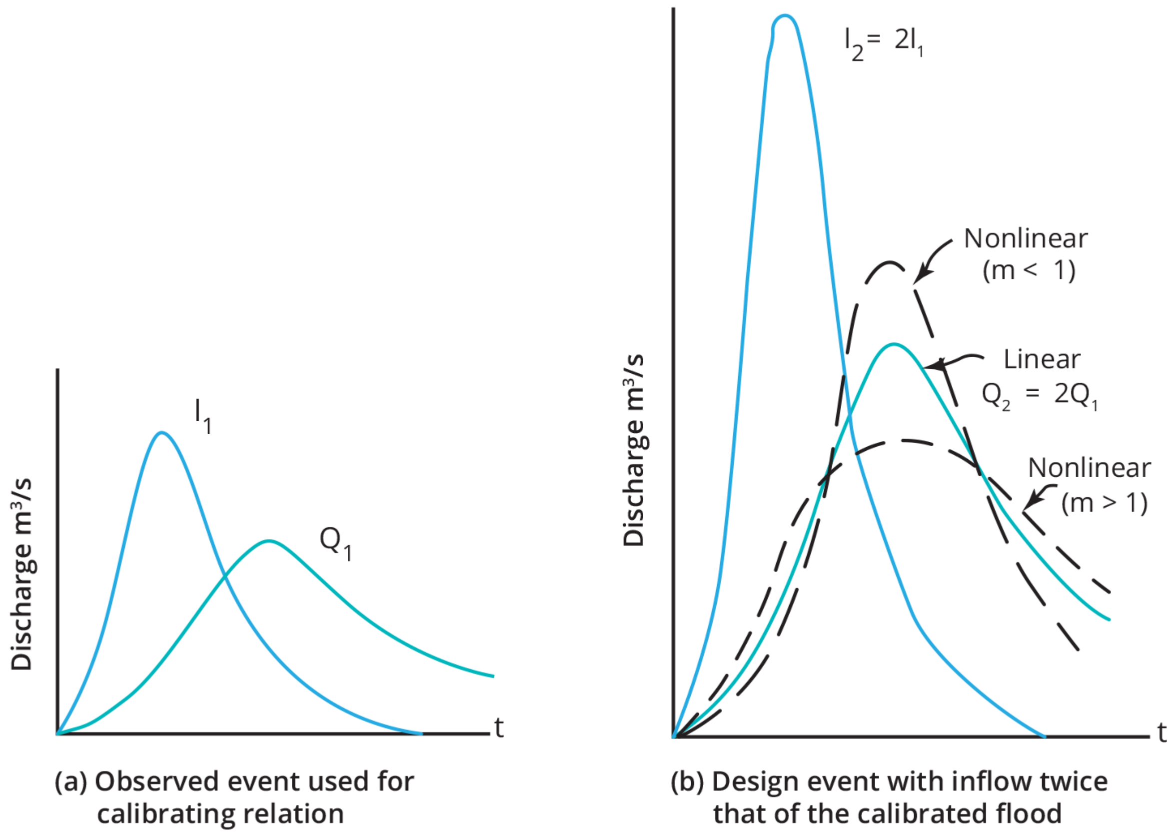


Figure 5.5.9. Effect of Non-linearity of Storage-Discharge Relationship on Routed Hydrographs (after Pilgrim (1987))

5.6. Hydraulic Routing Approaches

5.6.1. Introduction

The hydraulic routing approaches are based on various forms of unsteady flow equations. The *full dynamic wave equations* (or St Venant equations) introduced in Book 6, Chapter 2, Section 8 and Book 6, Chapter 4, Section 6 describe the conservation of mass (continuity equation) and the conservation of momentum (momentum equation).

For application in flood hydrograph estimation models it is most useful to present the one dimensional unsteady flow equations in terms of the discharge Q and stage z (Weinmann, 1977; Fread, 1985):

Continuity:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} - q = 0 \tag{5.5.41}$$

Momentum:

$$\frac{\partial A}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + gA\left(\frac{\partial z}{\partial x} + S_f\right) = 0 \tag{5.5.42}$$

where q is the rate of lateral inflow to the routing reach, g the gravitational acceleration, z the water level or stage and S_f the average friction slope of the routing reach. The friction slope can be determined from a uniform flow resistance formula (Book 6, Chapter 2, Section 5) as $S_f = Q^2/C^2$, where C is the conveyance of the cross-section.

This system of equations can be applied in flood hydrograph estimation models to track the movement of a flood hydrograph through river and floodplain reaches. The equations have no analytical solution and flood routing methods based on the full dynamic equations thus need to apply one of the numerical solution procedures described in Book 6, Chapter 4, Section 7. Explicit numerical solution schemes provide a more direct and more computationally efficient solution than implicit schemes but, to avoid computational stability problems, they require the time and space steps to be selected in accordance with the Courant stability criterion.

The particular advantage of the application of the full dynamic wave equations is in their ability to allow for backwater effects or tidal influences and to deal more accurately with rapidly rising or falling flood hydrographs. The flood routing models based on the full dynamic equations can also produce flood level hydrographs and rating curves at points of interest.

The application of hydraulic routing approaches requires the geometry of the channel and floodplain system to be defined by cross-sectional information obtained from river surveys or Digital Elevation Models. The representation of the actual river and floodplain system in the model is highly conceptualised, as the computational cross-sections are generally quite widely spaced and a smooth variation of the hydraulic characteristics over the model reach is assumed.

Traditionally the application of the full dynamic wave equations has been limited by the fact that their numerical solution is more demanding on computer resources but this is no longer an important factor.

Two dimensional forms of the unsteady flow equations are introduced in Book 6, Chapter 2, Section 9 and Book 6, Chapter 4, Section 7. This form of the dynamic wave equations (or a simplified form of the equations) is applied in the rainfall-on-grid models discussed in Book 5, Chapter 6, Section 5.

5.6.2. Kinematic and Diffusion Wave Routing

The basic equations used for kinematic wave and diffusion wave routing are derived from the full dynamic wave equations introduced in Book 5, Chapter 5, Section 6 by neglecting some terms of the complete momentum equation (Equation (5.5.42)).

5.6.2.1. Diffusion Wave Routing

By neglecting the first two terms of the momentum equation (Equation (5.5.42)), which represent the effects of local and convective acceleration respectively, but keeping the terms representing the pressure and friction forces, the following simplified form of the momentum equation is obtained:

$$S_f = S_0 - \frac{\partial z}{\partial x} \tag{5.5.43}$$

The inclusion of the pressure term ∂z/∂x allows for the effects of a downstream boundary condition (backwater, tidal influences) to be included in the routing computations.

By combining this simplified momentum equation with the continuity equation (Equation (5.5.41)) the form of the *convective-diffusion (diffusion wave) equation* (Equation (5.5.44)) used in hydrologic flood routing models is obtained:

$$\frac{\partial Q}{\partial t} + C_k \frac{\partial Q}{\partial x} = D \frac{\partial^2 Q}{\partial x^2} + C_k q \tag{5.5.44}$$

with $C_k = \frac{\partial Q}{\partial A}$ the kinematic wave celerity

and $D = \frac{Q}{2BS_o}$ the diffusion coefficient

where *Q* is the discharge, *q* the rate of lateral inflow, *x* the distance along the channel, *A* the cross-sectional area, *B* the water surface width and S_o the channel slope.

The diffusion term in Equation (5.5.44) allows explicitly for the diffusion and peak attenuation effects observed in the movement of flood waves through river and floodplain reaches. This is in contrast to the Muskingum Method where the diffusion effects are only introduced through judicious choice of the numerical solution scheme and determination of parameter values.

5.6.2.2. Kinematic Wave Routing

The kinematic wave equation is obtained from Equation (5.5.44) by omission of the diffusion term:

$$\frac{\partial Q}{\partial t} + C_k \frac{\partial Q}{\partial x} = C_k q \tag{5.5.45}$$

The term ‘kinematic wave’ was introduced by Lighthill and Witham (1955) to describe the motion of waves in time and space without considering mass and force. Equation (5.5.45) can be obtained from the full unsteady flow equations by replacing the momentum equation (Equation (5.5.42)) by a uniform flow relationship.

Kinematic waves are theoretically not dispersive (ie. they travel without attenuation) but the variation of the travel speed C_k with Q produces a change of wave form, resulting in a gradual steepening of the wave front as it travels downstream, eventually leading to a 'kinematic shock' (Henderson, 1966). Analytical solutions for the kinematic wave equations exist only for a few idealised situations (Miller, 1984). Numerical solution schemes for the kinematic wave equation introduce some degree of dispersion/attenuation of the flood wave, and thus match more closely the behaviour of actual flood waves.

As indicated in Book 5, Chapter 5, Section 4, the Muskingum Method can be understood to be a numerical solution scheme for the kinematic wave model. Various other numerical solution techniques are described in the literature and applied in practical flood routing models (e.g. Miller (1984), HEC (1993)) The application of kinematic wave principles in runoff routing models is discussed in Book 5, Chapter 6, Section 4.

5.7. References

Bedient, P.B., Huber, W.C. and Vieux, B.E. (2008), Hydrology and floodplain analysis, ed. 4, Upper Saddle River: Prentice Hall.

Boyd, M.J., Pilgrim, D.H., Knee, R.M. and Budiawan, D. (1989), Reverse Routing to Obtain Upstream Hydrographs. Hydrology and Water Resources Symposium. Christchurch, NZ, 23-30 November. Preprints of Papers (p: 372). Institution of Engineers, Australia.

Carroll, D.G. (2012), URBS (Unified River Basin Simulator) A Rainfall Runoff Routing Model for Flood Forecasting and Design. Version 5.00 Dec, 2012.

Chang, C.N., da Motta Singer, E. and Koussis, A.D. (1983), On the mathematics of storage routing, Jounral of Hydrology, 61(4), 357-370.

Chapra, S.C. and Canale, R.P. (2010), Numerical methods for engineers, New York: McGraw Hill.

Cunge, J.A. (1969), On the subject of a flood propagation method, Journal of Hydrology Research, 7(2), 205-230.

Fenton, J.D. (1992), Reservoir routing, Hydrological Sciences Journal, 37(3), 233-246.

Fread, D.L. (1985), Channel routing. In 'Hydrological Forecasting', Chichester: John Wiley and Sons Ltd.

HEC (1993). Introduction and application of kinematic wave routing techniques using HEC-1. US Army Corp of Engineers, Hydrologic Engineering Centre, http://www.hec.usace.army.mil/publications/TrainingDocuments/TD-10.pdf.

Henderson, F.M. (1966), Open Channel Flow, Macmillan Publishing Co., New York

Kalinin, G. P. and Miljukov, P. I. (1958), Approximate methods for computing unsteady flow movement of water masses (in Russian), Transactions Central Forecasting Institute, 66 p.

Koussis, A.D. (2009), Assessment and review of the hydraulics of storage routing 70 years after the presentation of the Muskingum method. Hydrological Sciences Journal, 54(1), 43-61, available at <http://dx.doi.org/10.1623/hysj.54.1.43>.

Laurenson, E.M. (1998), Hydrology CIV3263 Course Notes, Monash University, Department of Civil Engineering.

Laurenson, E.M. (1962), Hydrograph synthesis by runoff routing, Report No. 66, Sydney: Water Research Laboratory, University of New South Wales.

Laurenson, E.M. (1986), Variable time-step nonlinear flood routing, In Hydrosoft '86; Hydraulic Engineering Software, Proceeding sof the 2nd International Conference, Computational Mechanics Publns, Springer-Verlag, 9-12 September, pp: 61-72, Berlin.

Laurenson, E.M., Mein, R.G. and Nathan, R.J. (2010). RORB Version 6 Runoff Routing Program - User Manual.

Lighthill, M.J. and Witham, G.B. (1955), On kinematic waves I, Proceedings of the Royal Society of London, 229: 281-316.

McCarthy, G.T. (1938), The unit hydrograph and flood routing, Manuscript presented at a conference of the North Atlantic Division, United States Army Corps of Engineers, 24 June (unpublished).

Mein, R.G., Laurenson, E.M. and McMahon, T.A. (1975), Simple nonlinear model for flood estimation, Journal of Hydrology, 100(HY11), 1507-1518.

Miller, J.E. (1984), Basic concepts in kinematic wave models. U.S. Geological Survey Professional Paper 1302, Washington: United States Government Printing Office.

Nash, J.E. (1959), A note on the Muskingum flood-routing method, Journal of Geophysical Research, 64(8), 1053-1056.

Pilgrim, DH (ed) (1987) Australian Rainfall and Runoff - A Guide to Flood Estimation, Institution of Engineers, Australia, Barton, ACT, 1987.

Stephenson, D. (1979), Direct optimization of Muskingum routing coefficients, Journal of Hydrology, 41: 161-165.

Weinmann, P.E. (1977), Comparison of flood routing methods for natural rivers, Civil Engineering Research Report No.2/1977, Clayton: Monash University.

Wong, T.H.F. (1985), Reach subdivision for storage routing in rivers, Proceedings of the 21st IAHR Congress, 19-23 August, Melbourne.

Wong, T.H.F. and Laurenson E.M. (1983), Wave speed-discharge relations in natural channels, Water Resources Res., 19: 701-706.

Zoppou, C. (1999), Reverse routing of flood hydrographs using level pool routing, Journal of Hydrologic Engineering, 4(2), 184-188.

Chapter 6. Flood Hydrograph Modelling Approaches

James Ball, Erwin Weinmann

Chapter Status	Final
Date last updated	14/5/2019

6.1. Introduction

This chapter deals with a range of approaches available to calculate design flood hydrographs at the catchment outlet and other points of interest. It therefore integrates the previous chapters in Book 5 and also links to Book 7, where practical applications are discussed.

The time-area approaches (Book 5, Chapter 6, Section 2) and unit hydrograph approaches (Book 5, Chapter 6, Section 3) allow a relatively simple transformation of rainfall excess inputs to flood hydrograph outputs and are directly applicable to a lumped representation of the flood formation process in a catchment, where the inputs and processes can be assumed to be spatially uniform (or at least spatially consistent between different events). These “traditional” approaches have generally been replaced by more flexible approaches. However, they also find application to represent the overland flow phase of hydrograph formation in some of the node-link type models discussed in subsequent sections.

The most widely used flood hydrograph estimation models are based on the runoff routing approach, in which both the runoff production and hydrograph formation phases can be represented in a distributed fashion, reflecting the spatial variation of rainfall inputs and flood processes in a catchment. The two principal groups of models are the network (node-link type) models described in Book 5, Chapter 6, Section 4 and the rainfall-on-grid (or direct rainfall) models described in Book 5, Chapter 6, Section 5.

The routing methods incorporated in these models have their foundations in the open channel hydraulics introduced in Book 6 and apply the flood routing principles outlined in Book 5, Chapter 5 of this book. The descriptions in Book 5, Chapter 6, Section 4 and Book 5, Chapter 6, Section 5 focus on the specific way these principles are applied to represent different parts of the flood hydrograph formation process.

The discussion of flood hydrograph modelling in this chapter is intended to introduce readers to the different approaches used and the assumptions made in different modelling approaches and different runoff routing modelling systems. Guidance on the practical application of flood hydrograph models to different flood estimation problems, including estimation of model parameters, is provided in Book 7.

As with other chapters of Book 5, this chapter deals primarily with rural catchments, and while the principles apply also for urban catchments, urban catchment hydrology is covered in detail in Book 9.

6.2. Time-Area approaches

6.2.1. Time-Area Theory

Time-area approaches can be seen as an extension of the *travel time* concept used in the Rational Method. However, instead of using a single travel time or *time of concentration* for flow from the most remote point on the catchment to its outlet to calculate a peak discharge, time-area approaches use travel times from all parts of the catchment and calculate a complete flood hydrograph. Early development of the approach was carried out by Hawken (1921) and Ross (1921), while examples of time-area approaches are given in Bedient et al. (2008).

The basic principle of time-area approaches, as illustrated in Figure 5.6.1, is that rainfall excess at any time *t-tt* after the start of the storm that occurs at a point on the catchment with a travel time of *tt* to the catchment outlet will influence flow at the catchment outlet at time *t*. A fundamental assumption involved in this is that flow at the catchment outlet is influenced only when the runoff reaches the catchment outlet, i.e. when the individual water particles reach the catchment outlet.

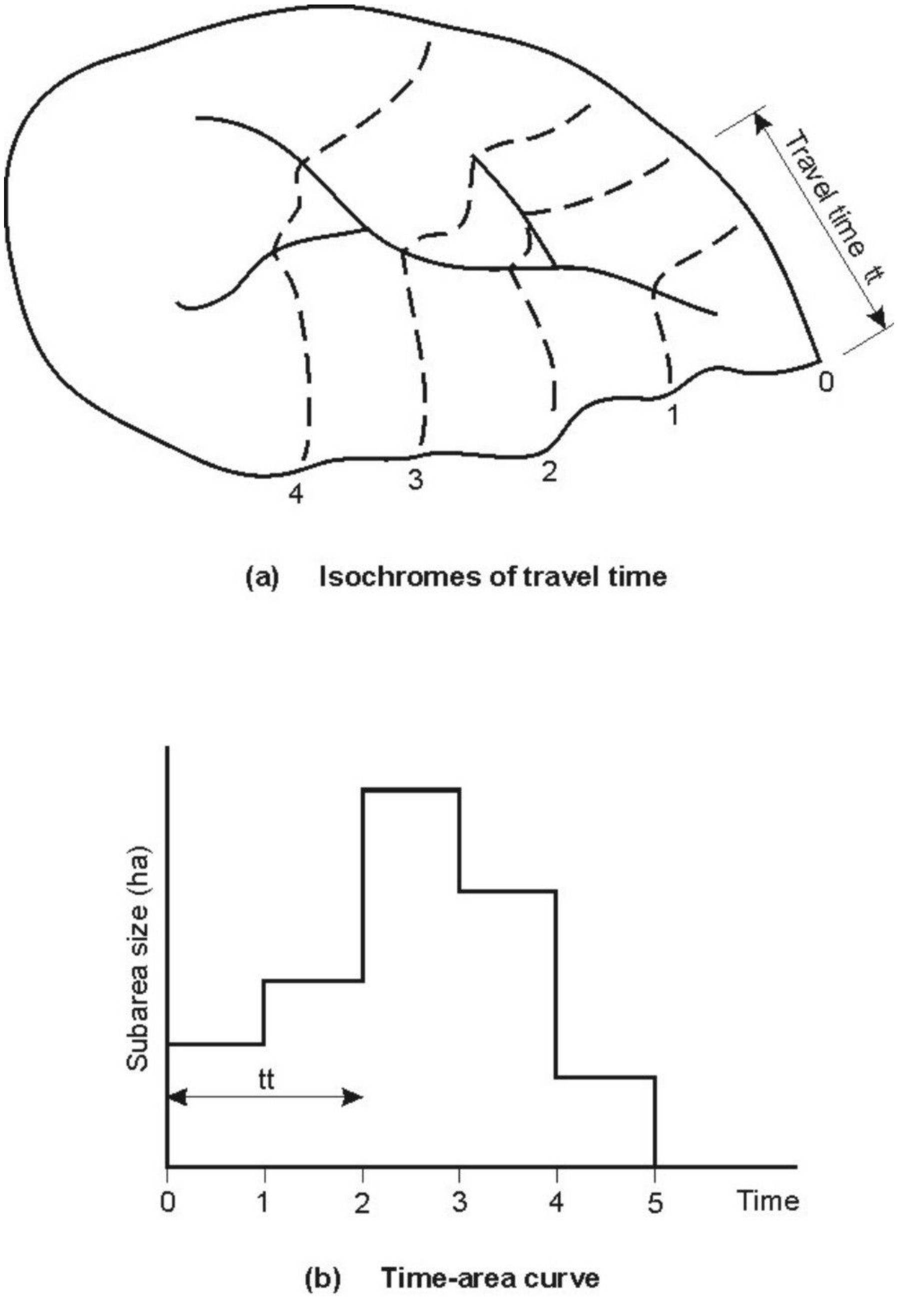


Figure 5.6.1. Isochrones and Time-Area Curve

Points on the catchment which have equal travel times to the outlet can be joined to form isochrones. Application of the time-area method requires isochrones to be drawn for the catchment being considered; note that many computerised applications assume that the area increases in a linear manner on small subcatchments to avoid the need for delineation of isochrones.

When construction of isochrones is required, a common assumption is that the travel time is related to the travel length (L) and slope (S) of the catchment by the following relationship:

$$t_t = \frac{L}{S^{0.5}} \tag{5.6.1}$$

Equation (5.6.1) follows directly from Manning's equation, in which flow velocity is related to the square root of stream slope which is assumed to be the same as the energy gradient. However, as the flow moves downstream to the catchment outlet, the flatter stream slopes are accompanied by greater water depths, which may compensate for the decreasing stream slope in the Manning equation. Studies by Leopold et al. (1964) and Pilgrim (1977) indicate that stream velocities essentially remain constant along the length of the stream and may even increase in a downstream direction. If velocities remain constant along the stream, the travel time would be directly related to travel distance by:

$$t_t = L \tag{5.6.2}$$

A plot of the area between adjacent isochrones against the travel time produces a time-area curve. An example of a time-area curve is presented in Figure 5.6.1.

Since there are many points on the catchment with travel times t_t and corresponding times t-t_t after the start of the storm, the flowrate at the outlet at time t is the sum of all possible combinations. As a simple example, if the catchment is divided into five segments using isochrones (A_i), and a storm has three periods of rainfall excess (P_j in mm/h) with both the isochrones and rainfall having the same time step, then the total time of the hydrograph is eight time steps. Also, the discharges from the catchment at successive time steps (equal to the isochrone interval) for this example are given by:

$$\begin{aligned}
&Q_0 = 0\\
&Q_1 = kA_1P_1\\
&Q_2 = k\left(A_1P_2 + A_2P_1\right)\\
&Q_3 = k\left(A_1P_3 + A_2P_2 + A_3P_1\right)\\
&Q_4 = k\left(A_2P_3 + A_3P_2 + A_4P_1\right)\\
&Q_5 = k\left(A_3P_3 + A_4P_2 + A_5P_1\right)\\
&Q_6 = k\left(A_4P_3 + A_5P_2\right)\\
&Q_7 = kA_5P_3\\
&Q_8 = 0
\end{aligned} \tag{5.6.3}$$

where k is an appropriate unit conversion factor (k varies with the units of Q and A).

In general, Equation (5.6.3) can be expressed as:

$$Q_t = k\sum_{i=1}^{t} A_iP_{j-i+1} \tag{5.6.4}$$

where Ai is the area between the i-1 and i isochrones, Pj is the rainfall excess depth in the jth period of the storm event, and k is a conversion factor. As the conversion factor (k) will vary with the isochrone interval, it is recommended that the intensity of rainfall excess (mm/h) be used; details of the conversion factors for different combinations of discharge, area units and rainfall excess intensity in mm/h are given in Table 5.6.1.

Table 5.6.1. Conversion Factors

Discharge	Area	Rainfall Excess Intensity	Conversion Factor
m^3/s	ha	mm/h	1/360
m^3/s	km^2	mm/h	1/3.6
L/s	ha	mm/h	1/0.36

6.2.2. Limitations of Time-Area Approaches

Despite the use of the time-area approach in various models, the time-area concept has several limitations, including:

- Isochrones of travel time usually are not known, except in a few experimental studies and must be estimated. These experimental catchments include those monitored by (Pilgrim, 1966a; Pilgrim, 1966b) using tracers to ascertain travel times. To overcome this disadvantage, many applications adopt a simplified time-area relationship. A common simplified relationship is based on a linear growth in area with time (in essence, an assumption of a rectangular shape with a length given by the response time and a width defined by the catchment area), and thus there is a need only to estimate a representative travel time for the conceptualised catchment.

- The time-area curve cannot be easily derived from recorded rainfall and streamflow data.

- Construction of the direct runoff hydrograph assumes that flow is translated to the outlet with a lag but without attenuation. In other words, a kinematic response is assumed. As a result of this, time area methods are more likely to be applicable to estimation of flows from small catchments and particularly to estimation of surface flows in urban catchments.

- The method is linear, i.e. a doubling of rainfall excess results is a doubling of predicted discharges, whereas data from many catchments, particularly the larger rural catchments, demonstrates a nonlinear response to changes in rainfall excess.

6.2.3. Worked Example

The example below illustrates the application of the time-area method to a small rural catchment. The steps in the method remain similar when applied to an urban catchment but the time-area diagram then needs to reflect differences in travel time over different types of catchment surfaces and different types of drainage systems.

Example - Hydrograph Calculation for Triangular Time-Area Curve

A 5 hectare catchment has a time of concentration of 15 minutes. The time-area curve is assumed to be triangular in shape. (This is similar to the time-area curve used with the Cordery-Webb approach for development of synthetic unit hydrographs). The surface runoff hydrograph is to be estimated for a 21 minute storm event with the details of this event shown in Table 5.6.2.

Since the storm event has 7 periods, each of 3 minutes duration, the catchment will be divided into 5 subareas by isochrones spaced at 3 minute intervals. The hydrograph base length is given by the catchment time of concentration plus the storm duration, ie 15 minutes + 21 minutes = 36 minutes. The total depth of rainfall excess is 8.1 mm (see Column 3 in Figure 5.6.2). Using this depth of rainfall excess and the catchment area, the volume of direct runoff from the catchment is 405 m^3.

The subarea sizes between each adjacent pair of 3 minute isochrones, proceeding from the outlet of the catchment to the top of the catchment, are 0.33, 0.67, 1.00, 1.33 and 1.67 ha. The resultant time-area relationship is shown in Figure 5.6.2.

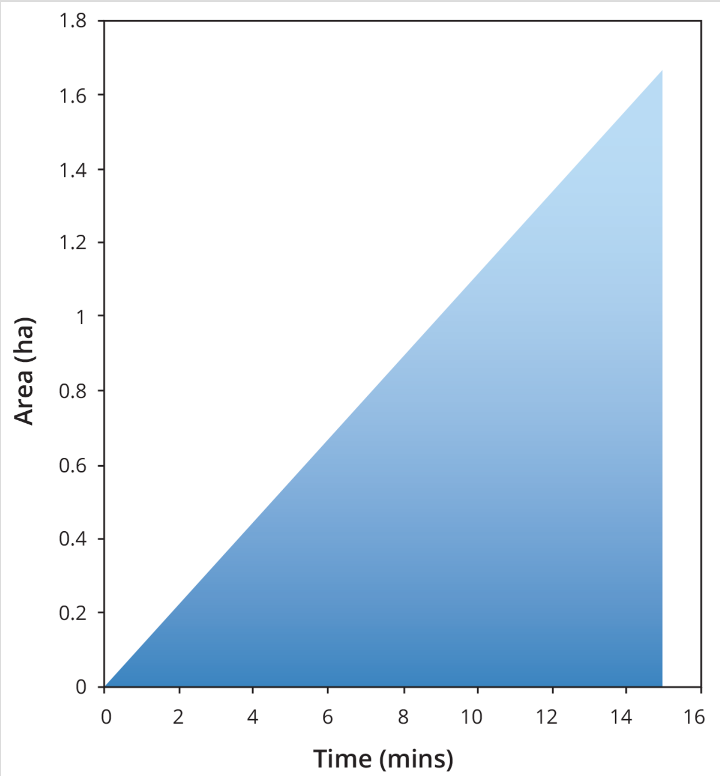


Figure 5.6.2. Time-Area Relationship for Example 2

Table 5.6.2. Calculation of Surface Runoff Hydrograph using Triangular Time-Area Curve

Time (minutes)	Subarea (ha)	Rainfall Excess (mm)	Rainfall Excess Intensity (mm/h)	Surface Runoff (m^3/s)
0	0	0	0	0
3	0.33	0.3	6	0.006
6	0.67	0.6	12	0.022
9	1	1.8	36	0.072
12	1.33	3.6	72	0.189
15	1.67	0.9	18	0.322
18		0.3	6	0.428
21		0.6	12	0.506
24				0.439
27				0.139
30				0.072
33				0.056
36				0
$\sum$		**8.1**		**2.25**

Application of Equation (5.6.4) allows the direct runoff hydrograph ordinates to be calculated (see Column 5 in Table 5.6.2). Figure 5.6.3 summarises the time-area calculations and shows the resulting surface runoff hydrograph.

As a check, from Column 5 in Table 5.6.2, the volume of the direct runoff hydrograph 1 is given by

$$\text{Volume} = \sum Q\Delta t = 2.25m^3/s \text{ x } 180s = 405m^3 \quad (5.6.5)$$

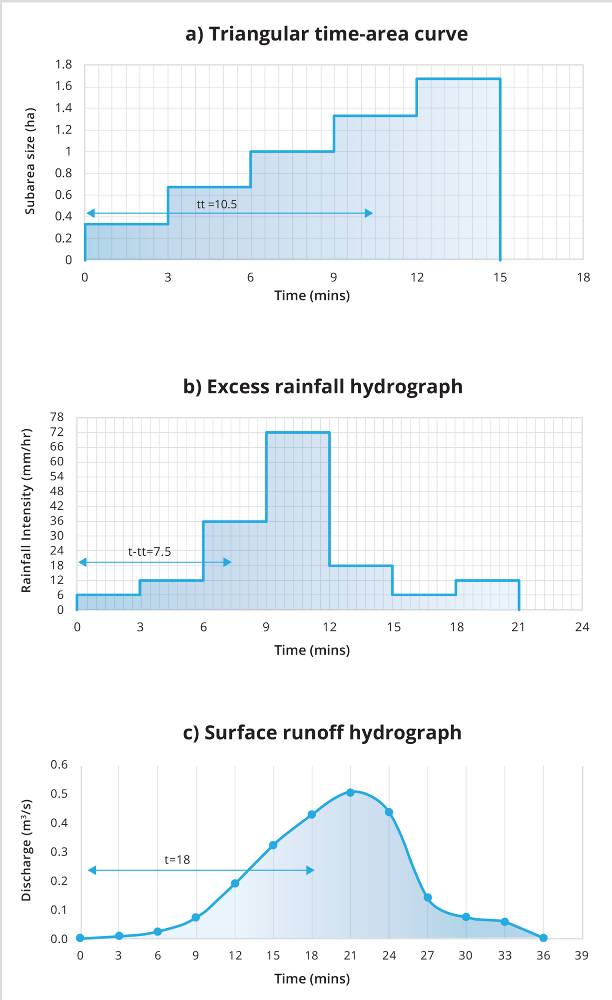


Figure 5.6.3. Calculation of Surface Runoff Hydrographs using Time-Area Approach

6.3. Unit Hydrograph Approaches

6.3.1. Introduction

This section of Australian Rainfall and Runoff is based on the chapter on unit hydrographs prepared by Cordery (1987) for the previous edition of Australian Rainfall and Runoff. However, as the unit hydrograph approaches are no longer widely applied in Australia, only a brief introduction is given here. For a more detailed description of the unit hydrograph method and worked examples the reader is referred to Cordery (1987).

6.3.2. Unit Hydrograph Theory

6.3.2.1. Basic Concepts

Development of the unit hydrograph approach is generally attributed to Sherman (1932). Unit hydrographs represent an advance over time-area procedures because, rather than constructing a time-area curve from isochrones of travel time, which requires assumptions regarding the travel times from all points on the catchment, the unit hydrograph represents the actual flood response of the catchment to rainfall, and can be directly determined or estimated from recorded rainfall and streamflow data. As a consequence, the resulting unit hydrograph incorporates the effects of both translation and attenuation, and so reduces the assumptions needed in the time-area approaches.

6.3.2.2. Definition of a Unit Hydrograph

A unit hydrograph is defined as the direct runoff hydrograph resulting from 1mm depth of rainfall excess, where the rainfall excess occurs over a particular time period and is uniformly distributed across the catchment. The time period of the rainfall excess sets the period of the unit hydrograph. Thus a rainfall burst which lasts 2 hours and has an average rainfall excess intensity of 0.5mm/h will produce a 2 hour unit hydrograph. It is important to note that the rainfall excess intensity should be uniform over the period of the burst, and that it also should be spatially uniform across the catchment.

Direct runoff hydrographs result from the interaction between two important factors:

- the time varying storm rainfall hyetograph; and
- the translation and attenuation properties of the catchment storage

In the special case of the unit hydrograph, a standardised rainfall excess (1 mm) is used, and the unit hydrograph, therefore, represents the effects of the catchment in delaying and attenuating rainfall excess as it flows from all points on the catchment to the catchment outlet. Use of this standardised rainfall excess provides the opportunity to relate the size and shape of the unit hydrograph to the catchment’s geophysical properties such as area, stream length and slope, and thus enables synthetic unit hydrographs to be estimated for catchments where no recorded streamflow data exists.

Each unit hydrograph reflects the unique geophysical properties of the catchment and, hence, each catchment should have its own unique unit hydrograph. Bernard (1935) pioneered this concept by developing a dimensionless unit hydrograph which reflected the geophysical properties of the catchment.

The basic principle of unit hydrograph theory is that the catchment responds in a linear manner to rainfall excess and, hence, superposition is feasible. As unit hydrographs form a linear system, the ordinates of the direct runoff hydrograph are linearly proportional to the depth of rainfall excess. For example, if the rainfall excess is doubled, each ordinate of the direct runoff hydrograph will be doubled. As another example, if a sequence of several periods of rainfall excess occurs, one after another, the resulting direct runoff hydrograph is equal to the sum of the runoff from each individual period of rain (see Figure 5.6.4).

6.3.2.3. The Specified Period of the Unit Hydrograph

The specified period of the unit hydrograph must be short enough to provide good definition of both the rainfall excess hyetograph and the resulting direct runoff hydrograph. This means the specified period should be short enough to provide a reasonable representation of all major changes of the rainfall excess intensity and should be less than a quarter of the time of rise of the unit hydrograph. Large rural catchments could have unit hydrographs with specified periods of 3 or more hours, whereas small urban catchments may require specified periods of 5 minutes or less.

For a given catchment the unit hydrograph for a particular specified period will be different from those with different time specified periods. For example, a 1 hour unit hydrograph results from 1mm of rainfall excess falling over 1 hour at a rate of 1mm/h, whereas a 2 hour unit hydrograph results from 1mm of rainfall excess falling over 2 hours at a rate of 0.5 mm/h. In general, the peak discharges of unit hydrographs with longer specified periods will be lower and will occur later in time. This decrease in peak discharge arises as a result of the lower intensity of rainfall excess as the specified period of the unit hydrograph increases.

The 1 hour unit hydrograph will be applied to a rainfall hyetograph with 1 hour rain periods, and will produce a direct runoff hydrograph with the ordinate values predicted every hour. The 2 hour unit hydrograph will be applied to a rainfall hyetograph with 2 hour rain periods.

6.3.2.4. Changing the Specified Period of a Unit Hydrograph

To change the specified time period of the unit hydrograph two approaches are available, depending on whether a longer or a shorter specified period is needed.

Calculation of a unit hydrograph of longer time period from one of a shorter period is accomplished by the addition of several short period unit hydrographs with each sequential unit hydrograph delayed by the specified period. Thus, the sum of four 15 minute unit hydrographs, each of which is delayed in time by 15 minutes, will produce a direct runoff hydrograph resulting from 4 mm of rainfall excess during a 1 hour period. Since a unit hydrograph is the result of 1 mm of rainfall excess during the specified time period, the 1 hour unit hydrograph is derived from this runoff hydrograph by dividing all ordinates in the runoff hydrograph by four.

Another situation in which the specified time period needs to be changed is where an instantaneous unit hydrograph has been obtained, as occurs in some synthetic unit hydrograph methods. The instantaneous unit hydrograph (IUH) represents the direct runoff hydrograph produced by 1 mm of rainfall excess which occurs at an instant in time. The ordinates of a specified period unit hydrograph at each time *t* are obtained by integrating the ordinates of the IUH over an interval from *t-T* to *T*, where *T* is the specified time period, then dividing this result by *T*. Practically, this is achieved by averaging the IUH ordinates at times *t* and *t-T*. For example, each ordinate of a 1 hour unit hydrograph is the average of the IUH ordinates at this time and 1 hour before this time (Boyd, 1982).

The derivation of a unit hydrograph of shorter specified time period from one of a longer period is less direct, but can be attempted using an S-curve. Data errors will often produce oscillations in the S curve and it is often difficult to obtain good results when this method is applied to real data. (Details of the S-curve method are given in many textbooks, e.g. Bedient et al. (2008))

6.3.3. Calculating Direct Runoff Hydrographs Using Unit Hydrographs

Since superposition is assumed to be feasible with unit hydrographs, a storm with *j* periods of rainfall excess will produce *j* direct runoff hydrographs, with the ordinates of each direct runoff hydrograph factored in proportion to the depth of rainfall excess. The direct runoff for the storm is then the sum of all *j* hydrographs. For the summation, the period of the unit hydrograph must be the same as incremental time period of the rainfall hyetograph.

If the unit hydrograph has *k* ordinates and there are *j* periods of rainfall excess, the number of direct runoff ordinates will be given by $n = j + k - 1$. Shown in Equation (5.6.6) is the determination of the direct runoff hydrograph for the case where there 3 periods of rainfall excess and 5 unit hydrograph ordinates.

$$
\begin{aligned}
Q_0 &= 0 \\
Q_1 &= P_1U_1 \\
Q_2 &= P_1U_2 + P_2U_1 \\
Q_3 &= P_1U_3 + P_2U_2 + P_3U_1 \\
Q_4 &= P_1U_4 + P_2U_3 + P_3U_2 \\
Q_5 &= P_1U_5 + P_2U_4 + P_3U_3 \\
Q_6 &= P_2U_4 + P_3U_4 \\
Q_7 &= P_3U_5 \\
Q_8 &= 0
\end{aligned}
\tag{5.6.6}
$$

The first column on the right hand side represents the direct runoff hydrograph from P_1 mm of rainfall excess in the first period of the storm, while the second and third columns represent the direct runoff from the subsequent periods in the storm. Note that each direct runoff hydrograph is delayed by one time period, because the rainfall excesses P_1, P_2 and P_3 occur in successive periods of the storm. Note also that the rainfall excess period, the period of the unit hydrograph, and the time step at which the unit hydrograph ordinates are listed, are all equal.

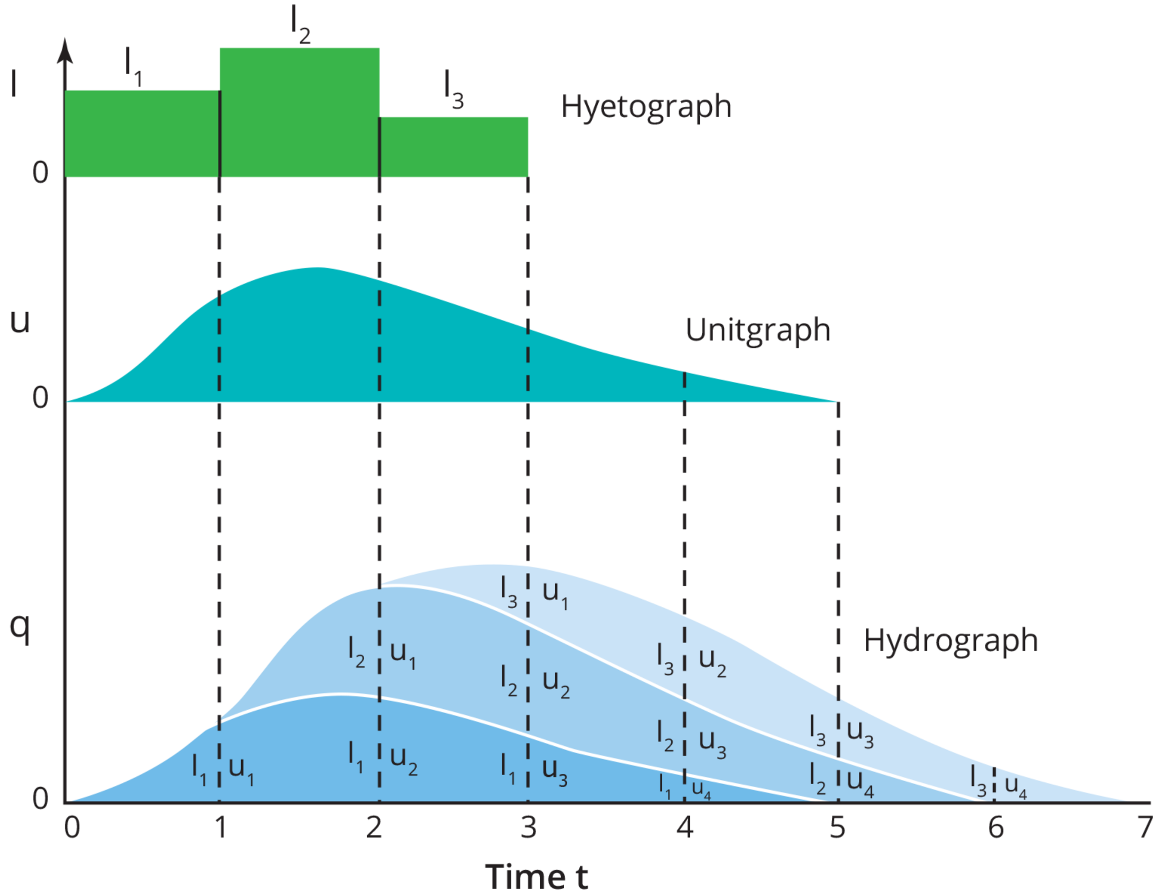


Figure 5.6.4. Unit Hydrograph Calculation – example with 3 periods of rainfall excess and unit hydrograph with 5 ordinates (after Laurenson (1998)).

This figure needs to be redrawn but still referenced to Laurenson.

Equation (5.6.6) can be written in a general form as

$$Q_m = \sum_{i=1}^{j} P_i U_{m-i+1} \tag{5.6.7}$$

where Q_m is any ordinate of the direct runoff hydrograph.

6.3.4. Derivation of Unit Hydrographs from Rainfall and Streamflow Data

6.3.4.1. General Concepts

Recorded rainfall and streamflow data can be used to derive unit hydrographs. Steps in the process are:

1. Select a range of significant flood events for which the direct runoff hydrograph and suitable rain gauge data are available. Only floods with a 50% AEP or less should be used in order to ensure that the data properly reflects the processes which occur during significant flood events. Furthermore, the rainfall during the selected events should be spatially and temporally uniform over the catchment. If an insufficient number of large events is available then it may be necessary to use smaller floods. In this case it must be borne in mind that small floods will tend to produce unit hydrographs which have lower peaks and longer times of rise than are appropriate for use in the estimation of large floods.

2. Separate baseflow as described in Book 5, Chapter 4 to obtain the direct runoff hydrograph.

3. Calculate the volume of direct runoff.

4. Calculate a spatial average rainfall hyetograph for the storm, using hyetographs from all available rainfall stations on or near to the catchment.

5. Calculate the rainfall excess hyetograph, by subtracting losses so that the depth of rainfall excess equals the depth of direct runoff. Rainfall losses can be assumed either as an initial loss-continuing loss model, or an initial loss-proportional loss model (see Book 5, Chapter 3).

Once the recorded streamflow and rainfall data have been analysed to extract the direct runoff hydrograph and rainfall excess hyetograph, derivation of the unit hydrograph can proceed. Unit hydrographs can be derived from single period storms, or from multi-period storms.

6.3.4.2. Selection of an Average Unit Hydrograph for the Catchment

In principle, every unit hydrograph derived from various recorded storms on a catchment should be the same (since each catchment is considered to have a unique unit hydrograph). In practice, however, various factors such as spatial variation in rainfall, errors in data, or limited data (for example insufficient rain gauge coverage of the catchment) means that the unit hydrographs derived from different storms will be somewhat different from one another. Titmarsh and Cordery (1991) found that the peak discharge of unit hydrographs derived from a range of storms on a catchment varied by a factor of 4, while Boyd (1975) found that the mean absolute deviation of the peak discharge was on average 31%.

It is important, therefore, to derive several unit hydrographs for a catchment, selecting the larger storms. All derived unit hydrographs should be compared for consistency, and inconsistent ones rechecked or deleted.

A plot of unit hydrograph peak discharge against the peak discharge of the recorded direct runoff hydrograph from which it was derived may reveal a trend for unit hydrograph peaks to increase for the larger floods. Any such trend is an indication that the catchment is not behaving in a linear manner. In these circumstances, it may be more appropriate to use an alternative technique for estimation of the direct runoff hydrograph. Catchments displaying a nonlinear response to storm events can still use the unit hydrograph approach, but it may be desirable to derive several unit hydrographs for the catchment, each one derived from, and being applicable to a particular range of flood sizes, as discussed by Body (1962).

The unit hydrographs which are considered to be acceptable can be averaged to produce a more representative unit hydrograph for the catchment. Averaging unit hydrographs in this way also has the benefit of reducing any oscillations on the recessions of unit hydrographs derived from multi-period storms.

The recommended approach to calculate the average unit hydrograph is to align the peaks of all unit hydrographs, then average their ordinates at each successive time step (Titmarsh and Cordery, 1991). This method produces a unit hydrograph whose time to peak is the average of all times to peak, and peak discharge which is the average of all peak discharges. A simple average of all unit hydrographs, without regard for the occurrence of the peak is not recommended, as this can produce an average unit hydrograph which is quite different from the individual unit hydrographs (see Figure 5.6.5).

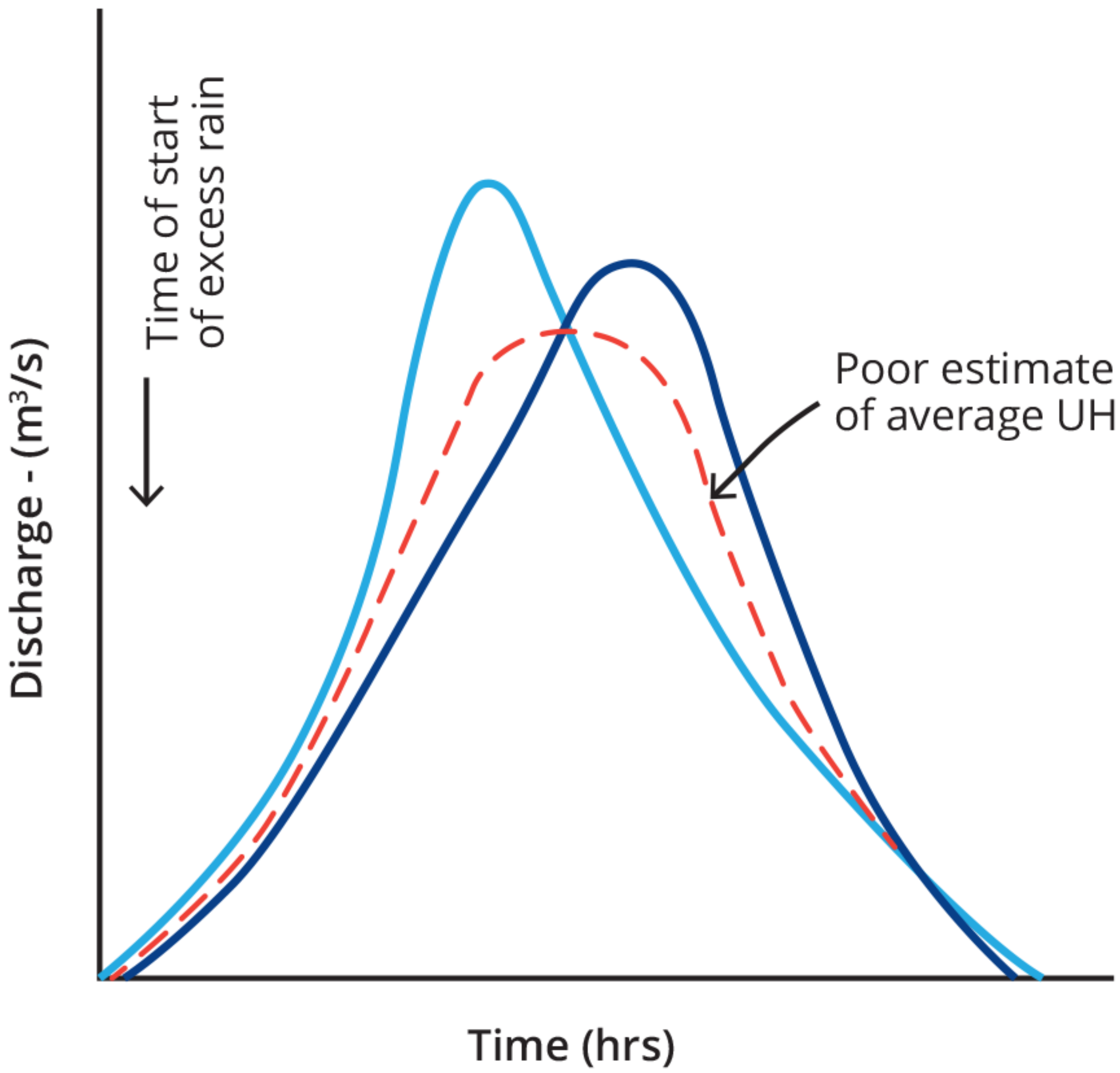


Figure 5.6.5. Poor Averaging of Unit Hydrographs

6.3.5. Synthetic Unit Hydrographs

The synthetic unit hydrograph approach provides a means of estimating unit hydrographs for ungauged catchments. In essence the approach involves estimating the parameters of the unit hydrograph from relationships between these parameters and the physical characteristics of the catchment. These relationships may be derived by considering a number of catchments in a reasonably homogeneous area (with similar climatic and geomorphologic characteristics) for which unit hydrographs have been derived from recorded rainfall and streamflow data. These relationships are empirical and as such cannot be expected to be universally applicable. In general their application should be restricted to the region in which the relationships were derived.

Of the various synthetic unit hydrograph approaches available, the only ones to have found widespread use in Australia are those based on the model of Clark (1945), commonly referred to as the Clark- Johnstone model. The Clark-Johnstone method has been simplified by Cordery and Webb (1974) to produce a model which is suitable for some limited applications.

The Clark-Johnstone model involves the translation of rainfall excess to the outlet and routing this translated flow through a lumped concentrated storage at that location. It has been used quite widely for synthetic unit hydrograph derivation and has been shown to be applicable to most of the east coast of Australia (Cordery et al., 1981). The basic assumption is that the shape of the unit hydrograph may be determined from two parameters, namely the base length of the time-area curve (C) and the catchment storage factor (K).

For more detailed description of these synthetic unit hydrograph approaches and examples of their application the reader is referred to Cordery (1987).

6.3.6. Limitations of the Unit Hydrograph Approach

The unit hydrograph is arguably the most direct way of characterising the response of a catchment to storm rainfall, as it takes account of all factors which influence the flood response to a particular rainfall excess input. The basic concept underlying the approach is simple to understand and easy to apply, being suited to both hand and spreadsheet calculations. The unit hydrograph approach can be applied with some confidence where its main assumptions are at least approximately satisfied: spatial uniformity of rainfall excess and linearity of catchment response.

However, the simplifying assumptions made in the unit hydrograph approach impose the following limitations for its application in many practical situations:

- Catchments generally respond in a non-linear fashion to rainfall excess inputs. This means that the travel time (or lag) varies with discharge or flood magnitude rather than being constant as implied by the linearity assumption. This limitation is particularly important when a significant degree of extrapolation is required from the magnitude of the observed events used in the derivation of the unit hydrograph to the magnitude of floods to be estimated.

- Particularly in larger catchments the spatial distribution of rainfall and rainfall excess is generally non-uniform, and in very large catchments heavy rainfall may only occur over part of the catchment. In such catchments the unit hydrograph approach would be likely to produce flood hydrographs that are biased both in terms of their peak flow and in their time to peak.

- In common with other lumped modelling approaches, the unit hydrograph approach produces only hydrographs at the catchment outlet and not at internal points of interest.

- The unit hydrograph approach and other lumped hydrograph estimation approaches are unsuitable for application in catchments where significant differences in flood response of different parts of the catchment require a more distributed modelling approach (e.g. urban catchments).

- Being based on a range of observed hydrographs for a particular catchment condition, the unit hydrograph approach is not suited to determine flood behaviour for changed catchment conditions (e.g. storage development, significant urbanisation or other major land use changes).

On the basis of these recognised limitations and the ready availability of more flexible runoff routing approaches (Book 5, Chapter 6, Section 4), the unit hydrograph approaches are not recommended for practical applications.

6.4. Runoff Routing Approaches

6.4.1. Introduction

The general term ‘runoff routing’ refers to flood hydrograph modelling approaches where a simplified conceptual representation is used to model the actual processes involved in the conversion of rainfall inputs to direct runoff (using a loss model – Book 5, Chapter 3), the contributions of baseflow (Book 5, Chapter 4) and the translation of runoff from different points in the catchment to a flood hydrograph at the catchment outlet (using a routing model – Book 5, Chapter 5).

The actual flood formation processes in a catchment are complex and highly distributed in nature. Direct runoff generated from storm rainfall in the upper parts of the catchment initially moves downhill as shallow overland flow and is modified by the effects of various forms of detention storage as it moves over the catchment surface. It is then gradually concentrated into minor drainage pathways and successively combined with baseflows and flows from other pathways. These flows eventually reach well defined water courses, creeks or rivers and move downstream, being combined with other tributary flows on their way to the catchment outlet.

A significant degree of simplification in the representation of the actual processes in models is made possible by the fact that catchments act on rainfall inputs as systems with a high degree of damping. This means that the streamflow hydrograph output at the catchment outlet does not reflect the ‘high frequency’ variations of the input in either the time or space dimensions. Similarly, small errors in modelling the various catchment processes may have little effect on the outflow hydrograph. This enables surprisingly accurate and useful results to be obtained from relatively simple models (Laurenson, 1975).

The different groups of models developed in Australia and in other countries have adopted different conceptualisations of the actual flood formation processes, with different levels of simplifications and assumptions in terms of:

- areal variability of runoff inputs over the catchment – lumped, semi-distributed and fully distributed models (see Book 5, Chapter 2)
- variation of routing processes from hillslopes to channel and floodplain reaches (Book 5, Chapter 2, Section 3)
- flood routing techniques (Book 5, Chapter 2, Section 3 and Book 5, Chapter 6, Section 5)
- model parameters and links with physical catchment characteristics.

However, all models are only approximations of reality and require care and expertise in their application and interpretation.

The major types of runoff routing models are described firstly in terms of how they deal with the distributed nature of the flood formation and the variation of routing processes along the flow path from the top to the bottom of the catchment (Book 5, Chapter 6, Section 4). The different model representations of the hillslope or overland flow phase of the hydrograph formation are introduced in Book 5, Chapter 6, Section 4, while Book 5, Chapter 6, Section 4 deals with flood routing in the various forms of channel, natural stream and floodplain segments of the catchment. Finally, Book 5, Chapter 6, Section 4 describes how areas of significant extra flood storage, such as natural lakes or swamps, extensive floodplain areas, reservoirs or flood retention/detention basins are modelled.

6.4.2. Representing the Distributed Nature of Flood Formation

6.4.2.1. Relative Importance of Overland Flow and Channel Routing Phases

As described in Book 5, Chapter 2, the detail of catchment representation in flood hydrograph estimation models ranges from lumped models to fully distributed models. This section focuses on semi-distributed or node-link type models introduced in Book 5, Chapter 2, Section 3.

A number of investigators (e.g. Robinson et al. (1995)) have examined the relative roles of 'hillslope' (overland flow) processes and channel routing in the modelling of hydrologic response. Their conclusions indicate that in *relatively small catchments* the emphasis should be on appropriate modelling of the *hillslope response* to rainfall inputs, while the spatial variation of rainfall inputs and hillslope responses is less important. Lumped models (Book 5, Chapter 6, Section 4) or semi-distributed models with relatively simplistic representation of spatial variations (Book 5, Chapter 6, Section 4) can thus produce acceptable flood hydrograph estimates.

In contrast, in *large (>1000 km^2) to very large (>10,000 km^2) catchments* the flood response is governed primarily by the *network geomorphology* and the *spatial distribution of runoff inputs*. In these larger catchments, and in catchments with significant storage development, it is thus important to model the distributed nature of runoff inputs and to give a realistic representation of the actual drainage network in node-link type models (Book 5, Chapter 6, Section 4).

In catchments of intermediate size, the overland flow and channel routing phases may be of similar importance in the overall catchment response to rainfall inputs, and it is thus desirable to model the flow routing in the overland flow segments separately from the flow routing in the drainage network segments. Similar considerations apply in catchments with significantly different land uses (e.g. urban or partly urbanised catchments), where runoff from subareas of different type is routed separately before routing in the pipe or channel network.

6.4.2.2. Lumped Runoff Routing Models

By definition, lumped models do not allow for the distributed nature of flood hydrograph formation in catchments. At the time of their development the application of hydrograph estimation models was restricted to hand calculations, requiring relatively simple models. However, lumped models can still play a useful role as a component of node-link type models, to represent the formation of runoff hydrographs from hillslope or overland flow segments as an input to the streamflow network.

Apart from the Time-Area Method (Book 5, Chapter 6, Section 2) and the Unit Hydrograph Method (Book 5, Chapter 6, Section 3), the best known lumped runoff routing models are the Clark model (Clark, 1945) and the Nash model (Nash, 1960) illustrated in Figure 5.6.6. The Clark model represents the translation and attenuation through a linear storage placed at the catchment outlet. By placing a number of linear storages in series and routing the rainfall excess input successively through this cascade of storages, the Nash model provides more flexibility in matching the routing response of the model to both the hydrograph translation and attenuation characteristics of the catchment.

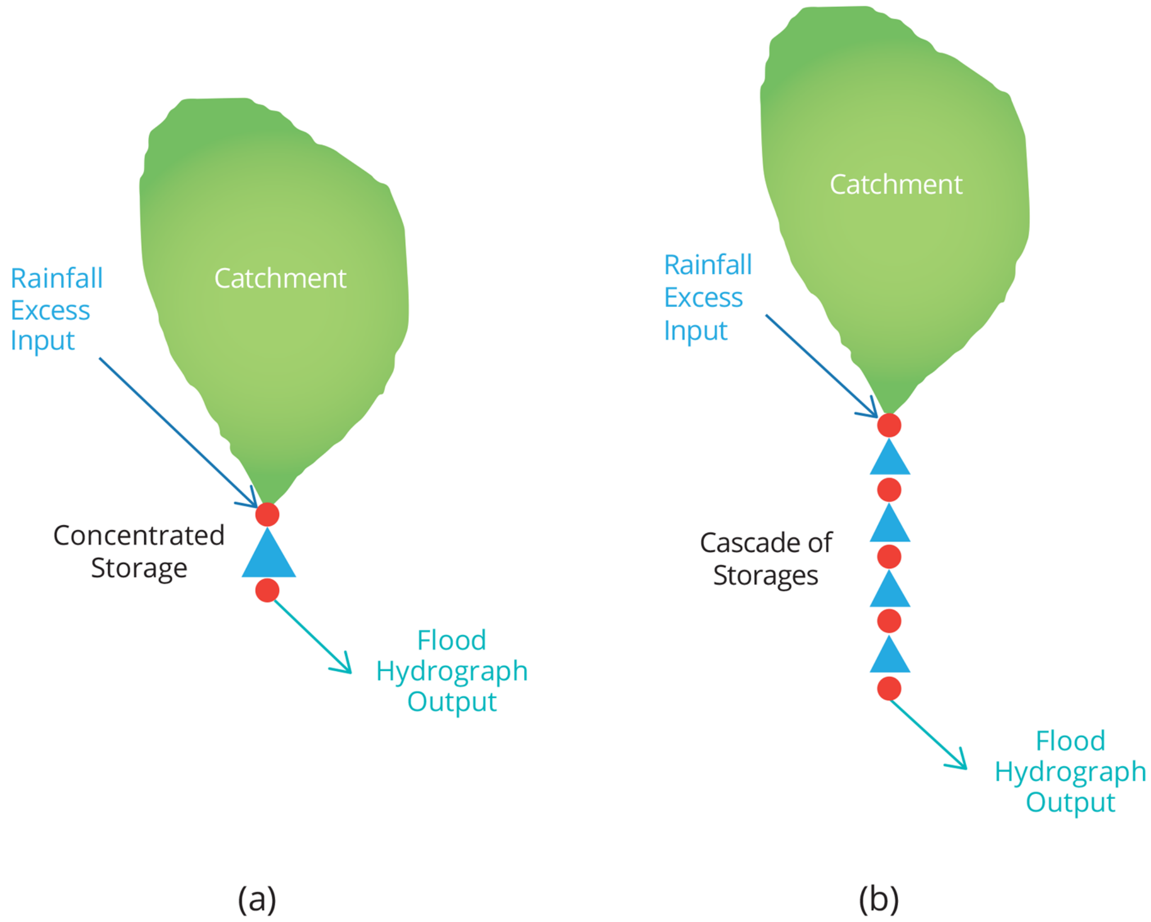


Figure 5.6.6. (a) Clark and (b) Nash models of runoff routing

The example below illustrates the application of a simple runoff routing model to estimate design flood hydrographs for an ungauged catchment of medium size. In most practical applications the simple Clark model would be replaced by a node-link type runoff routing model, implemented through one of the readily available runoff routing modelling systems referred to in Table 5.6.3 and Table 5.6.4. However, the steps of estimating model parameters and model inputs for the design application remain similar. The example also illustrates the application of the critical rainfall duration concept.

Example: Clark Runoff Routing Model

An ungauged catchment west of Melbourne has an area of 56 km^2 and a main stream length of 20 km. A flood hydrograph for a 1% AEP design event is required as the basis of defining flood prone land for a planned subdivision.

The Clark runoff routing model is being used to obtain an initial flood estimate for planning purposes. For the concentrated linear storage representing the catchment's routing characteristics in this model, the Muskingum routing equation (Equation (5.5.18)) can be written as

$$O_{n+1} = 2C_1\left(\frac{I_n + I_{n+1}}{2}\right) + C_3O_n \quad (5.6.8)$$

C_1 and C_3 are calculated from Equation (5.5.19) which for $X = 0$ simplifies to

$C_1 = \frac{0.5\Delta t}{K + 0.5\Delta t}$ and $C_3 = \frac{K - 0.5\Delta t}{K + 0.5\Delta t}$

The coefficient K of the can be calculated as a function of the main stream length to the catchment boundary (L) using the equation proposed by Cordery et al. (1981):

$$K = 0.70L^{0.57} \quad (5.6.9)$$

For a main stream length of 20 km this gives a value of K = 3.86 hours.

The critical rainfall duration for this catchment is not known *a priori*, so a range of rainfall durations from 3 to 24 hours are trialled to find the duration than produces the highest peak flow. The design rainfall depths and temporal patterns have been selected in accordance with Book 2.

Based on experience with neighbouring catchments, the following design loss values have been adopted: IL = 15 mm, CL = 2.5 mm/h.

The resulting hydrographs for the five different durations are shown in the Figure below. This indicates that the critical rainfall duration for this catchment is about 9 hours. The estimated peak flow for the 1% AEP design flood event is 155 m^3/s.

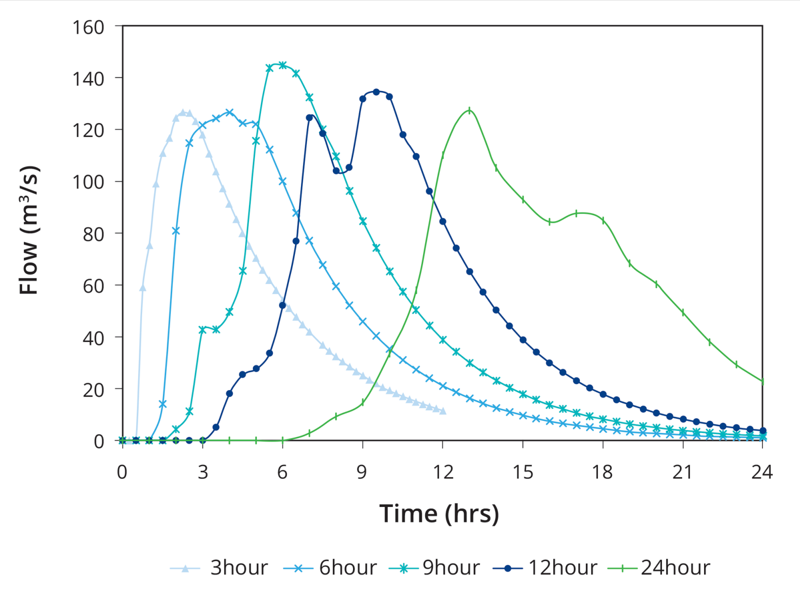


Figure 5.6.7. Clark Runoff Routing Example- resulting hydrographs for the five different durations

6.4.2.3. Simple Semi-distributed Models

The original Laurenson Runoff Routing Model – LRRM (Laurenson, 1962; Laurenson, 1964) illustrated in Figure 5.6.8 is an example of simple semi-distributed flood hydrograph estimation models. Similar to the time-area method, the LRRM divides the catchment into a number of sub-areas (typically 10) on the basis of equal travel time to the catchment outlet (isochrones). However, it assigns a separate storage to each of the subareas, and runoff from a sub-area is then routed through the series of downstream storages to the catchment outlet. The division into sub-areas and the detailed representation of travel time in the catchment allows the effects of spatial variation of catchment rainfall to be modelled explicitly.

The model uses nonlinear storages, with the relationship between discharge and storage represented by a power function, as discussed in Book 5, Chapter 5, Section 5 and expressed by Equation (5.5.32).

The catchment representation in the LRRM can be regarded as a linear network of ten rainfall input nodes, ten routing links (each with a nonlinear concentrated storage) and one output node. While the LRRM was originally conceptualised as a runoff routing model for the whole catchment, it is now more typically used as model to represent the routing of overland flow in a hillslope segment to a channel network node, e.g. in the XPRAFTS model (xpsolutions, 2016).

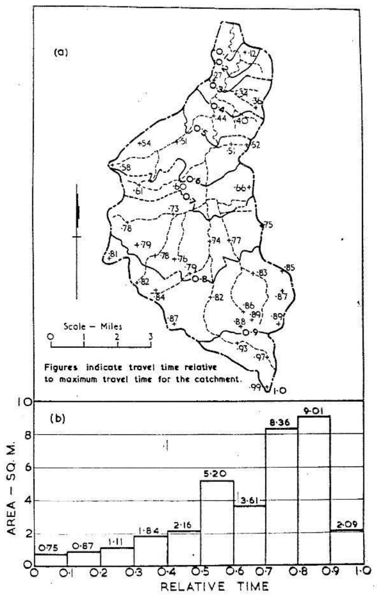


Figure 5.6.8. Original Laurenson Runoff Routing Model (South Creek catchment) (a) Isochrones of storage delay time (b) Time-area diagram

6.4.2.4. Node-link Type Models

The representation of the runoff routing process in a typical node-link type model is shown in Book 5, Chapter 6, Section 4. As shown on the left, the catchment is divided into a number of subareas within which the spatial rainfall distribution can be assumed to be uniform, and the actual drainage network is represented by a simplified network of the main tributary streams. The conceptual representation of this catchment in a runoff routing model is then by a set of nodes and links, as shown for the examples of RORB (centre) and WBNM (right).

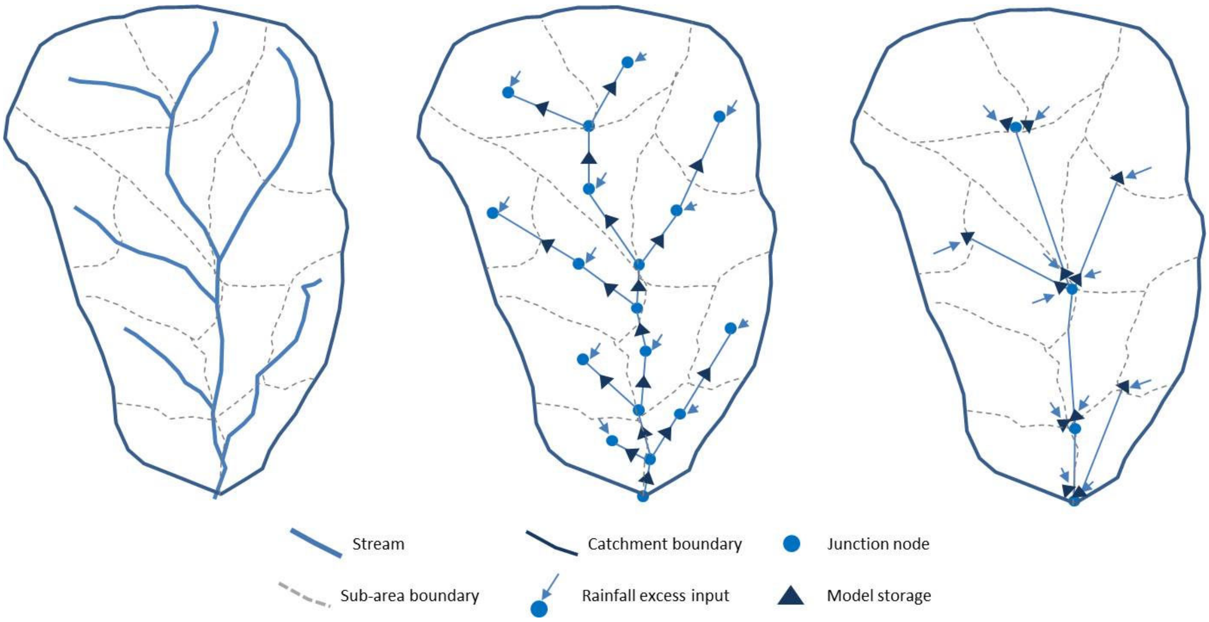


Figure 5.6.9. Node-link type representation of a catchment in runoff routing models: map view and schematic representation of node-link network in RORB and WBNM

As explained in Book 5, Chapter 6, Section 4, there are two distinctly different ways to convert the distributed rainfall excess inputs over a subarea into a runoff hydrograph at the subarea node (placed near the centroid of the subarea). These subarea runoff hydrographs are then routed progressively from one node through a routing link to the next node in the drainage network and eventually to the catchment outlet (Book 5, Chapter 6, Section 4). The concentrated or distributed storages used in these routing links are shown in Book 5, Chapter 6, Section 4 as small black triangles.

Some of the routing links receive the outflow hydrograph from an upstream link as well as a subarea runoff hydrograph. In these links the two hydrographs are combined before they are routed through to the next node in the network. At stream junction nodes the two (or more) tributary hydrographs are combined by simple addition.

The drainage network may also have a branched structure, where flows are diverted by natural or artificial features into a system of distributary or diversion channels, and these flows may or may not join up again with flows in the main channel. Most runoff routing modelling systems have the capability to represent the features controlling diversion and return flows.

If the catchment includes areas with significant extra flood storage, such as natural lakes or swamps, extensive floodplain areas, reservoirs or flood retention/detention basins, these can also be included in the drainage network as 'special storage' nodes or links, with separately defined storage and discharge characteristics (Book 5, Chapter 6, Section 4).

The baseflow contributions to the total flood runoff hydrograph (Book 5, Chapter 4) are typically modelled in a lumped fashion and added to the routed hydrograph at the catchment outlet. However, in more complex catchment situations with significant baseflow contributions it is desirable to model the distributed nature of baseflow contributions, by adding them at each runoff input node and routing the combined runoff hydrograph through the drainage network.

Figure 5.6.10 shows schematically how the different modelling components can be combined to convert the distributed rainfall inputs to a hydrograph at the catchment outlet.

The Book 5 chapters and sections providing more detail of the individual components are also indicated in the figure.

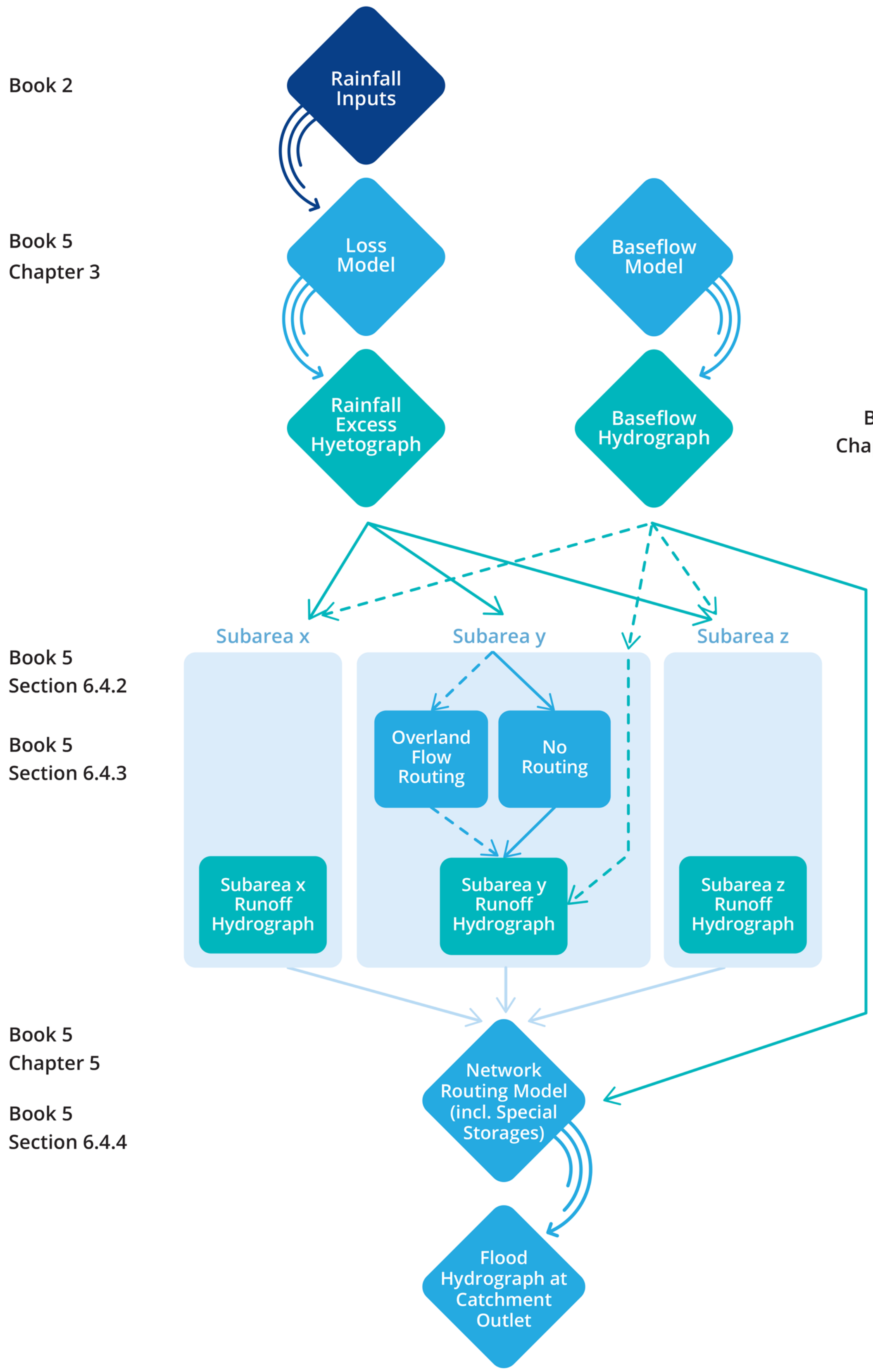


Figure 5.6.10. Components of a runoff routing model

Book 7 provides practical guidance on how these different model elements can best be used to represent the important features of a specific catchment.

6.4.3. Modelling of Hillslope (Overland Flow) Phase

The individual subareas of semi-distributed runoff routing models represent the runoff contributions from a relatively large part of a catchment (see Figure 5.6.9). Runoff from these subareas is the result of a complex and spatially varying set of processes which divide the rainfall inputs into a number of runoff components. These components may then follow different pathways to the model node used to represent their combined input to the modelled stream or channel network (Kemp, 2002). The mix and relative importance of different processes depends on the runoff production characteristics of a particular catchment. The scale of the modelled catchment also plays an important role, as with increasing size of the subareas a greater degree of averaging of the effects of different processes and modification of the runoff hydrograph though routing processes will occur,

As discussed in more detail in Book 4, Chapter 2, the runoff routing models routinely used in Australia employ very simplified representations of the processes involved in runoff production, generally dividing the rainfall inputs into a loss and rainfall excess component, and dealing only indirectly with baseflow contributions. The main distinction in the modelling of runoff from contributing areas is in how the routing of runoff within the subareas is treated.

6.4.3.1. Combined Subarea and Stream/Channel Network Routing

In this form of model conceptualisation the hillslope or overland flow phase of the hydrograph formation is modelled by simply converting the rainfall excess hyetograph into a direct runoff hydrograph:

$$1/3.6 * \text{Rainfall Excess (mm/h)} * \text{Catchment Area}\left(km^2\right) = \text{Runoff}\left(m^3/s\right) \qquad (5.6.10)$$

(6.4.1)

This subarea runoff hydrograph is input at a stream network node near the centroid of the subarea and then routed successively along the stream network to the catchment outlet. The modification of the hydrograph as it travels through the stream or channel network is generally modelled by a linear or nonlinear storage for each routing link (Book 5, Chapter 5, Section 4 and Book 5, Chapter 5, Section 5).

Examples of the application of the combined subarea and stream network routing approach include the RORB Runoff Routing Model (Laurenson et al., 2010) and the Basic Model version of URBS (Carroll, 2012).

The key feature of this runoff routing conceptualisation is that all the translation and attenuation effects experienced by the runoff inputs on their way to the catchment outlet have to be represented in the routing through the channel links.

The justification for the combined treatment of overland flow and stream/channel network routing is firstly that the separation of these two phases is somewhat arbitrary, in that the change from shallow distributed flow over hillslope surfaces to concentrated flow in water courses and streams is very gradual. Secondly, if the interest is only on hydrographs at the catchment outlet, the internal separation into different processes is of secondary importance, as long as the overall routing and delay characteristics of the catchment and their variation with flood magnitude can be adequately represented. As discussed in Book 5, Chapter 6, Section 4, this condition is likely to be satisfied in large catchments with relatively uniform land use.

The main limitation of this modelling approach is that it is designed to produce only an integrated catchment response hydrograph at or near the catchment outlet. As demonstrated by Yu and Ford (1989), this modelling approach does not satisfy the principles of 'self-consistency', as the storage parameter of an individual routing element depends on the size of the total catchment being modelled. While the model can output hydrographs at any internal node, hydrographs produced for the upper parts of the catchment are likely to show positive bias, as they tend to underestimate the degree of attenuation in the routing process.

6.4.3.2. Separate Overland Flow and Stream/Channel Network Routing

The node-link conceptualisation of the catchment in these runoff routing approaches is similar to the one shown in Figure 5.6.9 but the subarea rainfall excess inputs are now first routed through a storage element (or a kinematic wave routing element) to produce the runoff hydrograph inputs to the stream network. Different models use different methods to derive the runoff hydrographs from the hillslope segments, as summarised in Table 5.6.3.

Table 5.6.3. Methods used in different runoff routing modelling systems to derive the overland flow hydrograph

Method	Example	Reference	ARR Section
Time-area method	ILSAX, DRAINS	O'Loughlin and Slack (2014)	Book 5, Chapter 6, Section 2
Unit hydrograph convolution	HEC-HMS	HEC (2000)	Book 5, Chapter 6, Section 3
Cascade of non-linear storages	XPRAFTS	xpsolutions (2016)	Book 5, Chapter 5, Section 5 Book 5, Chapter 6, Section 4
Nonlinear storage routing	SWMM URBS WBNM2000	EPA (2016) Carroll (2012) Boyd et al. (2002)	Book 5, Chapter 5, Section 5 Book 5, Chapter 6, Section 4
Kinematic wave routing	HEC-HMS (Option)	HEC (2000)	Book 5, Chapter 5, Section 6 Book 5, Chapter 6, Section 4

The hydrograph formation methods used by the first three groups of modelling systems have been described in previous chapters, as indicated in the last column of Table 5.6.3, and the methods used to estimate their parameters are described in the relevant user manuals. The catchment conceptualisation used in nonlinear storage routing models and kinematic wave routing models warrants some additional discussion (Book 5, Chapter 6, Section 4 and Book 5, Chapter 6, Section 4 respectively).

The main advantage of modelling the overland flow phase separately is that this modelling approach can deal with different land uses in different parts of the catchment and changes to these land uses, such as substantial urbanisation. If the variation of the routing response with flood magnitude is quite different for the overland flow and channel routing phases, then a separate modelling approach lends itself better to extrapolation to Very Rare to Extreme flood events.

The disadvantage of the separate overland and channel flow routing approach is that it requires additional parameters to model the contributing area or overland flow component of the overall catchment routing process. If appropriate information is not available to allow separate calibration of the parameters to the overland and channel routing processes, then it may be more appropriate to use a combined routing approach, as described above.

More detailed approaches for modelling runoff from the overland flow segment have also been proposed and applied. The RRR model (Kemp and Daniell, 1996) allows for the generation of the runoff hydrograph by two or more different processes, with different losses and subarea routing delays being applied to each runoff component. Kemp (2002) postulated three different conceptual processes that can contribute to runoff at the subarea scale: baseflow, ‘slow flow’ and ‘fast flow’.

For urban catchments, further sub-division of the overland component on a spatial basis to allotment-size units and subsequent scaling up to the subareas has been proposed by Goyen (2005).

6.4.3.3. Nonlinear Storage Routing of Overland Flow

In the nonlinear storage routing models the attenuation and delay experienced by runoff from a subarea (i.e. a hillslope or overland flow segment) are modelled by routing through a nonlinear storage of the form

$$S = kQ^m \tag{5.6.11}$$

where the coefficient k is a delay or lag time parameter related to the lag parameter K in linear storage routing models (Book 5, Chapter 5, Section 4) by the following equation

$$k = KQ^{1-m} \tag{5.6.12}$$

and the exponent m expresses the degree of nonlinearity of the routing response. Exponent values in the range of 0.6 to 0.8 are typically used.

The coefficient k has been shown to be a function of catchment area A

$$k = CA^b \tag{5.6.13}$$

where C is a lag parameter for the subarea of the catchment. Equation (5.6.13) with an exponent value b = 0.57 is the basis of the subarea routing elements used in the WBNM2000 model (Boyd et al., 2002).

A similar expression for k is used in the URBS model, with an exponent value b = 0.5 (Carroll, 2012). URBS also allows adjustments of k for the degree of forestation of the subarea (increasing the value of k) and for the fraction of the subarea being urbanised (reducing the value of k).

The detailed form of the equations used and the adopted numerical solution method are described in the manuals of the respective runoff routing modelling systems.

6.4.3.4. Kinematic Wave Routing of Overland Flow

In the kinematic wave routing approaches the 'hillslopes' are generally conceptualised as two symmetrical rectangular planes of width *W*, inclined at slope *S*, discharging into the stream channel, as shown in Book 5, Chapter 6, Section 4. This simplified representation of the complex hillslope topography focuses on the average properties of the hillslope relevant to runoff generation rather than reflecting the actual physical processes at a smaller scale.

The overland flow discharging from the hillslope segment into the channel (at a node) is computed as flow in a wide rectangular channel, giving the simplified expression

$$q = \frac{s^{\frac{1}{2}}}{n} y^{\frac{5}{3}} \qquad (5.6.14)$$

where *q* is the discharge per unit width of the hillslope, *S* is the slope of the hillslope plane, *n* a roughness coefficient and *y* the average flow depth over the plane. It should be noted that the roughness coefficient *n* for overland flow over a particular type of surface and ground cover is typically higher than for channels (Bedient et al., 2008).

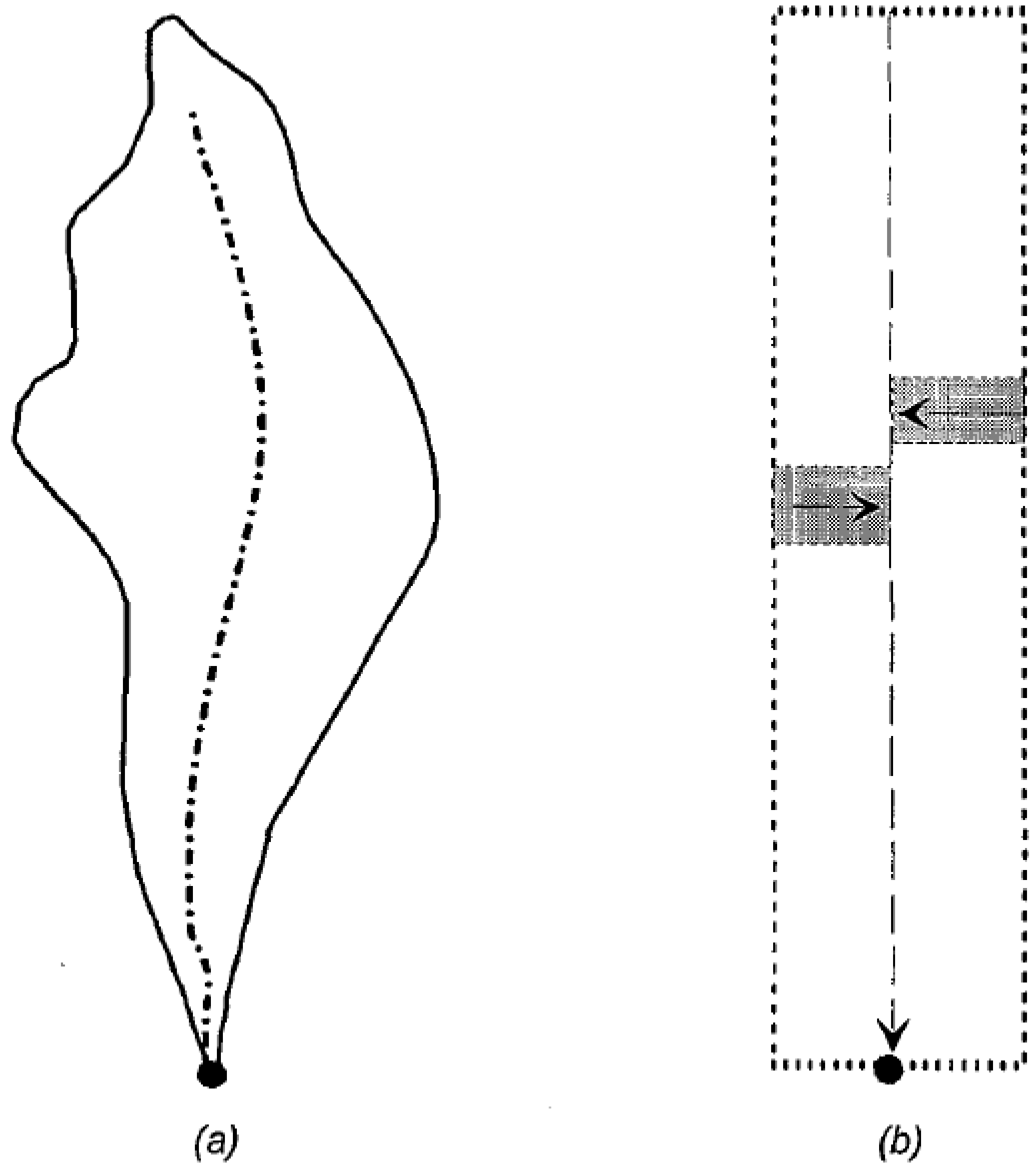


Figure 5.6.11. Hillslope representation in kinematic wave routing models (a) actual catchment (b) model representation (from HEC-HMS Manual)

Equation (5.6.15) is substituted in the appropriate form of the kinematic wave equation

$$\frac{\partial y}{\partial t} + \frac{\partial q}{\partial x} = i \qquad (5.6.15)$$

where is x is the distance in the direction of the overland flow and i the rate of rainfall excess (mm/h) on the hillslope plane.

While analytical solutions are available to solve the overland flow equations, runoff routing modelling systems such as HEC-HMS (HEC, 2000) use a numerical solution scheme to solve the kinematic wave equation for y at each time step. This value is then substituted into Equation (5.6.14) and the flow rate q per unit width (from one of the planes) is multiplied by twice the width of the hillslope (measured parallel to the channel) to determine the total overland flow hydrograph from the subarea.

A more detailed description of kinematic wave routing techniques and their application in a flood hydrograph estimation model (HEC-1 or HEC-HMS) is given in HEC (1993).

6.4.4. Routing Through Channel, Stream and Floodplain Reaches

As shown in Figure 5.6.9, the inflow hydrographs from the various catchment subareas have to be routed through the drainage network formed by the channel, stream and floodplain reaches. Table 5.6.4 gives an overview of the routing methods of varying complexity available in different runoff routing modelling systems, with corresponding references and links to the relevant sections of Book 5, Chapter 5 that describe these routing methods in more detail.

Table 5.6.4. Routing models used in different runoff routing modelling systems to route flows through channel, stream and floodplain reaches

Routing Model	Example	Reference	ARR Section
Simple lag, Lag and route	RORB (Option) SWMM (Option) XPRAFTS Option)	Laurenson et al. (2010) EPA (2015) xpsolutions (2016)	Book 5, Chapter 5, Section 3
Muskingum-Cunge Method (linear)	XPRAFTS	xpsolutions (2016)	Book 5, Chapter 5, Section 4
Concentrated non-linear storages	ILSAX, DRAINS RORB WBNM2000	O'Loughlin and Slack (2014) Laurenson et al. (2010) Boyd et al. (2002)	Book 5, Chapter 5, Section 5
Nonlinear Muskingum	SWMM URBS	EPA (2015) Carroll (2012)	Book 5, Chapter 5, Section 5
Kinematic wave routing	HEC-HMS (Option) ILSAX, DRAINS XPRAFTS (Option)	HEC (2000) O'Loughlin and Slack (2014) xpsolutions (2016)	Book 5, Chapter 5, Section 6
Dynamic wave routing	SWMM XPRAFTS (Option)	EPA (2015) xpsolutions (2016)	Book 5, Chapter 5, Section 6

The parameters of the simple lag and storage routing models are generally estimated by analysis of or calibration to observed hydrographs in the catchment being modelled, or by transfer of information from gauged catchments in regions with similar streamflow characteristics. In these methods it is generally assumed that the same parameter set applies to the different routing links, except for an adjustment to reflect the different time lag associated with routing reaches of different lengths.

In the Muskingum-Cunge Method, the kinematic wave and the dynamic routing methods the routing parameters can be determined from direct links with stream survey data and hydraulic flow characteristics, e.g. channel slope and hydraulic roughness (Equation (5.5.25) and Equation (5.5.26)). These parameters will thus vary naturally with the topographic and hydraulic characteristics of the routing reaches.

In the modelling systems using the nonlinear storage routing methods described in Book 5, Chapter 5, Section 5, the value of the exponent m in the nonlinear storage-discharge relationship found from calibration to observed hydrographs typically varies in the range from 0.6 to 0.8. These values imply that with increasing flood magnitude discharge increases more rapidly than storage. A value of m = 1.0 (linear storage) would imply that discharge and storage increase at the same rate).

The expected variation of the exponent with flood magnitude is particularly important when appropriate routing parameter values for the estimation of Very Rare to Extreme floods need to be selected. This question is discussed in more detail in Book 8, Chapter 5, Section 4.

As indicated in Book 5, Chapter 5, Section 4 and Book 5, Chapter 5, Section 5, the translation and attenuation effects of distributed storage, as modelled by the Muskingum-Cunge method, can also be represented by routing through a series of concentrated storages. Figure 5.6.12 illustrates the effect of successive routing of a rainfall excess hydrograph from subarea A through three concentrated nonlinear storages. The peak of the input hydrograph is progressively translated and attenuated on its movement along the channel network.

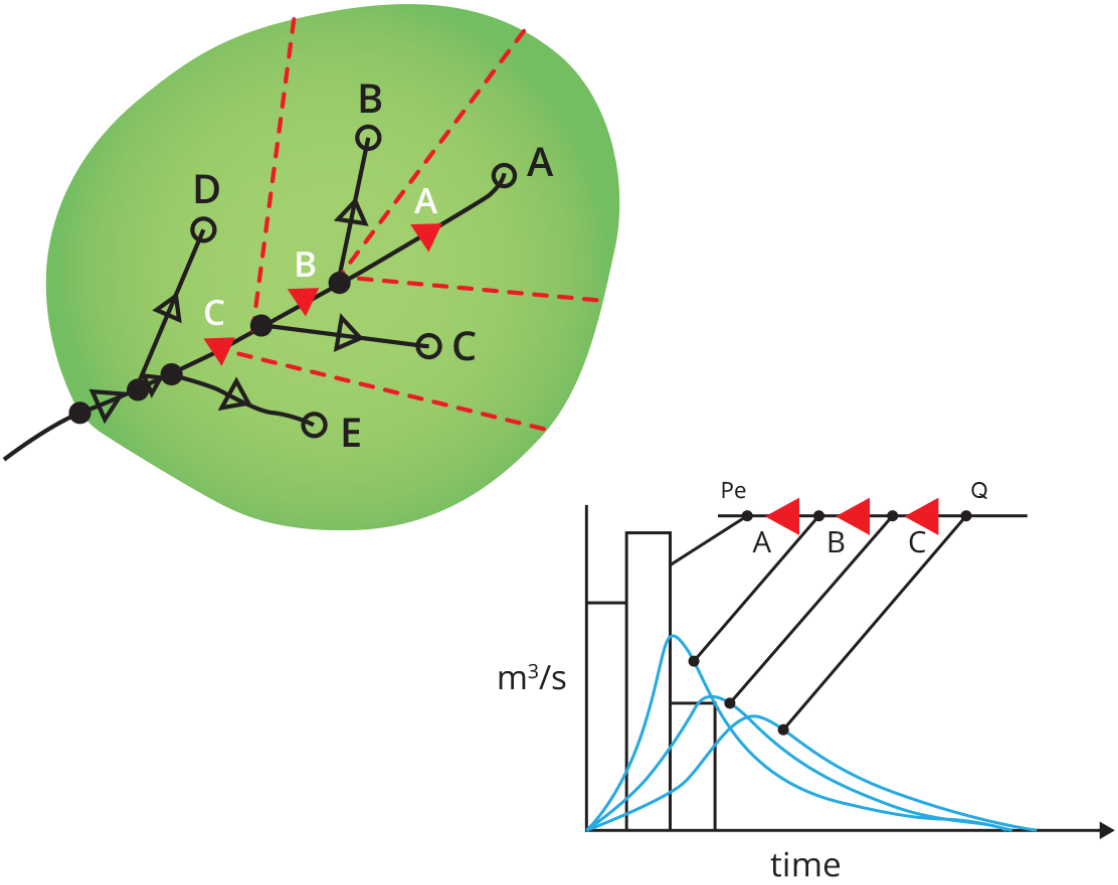


Figure 5.6.12. Routing of rainfall excess hydrograph through a series of nonlinear storages (after Laurenson et al. (2010))

The characteristic reach length criterion expressed by Equation (5.5.28) means that the degree of subdivision of the drainage network into sub-reaches represented by concentrated storages cannot be chosen arbitrarily. Too few or too many sub-reaches will make it difficult to accurately reflect both the translation and attenuation effects experienced by flood waves as they move through the drainage network of the actual catchment. Boyd (1985) has shown empirically that the optimum degree of subdivision of a catchment into subareas and routing reaches increases approximately with the square root of catchment area.

The different methods available to estimate the routing parameters for different runoff routing modelling systems are discussed in Book 7, Chapter 5.

6.4.5. Routing Through Special Storages

The methods used to route hydrographs through channel, stream and floodplain reaches assume similar routing characteristics in different routing links. If the catchment includes areas with significant extra flood storage (e.g. natural lakes or swamps, extensive floodplain areas, reservoirs or flood retention/detention basins), this assumption is no longer satisfied, and these special features will require separate representation in the model. They can be included in the node-link network as ‘special storage’ nodes or links, with separately defined storage-discharge relationships.

The approaches used to represent the routing effects through a special storage range from linear reservoir routing methods (Book 5, Chapter 5, Section 4) to non-linear storage routing methods (Book 5, Chapter 5, Section 5), with different methods being applied to define the S-Q relationship for the storage:

- a linear or nonlinear S-Q relationship derived by calibration to observed hydrographs (after the routing parameters of the normal channel routing links have been determined)
- a nonlinear S-Q relationship for a special channel or floodplain routing link determined from calculations of storage volumes in the link and corresponding flow through the link for different flood magnitudes, or from the analysis of hydraulic modelling results
- a nonlinear S-Q relationship for reservoir or pond determined by combining a stage-storage relationship (e.g. a reservoir storage capacity curve) with a stage-discharge relationship (e.g. a spillway rating curve)
- a set of S-Q relationships for regulated storages with information on the triggers for the application of the individual S-Q relationships
- separate specification of a stage-storage relationship together with details of the levels and hydraulic characteristics that determine the different forms of outflows from the storage (e.g. for flood detention basins)

Details of these different options are provided in the user manuals of the different runoff routing modelling systems.

6.5. Rainfall on Grid Modelling Approaches

6.5.1. Introduction

The following description of the 'rainfall-on-grid' or 'direct rainfall' modelling approaches to runoff routing is based mainly on the Stage1/2 report from ARR Revision Project 15 'Two-dimensional (2D) modelling (Babister and Barton, 2016)

In contrast to the traditional rainfall runoff modelling approaches, the rainfall-on-grid approaches do not require the specification of a node-link type representation of the catchment and its drainage network. Instead they use information from a digital elevation model and hydraulic modelling to define the drainage paths used by floodwaters on their way towards the catchment outlet. In the flatter parts of a catchment the drainage paths may thus change adaptively during a flood event.

The catchment is represented by a large number of grid cells, each with its individual rainfall input and runoff output – in other words, the cells act as the equivalent to subareas in node-link type models. However, the different scales of these basic catchment elements have important implications for the modelling of flood runoff, as they require different representations of the typical properties of the catchment elements. The non-linear nature of runoff generation means that the average response from many different small scale catchment elements cannot be expected to correspond to the lumped response of a subarea with average properties (e.g. a hillslope in a kinematic wave model).

The rainfall-on-grid approaches can be applied in two ways:

- the whole catchment is represented by a 2D grid, or

- a ‘hybrid approach’, where only the flatter parts of the catchment with more complex floodplain topography are represented by a 2D grid; they receive inflow hydrographs from the rest of the catchment produced by a traditional runoff routing approach.

The computational basis of the rainfall-on-grid approaches are the two-dimensional unsteady flow equations introduced in Book 6, Chapter 4, Section 7, or simplified forms of these equations.

Figure 5.6.13 shows how the generation of runoff from a grid cell is conceptualised. Different direct rainfall models vary in the degree of detail adopted in modelling the runoff generated on a grid element, as discussed in Book 5, Chapter 6, Section 5.

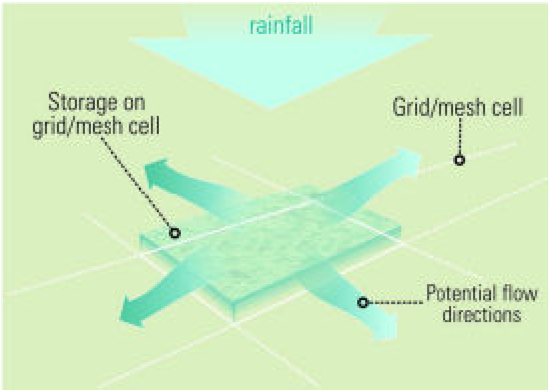


Figure 5.6.13. Conceptualisation of generation of runoff hydrograph from a grid cell

The successive routing of the runoff through other grid cells to the catchment outlet then uses the hydraulic routing approaches incorporated into the modelling package.

6.5.2. Modelling of Runoff from Individual Cells

Similar to traditional runoff routing models, runoff from a grid cell depends on the following factors:

- the area of the grid cell
- the rainfall depth
- the losses and
- the storage volume in the cell

However the amount and direction of outflow from the cell here also depends on additional factors:

- the hydraulic roughness of the cell
- the slope between neighbouring grid cells and
- the water level in neighbouring cells
- inflows from other cells

In principle the model can accept a detailed space-time distribution of the rainfall inputs but in practice limited data availability means that more discretised rainfall inputs need to be used.

The traditional loss models described in Book 5, Chapter 3 can be used but the way these losses are applied may vary in different models, and the traditional design loss values may thus not be directly applicable. Loss models that have a more direct physical basis could also be applied to reflect the varying infiltration, depression storage and transmission losses that reduce the volume of rainfall inputs but the required data and parameters estimates are not readily available. Finally, at least in theory, a groundwater model could be integrated to allow a consistent estimation of losses and baseflow contributions when modelling over an extending time period.

The topographic information included in the model means that the model can include relatively large depression storage areas which interact with losses. A process of 'pre-wetting' these storage areas (priming the model with an artificial rainfall burst to fill depression storages) may have to be used to prevent low bias in modelled flood hydrographs.

The model outputs are quite sensitive to the selection of the hydraulic roughness parameters. Because of the differences in conceptualisation and scale, these grid cell roughness parameters will generally be different from the values used in traditional channel hydraulics. Use of depth varying roughness parameters rather than constant ones may be necessary to reflect the changing hydraulic characteristics of catchment surfaces with flow depth.

The modelling of urban areas requires consideration of the impacts of a mix of different catchment surfaces, buildings, drainage systems and other infrastructure. The model resolution will generally not allow these features to be represented in detail, thus representative cell characteristics need to be adopted, using some form of averaging of the detailed urban area characteristics.

6.5.3. Advantages and Limitations of the Rainfall-on-Grid Approaches

As summarised in ARR (2012) the use of rainfall-on-grid approaches has following advantages and limitations:

Advantages of rainfall-on-grid approaches:

- Where a 2D model extends over the whole catchment, there is no need to develop and calibrate a separate hydraulic model, allowing seamless simulation of flood level outputs from rainfall inputs.
- Assumptions on sub-catchment delineation are not required as these are automatically defined by the topographic information for the cells.
- Overland flow is modelled directly.
- Drainage paths and flow direction do not need to be predefined, which makes the approaches particularly useful for runoff routing in flat areas and where catchment boundaries are not well defined.

Challenges and limitations of rainfall-on-grid approaches:

- Direct rainfall modelling is a new technique, with limited calibration or verification to gauged data. Caution and detailed checking is needed in the application of this approach.
- Potential significant increases in model run times. Hydrological models on their own generate peak flows significantly faster than direct rainfall, which facilitate their use in simulation frameworks that aim to ensure probability neutrality in the transformation of rainfall into floods (as discussed in Book 4, Chapter 3).
- Require digital terrain information. Depending on the accuracy of the results required, there may be a need for extensive survey data, such as aerial survey data.
- Insufficient resolution of smaller flowpaths may impact upon timing of routed flows. The smaller flowpaths higher up in the catchment may not be as well-represented by the 2D model as they may exist on a sub-grid scale. This may affect timing of runoff routing.
- The shallow flows generated in the direct rainfall approach may be outside the typical range of application of Mannings 'n' roughness parameters and will thus require special consideration.

ARR (2012) discusses these challenges/limitations and possible ways to deal with them in more detail.

It is important for both users of rainfall-on-grid models and their clients to realise that greater detail in the representation of catchment physiography can only be expected to translate to greater accuracy of flood estimation results if this is accompanied by appropriate representation of hydrologic flood formation processes at the adopted special scale. Given the present simple representation of such processes and the difficulties of realistically representing shallow overland flows, it is considered that at present the main value of rain-on-grid models is their ability to accommodate the influences of hydraulic controls on flow conditions.

At the current stage of development of these models and with the limited level of experience gained with their practical application, it is considered premature to recommend the general use of rainfall-on-grid models in these guidelines.

However, it is expected that further development and testing will allow rainfall-on-grid models to be more widely applied.

6.6. References

ARR (2012). Australian Rainfall and Runoff Revision Project 15: Two-dimensional modelling in urban and rural floodplains, Stage 1 and Stage 2 Report.

Babister, M. and Barton, C. (eds) (2016). Australian Rainfall and Runoff Support Document: Two dimensional modelling in urban and rural floodplains. Project 15

Bedient, P.B., Huber, W.C. and Vieux, B.E. (2008), Hydrology and floodplain analysis, ed. 4, Upper Saddle River: Prentice Hall.

Bernard, M.M. (1935), An approach to determinate streamflow, Transactions of the American Society of Civil Engineers, 100: 347-395.

Body, D.N. (1962), Significance of peak runoff intensity in the application of the unit graph method to flood estimation, Journal of the Institution of Engineers, Australia, 34: 25-31.

Boyd, M.J. (1982). A note on the Cordery-Webb synthetic unit hydrograph, IEAust. Civil Engineering Transactions, CE24(1), 107-108

Boyd, M.J. (1975), Hydrograph synthesis using mathematical models, Proceedings of the 1975 Hydrology Symposium, Institution of Engineers, Australia, NCP 75/3, pp: 117-121.

Boyd, M.J. (1985), Effect of catchment subdivision on runoff routing models, Civil Engineering Transactions, Institution of Engineers, Australia, 27: 403-410.

Boyd, M.J., Rigby, E.H., VanDrie, R. and Schymitzek, I. (2002), WBMN 2000 for flood studies on natural and urban catchments, In 'Mathematical Models of Small Watershed Hydrology and Applications', pp: 225-258.

Carroll, D.G. (2012), URBS (Unified River Basin Simulator) A Rainfall Runoff Routing Model for Flood Forecasting and Design. Version 5.00 Dec, 2012.

Clark, C.O. (1945), Storage and the unit hydrograph, Transactions of the American Society of Civil Engineers, volume 110, pp.1419-1488.

Cordery, I. (1987), The unit hydrograph method of flood estimation, Chapter 8 in Australian Rainfall & Runoff - A Guide to Flood Estimation, Barton: Engineers Australia.

Cordery, I. and Webb, S.N. (1974), Flood estimation in eastern New South Wales - a design method, Civil Engineering Transactions, Institution of Engineers, Australia, 16: 87-93.

Cordery, I., Pilgrim, D.H. and Baron, B.C. (1981), Validity of use of small catchment research results for large basins, Civil Engineering Transactions, Institution of Engineers, Australia, CE23: 131-137.

United States Environmental Protection Agency (2016), Storm water management model - reference manual, volume 1 - Hydrology (Revised), Washington: United States Environmental Protection Agency, available at: <http://nepis.epa.gov/Exe/ZyPDF.cgi?Dockey=P100NYRA.txt>.

United States Environmental Protection Agency (2015), Storm water management model - user manual version5.1, Washington: United States Environmental Protection Agency, available at: <http://nepis.epa.gov/Exe/ZyPDF.cgi?Dockey=P100N3J6.TXT>.

Goyen, A. (2005), Changes to urban stormwater management from ARR1958 to ARR2008? - 2005 Munro Oration, delivered at the 29th Hydrology and Water Resources Symposium, 20-23 February, Canberra.

HEC (2000), Hydrologic Modelling System HEC-HMS Technical Reference Manual, United States Army Corp of Engineers, Hydrologic Engineering Centre, available at: <http://www.hec.usace.army.mil/software/hec-hms/documentation/HEC-HMS_Technical%20Reference%20Manual_%28CPD-74B%29.pdf>.

HEC (1993). Introduction and application of kinematic wave routing techniques using HEC-1. US Army Corp of Engineers, Hydrologic Engineering Centre, http://www.hec.usace.army.mil/publications/TrainingDocuments/TD-10.pdf.

Hawken, R.W.H. (1921), An analysis of maximum run-off and rainfall intensity, Transactions of the Institution of Engineers, Australia, 2: 193-215.

Kemp, D.J. (2002), The development of a rainfall-runoff-routing (RRR) model. Thesis submitted to the Department of Civil and Environmental Engineering, University of Adelaide.

Kemp, D.J. and Daniell, T.M. (1996), A proposal for a rainfall runoff routing (RRR) model. I.E.Aust. Proceedings of the 23rd Hydrology and Water Resources Symposium, May, Hobart, pp: 15-20.

Laurenson, E.M. (1998). Hydrology CIV3263 Course Notes, Monash University, Department of Civil Engineering.

Laurenson, E.M. (1975), Streamflow in catchment modeling, Proceedings of Prediction in Catchment Hydrology - a national symposium on Hydrology, sponsored by the Australian Academy of Science, 25-27 November, Canberra.

Laurenson, E.M. (1962), Hydrograph synthesis by runoff routing, Research Report 66, Sydney: Water Research Laboratory, University of New South Wales.

Laurenson, E.M. (1964), A catchment storage model for a runoff routing, Journal of Hydrology, 2: 141-163.

Laurenson, E.M., Mein, R.G. and Nathan, R.J. (2010). RORB Version 6 Runoff Routing Program - User Manual.

Leopold, L.B., Wolman, M.G. and Miller, J.P. (1964), Fluvial processes in geomorphology, San Francisco: W.H. Freeman.

Nash, J.E. (1960), A unit hydrograph study with particular reference to British catchments, Proceedings of the Institute Civil Engineers, 17: 249-282.

O'Loughlin, G. and Stack, R. (2014), DRAINS User Manual - a manual on the DRAINS program for urban stormwater drainage system design and analysis, Available at: <http://www.watercom.com.au/DRAINS%20Manual.pdf>.

Pilgrim, D.H. (1977), Isochrones of travel time and distribution of flood storage from a tracer study on a small watershed, Water Resources Research, 13(3), 587-596.

Pilgrim, D.H. (1966a), Radioactive tracing of storm runoff on a small catchment - 1. Experimental technique, Journal of Hydrology, 4: 289-305.

Pilgrim, D.H. (1966b), Radioactive tracing of storm runoff on a small catchment - 2. Discussion of results, Journal of Hydrology, 4: 306-326.

Robinson, J. S., Sivapalan, M. and Snell J. D. (1995), On the relative roles of hillslope processes, channel routing, and network morphology in the hydrologic response of natural catchments, Water Resour. Res., 30(12), 3089-3101.

Ross, C.N. (1921), The calculation of flood discharges by the use of a time contour plan, Transations of the Institution of Engineers, Australia, 2: 85-92.

Sherman, L.K. (1932), Streamflow from rainfall by unitgraph method, Engineering News Record, 108: 501-505.

Titmarsh, G.W. and Cordery, I. (1991), An examination of linear and non linear flood estimation methods for small agricultural catchments, Australian Civl Engineering Transactions, 33(4), 233-242.

Yu, B and Ford, B.R. (1989). Self-Consistency in Runoff Routing Models. IEAust Civil Engg. Trans., CE31(1), 47-53.

xpsolutions (2016), Rafts: Urban and rural runoff routing application, Belconnen, available at: <http://xpsolutions.com/assets/dms/XPRAFTS-tech-desc.pdf>.

BOOK 6

Flood Hydraulics

Flood Hydraulics

Table of Contents

1. Introduction 1
1.1. Objectives and Scope 1
1.2. References 1
2. Open Channel Hydraulics 3
2.1. Introduction 3
2.2. General Characteristics of Open Channels 4
2.2.1. The one dimensional energy equations 10
2.3. Classification of Free Surface Flows 11
2.4. Uniform Flow and Critical Flow 12
2.4.1. Uniform Flow 12
2.4.2. Critical Flow 13
2.4.3. Bed Shear Stress 17
2.5. Uniform Flow Resistance Formulas 18
2.5.1. Manning Formula 18
2.5.2. Application of the Manning Equation 19
2.5.3. Factors Affecting the Manning Roughness Parameter 25
2.5.4. Chezy Formula 25
2.5.5. Application of the Chezy Equation 28
2.5.6. The link between between the Manning Roughness coefficent and the roughness height 28
2.5.7. Uniform Flow in Channels of Compound Cross-Section 30
2.6. Classification of the 1D Backwater and Drawdown Water Surface Profiles 35
2.7. Methods for Calculating Steady State Backwater and Drawdown Curves 38
2.7.1. Direct Step Method for Calculating Backwater and Drawdown Curves 39
2.7.2. Standard Step Method for Calculating Backwater and Drawdown Curves 40
2.7.3. Averaging Required in Water Surface Profile Calculations 41
2.8. One Dimensional Unsteady Flow Equations 42
2.8.1. Derivation of the Continuity Equation for Gradually Varied Unsteady Flow in an Open Channel 43
2.8.2. Derivation of the Momentum Equation for Gradually Varied Unsteady Flow in an Open Channel 43
2.8.3. Why is the time step so important? 45
2.8.4. Steady Flow Equations 47
2.8.5. Simplifying from Gradually Varied to Steady Uniform Flow 49
2.9. Numerical Modelling - Two Dimensional Models of Flood Flows 49
2.9.1. The Mass Equation 50
2.9.2. The Momentum Equations 50
2.9.3. Assumptions 51
2.10. Extension of the Equations for Modelling Applications 52
2.10.1. Extension of the Mass Equation 52
2.10.2. Extension of the Momentum Equations 52
2.10.3. Final Forms of the Equations 54
2.10.4. Modelling Requirements and Simplifications 54
2.11. Three Dimensional Flow Equations 56
2.12. Physical Modelling 59
2.12.1. The Basis for Physical Model Design 59
2.12.2. Model Scales 62
2.12.3. Distorted Models 62
2.12.4. Model Scales in Froude Models 63

2.12.5. Model Roughness in Froude Models 63
2.12.6. Mobile Bed Models 64
2.12.7. Stages of an Investigation by Physical Models 65
2.12.8. Physical Models Development And Non-dimensional Analysis 66
2.12.9. Model Criteria - Dimensional Analysis and Process Functions 66
2.12.10. Scale Effects 69
2.12.11. Some Issues for the Future 71
2.13. References 72
3. Hydraulic Structures 78
3.1. Introduction 78
3.2. Flood Bypass Channels 78
3.3. Control structures – Gates, weirs and flumes and spillways 81
3.3.1. Sluice gates and other control gates 81
3.3.2. Free Outflow 83
3.3.3. Drowned Outflow 84
3.3.4. Weirs and flumes 85
3.3.5. Spillways 86
3.4. Levees 90
3.4.1. Cutoffs 92
3.4.2. Riverside Blankets 92
3.4.3. Landside Seepage Berms 92
3.5. Culverts 93
3.5.1. Culvert Flow Principles 93
3.5.2. Inlet Design 97
3.5.3. Outlet Design 97
3.6. Bridge Waterways 102
3.6.1. Introduction 102
3.6.2. Energy Losses at Bridges 102
3.6.3. Modelling Approaches for Non-Standard Bridge Crossings 104
3.7. Floodways on Roads 110
3.7.1. Introduction 110
3.8. Scour 114
3.8.1. Scour at Bridges 114
3.8.2. Design for scour at bridges 122
3.8.3. Countermeasures for existing scour susceptible bridges 123
3.8.4. Scour protection for culverts 124
3.9. Flow Measurement Structures 125
3.9.1. Introduction 125
3.9.2. Rectangular Sharp-Crested Weir 126
3.9.3. V-Notch Sharp-Crested Weir 128
3.9.4. Submerged Weirs 129
3.9.5. Broad-crested Weirs and Long-Throated Flumes 130
3.9.6. Advantages 131
3.9.7. Disadvantages 131
3.9.8. Analysis 133
3.9.9. Uncertainty in Flow Measurement Structures 136
3.10. References 138
4. Numerical Models 140
4.1. Introduction 140
4.2. Development Stages for Numerical Hydraulic Models 141
4.2.1. Review of the Physical System 142
4.2.2. Selection of the Mathematical Model(s) 142

4.2.3. Selection of the Numerical Model ... 143
4.2.4. Development of the Site-Specific Model ... 143
4.3. Model Reliability ... 143
4.3.1. Model Verification ... 143
4.3.2. Model Calibration and Validation ... 144
4.4. Steady vs Unsteady Hydraulic Models ... 145
4.4.1. Steady Flow Models ... 145
4.4.2. Unsteady Flow Models ... 146
4.5. Types of Unsteady Flow Models ... 147
4.5.1. 1D Models ... 148
4.5.2. 2D Models ... 148
4.5.3. Coupled 1D/2D Models ... 148
4.5.4. 3D Models ... 149
4.5.5. 1D, 2D and 3D Model Limitations ... 149
4.5.6. CFD and other Non-Hydrostatic Models ... 150
4.6. 1D Unsteady Hydraulic Models ... 150
4.6.1. 1D Equations of Motion ... 151
4.6.2. Simplified Forms of the 1D Momentum Equation ... 153
4.6.3. Model Set-up ... 154
4.6.4. Data Requirements ... 157
4.6.5. Advantages and Limitations ... 159
4.7. 2D Unsteady Hydraulic Models ... 159
4.7.1. Model Equations ... 160
4.7.2. Assumptions ... 160
4.7.3. Other Forms of the Equations ... 161
4.7.4. Numerical Solution Procedures ... 161
4.7.5. Discretisation Errors ... 164
4.7.6. Model Applications ... 167
4.7.7. Site Specific Model Development ... 168
4.7.8. Advantages and Disadvantages ... 174
4.7.9. Coupled 1D/2D Hydraulic Models ... 175
4.7.10. Direct Rainfall Models ... 176
4.7.11. Limitations of 2D Hydraulic Models ... 177
4.8. Summary and Conclusions ... 177
4.9. References ... 178
5. Interaction of Coastal and Catchment Flooding ... 181
5.1. Introduction ... 181
5.2. Background to Flood Processes in Estuarine Areas ... 182
5.3. Flood Estimation Approaches for the Joint Probability Zone ... 183
5.4. Theory of Joint Probability ... 186
5.4.1. Joint, Marginal and Conditional Distributions ... 186
5.4.2. Representations of Univariate and Multivariate Extremes ... 188
5.4.3. External Dependence ... 191
5.4.4. Design Variable Method ... 194
5.4.5. Illustration of Joint Probability Concepts ... 197
5.5. The Design Variable Method ... 201
5.6. Worked Example 1 — Hawkesbury/Nepean River ... 208
5.7. Worked Example 2 — Nambucca River ... 213
5.8. Summary ... 220
5.9. References ... 220
6. Blockage of Hydraulic Structures ... 222
6.1. Introduction ... 222

6.1.1. Background and Scope ... 222
6.1.2. Limitations of the Procedure ... 222
6.2. Types of Structures and Drainage Systems ... 223
6.3. Factors Influencing Blockage ... 224
6.3.1. Overview ... 224
6.3.2. Debris Type and Dimensions ... 224
6.3.3. Debris Availability ... 226
6.3.4. Debris Mobility ... 227
6.3.5. Debris Transportability ... 227
6.3.6. Structure Interaction ... 228
6.3.7. Random Chance ... 228
6.4. Assessment of Design Blockage Levels ... 228
6.4.1. Overview ... 228
6.4.2. Appropriate Investigation ... 229
6.4.3. History of Blockage ... 230
6.4.4. Assessment Procedure ... 230
6.4.5. All Clear ... 237
6.4.6. Implementation ... 238
6.5. Hydraulic Analysis of Blocked Structures ... 238
6.5.1. Overview ... 238
6.5.2. Blockage Types ... 238
6.5.3. Blockage Mechanisms ... 239
6.6. Management of Blockage ... 241
6.6.1. Design Considerations ... 241
6.6.2. Retro-fitting Existing Structures ... 244
6.6.3. Debris control structures ... 244
6.7. Conclusion ... 245
6.8. References ... 245
7. Safety Design Criteria ... 246
7.1. Introduction ... 246
7.2. Flood Hazard ... 247
7.2.1. General Introduction ... 247
7.2.2. Flood Hazard Assessment ... 248
7.2.3. People Stability ... 250
7.2.4. Vehicle stability ... 254
7.2.5. Building Stability ... 257
7.2.6. ... 259
7.2.7. General Flood Hazard Curves ... 260
7.2.8. Isolation, Effective Warning Time, Rate of Rise and Time of Day ... 261
7.3. Examples of Hazard Assessment ... 262
7.3.1. Example - Warehouse Car Park ... 262
7.3.2. Flood Mitigation - Warehouse Car Park ... 264
7.3.3. Example - Detention Basin ... 270
7.4. Conclusions and Recommendations ... 272
7.5. References ... 272

List of Figures

6.2.1. Illustration of the river main channel and floodplains 4
6.2.2. The River Murray and its floodplain near Waikerie in South Australia, Courtesy of Martin Lambert 4
6.2.3. River bend showing areas of deposition and erosion and characteristic cross-section. 5
6.2.4. Parameters characterising flows in open channels 6
6.2.5. Variation of cross-sectional properties in natural channel 7
6.2.6. Cross-section velocity distribution in a small straight laboratory channel (velocities shown as a ratio of the mean) 8
6.2.7. Typical velocity distribution in a natural channel cross-section 8
6.2.8. Typical vertical velocity distribution 9
6.2.9. Variation of the total energy grade line across the compound channel section assuming a horizontal water surface. 10
6.2.10. Opposing forces acting on a fluid element down the channel cancel thereby producing uniform flow. 13
6.2.11. Roughness coefficient data for Esopus Creek with n = 0.030 (Page 34 in Barnes (1967)). 22
6.2.12. Esopus Creek 23
6.2.13. Relative height of the roughness projection elements and the thickness of the viscous sublayer. 28
6.2.14. Comparison between Equation (6.2.31) and the power law approximation presented in Equation (6.2.32) 29
6.2.15. Typical compound channel with floodplains of greater roughness than the main channel 30
6.2.16. Imaginary division of a compound channel assumed by Horton (1933) to give the same average velocity on the floodplains and in the main channel. 31
6.2.17. Vertical division of a compound channel into floodplain and main channel subsections 31
6.2.18. Alternative approaches to subdividing a compound channel cross-section. 32
6.2.19. Drawdown Water Surfaces 37
6.2.20. Lateral inflow 40
6.2.21. Control volume used to derive the gradually varying unsteady flow equations 43
6.2.22. Definition of symbols 49
6.2.23. Process Function Diagram for Friction 68
6.2.24. Data for Cut-throat flumes (a) Uncorrected, (b) Corrected for Scale Effects (after Keller (1984b)) 71
6.3.1. Schematic of Meander, Floodway, and weir 80
6.3.2. Types of Underflow Gates (a) Vertical (b) Radial (c) Drum 82

6.3.3. Schematic for flow under a Sluice Gate 83

6.3.4. Schematic of Drowned Vertical Sluice Gate 85

6.3.5. Schematic of USBR Type 2 Stilling Basin 89

6.3.6. Schematic of USBR Type 3 Stilling Basin 90

6.3.7. Example of incorrect and correct berm length according to existing foundation conditions (USACE, 2000) 93

6.3.8. Culvert Flow Types 96

6.3.9. Schematics of Improved Inlets 99

6.3.10. Examples of Culvert Outlets 100

6.3.11. Schematic of Flow at Unsubmerged Outlet 101

6.3.12. Energy Losses at a Bridge Site 103

6.3.13. Schematic of a Perched Bridge 104

6.3.14. Schematic of a Submerged Bridge 105

6.3.15. Schematic of a Skewed Bridge 106

6.3.16. Schematic of Parallel Bridges 107

6.3.17. Schematic of Bridge with Multiple Openings 109

6.3.18. Illustration of Divided Flow Approach 110

6.3.19. Floodway flows (DTMR) 113

6.3.20. Some Effects of Scour 115

6.3.21. Development of Clear-water and Live-bed scour with Time 116

6.3.22. Contraction scour (QDTMR, 2013) 118

6.3.23. Vertical contraction scour (QDTMR, 2013) 119

6.3.24. Schematic of Local Scour at a Bridge Pier 120

6.3.25. Schematic of Abutment Scour 121

6.3.26. Schematic of Flow Over Rectangular Sharp-Crested Weir 126

6.3.27. Schematic of Flow Over V-notch Sharp-Crested Weir 129

6.3.28. Schematic of Submerged Weir 130

6.3.29. Schematic of Long-Throated Flume 132

6.3.30. Flow Profile Through a Long-Throated Flume 133

6.3.31. C_v Relationship for Rectangular Cross-Sections 136

6.4.1. Stages in Numerical Hydraulic Model Conceptualisation and Development 142

6.4.2. Storage Effects on the Rating Curve; (a) Flood Hydrograph, (b) Corresponding Rating Curve (Cunge et al., 1980) 147

6.4.3. The main dependent variables for the 1D model equations 151

6.4.4. The Computational Grid 155

6.4.5. Schematisation of a Channel and Structure Grid 156

6.4.6. Different Types of Flood Plain Flow 157

6.4.7. Schematisation of a Simple River Channel/Floodplain Branch System 157

6.4.8. A Branched 1D Model Network of the Lindsay and Murray River Systems 158

6.4.9. Lindsay River 159

6.4.10. Example of a Computational Grid 165

6.4.11. Figures showing (a) flow separation and eddy formation, and (b) suppression of flow separation with numerical eddy viscosity form a first order upwind scheme 167

6.4.12. Showing (a) poor resolution of a sinuous flow path on a coarse square grid, and (b) improved, but still relatively poor resolution at a finer grid scale. 169

6.4.13. Showing (a) improved resolution of a sinuous flow path using a flexible mesh, relative to (b) a fine square grid. 170

6.4.14. Topography of The Lindsay and Murray River System 175

6.4.15. Example of a Coupled 1D/2D Model of the Lindsay River System 176

6.5.1. Schematic of a longitudinal section of an estuary, which shows two hypothetical water levels: the level obtained by assuming that fluvial floods will always coincide with storm tides of the same exceedance probability (upper curve); and the level assuming fluvial processes and ocean processes are completely independent and thus will almost never coincide (lower curve). 181

6.5.2. Timing factors affecting the magnitude of a flood in the joint probability zone 182

6.5.3. Joint, Marginal and Conditional Probability Density Functions 188

6.5.4. Three Representations of 'Extreme Values' Following Different Extreme Value Methods (After (Zheng et al., 2014b)) 190

6.5.5. (a) Pairwise Plot of Daily Maximum Storm Tide and Daily Rainfall; (b) Application of Marginal Thresholds (Based on the 1% Daily Exceedance Probability for Each Margin), with Events Below the Radial Threshold r_0 Shaded in Blue; (c) Transformation of Events to Unit Fréchet Scale; and (d) Fitting the Joint Probability Distribution 193

6.5.6. The Relationship Between the Dependence Parameter and the Number of Joint Extreme Events per 10 000 days 194

6.5.7. Exceedance Probabilities Obtained from a Univariate Analysis (top panel) and a Bivariate Analysis (bottom panel) 196

6.5.8. Illustrating the Probability of Two Independent Events Z = X>x or Y>y 198

6.5.9. Illustrating the Probability of Two Independent Events Z = X>x and Y>y 199

6.5.10. Conceptual Diagram (a) Probability of Floods Caused by Either a Significant Rainfall Event or a Significant Storm Tide Event, and (b) Additional Probability of Floods Produced by Combinations of Smaller Rainfall and Storm Tide Events 201

6.5.11. The Design Variable Method 202

6.5.12. Pre-Screening Step, which Involves Calculating the Outer Envelope of the Possible Flood Levels. 204

6.5.13. Dependence Parameter (α) Map for the Basins of the Australian Coastline - shorter than 12 hours, 12 to 48 hours, and 48 to 168 hours 205

6.5.14. Interim Approach to Account for the Effects of Climate Change 208

6.5.15. Interpolated Contours for Olga Bay (left) and Spencer (right) Corresponding to the Water Levels in Table 6.5.11 and Table 6.5.12. 212

6.5.16. Water Levels at Olga Bay (left) and Spencer (right) Corresponding to Cases of Complete Dependence, Complete Independence and the Best Estimate when α=0.9 212

6.5.17. Longitudinal Comparison of 1% AEP and 10% AEP Water Levels 213

6.5.18. Water Levels at Macksville Corresponding to Cases of Complete Dependence, Complete Independence and the Best Estimate when α=0.95 215

6.5.19. Comparison of Observed Water Levels at Macksville with Range of Estimates from Design Variable Method from Complete Dependence to Complete Independence 216

6.5.20. Comparison of Observed Water Levels to 90% Confidence Limits from Generalised Extreme Value Distribution and Design Variable Method 217

6.5.21. Comparison of Observed Water Levels at Macksville to Best Fit Estimates from Design Variable Method Assuming Correction to Frequent Rainfall AEPs 218

6.6.1. Series of Floodplain Culverts 242

6.6.2. Floodplain Culvert 242

6.6.3. Debris Deflector Walls 242

6.6.4. Post Flood Collection of Debris on Top of Deflector Walls 243

6.6.5. Sediment Training Walls Incorporated with Debris Deflector Walls (Catchments & Creeks Pty Ltd) 243

6.6.6. Multi-Cell Culvert with Different Invert Levels 244

6.6.7. Debris Deflector Walls and Sediment Training Wall Added to Existing Culvert 244

6.7.1. Example of a Flood Study Depth Map (Smith andWasko (2012)) 249

6.7.2. Comparison of Provisional Flood Hazard Estimates from Numerical Models at Differing Grid Resolutions (after Smith and Wasko, 2012) 250

6.7.3. Typical Modes of Human Instability in Floods (Cox et al., 2010) 251

6.7.4. Safety Criteria for People in Variable Flow Conditions Cox et al. (2010) 253

6.7.5. Typical Modes of Vehicle Instability (Shand et al., 2011) 254

6.7.6. Interim Safety Criteria for Vehicles in Variable Flow Conditions (Shand et al., 2011) 256

6.7.7. Comparison of Building Stability Curves 257

6.7.8. Comparison of Updated Hazard Curves (after Smith et al. 2014) 259

6.7.9. Combined Flood Hazard Curves (Smith et al., 2014) 260

6.7.10. Flood Depth Map From Numerical Model Output (Courtesy WMAwater Pty Ltd) . 263

6.7.11. Flood Hazard Classification From Numerical Model Output (Courtesy WMAwater Pty Ltd) 264

6.7.12. Schematic of Proposed Warehouse Development 265

6.7.13. 1% AEP Flood Depth Map – Existing Site 266

6.7.14. 1% AEP Provisional Flood Hazard Map – Existing Car Park 267

6.7.15. Comparison of Car Park Cross Sections (A-A) 268

6.7.16. 1% AEP Flood Depth Map – Revised Car Park 269

6.7.17. 1% AEP Provisional Flood Hazard Map – Revised Car Park 270
6.7.18. 1% AEP Flood Depth - Proposed Flood Detention Basin 270
6.7.19. 1% AEP Provisional Flood Hazard Map - Proposed Flood Detention Basin 271
6.7.20. Basin Overflow Spillway – Flood Depth 271
6.7.21. Provisional Flood Hazard Map – Basin Overflow Spillway 272

List of Tables

6.2.1. Values of Roughness Coefficient n for different channel conditions (Sellin 1961) 19

6.2.2. Valid Manning ‘n’ Ranges for Different Land Use Types 24

6.2.3. Values of the roughness projection height k_s and Manning n for straight, clean pipes concentrically jointed. 27

6.2.4. Gradually varied flow classification system (modified from table on p35 of Fenton (2007)) 36

6.2.5. Comparison of the direct step and standard step methods 38

6.3.1. Culvert Flow Characteristics 96

6.5.1. Comparison design flood estimation methods in the joint probability zone 185

6.5.2. Advantages and Disadvantages of Alternative Representations of Joint Extremes(based on Zheng et al (Zheng et al., 2014b)) 191

6.5.3. Worked Examples of the Probability of Two Independent Events Z = X>x or Y>y 198

6.5.4. Worked Examples of the Probability of Two Independent Events Z = X>x and Y>y 199

6.5.5. Calculating the Probability of Two Independent Events Z = X>x and Y>y, When Adding the Constraint that Both Events Must Occur on the Same Day 200

6.5.6. Flood Levels of Different Combinations of Rainfall and Storm Tide in Terms of Annual Exceedance Probability, for a Particular Storm Burst Duration. Only the highlighted cells need to be evaluated. 203

6.5.7. Flood Levels of Different Combinations of Rainfall and Storm Tide in Terms of Annual Exceedance Probability with a Particular Storm Burst Duration 205

6.5.8. Model-Derived Water Levels (mAHD) for Given Pairs of Tide and Rainfall Boundary Input Conditions for a Cross-section Located at Liverpool (Chainage–80 300) 209

6.5.9. Pre-Screen Analysis Pairs at Olga Bay (Chainage–20 400) 210

6.5.10. Pre-Screen Analysis Pairs at Spencer (Chainage–34 700) 210

6.5.11. Model-Derived Water Levels (mAHD) for Given Pairs of Storm Tide and Rainfall Boundary Input Conditions for a Cross-section Located at Olga Bay (Chainage–20 400). 210

6.5.12. Model-Derived Water Levels (mAHD) for Given Pairs of Storm Tide and Rainfall Boundary Input Conditions for a Cross-section Located at Spencer (Chainage–34 700) 211

6.5.13. Pre-Screening Analysis Pairs for Macksville 213

6.5.14. Flood Levels for Various Combinations of Rainfall and Tide Levels at Macksville (Pacific Highway Bridge) Nambucca River 214

6.5.15. Hydraulic Response Table for cell (294, 200) 219

6.6.1. Debris Availability - in Source Area of a Particular Type/Size of Debris 232

6.6.2. Debris Mobility - Ability of a Particular Type/Size of Debris to be Moved into Streams 232

6.6.3. Debris Transportability - Ability of a Stream to Transport Debris Down to the Structure 233

6.6.4. 1% AEP Debris Potential 233

6.6.5. AEP Adjusted Debris Potential 234

6.6.6. Most Likely Inlet Blockage Levels - B_{DES}% 234

6.6.7. Likelihood of Sediment Being Deposited in Barrel/Waterway (HML) 235

6.6.8. Most Likely Depositional Blockage Levels – B_{DES}% 235

6.6.9. Likely Blockage Timing and Extents 240

6.7.1. Flow Hazard Regimes for People (Cox et al., 2010) 252

6.7.2. Interim Flow Hazard Regimes for Vehicles (Shand et al., 2011) 256

6.7.3. Combined Hazard Curves - Vulnerability Thresholds (Smith et al., 2014) 261

6.7.4. Combined Hazard Curves - Vulnerability Thresholds Classification Limits (Smith et al., 2014) 261

Chapter 1. Introduction

Martin Lambert

Chapter Status	Final
Date last updated	14/5/2019

1.1. Objectives and Scope

Much of Australian Rainfall and Runoff is dedicated to the determination of peak discharge, discharge hydrographs and flood volumes. This chapter deals with translating these discharge estimates into flood levels, flow velocities and the extent of flood inundation needed to determine flood damage and flood hazard. The importance of the inter-relationship between hydrology and hydraulics should not be under-estimated and there is often a blurring of the boundaries between the two areas. For example, the flood routing used to obtain a flow hydrograph and shallow flow surface runoff expressions are both founded in hydraulic engineering principles. In recent approaches to urban flooding, rainfall is a direct input to the hydraulic numerical grid hence forcing the hydraulic model to deal with the hydrology and the hydraulics simultaneously.

The primary objective of this book is to provide a document which provides background information to assist practitioners to carry out calculations or hydraulic investigations related to free surface flows. The needs for such calculations or investigations may be related to floods (inundation levels, concentration of flows to endanger life, the power of flood flows to threaten or impair structural integrity or even wash away structures, and bed level scour), stormwater disposal, water supply distribution systems and sewerage collection systems (the installation of new systems or augmentation of existing systems).

The present document concentrates on free surface flows. Textbooks are available which cover the basics of open channel flow. The traditional texts in this area are Henderson (1966) and Chow (1959) but more modern books include Chaudhry (2007) and Sturm (2009). Pipe flow is covered in fluid mechanics textbooks like Streeter and Wylie (1981) and Elger et al. (2014). There are numerous journals which publish research dealing with open channel, pipe flow and river flood flows and these include the ASCE - Journal of Hydraulic Engineering, IAHR - Journal of Hydraulic Research and the Journal of Flood Risk Management. Those practicing and working in this area are encouraged to keep abreast of new developments as this is an area that is evolving with increases in computational power.

1.2. References

Chaudhry, M.H. (2007), Open Channel Flow, Springer Publishing Company.

Chow, V.T. (1959), Open-Channel Hydraulics. McGraw-Hill Book Company, New York.

Elger, D.F. Williams, B.C. Clayton T. Crowe, C.T. and Roberson, J.A. (2014), Engineering Fluid Mechanics, 10th Edition.

Henderson, F.M. (1966), Open Channel Flow, Macmillan Publishing Co., New York.

Streeter, V.L. and Wylie E.B. (1981), Fluid Mechanics. McGraw-Hill Ryerson Ltd., Singapore.

Sturm, T. (2009), Open Channel Hydraulics Sturm, McGraw-Hill Water Resources and Environmental Engineering Series, New York.

Chapter 2. Open Channel Hydraulics

Martin Lambert, Bruce Cathers, Robert Keller

Chapter Status	Final
Date last updated	14/5/2019

2.1. Introduction

All hydraulic flows are three dimensional in nature and involve complex turbulent flow motion. This is not unlike hydrology where the three dimensional and turbulent nature of atmospheric flows create temporal and spatial variability in rainfall and runoff which must be dealt with. The type of hydraulic computations to be undertaken will depend on the problem to be examined and on the data that is available. As a result, some thought is needed to determine the appropriate analysis techniques. For example, an unsteady flow analysis will not be possible if only peak discharges are known.

Free surface flows are driven by gravity and resisted by shear forces on the channel bed and drag forces on objects such as vegetation and obstructions. The free surface that exists in open channel flow means that the flow depth and flow area will most likely change with distance and/or time. In contrast, closed conduits that are operating under pressure have a fixed cross-sectional area and are driven by the pressure gradient. A combination of free surface flow in conjunction with closed conduit flow is not uncommon. Hydraulic modelling for the purposes of flood estimation typically assumes that the flow is composed of water that is incompressible but the reality maybe that it is sediment or debris laden and in some cases multi-phase.

Hydraulic computations are usually carried out to determine flood characteristics such as:

- flow depths or water levels (e.g. for recorded floods or synthesised floods of a particular magnitude),
- flow velocities (including flow direction),
- flood or inundation extent,
- the timing of transport network disruptions,
- sediment scour, and sometimes also,
- energy losses, be they :

i. friction losses which are cumulative and can be significant over long distances or

ii. local losses such as those which occur due flow constrictions and expansions imposed by hydraulic structures or flow around bends).

Hydraulic computations can be undertaken using a range of analytical or numerical model approaches often involving spreadsheets or computer programs (freeware or proprietary software). The level of detail in which the calculations are completed will depend upon the nature of the hydraulic investigation (including the risks to life and property and the complexity of the flows) and the availability of time and data to undertake the investigation.

In some circumstances, it is desirable to make recourse to physical models rather than numerical models.

2.2. General Characteristics of Open Channels

Open channels can be natural or man-made. The cross-sections of natural channels are irregular, usually broader than they are deep and often consisting of a deeper in-bank channel as well as a shallower over-bank area (floodplain). These are often referred to as compound channels. During floods, the flow frequently leaves the in-bank channel and enters the over-bank area as shown in Figure 6.2.1. An illustration of a complex floodplain associated with the River Murray in South Australia is shown in Figure 6.2.2. The increase in roughness on the floodplain due to vegetation means that the floodplain flows are typically shallower and slower than the flow in the main channel. This can lead to a complex interaction between the two flows and in meandering channels the flow direction may be different from the main channel leading to additional interaction and momentum exchange.

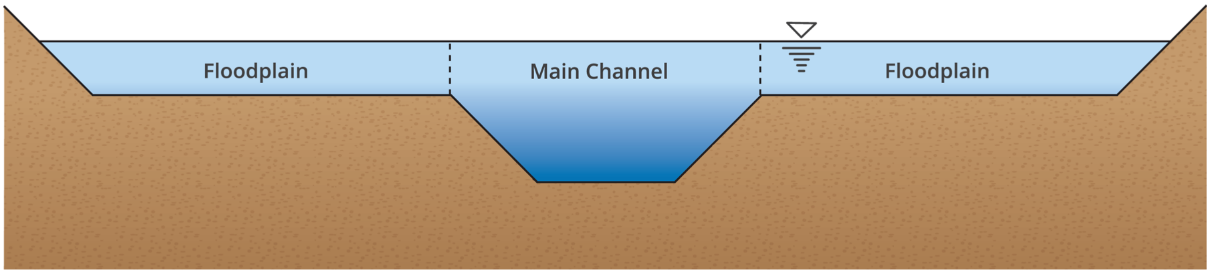


Figure 6.2.1. Illustration of the river main channel and floodplains

Figure 6.2.2. The River Murray and its floodplain near Waikerie in South Australia, Courtesy of Martin Lambert

At bends in the water course, natural cross-sections are asymmetrical; they tend to be deeper on the outside of the bend due to the effect of helicoidal secondary currents which tend to scour the outside of the bend and deposit sediment at the inside of the bend as illustrated in Figure 6.2.3.

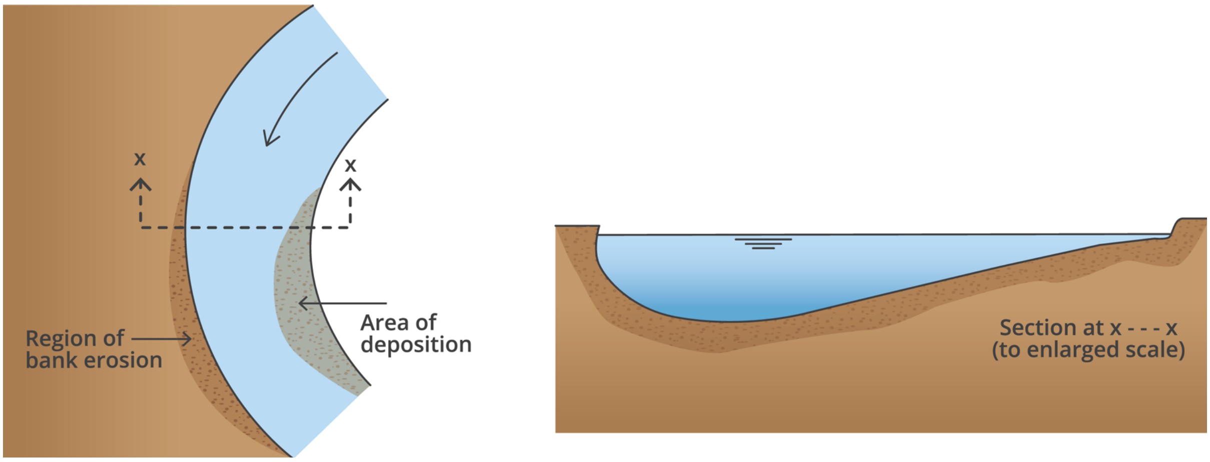


Figure 6.2.3. River bend showing areas of deposition and erosion and characteristic cross-section.

The cross-sections of man-made channels are usually rectangular, trapezoidal, triangular or circular. Stormwater channels often have a small deeper indentation along the centreline of the channel for easy cleaning and containing low flows.

Since the channel roughness affects the flow, it is important to be able to quantify the roughness. The roughness in channels is determined by the materials from which the channel is cut or made, including any vegetation which is growing (or lodged) in the channel. In man-made channels, the roughness must also include the effects of any jointing between panels, slabs or pipes. In channels with significant sediment transport, the roughness maybe changing with flow as the bedforms change in dimension. The challenge of determining the appropriate roughness to use in flood level computations should not be under-estimated and is often based on experience and should be validated or calibrated where possible.

Apart from channel roughness, the main parameters associated with the channels are:

i. the cross-sectional area (measured in a cross-section at right angles to the flow direction A),

ii. the depth y (especially for man-made channels),

iii. the top width B (sometimes also called the storage width),

iv. the wetted perimeter P,

v. the hydraulic radius R and

vi. the stage h.

Depth is usually measured vertically up from the bottom point in the cross-section (rather than at right angles to the bed), while the stage is measured vertically from a datum to the water surface. See Figure 6.2.4.

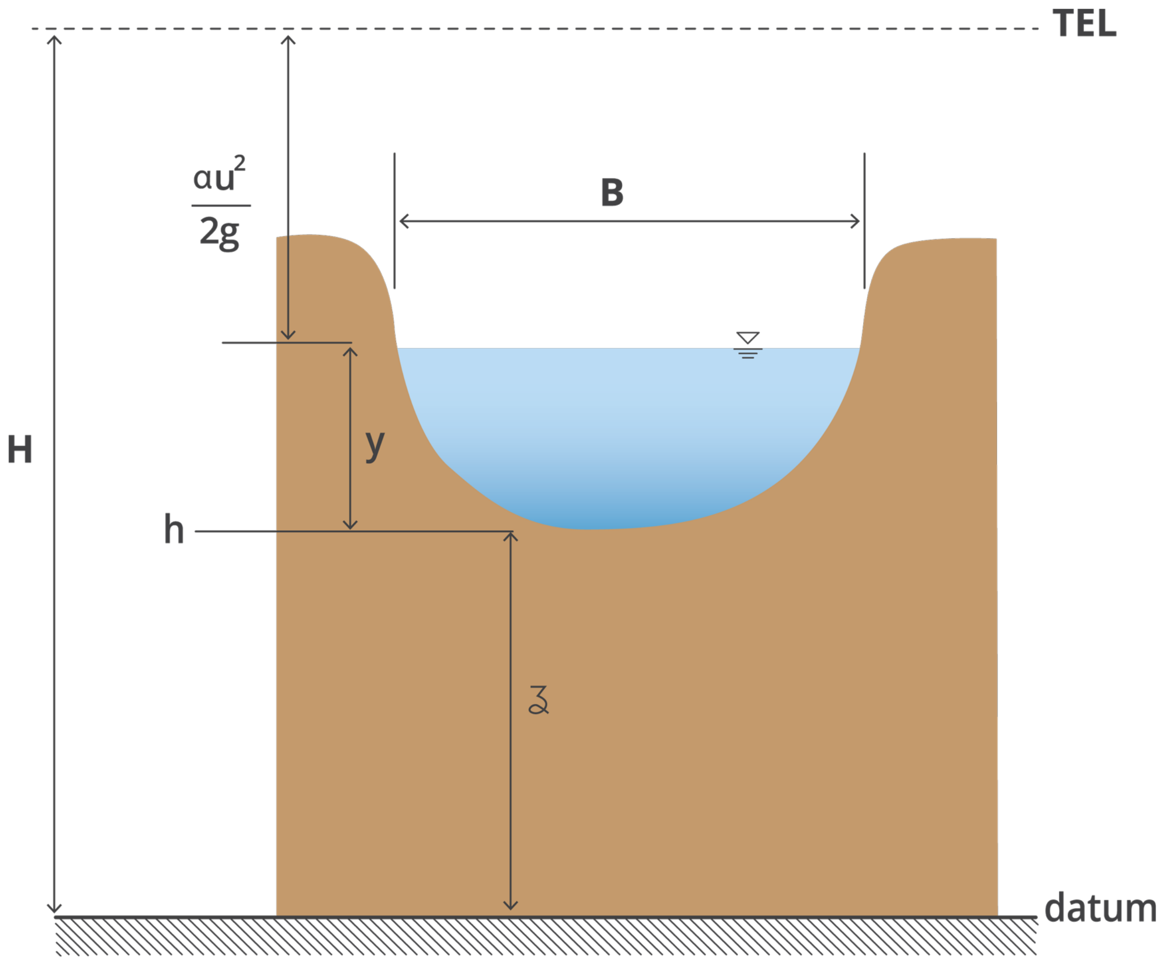


Figure 6.2.4. Parameters characterising flows in open channels

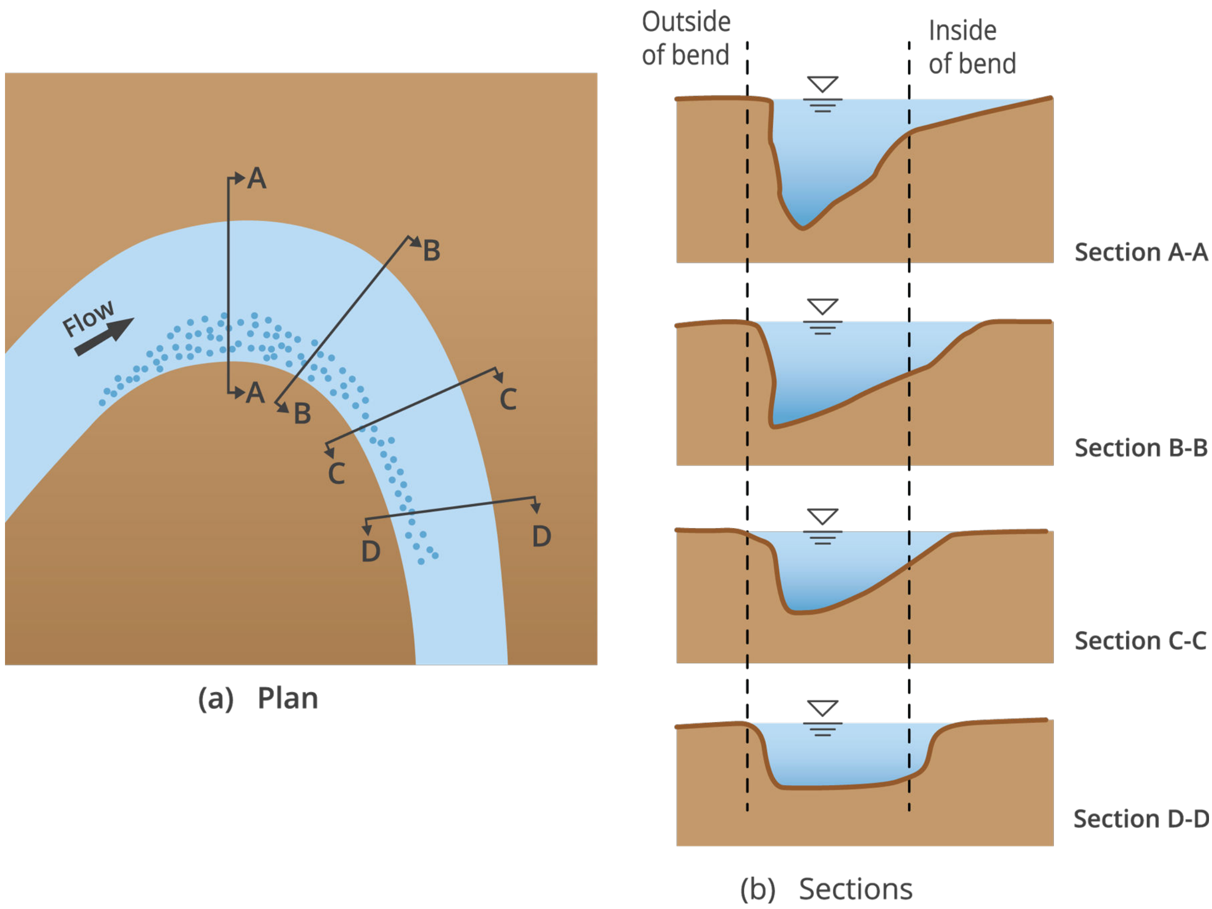


Figure 6.2.5. Variation of cross-sectional properties in natural channel

In natural channels the cross-sectional parameters vary with distance along the channel as shown in Figure 6.2.5.

Open Channel Flows are three dimensional but often treated as one dimensional

Open channel flows are three dimensional in nature and must satisfy the fundamental equations of fluid motion governed by the Navier-Stokes equations. A solution of these three-dimensional equations using direct numerical simulation, however, is not feasible for the spatial scales dealt with in flood estimation. Three-dimensional simulations using models which approximate the turbulence behaviour are feasible for small sections of a river reach and are seeing some use. Advances in computational power and numerical methods have allowed common usage of unsteady two-dimensional (2D) depth averaged models in flood hydraulics simulation. These approaches have largely replaced traditional one-dimensional (1D steady and unsteady) flow analysis for most flood studies even where the flood behaviour is essentially 1D due to ease that they produce flood inundation maps. While the treatment of river flows as a one-dimensional flow does make considerable assumptions about the flow field, this approach has served the engineering community well for over a century and is still a powerful tool provided the assumptions utilised are appropriate. The extent of some rivers are so large that even two dimensional models are infeasible, and methods have been developed which allow the 1D and 2D approaches to work together. When a fluid with a viscosity such as water flows in an open channel, boundary shear stresses resist the flow and prevent the unchecked acceleration of the water in the downhill direction. These resistive forces are transmitted throughout the main body of the flow by either viscous or turbulent shear stresses generated by velocity gradients over the cross-section. As a result, a uniform flow cannot have a uniform velocity distribution.

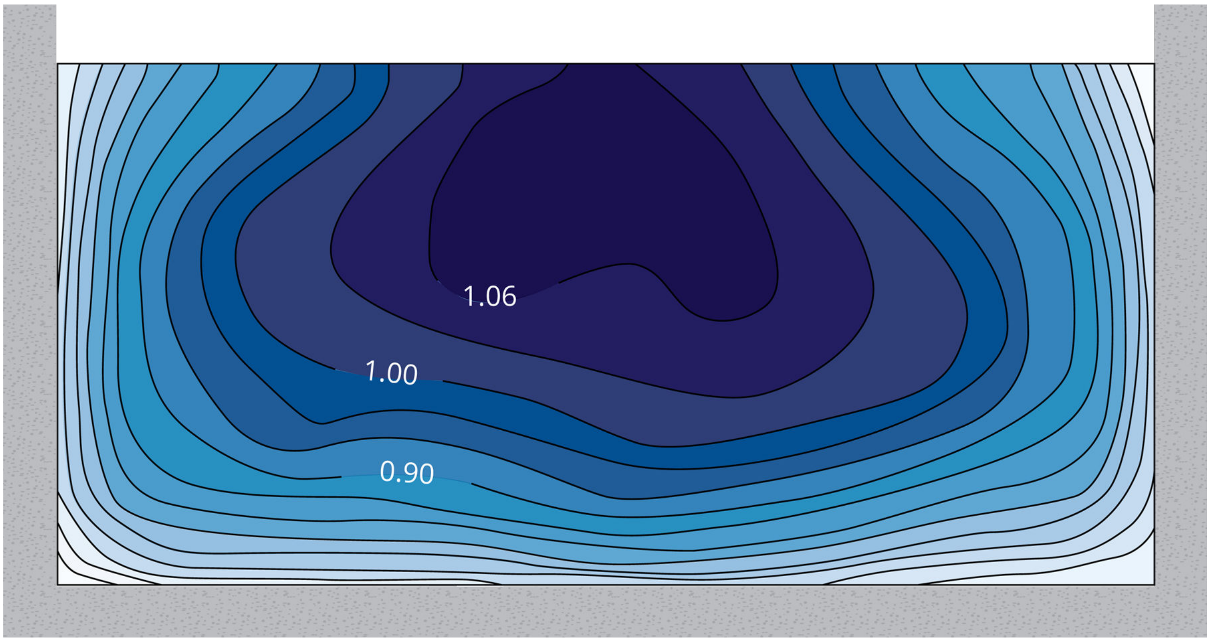


Figure 6.2.6. Cross-section velocity distribution in a small straight laboratory channel (velocities shown as a ratio of the mean)

The distribution of longitudinal velocity in a cross-section is controlled by the channel shape and the location of the free surface and boundary roughness. Figure 6.2.6 shows by lines of equal velocity the distribution of velocity in a straight laboratory channel. Figure 6.2.7 shows, for comparison, the velocity distribution in a typical river cross-section.

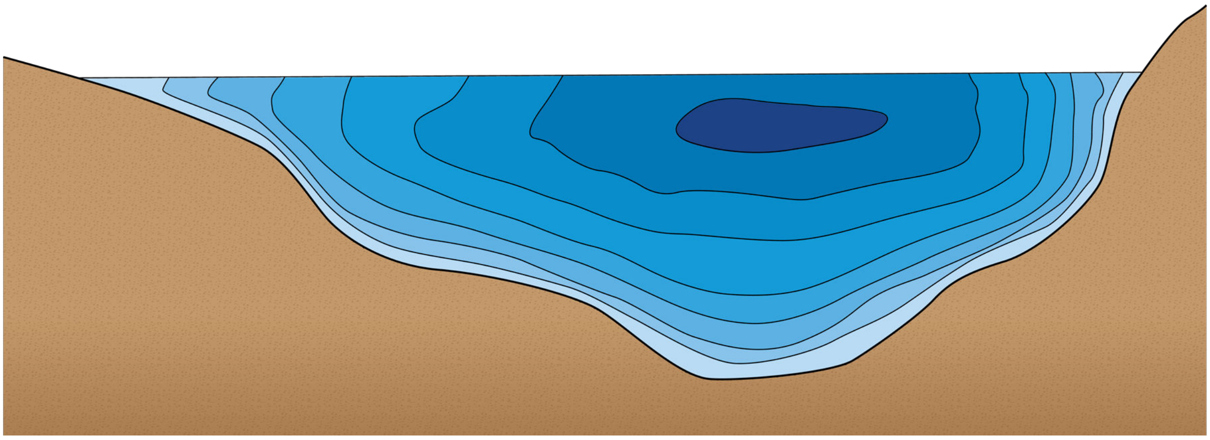


Figure 6.2.7. Typical velocity distribution in a natural channel cross-section

The velocity distribution is shown in Figure 6.2.8 where (a) shows the vertical velocity distribution on the centreline of a rectangular channel in which the depth is equal to one half of the breadth. In the same figure, curve (b) shows the vertical distribution of mean velocity; each point on this curve represents the average velocity in a horizontal line across the section at that level. Secondary currents in the plane of the cross-section produce circulation which account for both the depression of the maximum velocity filament and the observed movement of floating material towards the centre of the channel surface. This is another example of the three dimensional nature of the flows.

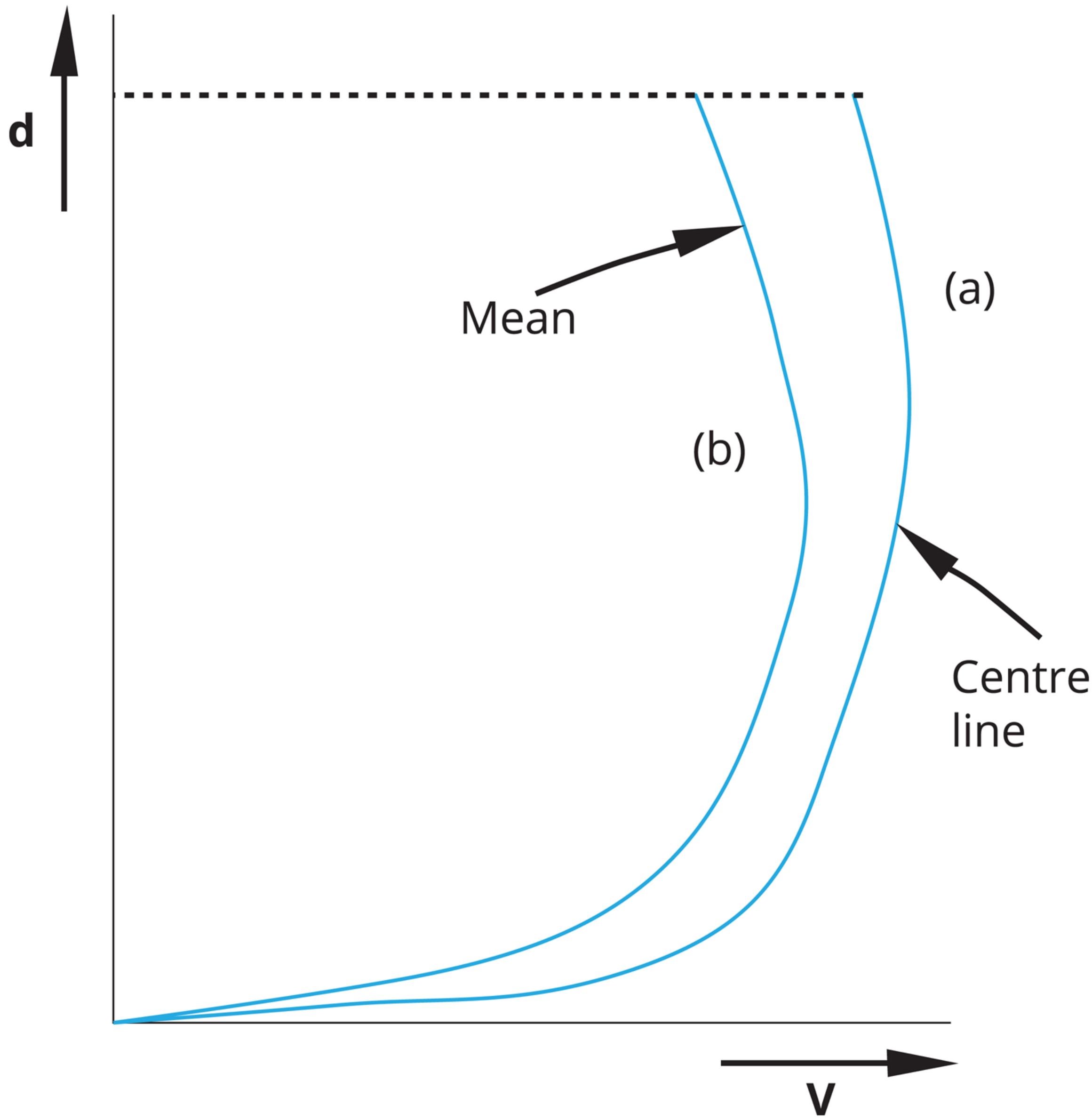


Figure 6.2.8. Typical vertical velocity distribution

A traditional one dimensional approach to open channel flow uses a single value to express the velocity at a cross-section. This is normally the average velocity V defined as the discharge divided by the cross-section area and forms the basis of the continuity equation.

$$V = \frac{Q}{A} \qquad (6.2.1)$$

This simplification leads to an error in any calculations of kinetic energy head since the mean of the squares of individual values is always larger than the square of the mean value. To make allowance for this effect an energy coefficient a is normally introduced so that the kinetic energy head at a cross-section is then $\alpha\frac{V^2}{2g}$. For complex cross-sections, such as compound channels or close to constrictions like bridge piers and weirs, the value of α can be significant. Figure 6.2.9 shows the variation in the total energy line between the main channel and the floodplain if the water surface is horizontal in the channel cross-section.

Also shown is the average total energy line given by energy correction coefficient. For a detailed discussion of this and other velocity coefficients see Chow (1959).

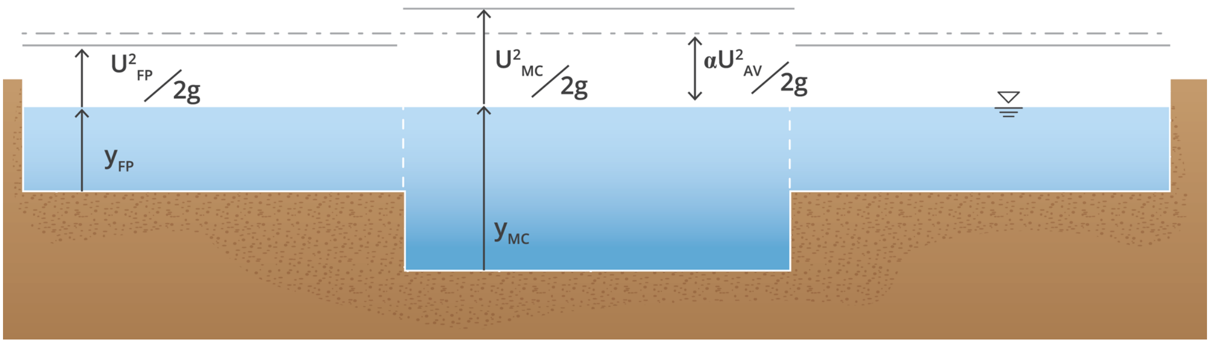


Figure 6.2.9. Variation of the total energy grade line across the compound channel section assuming a horizontal water surface.

It is frequently sufficient when analysing prismatic open channels, particularly man-made rectangular and trapezoidal channels, to assume a value of unity for the energy coefficient (α). This can be justified by consideration of the error introduced in relation to the low order of accuracy inherent in many other factors involved with open channel flow characteristics.

2.2.1. The one dimensional energy equations

The total head (H) associated with a free surface flow has dimensions of energy per unit weight of fluid (i.e. length) and units of Joules per Newton (i.e. metres) and is given by:

$$H = z + y + \alpha\frac{u^2}{2g} \tag{6.2.2}$$

where... z = vertical distance from the datum to the channel invert or potential energy (J) per unit weight of fluid (m), y = depth of flow (m), u = cross-sectionally averaged flow velocity (m/s), g = acceleration due to gravity =9.806m^2/s, $\alpha =$ dimensionless kinetic energy coefficient. The term

$$\alpha\frac{u^2}{2g} \tag{6.2.3}$$

represents the kinetic energy per unit weight of fluid (m).

The need for the kinetic energy coefficient (α) in Equation (6.2.2) and Equation (6.2.3) arises whenever the velocity is non-uniform over the cross-section. In the case of a uniform velocity over a cross-section, $\alpha = 1$. Departures from a uniform velocity over a cross-section, result in the cube of the mean velocity over the cross-section having a different value from the cube of the velocity (at each point in the cross-section) when averaged over the cross-section.

Because the cube of the cross-sectionally averaged velocity will be less that the average of the local velocity cubed, the parameter α is introduced as:

$$\alpha = \frac{\sum Q_i V_i^2}{QV^2} \tag{6.2.4}$$

$$\alpha = \frac{\sum A_i V_i^3}{AV^3} \tag{6.2.5}$$

where V_i = velocity through cross-sectional area A_i, V = cross-section averaged velocity And A = total flow area.

In reality, the flow velocity varies over the cross-section and where measurements are available it is possible to evaluate the parameter α. In practice, for turbulent flow in pipes, $\alpha = 1.03 - 1.06$ and is normally set to unity, but for open channels, particularly compound channels the value of α can depart significantly further from unity. However, it is not an uncommon practice to also adopt $\alpha = 1$ in computations for simple prismatic open channel flows.

The specific head or specific energy (*E*) is the total head with respect to the channel invert and is given by:

$$E = y + \alpha\frac{u^2}{2g} \tag{6.2.6}$$

Specific energy is a concept which is useful for determining the water surface profile through smooth transitions such as a channel narrowing, a smooth hump or smooth step in the channel or a combination of these channel transitions. This equation is often differentiated to give the minimum specific energy (critical flow) however the fact that α varies with depth is a complication. In a compound channel (α) can be significantly greater than 1 and must be considered.

The equations defining total head Equation (6.2.2) and specific energy Equation (6.2.6) have two assumptions built into them:

i. The pressure distribution is hydrostatic since the streamlines are straight and parallel, and

ii. that depth measured vertically (*y*) which is a good approximation to the actual pressure head ($y\cos^2\theta$) in an open channel flow with a bed inclined at θ to the horizontal. For example, a bedslope (S = $\tan\theta$) which is as steep as 1V:10H (or θ = 5.7°) only gives rise to an error which is close to 1% of the depth i.e. y. $\cos^2$(5.7°) = 0.99*y*.

2.3. Classification of Free Surface Flows

Free surface flows are flows in which the top water surface is subject to atmospheric pressure but is free from shear stresses exerted by any containing flow boundaries from the channel or pipe.

Free surface flows occur:

- as overland flows (across grasslands, bitumen surfaces or paved areas),
- in natural channels (as in-bank and overbank flows of rivers, streams, and creeks),
- in manmade channels (in stormwater and wastewater treatment plants, in urban drainage systems),
- other waterways (such as in estuaries or wetlands), and
- pipes and closed conduits (as in sewerage channels, pipelines and culverts).

Free surface flows can be classified:

- according to their time variation (as steady or unsteady),
- according to their spatial variation (as 1D, 2D or Three Dimensional (3D) - even though the small scale turbulent motions are always in 3D),
- as laminar or turbulent flows (depending upon the Reynolds number, *Re),*
- as subcritical or supercritical flows (depending upon the Froude number, *Fr*), or
- as gradually varying flow (as in backwater and drawdown longitudinal water surface profiles) or rapidly varying flows (as in hydraulic jumps). There is no sharp line of demarcation separating gradually varying flow and rapidly varying flow but the distinction between them can be put in terms of:

i. a comparison between the radius of curvature of the streamlines and the depth of flow, or

ii. whether the pressure distribution through the vertical can be approximated as being hydrostatic or not.

2.4. Uniform Flow and Critical Flow

There are two key flow conditions in steady state open channel flow calculations:

- uniform flow, and
- critical flow.

From a theoretical perspective, these form important bounding conditions to water surface profiles and allow the classification of the flow profiles. This water surface profile classification allows for a greater understanding of the flow, what controls it and how should it behave. For example, is the channel hydraulically mild or steep and where are the controls. In addition, it also gives insight into where a hydraulic jump might occur if the flow is constricted or controlled in some way downstream. A detailed discussion of flow profile classification is given in Henderson (1966). Detailed knowledge and understanding of these flow classifications and their implications is essential when interpreting outputs of water surface profile calculation numerical models. It is also good practice to verify and validate complex models with simpler water surface profile calculations before application to the more complex topologic problems often encountered in real flood studies. For example, can the model maintain a uniform flow, is the water surface profile correct in a variety of gradually varied flow situations, how does it handle controls, can it compute both sub-critical and super-critical flows in simple channels and can it locate hydraulic jumps correctly? These are good checks to ensure that 1) the model is working correctly and 2) that the user is using the model correctly.

2.4.1. Uniform Flow

Uniform flow is a useful reference condition in which the depth of flow (as well as the flow velocity) remains constant down a prismatic channel. The water is neither accelerating nor decelerating. Such a condition is brought about by the equilibration of two opposing forces,

- the weight or gravity force resolved down the channel ($W \sin\theta$) which tends to accelerate the flow, and

- the frictional shear force ($F\tau$) which opposes the motion of the water and acts in the longitudinal direction, tangential to the flow boundary of the channel in contact with the water (see Figure 6.2.10). The shear force acts around the wetted perimeter of the channel cross-section.

Figure 6.2.10 depicts the dominant forces acting on an elemental volume of water. For the flow condition of uniform flow, all the forces cancel out and there is no net force acting on the fluid element since $W.\sin\theta = F\tau$.

Because the hydrostatic pressure forces on the two ends of the control volume cancel each other out, they do not enter the picture here. In uniform flow, the bed slope S_o, the slope of the water surface S_{ws} and the slope of the total head line or friction slope S_f are all equal.

While uniform flow rarely (if ever) occurs in nature, it may be a reasonable approximation for the flow down a long, prismatic, manmade channel or as a first pass for estimating a flow depth in a natural channel given a discharge.

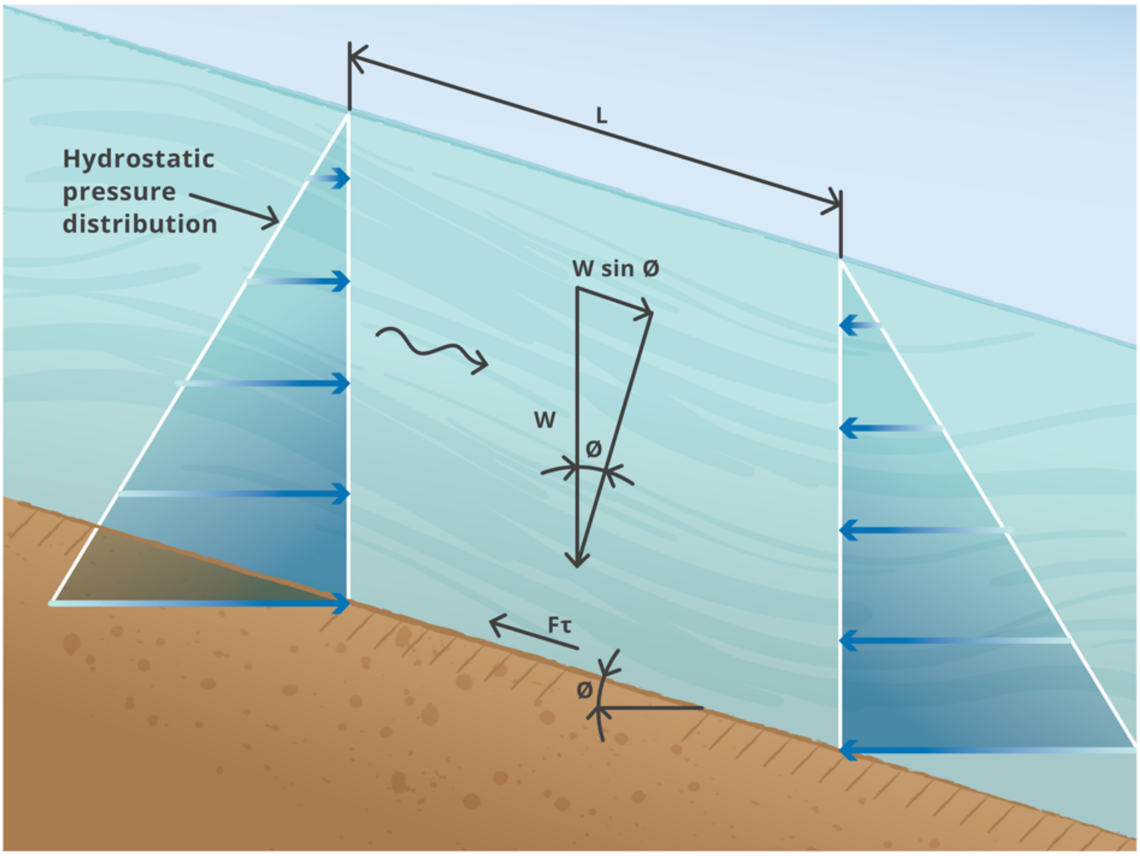


Figure 6.2.10. Opposing forces acting on a fluid element down the channel cancel thereby producing uniform flow.

2.4.2. Critical Flow

Critical flow occurs when the specific energy (E) at a cross-section is a minimum for a fixed flow, and this corresponds to the Froude number (Fr) having the value of unity at that cross-section. Here, we consider two cases:

i. channel with a general cross-section, and

ii. channel with a rectangular cross-section which is a particular but useful case.

2.4.2.1. Channel with a General Cross-Section

For a channel with a general (either natural or prismatic but not compound) cross-section as shown in Figure 6.2.7, critical flow is defined by setting the Froude number to unity.

$$Fr = \sqrt{\frac{Q^2 B}{g A^3}} = 1 \tag{6.2.7}$$

where. . . B = top width of the cross-section (*m*)

2.4.2.2. Channel with a Rectangular Cross-Section

Only for channels with a rectangular cross-section, is it convenient to work in terms of the flow per unit width of channel i.e. $q = Q/b$ in units of (m^2/s).

At critical flow, the value of the Froude number is unity and the resulting expressions are:

$$Fr = \frac{q}{\sqrt{g y_c^3}} = \frac{V}{\sqrt{g y_c}} = 1 \tag{6.2.8}$$

$$y_c = \left(\frac{q^2}{g}\right)^{\frac{1}{3}} \tag{6.2.9}$$

where q = flow per unit width in channel with rectangular cross-section (m^2/s) y_c = critical flow depth in channel with rectangular cross-section (m), V = flow velocity (m/s) when the depth is y_c.

Cross-sections at which the flow is critical are special in that there is a unique relation between velocity (or rate of flow) and depth, irrespective of the channel roughness or bedslope. Such cross-sections are technically known as controls and examples of locations where they occur might are:

i. a sudden steepening of the channel bedslope,

ii. a brink (or overflow),

iii. a hump of sufficient height,

iv. an upwards step of sufficient height,

v. a narrowing of the channel which is sufficiently constricting,

vi. flow from a reservoir or lake into a channel, or

vii. a hydraulic structure such as a broad crested weir. Some hydraulic structures that consist of control gates (such as sluice or radial gates) also provide a unique relationship between depth and discharge, but critical depth is not present.

In an open channel with various features such those listed above, it is possible that if the flow changes, so can the control(s). For a given flow, a channel can have more than one control. The actual controls can only be identified through a trial and error process in which channel feature(s) is/are assumed to be control(s), and the computations on the backwater and drawdown curves completed. If the resulting water surface profile is compatible with the known boundary conditions at both ends of the channel, then the assumed control(s) are valid. If, on the other hand, an incompatibility is reached, it means that one of more of the assumptions regarding the controls needs to be changed and the solution process of calculating the water surface profiles is repeated until compatibility with all imposed boundary conditions is achieved. In a channel with some potential controls, the above solution process can be lengthy and require a number of trials.

2.4.2.3. Channels with a compound cross-section

While much research attention has been focused on uniform flow in compound channels in the past, little work has been done on predicting critical depth in such channel configurations. Critical depth in an open channel is most commonly defined as the point of minimum specific energy, the point of minimum specific force, or the transition between supercritical and subcritical flow, where the celerity of a surface wave is equal to the velocity of the flow.

Petryk and Grant (1978) were the first to propose an alternative Froude number for compound channels. This Froude number was based on a discharge weighted average of each subsections Froude number calculated from simple channel procedures. While this method was simple in its application, it was shown to have limitations (Blalock and Sturm, 1981).

Blalock and Sturm (1981) developed a method of calculating the critical depth in a compound channel based on the definition of minimum specific energy including the kinectic energy corrections coefficient.

$$E = \alpha \frac{V^2}{2g} + y \tag{6.2.10}$$

where, α is the kinetic energy correction factor, u is the average velocity within the cross section and y is the depth of flow.

The minimum specific energy is found by setting the derivative with respect to the depth of Equation (6.2.11) to zero. This yields the following general Froude number for open channel flow:

$$Fr_c = \left[\frac{\alpha Q^2 T}{g A^3} - \frac{Q^2}{2 g A^2} \frac{d\alpha}{dy} \right]^{\frac{1}{2}} \tag{6.2.11}$$

where, Q is the total discharge, A is the total cross-sectional area and T is the channel top width.

Blalock and Sturm (1981) stated that early works on compound channel sections assumed a value of unity for the value of kinetic energy correction factor, α, which gives:

$$Fr_c = \sqrt{\frac{Q^2 B}{g A^3}} \tag{6.2.12}$$

This is the equation that would be used in a prismatic channel where the average velocity is a good representation of the flow velocity distribution. Equation (6.2.12) considers the compound channel as a single unit. There are some similarities here to the Single Channel Method (SCM) used to calculate uniform flow in compound channels (Lambert and Myers, 1998). Equation (6.2.12) would be appropriate in a compound channel if the velocities in the different sub-areas were of a similar magnitude (Lee et al., 2002)

Blalock and Sturm (1981) found that the value of a is not a constant value, but varies as a function of depth in compound channels. Blalock and Sturm (1981) evaluated the kinetic energy correction factor using the traditional divided channel method with vertical divisions (Chow, 1959). The channel was split into three independent sections, the main channel, and the two adjacent floodplains. The assumption was that the non-uniformity in the velocity profile occurs predominantly as a result of the difference in velocities between sections, and the variation within each section was seen to be negligible. The Manning formula was used to calculate the conveyance of each section and the boundary between the sub-sections was not included in the wetted perimeter of the sub-sections. Using this method, the value of α in a compound channel was derived to be:

$$\alpha = \frac{\sum_{i=1}^{3} \frac{K_i^3}{A_T^2}}{\frac{K_T^3}{A_T^2}} \tag{6.2.13}$$

where, K_i is the conveyance of the ith subsection given by $\left(\frac{1}{n} A_i R_i^{\frac{2}{3}}\right)$, A_i is the cross-sectional area, n_i is the Manning surface roughness coefficient, K_i is the hydraulic radius of the cross-section and K_T, A_T are the total conveyance and cross-sectional area for the entire channel respectively.

Differentiating Equation (6.2.13) with respect to the depth *y* yields

$$\frac{\partial \alpha}{\partial y} = \left(\frac{A_T 2\sigma_1}{K_T{}^3} + \sigma_2\left(\frac{2A_T T}{K_T{}^3} - \frac{A_T 2\sigma_3}{K_T{}^4}\right)\right) \tag{6.2.14}$$

where σ_n is the nth subsection property defined by Blalock and Sturm (1981) as:

$$\sigma_1 = \sum_{i=1}^{3}\left[\left(\frac{K_i}{A_i}\right)\left(3T_i - 2R_i\frac{\partial P_i}{\partial y}\right)\right] \tag{6.2.15}$$

$$\sigma_2 = \sum_{i=1}^{3} \frac{K_i^3}{A_i^2} \tag{6.2.16}$$

$$\sigma_3 = \sum_{i=1}^{3}\left[\left(\frac{K_i^3}{A_i^2}\right)\left(5T_i - 2R_i\frac{\partial P_i}{\partial y}\right)\right] \tag{6.2.17}$$

and, T_i is the top width of flow and P_i is the wetted perimeter for the ith subsection.

The Froude number for a compound channel, based on the definition of critical flow as the point of minimum specific energy is given by substituting Equation (6.2.13) and Equation (6.2.14) into Equation (6.2.11):

$$Fr_c = \frac{Q^2}{2gK_T{}^3}\left(\frac{\sigma_2\sigma_1}{K_T} - \sigma_1\right)^{\frac{1}{2}} \tag{6.2.18}$$

This approach can result in multiple values for critical depth in a compound channel. Typically, one crtitical depth is within the main channel and the other is above. Exactly which value should be used is somewhat unclear and is often found by a numerical minimisation method.

For any given overbank depth a discharge and velocity can be computed using the divided channel method with vertical divisions, and a subsection Froude number could be computed based on these values. This is consistent with treating these sub-sections as independent channels. Further discussion of this is given in Lee et al. (2002) who suggested that there may be a transition zone between subcritical and supercritical flow where a mixed flow regime is possible and that the concept of specific value of critical depth given above may not be meaningful.

2.4.3. Bed Shear Stress

As soon as there is movement of water through a channel or pipe with slope, $S = \tan\theta$, a resistive shear force is mobilised which opposes the motion of the water down the channel. This shear force (or friction force) is developed around the wetted perimeter or periphery (P) of the channel or conduit and has its origins in the (albeit small) viscosity of the water. The shear force arises from a combination of:

i. the viscous skin drag between the moving fluid and the flow boundary, and

ii. the form drag on the roughness projections of the flow boundary.

While a shear force is also developed at the air-water interface, is generally small (except for strong winds over large surface areas) in comparison to the shear force around the interface of the water and channel (or water and pipe wall).

An expression for the bed shear stress for uniform flow (t) can be determined by equating the two forces acting on the fluid element of length (L) in Figure 6.2.10.

$$W\sin\theta = F_T \tag{6.2.19}$$

$$\rho g L A \sin\theta = L P \tau_o \tag{6.2.20}$$

$$\tau_o = \rho g\left(\frac{A}{P}\right)\sin\theta \tag{6.2.21}$$

$$\approx \rho g R_h \tan\theta \tag{6.2.22}$$

$$= \rho g S R_h \tag{6.2.23}$$

$$where\ldots R_h = \frac{A}{P} \tag{6.2.24}$$

R_h= hydraulic radius (m)

The approximation $\sin\theta \approx \tan\theta$ is only in error by 1% when the channel slope is as steep as about 8° or 7H:1V since $\left(\frac{\sin(8°)}{\tan(8°)}\right) = 0.99$.

Equation (6.2.21) applies to uniform flow, but it can be generalised to include gradually varying flow by replacing the slope, S by the friction slope, Sf. For gradually varying flow, the bed shear stress is given by:

$$\tau_o = \rho g R_h S_f \qquad (6.2.25)$$

The bed shear stress is important when considering the flow velocities necessary for scour and deposition including:

i. the initiation of sediment motion,

ii. sediment motion in alluvial channels,

iii. avoiding deposition of sediment through suspension by fluid turbulence,

iv. scour at bridges, and

v. scouring conduits free of sediment.

2.5. Uniform Flow Resistance Formulas

In turbulent flow, there are three equations which are commonly used to quantify the effects of boundary resistance when a flow is passing down the channel:

- Manning equation
- Chezy equation, and
- Darcy-Weisbach equation in conjunction with the Colebrook-White equation to determine the Darcy friction factor, f.

While the Chezy and Manning formulas are more commonly applied to open channel flows, the Darcy-Weisbach and Colebrook-White equations are more commonly applied to pipe flows.

Both the Chezy and Manning formulas relate the cross-sectional averaged velocity to the channel slope, the hydraulic radius and an empirical parameter which is used to encapsulate the effects of the resistance to flow. The two formulas have a different form to each other. A discussion of the resistance equations is given in the American Society of Civil Engineers Task Force on Friction Factors in Open Channels (ASCE Task Force, 1963).

2.5.1. Manning Formula

The formula known as the Manning formula is the result of several modifications of a formula originally published by Manning in 1889. The Manning formula shown below is popular internationally and is the most used approach in practice in Australia:

$$V = \frac{R_h^{\frac{2}{3}} S^{\frac{1}{2}}}{n} \qquad (6.2.26)$$

where V = cross-sectional averaged flow velocity ($m.s^{-1}$) = Q/A, Q = flow (m^3/s), R_h = hydraulic radius of the cross-section (m) = A/P, S = longitudinal slope of the channel ($m.m^{-1}$)

The Manning's equation Equation (6.2.26) is not dimensionally homogeneous and this has led to confusion about the dimensions of the Manning roughness parameter n. There are three interpretations of the dimensions of Manning's n (See Section 5-6 in Chow (1959)):

- $L^{-\frac{1}{3}}T^{1}$ -These dimensions result directly from the Manning equation Equation (6.2.26),
- $L^{0}T^{0}$ - i.e. dimensionless. Consequently, the unstated coefficient of unity at the front of the right hand side of Equation (6.2.26) must have the dimensions of $L^{-1/3.T1}$,
- $L^{\frac{1}{6}}$ - In this case, the dimensions of n are independent of time, as would be expected of a roughness parameter and only involve length. As n is a measure of the absolute roughness of the channel surface. In this case, it is assumed that there is a coefficient of unity equal to $\sqrt{\frac{g}{9.81}}$ attached to the right hand side of the Manning equation Equation (6.2.26). In engineering practice, this is perhaps the most commonly adopted version for the units of Manning n.

2.5.2. Application of the Manning Equation

The Manning equation Equation (6.2.26) can be applied in two modes:

1. With Manning n being held constant for a particular channel, irrespective of the flows down the channel. This case corresponds to a direct relationship between the square of the flow velocity and the slope (i.e. S˜ v2) and is a characteristic of a fully rough turbulent flow.

2. With Manning n varying in some prescribed fashion according to the depth of flow (or stage or hydraulic radius). The actual variation of Manning n will normally have been determined from measurements and back-calculated through Manning equation. When used in numerical modelling of flood studies, the variation may be stored as a table of values for varying depth or stage.

Estimates for the values of Manning n can be found from various sources. An example of a table of Manning n is give in the table below.

Table 6.2.1. Values of Roughness Coefficient n for different channel conditions (Sellin 1961)

Description of channel	Minimum	Normal	Maximum
Glass, plastic, machined metal	0.009	0.010	0.013
Fabricated steel channels	0.011	0.012	0.017
Planed timber, joints flush	0.010	0.012	0.014
Sawn timber, joints uneven	0.011	0.013	0.015
Concrete, trowel finished	0.011	0.013	0.015
Concrete, shuttering	0.012	0.014	0.017

Description of channel	Minimum	Normal	Maximum
Brickwork	0.012	0.015	0.018
***Excavated channels:**			
earth, clean	0.016	0.022	0.030
gravel	0.022	0.025	0.030
rock cut, smooth	0.025	0.035	0.040
rock cut, jagged	0.035	0.040	0.060
***Natural channels:**			
clean, regular section	0.025	0.030	0.040
some stones and weeds	0.030	0.035	0.045
some rocks and/or brushwood	0.050	0.070	0.080
very rocky or with standing timber	0.075	0.100	0.150
Flood plains:			
short grass pasture	0.025	0.030	0.035
mature crops	0.025	0.035	0.045
brushwood	0.035	0.050	0.070
heavy timber or other obstacles	0.050	0.100	0.160

1. Tables of Manning n can be found in numerous references for different surfaces such as asbestos cement, concrete (centrifugally spun), ductile iron and steel (e.g. Table 2 in AS 2200 (2006) and Table 5-6 in Chow (1959)). References often give a range of values for minimum and maximum values corresponding to pipes in good to poor condition.

2. Twenty four black and white photographs of manmade and natural river channels can be found in Figure 5-5 of reference Chow (1959) with corresponding values of Manning roughness parameter ranging from n = 0.012 to n = 0.150. No information on the stream or river geometry (i.e. plan view or cross-sections) is given in this reference. The main difficulty with acquiring estimates of n from these photos is that one is relying on the brief caption for each photo and the view of the exposed part of the bank or channel to gauge the roughness (and hence the Manning's n) of the submerged part of the channel or of the channel under flood conditions.

3. Colour photos of fifty natural rivers, two to seven (but typically three or four) cross-sections of each river channel and a plan view can be found in Barnes (1967). This information was assembled by the U.S. Geological Survey over a 15 year period. Although this reference is old with its quantitative information in imperial units, it is perhaps the best known and complete compendium on Manning n for natural streams and rivers. The Manning n values range from n = 0.024 through to n = 0.075 and all measurements in this reference were made during the peaks of documented floods. The locations of the cross-sections are indicated on an accompanying plan view of the stream or river. It is clear from the plan views of all the channel reaches that where the Manning n values refer to are straight or gently curving. Other information which can be found in this

reference includes the river name, geographical location, date of the flood, the flow, a description of the bed material and condition of both river banks, and a table of cross-sectional flow area, top width, mean depth, hydraulic radius, mean flow velocity, length between cross-sections, and the slope between sections.

An example of the information contained in this reference is reproduced in Figure 6.2.11 to Figure 6.2.12 which refers to Esopus Creek. (Esopus Creek is one of three waterways with Manning n = 0.030 included in this reference.)

Interestingly, rivers and streams with sandy beds were not included in this reference because Manning n values for streams and rivers of this type, which depend upon the size of the bed material and bedforms, can be found elsewhere. The beds of the streams and rivers included in this reference ranged from boulder strewn mountain streams to heavily vegetated streams and rivers.

1. Other references with pictures of channels with varying channel roughness can be found from various web sites on the internet (Phillips and Ingersoll, 1998) or other references from the U.S. Geological Survey.

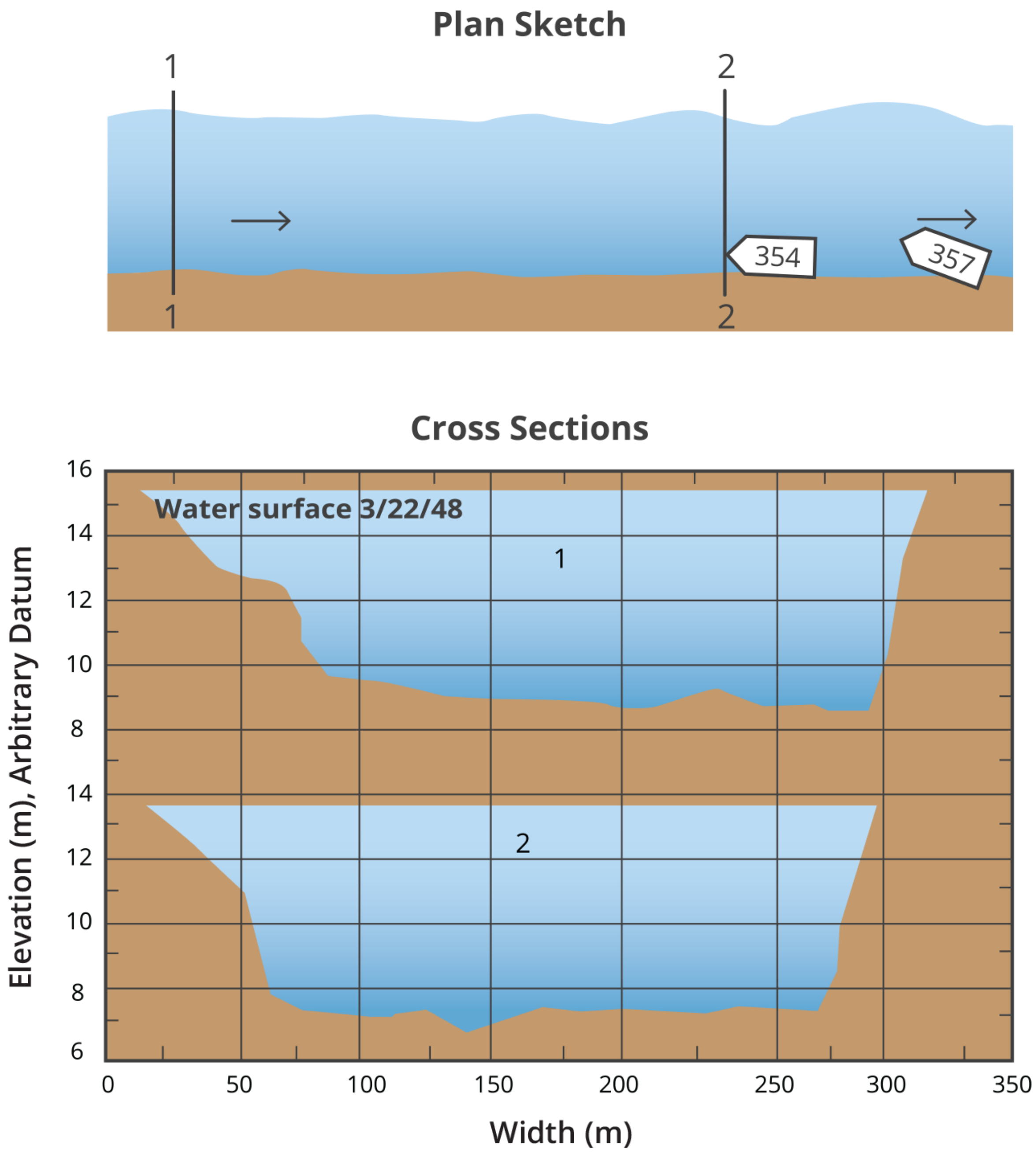


Figure 6.2.11. Roughness coefficient data for Esopus Creek with n = 0.030 (Page 34 in Barnes (1967)).

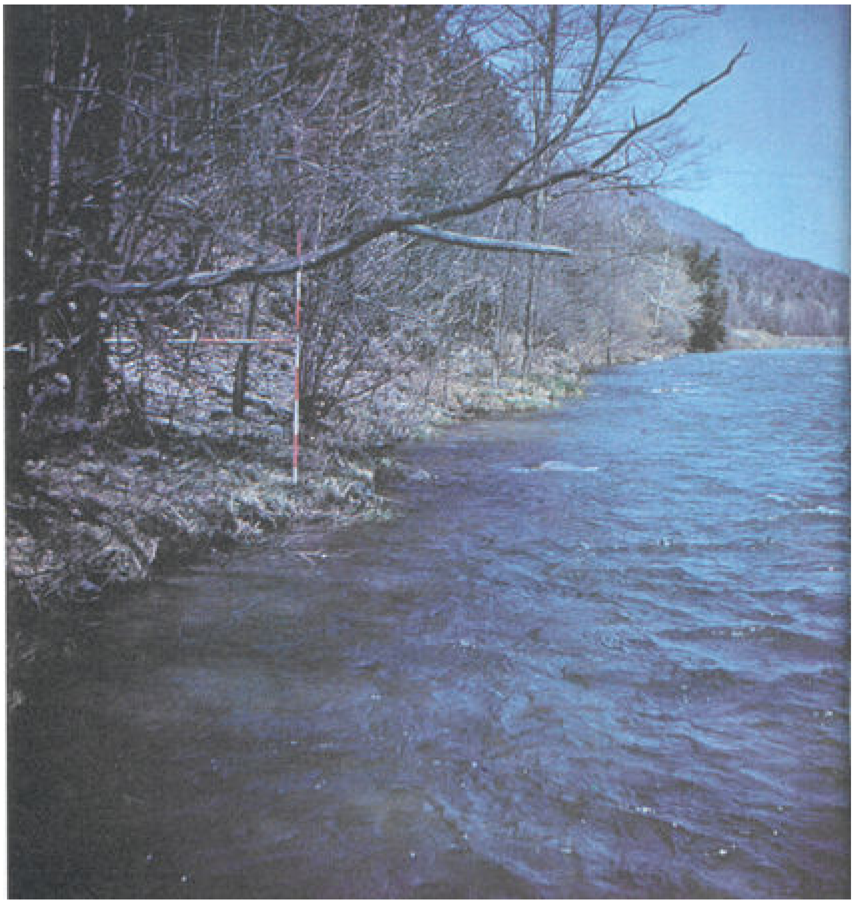


Figure 6.2.12. Esopus Creek

In a 1D model the roughness parameter can account for:

1. Friction losses associated with the bed material of a channel/floodplain,

2. Drag losses associated with vegetation or other obstructions in the channel/floodplain,

3. Losses due to turbulence in a channel/floodplain due to channel geometry,

4. Variations in geometry and associated form losses between cross-sections,

5. Bend losses in a channel.

Developing Hydraulic Roughness

As two dimensional models account for some non-boundary energy losses they often use slightly lower Manning roughness values than one dimensionalmodels listed above. In a 1D model roughness is typically assigned according to the cross-sections or along particular branches. For 2D models, roughness is generally specified as a spatially varying grid/mesh over the 2D model domain. It is important to note that the loss processes embedded in hydraulic roughness parameters for 1D and 2D models, whilst closely related, are not exactly the same.

In a 2D model, some of the above losses are to some degree accounted for by the numerical scheme. For example, some aspects of bend losses due to change in directional momentum are explicitly modelled in a full 2D solution. Similarly, part of the form loss due to variations in geometry will be explicitly modelled in 2D scheme, depending on the grid/mesh resolution. Whilst the 2D roughness parameter nominally represents friction loss due to the ground surface material in each grid/mesh element, in practice there are still many sub-element loss processes that are not explicitly described, such as vegetation resistance (trees, shrubs), physical obstructions (fences, cars, poles etc) and local variability in topography. In effect, roughness parameters in a 2D domain also need to account for some losses in addition to the bed frictional losses, but less so than a 1D domain applied over the same area. In general, increasing grid/mesh resolution in the 2D domain can result in more of the additional losses being accounted for within that domain and less needing to be compensated for within the roughness parameter.

In urban areas, the way in which buildings are represented in the model has a significant bearing on the specification of roughness. In areas where buildings are explicitly represented as obstructions in the model topography, roughness in the surrounding areas should only account for the nature of the land-use (such as grass, paved, or vegetated areas). Alternatively, where buildings or other major obstructions are not explicitly modelled, the impact of these features on losses can be incorporated into the roughness parameter, using a significantly higher value than would otherwise be the case. Book 6, Chapter 4 contains further discussion on incorporating buildings, fences and other urban features within the 2D domain.

Applicable ranges for hydraulic roughness in 1D models have been well established and defined in numerous references over the last 50+ years, such as Chow et al. (1988). Values that represent average conditions within and between cross-sections are applied either at the cross-section or along a part or all of the branch.

Roughness for 2D models is generally specified as a map and based on land-use information that can be derived from aerial photography, satellite images, planning zone maps or field observations. Different areas can be digitised into land-use polygons representing zones of similar loss characteristics (e.g., vegetation or impervious surface type). This is typically conducted in a GIS environment and then transferred to the required format for a specific model package.

Roughness maps have also been generated from an auto image or LiDAR processing in some areas. However this is not a commonly adopted technique at present.

2D roughness is generally parameterised in terms of Manning ‘n’, or similar related parameterisation of bed friction. Typical ranges of 2D roughness parameters for various land-use types are listed in Table 6.2.2.

Table 6.2.2. Valid Manning ‘n’ Ranges for Different Land Use Types

Land Use Type	Manning ‘n’
Residential areas – high density	0.2 – 0.5
Residential areas – low density	0.1 – 0.2
Industrial/commercial	0.2 – 0.5
Open pervious areas, minimal vegetation (grassed)	0.03 – 0.05
Open pervious areas, moderate vegetation (shrubs)	0.05 – 0.07
Open pervious areas, thick vegetation (trees)	0.07 – 0.12

Land Use Type	Manning 'n'
Waterways/channels – minimal vegetation	0.02 - 0.04
Waterways/channels – vegetated	0.04 – 0.1
Concrete lined channels	0.015 – 0.02
Paved roads/car park/driveways	0.02 – 0.03
Lakes (no emergent vegetation)	0.015 – 0.35
Wetlands (emergent vegetation)	0.05 – 0.08
Estuaries/Oceans	0.02 – 0.04

2.5.3. Factors Affecting the Manning Roughness Parameter

In practice, the Manning roughness coefficient n is used to encapsulate energy losses which may arise from several sources apart from boundary roughness. Other possible causes of energy losses are:

- flow expansions and contractions which may cause the flow to separate from the flow boundary and form a recirculation bubble in which energy is continuously dissipated in the eddy,
- vegetation (be it grasses, macrophytes or algae), and
- channel sinuosity which give rise to secondary currents. (No natural channel runs straight for more than ten times its width.)

If the flow expansion or contraction is sudden, it is more appropriate to formulate the resulting energy losses in any calculations or numerical model as a local loss. Chow (1959) and James and Wark (1992) discusses this in more detail.

2.5.4. Chezy Formula

The Chezy formula was proposed by Chezy in 1769 and is:

$$u = C\sqrt{SR_h} \tag{6.2.27}$$

where. . . u = cross-sectionally averaged velocity (m/s) = Q/A, Q = flow ($m^3.s^{-1}$) R = hydraulic radius of the cross-section (m) = A/P S = longitudinal slope of the channel ($m.m^{-1}$) A = flow area (m^2) P = wetted perimeter (m) C = Chezy coefficient ($m^{1/2}.s^{-1}$)

The Chezy coefficient is a measure of the smoothness of the channel since the smoother the channel, the greater is the value of the Chezy coefficient. However, in general, the resistance to flow depends on the viscosity of the water, as well as the roughness of the surface of the channel. The general semi-empirical expression for the Chezy coefficient is:

$$C = \sqrt{32g}\log\left(\frac{12R_h}{2\delta_s/7 + k_s}\right) \tag{6.2.28}$$

$$= 18\log\left(\frac{12R_h}{2\delta_s/7 + k_s}\right) \tag{6.2.29}$$

where. . . δ_s = viscous sublayer thickness (m) = 11.6 (v/ v*v*) = shear velocity ($m.s^{-1}$) = $\sqrt{gR_hS}$ = kinematic viscosity of water ($m^2.s^{-1}$) = 10^{-6} ($m^2.s^{-1}$) for water at 20°k_s = boundary roughness projection height (m)

A few more words on k_s as it relates to pipe wall roughness and to bedforms are in order.

- When k_s refers to the roughness of a pipe or a manmade surface such as concrete, k_s is also known as the equivalent sand grain roughness, after the definitive experimental work of Nikuradse (1933). Nikuradse conducted experiments aimed at measuring the head losses in pipes of varying roughness. In each experiment Nikuradse conducted, the inside wall of the pipe was coated with sand grains of approximately the same size. Nikuradse tested pipes with internal coatings of various discrete grain sizes, lengths and diameters and at various Reynolds numbers for the flow. In each case, the resulting head loss along the pipe was measured.

For commercial pipes (with their various pipe-to-pipe joints), the wall roughness will have a different geometry and distribution of roughness projection heights compared to Nikuradse's sand coated pipes.

The commercial pipe with an equivalent sand grain roughness is the pipe with the same size and length which yields the same head loss as a pipe coated with sand of a particular size. Data on commercial pipes (or other manmade surfaces) will include the equivalent sand grain roughness.

As pipes in service age, their walls become rougher through deterioration of the pipe wall and/or build-up of algal slimes. The equivalent sand grain roughness of pipes tends to increase with time and only through cleaning the pipes can the roughness (ks) be reduced.

1. In the case of bedforms on a mobile bed of sediment, it is recommended that k_s be taken as half the height of the bedform. For example, for a river bed with dunes of height 10 cm engineering practice would be to set k_s = 5 cm.

Retardation of the flow in an open channel is due to two forces which both oppose the flow:

1. the viscous force between the fluid and the boundary, and

2. the drag force due to the protuberances of the channel (or wall) roughness projections.

The effect of the roughness of the boundary surface on the value of the Chezy coefficient is quantified by the (wall) roughness (projection height) parameter (k_s) while the effect of the viscosity of the water is incorporated in the thickness of the viscous sublayer (δ_s). The viscous sublayer is the very thin region of chaotic flow dominated by the viscous force and which separates the stationary fluid immediately in contact with the stationary flow boundary and the overlying turbulent boundary layer.

It is evident from the denominator in Equation (6.2.28) that there are two extremes of turbulent flow:

- the viscous force dominates i.e. δ_s k_s; the roughness projection elements are fully immersed in the viscous sublayer (see Figure 6.2.13) and the value of the Chezy coefficient is independent of the wall roughness projection height k_s. This extreme flow condition is known as hydraulically smooth turbulent flow and is unlikely in most practical situations.

- the drag forces on the roughness projection protuberances dominate i.e. $k_s >> \delta_s$; the roughness projection elements are sufficiently long to break down or protrude through and rupture the viscous sublayer and the value of the Chezy coefficient (and the uniform flow velocity) is independent of the fluid viscosity. This extreme flow condition is known as hydraulically rough turbulent flow and is normally the case when the Reynolds number Re > 10^6 (approximately) and would commonly occur in earthen or natural channels. For hydraulically rough flow, Equation (6.2.29) can be simplified and the value of the Chezy coefficient can then be determined from:

$$C = 18\log\left(\frac{12R_h}{k_s}\right) \tag{6.2.30}$$

Values of the boundary roughness (k_s) vary according to the nature of the surface of the channel boundary. Three examples are earthen channels, brickwork, and concrete. Amongst other references, useful tabulations of the values for ks for various channel surfaces may be found in Table 5-6 of Chow (1959) or Table 4-1 in Henderson (1966). The values in Table 6.2.1 were extracted from Hydraulics Research Ltd. (1990) Table 3 in Charts for the Hydraulic Design of Channels and Pipes.

Table 6.2.3. Values of the roughness projection height k_s and Manning n for straight, clean pipes concentrically jointed.

Type of Pipe	Roughness Coefficient	
	Rouchness Projection Height k_s (mm)	Manning n
Asbestos cement	0.015 - 0.06	0.008 - 0.011
Bitumen-lined concrete	0.06 - 0.15	0.009 - 0.012
Spun bitumen-lined steel	0.03 - 0.06	0.009 - 0.010
Brass	0.003 - 0.015	0.008 - 0.009
Cast iron (unlined)	0.015 - 0.6	0.010 - 0.013
Cement-mortar lined (in-situ)	0.03 - 0.15	0.009 - 0.012
Coal-tar enamel lined steel	0.03 - 0.15	0.009 - 0.011
Concrete, centrifugally spun	0.03 - 0.15	0.009 - 0.012
Copper	0.003 - 0.15	0.008 - 0.009
Zinc-coated (galvanised) steel	0.03 - 0.15	0.009 - 0.011
Thermoplastics	0.003 - 0.15	0.008 - 0.009
Thermosetting plastics	0.003 - 0.15	0.008 - 0.009
Vitrified clay	0.15 - 0.6	0.010 - 0.013
Fibre cement	0.015 - 0.06	0.008 - 0.009
Ductile iron, bitumen lined	0.06 - 0.3	0.009 - 0.012
Ductile iron and steel, cement mortar lined with or without seal coats	0.01 - 0.06	0.006 - 0.011
Ductile iron and steel	0.01 - 0.03	0.006 - 0.009
Steel, polyethylene lined	0.003 - 0.15	0.008 - 0.009

2.5.5. Application of the Chezy Equation

In hydraulic investigations, the Chezy equation (Equation (6.2.27)) can be applied in two modes:

1. with the Chezy coefficient being held constant for a particular channel, irrespective of the flows down the channel. This case corresponds to a direct relationship between the square of the velocity and the slope (i.e. S ~ u^2)and is a characteristic of a fully rough turbulent flow.

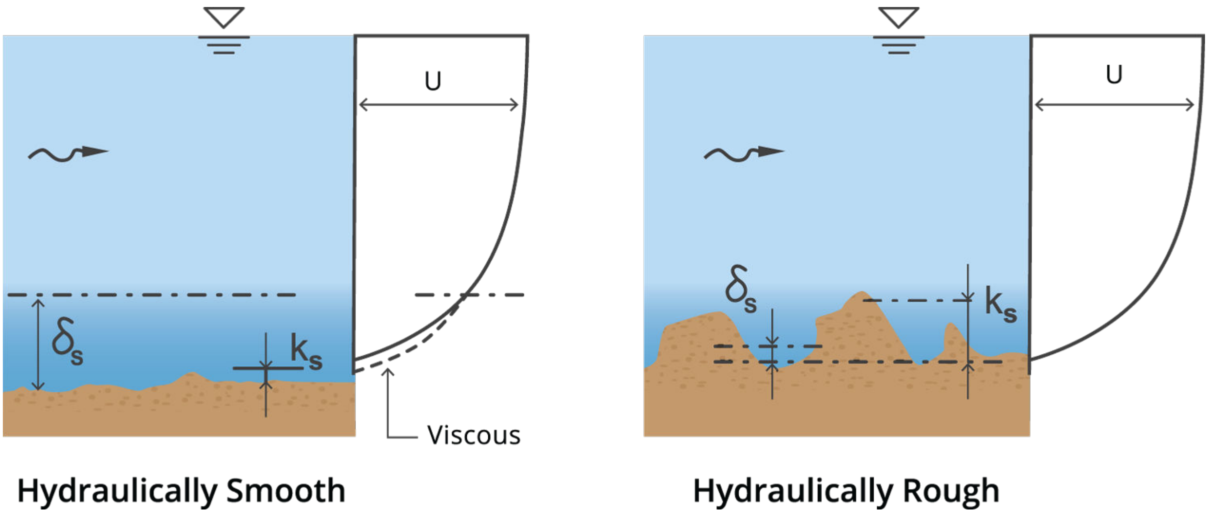


Figure 6.2.13. Relative height of the roughness projection elements and the thickness of the viscous sublayer.

1. with the Chezy coefficient varying according to Equation (6.2.28). In this case, the Chezy coefficient depends on the flow down the channel. Consequently, the flows capable of being simulated can range from hydraulically smooth turbulent flow to hydraulically rough turbulent flow. In effect, Equation (6.2.28) is closely related to the Colebrook-White equation of pipe flow.

2.5.6. The link between between the Manning Roughness coefficent and the roughness height

Some justification may be given for use of the Manning equation in terms of the dimensionally consistent Darcy-Weisbach friction factor. Figure 6.2.14 shows the logarithmic relationship for rough non-circular sections given in Equation (6.2.31) which is plotted against the power law approximation presented in Equation (6.2.32).

$$\frac{1}{\sqrt{f}} = 2\log\left(\frac{R}{ks}\right) + 2.34 \tag{6.2.31}$$

$$\frac{1}{\sqrt{f}} = 2.9\left(\frac{R}{k_s}\right)^{\frac{1}{6}} \tag{6.2.32}$$

Equation (6.2.32) provides an adequate description of Equation (6.2.31), giving an error of ± 5 %, within the range 5 < < 500. Given that the Manning roughness coefficient is related to the Darcy-Weisbach friction factor by n = $\frac{1}{\sqrt{f}}R^{\frac{1}{6}}$ or $\frac{R^{\frac{1}{6}}}{C}$ Substitution of Equation (6.2.32) into this expression produces:

$$n = 0.039k_s^{\frac{1}{6}} \tag{6.2.33}$$

where k_s is expressed in metres. Equation (6.2.33) relates the Manning n to the typical roughness height (k_s) used in Equation (6.2.31). The roughness height ks is often taken to be the D_{75} value (diameter which more than 75% material passes through) of the gravel bed material. The Manning equation has also been applied to open channels which are not hydraulically rough with some success. Ackers (1991) discusses reasons for this by comparing the Manning equation to the Blasius equation (Streeter and Wylie, 1981) for the friction factor in smooth wall turbulent flow.

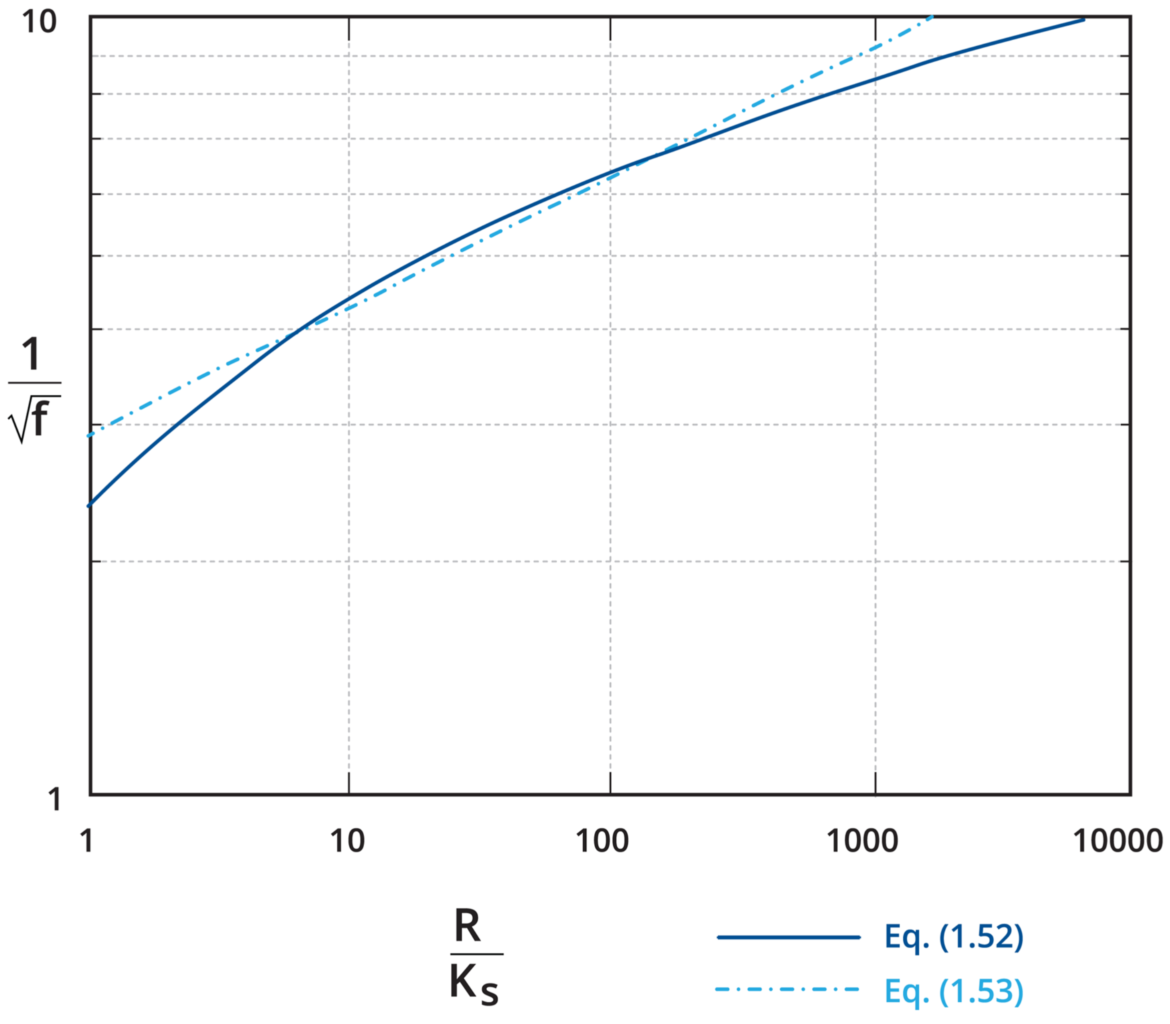


Figure 6.2.14. Comparison between Equation (6.2.31) and the power law approximation presented in Equation (6.2.32)

2.5.7. Uniform Flow in Channels of Compound Cross-Section

Compound channels are open channels with cross-sections which have berms or floodplains adjacent to a main channel that convey water at stages which exceed the bankfull depth as shown in Figure 6.2.15.

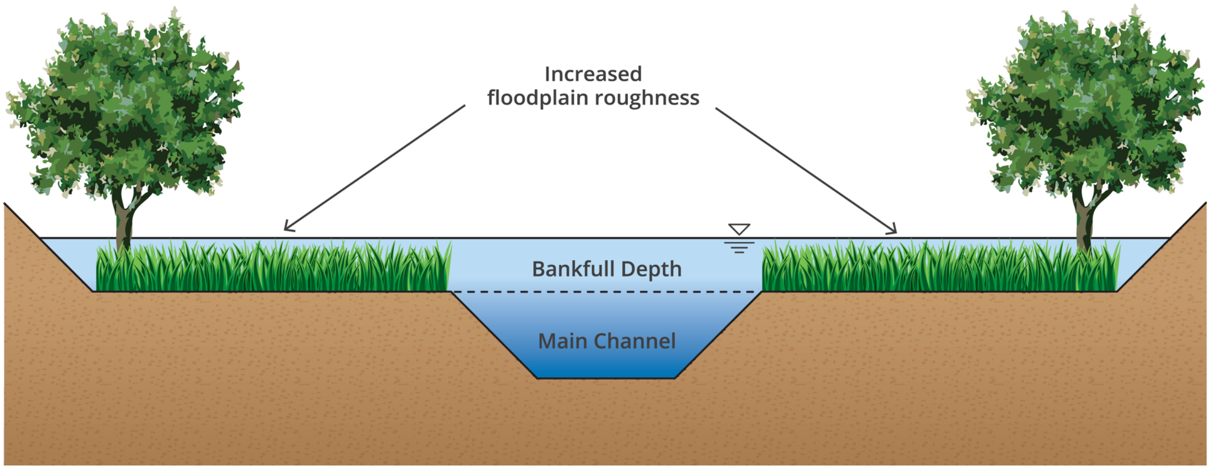


Figure 6.2.15. Typical compound channel with floodplains of greater roughness than the main channel

Typically the floodplains may be rougher than the main channel presenting the additional problem that the channel may have composite roughness as well as a complex or compound geometry. These features tend to set compound channels apart from simple prismatic channels for which the uniform flow depth can be predicted with some degree of accuracy.

Historically, compound channels were treated in the same manner as simple prismatic channels in which the overall hydraulic characteristics were used to calculate the discharge. This approach was formalised by Horton (1933) who gave the following relationship to calculate an equivalent Manning n (n_e) in simple channels where the roughness varied along the wetted perimeter. The same approach can be used for compound channels by employing Equation (6.2.34).

$$n_e = \left(\frac{\sum_{j=1}^{N} P_j n_j^{1.5}}{P} \right)^{\frac{2}{3}} \tag{6.2.34}$$

where P is the total wetted perimeter, P_j is the length of wetted perimeter associated with n_j and N is the number of different roughnesses. Both Horton (1933) and Einstein (1934) assumed that the water area is divided imaginatively into N parts (see Figure 6.2.16) for each different roughness. They then assumed that each part had the same velocity which is also equal to the average velocity of the whole section (ie. $u_1 = u_2 = u_3 = u$).

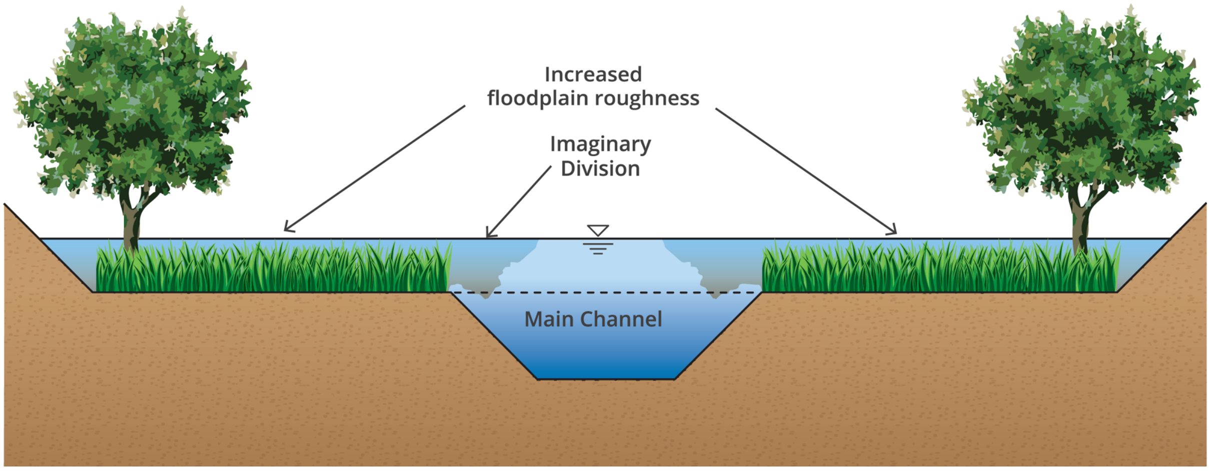


Figure 6.2.16. Imaginary division of a compound channel assumed by Horton (1933) to give the same average velocity on the floodplains and in the main channel.

While Horton's assumption is obviously invalid at low overbank depths when there is a large difference between the velocities of the two areas, work by Myers (1987) and Ackers (1991) shows that this approach produces satisfactory results for high overbank stages where the compound channel is once again tending to act as a single unit. This approach will be referred to as the Single Channel Method.

Lotter (1933) assumed that the total discharge is equal to the sum of the discharges in each sub-area. Lotter's approach, like Horton's, was developed to predict the discharge in a simple prismatic cross-section with varying roughness around the channel perimeter. In simple prismatic channels, but not in compound channels, it may be assumed that the hydraulic radius of each sub-area is equal. This however, does not restrict its application to compound channel flows. When the method suggested by Lotter (1933) is applied to compound channels some decisions need to be made regarding the subdivision of the channel cross-section. Typically for compound channels, a vertical division is used to separate the floodplain from the main channel as shown by the dashed lines in Figure 6.2.17.

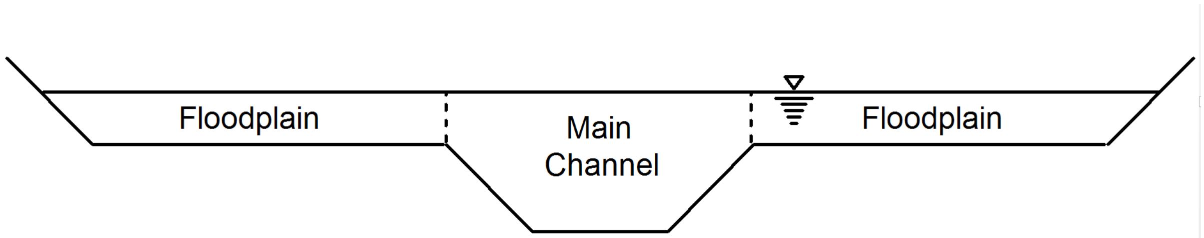


Figure 6.2.17. Vertical division of a compound channel into floodplain and main channel subsections

In compound channels, this leads to the assumption that the different sub-areas act independently of each other. As a result, the flood plain subsections and the main channel are treated as individual simple prismatic channels, and the discharge is obtained for each subsection by applying an appropriate resistance law, such as the Manning equation, to each subsection in turn. An expression for an equivalent Manning roughness coefficient (n_e) can also be obtained by this approach.

$$n_e = \frac{PR^{\frac{5}{3}}}{\sum_{j=1}^{N} \frac{P_j R_j^{\frac{5}{3}}}{n_j}} \tag{6.2.35}$$

where P and R are the overall wetted perimeter and hydraulic radius respectively and P_j, R_j and n_j are the wetted perimeter, hydraulic radius and Manning roughness coefficient of the j^{th} sub-area. This approach, along with the use of vertical divisions at the edge of the main channel, has become the most popular method of dealing with compound channels. The methods used to divide the channel into the individual subsections is however, somewhat arbitrary. While it does seem logical to separate the floodplains (a lower average velocity typically) from the main channel, it does assume that they act independently of each other.

As a result of the need to subdivide a compound channel into different sub-areas, the following horizontal and diagonal divisions as illustrated below are equally valid.

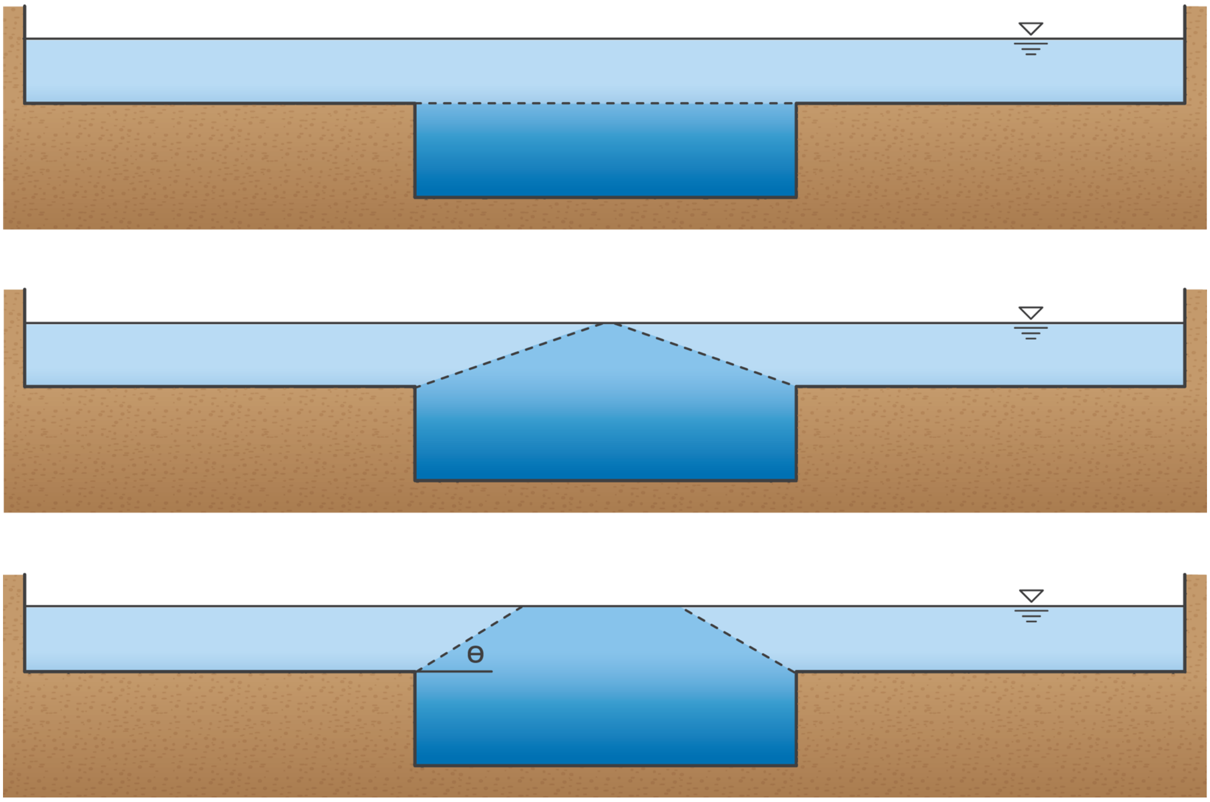


Figure 6.2.18. Alternative approaches to subdividing a compound channel cross-section.

Since 1964 evidence has been presented which demonstrates that flows in the different sub-areas do not act independently. The interaction between the faster moving water in the main channel and the slower moving water on the floodplain has the effect of reducing the overall discharge in the compound channel below the value that would be calculated assuming that they act independently. The first papers describing this phenomenon were Sellin (1964) and Zheleznyakov (1965).

Sellin (1964) provided photographic evidence of vortices which were believed to be the source of the interaction between the main channel and the floodplain. Sellin also provided experimental results which showed that the mean velocity in a channel with floodplains was approximately 30 % less than the same channel with the flood plains removed. Additionally, Sellin showed that the discharge in the channel was over-predicted by approximately 10 - 12 %. Even though these experiments were carried out at a relatively small scale they did serve to illustrate that the prediction of the discharge capacity of a compound channel was not as straightforward as originally thought.

In the time since 1964, a great deal of experimental research has been carried out on this phenomenon in straight compound channels. A lot of the work has concentrated on discharge assessment, boundary shear stress distribution, velocity distribution, momentum transfer and apparent shear stress, as well as the structure of the turbulent flow.

Several other investigators have conducted field tests on compound channels including Bhowmik and Demissie (1982), Sellin and Giles (1988), Myers and Lyness (1989) and Martin and Myers (1991). Bhowmik and Demissie (1982) showed that above the bankfull level the floodplain velocity increased with stage, but the main channel velocity first reduces with increasing stage and later increases. Work carried out by Sellin and Giles (1988) on the River Roding in the United Kingdom and (Myers and Lyness, 1989) along with (Martin and Myers, 1991) on the River Main in Northern Ireland found similar reductions in discharge capacity above the bankfull level.

Many of the above authors have attempted to quantify, by various means, the effect of the interaction between the main channel and the floodplain on the overall discharge, component discharges and boundary shear stress distribution. The methods that have been used to date can be broadly classified as follows:

1. Using the single channel method with modified resistance coefficients or interaction factors.

2. Adjusting the subdivision boundaries between the main channel and the floodplain, sometimes coupled with the inclusion of the internal subdivision boundaries in the wetted perimeter.

3. Applying correction factors to the discharge which are determined from experimental research.

4. Using experimental research to assess the apparent shear force on the assumed subdivision boundary. The discharge is then estimated by incorporating the apparent shear stress in the external force balance required for uniform flow in the main channel and floodplain sub-areas.

5. Using turbulence models to predict the lateral spread of the interaction zone in the compound channel resulting in the determination of the lateral velocity profile.

In the absence of other information, the vertical divisions at the edge of the main channel are still the favoured technique because it is easy to apply and calculate and divides the zones in a practical way. It is used in many water surface profiles calculation packages. Lambert and Sellin (Lambert and Sellin, 1996a) illustrate the use of (Point no. 5 above) and an approach for determining the interactions between the different regions.

2.5.7.1. Flow in Non-Straight and Meandering Compound Channels

While this section has dealt primarily with the flow in straight and uniform channels not all natural channels can be modelled in this manner and additional parameters may become important in the determination of the stage-discharge relationship.

2.5.7.1.1. The Effect of Skewness on Flow in Compound Channels

Experimental work by Elliot and Sellin (1990) showed that skewing of the main channel relative to the floodplains by only 5° to 10° was enough to introduce some deviation in the stage-discharge relationship that would be expected for a similar straight channel. They also found that the region of maximum velocity is shifted in the direction of the cross-flow and that the secondary currents appeared to be stronger and more complex than in a straight compound channel. Additionally, a strong peak in the boundary shear stress distribution occurred where the cross-flow left the main channel and moved onto the floodplain. This situation common with meandering streams where the floodplain switches sides of the main channel.

2.5.7.1.2. Meandering Compound Channels

The earliest report on the effect of main channel sinuosity on the stage-discharge relationship in a compound channel appears to be by Lipscomb (1956). Lipscomb concluded that an increase in sinuosity results in a decrease in discharge.

Similar experiments by Toebes and Sooky (1966) concluded that energy losses in the model depended on both the Reynolds number and Froude number and that energy losses per unit length for the meandering channel were up to 2.5 times as large as those for a uniform channel of the same hydraulic radius and discharge. The work was extended by James and Brown (1977) who also found that with increasing sinuosity, the resistance to flow increases and the velocity profiles become more distorted.

Smith (1978) carried out an experimental investigation into the effect of channel meanders on flood stage. In this investigation, he compared the stage-discharge relationship of a meandering compound channel to the stage-discharge relationship of the floodplain alone where the main channel was filled in and sealed with cement mortar. In doing this Smith found that at high stages the floodplain without the main channel had a larger discharge capacity than the combined meandering main channel and floodplain system. This demonstrated, for this channel geometry, that at high stages (Dr > 0.41) the addition of a main channel did not contribute to the discharge capacity, on the contrary, it decreased the discharge capacity. At lower relative depths this was not the case but the meandering compound channel still showed evidence of the interaction between the main channel and the floodplain.

Rajaratnam and Ahmadi (1983) undertook experiments on a curved main channel constructed inside a tilting rectangular channel 1.2 m wide and 18 m long. Rajaratnam and Ahmadi (1983) considered two relative depths $\left(\frac{H-h}{H}\right)$, one equal to 0.37 and the other equal to 0.45. From this work Rajaratnam and Ahmadi (1983) concluded:

1. The main channel was not exclusively the location of the highest velocities in the section.

2. The maximum velocity filament (also observed by Toebes and Sooky (1966)) tended to roughly follow the inner side walls of the main channel.

3. For the floodplain flow the velocity varied continuously with distance above the floodplain whereas the main channel velocity remained almost constant with distance above the floodplain level.

More recently, interest in meandering compound channels has provided the impetus for more detailed experimental studies. One of these studies followed the Series A experiments (Knight and Sellin, 1987) on straight compound channels at the SERC-Flood Channel

Facility at HR Wallingford. Details of this experimental program (Series B) can be found in Greenhill (1992) and Sellin et al. (1993). A consequence of this experimental work was the commissioning of HR Wallingford to undertake the production of a hydraulic manual for discharge assessment in meandering compound channels. This manual (Wallingford, 1992) is intended to provide engineers with a more accurate method of estimating the stage-discharge relationships in meandering compound channels. The method is based mainly on the SERC-Flood Channel Facility data on meandering compound channels but also includes other suitable data sources.

Subsequently, a more detailed record of this study was published by James and Wark (1992) which considered both in-bank and overbank flows. For in-bank flow conditions, a modification of an existing method (U.S. Department of Agriculture, 1963) was found to give satisfactory results. For overbank flow, a new approach was adopted which quantified the loss mechanisms which occur in meandering compound channels. The new method splits the flow into four flow zones:

• the inner channel below the bankfull level (1)

• The floodplain within the meander belt (2).

• The floodplains either side of the inner channel and outside the meander belt (3-4).

and then adopts an empirical approach but using, where possible, parameter groups to represent the known flow mechanisms in each zone. The discharge is then calculated as the sum of the zonal discharges. It should be noted however that this approach is similar to that suggested by Ervine and Ellis (1987).

The increased use of 2D flood models now provides much more flexibility to capture the complex nature of these flows and how they vary across the cross-section than previously existed with 1D approaches. However, it should be remembered that meandering compound channels flow can be highly three-dimensional in nature, particularly at the cross-over if the floodplain switches sides of the main channel and water must flow across the main channel to get to the downstream floodplain as shown in Lambert and Sellin (Lambert and Sellin, 1996b). The assumptions that form the basis of the depth-averaged 2D approaches break down in this case and these assumptions need to be checked.

2.6. Classification of the 1D Backwater and Drawdown Water Surface Profiles

When the gradually varied flow equation is applied to a steady flow down a prismatic channel, it can be shown that there are 12 generic, gradually varying flow, water surface profiles (apart from uniform flow). These profiles can be classified according to (Fenton, 2007):

- 5 conditions which compare the normal depth (y_o) with the critical depth (y_c). This results in the classification of 5 bedslopes.

- 3 conditions which compare the actual depth (y) with the normal depth (y_o) and the critical depth (y_c). This results in 3 zones for the depth.

Table 6.2.4 contains the bedslope and depth classifications.

Table 6.2.4. Gradually varied flow classification system (modified from table on p35 of Fenton (2007))

Bedslope Classification		
S	steep	$y_o < y_c$
C	critical	$y_o = y_c$
M	mild	$y_o > y$
H	horizontal ($S_o = 0$)	$y_o = y_c$
A	adverse (($S_o < 0$)	y_o does not exist
Depth Classification		
Zone 1		$y > y_o$ and $y > y_c$
Zone 2		y is between y_o and y_c
Zone 3		$y < y_o$ and $y < y_c$

The 12 gradually varied flow profiles are illustrated in Figure 6.2.19.

The 12 generic profiles are curves of increasing or decreasing curvature in either the downstream or upstream direction. When some simplifying assumptions are made (IF^2 <<1 and a linearisation about normal flow depth using a Taylor series has been employed), it has been shown that the departure of the actual depth (y) from the normal flow depth (y_0) follows an exponential variation in space (Samuels, 1989; Fenton, 2007); this is of the form y $-y_o$~e-dS_f /d_x x provided IF << 1 and where S_r can be found from the Manning or Chezy equation.

A consequence of the nature of thewater surface profiles is that any errors introduced during a backwater computation tend to be systematic. As the computations proceed in one direction (be it downstream or upstream), the flow curvature continuously increases or decreases.

All the water surface profiles schematised in Figure 6.2.19 can be classified as a backwater curve or a drawdown curve:

- backwater curve - the flow depths continuously increase in the downstream direction; the flow is one of deceleration.
- drawdown curve - the flow depths continuously decrease in the downstream direction; the flow is one of acceleration.

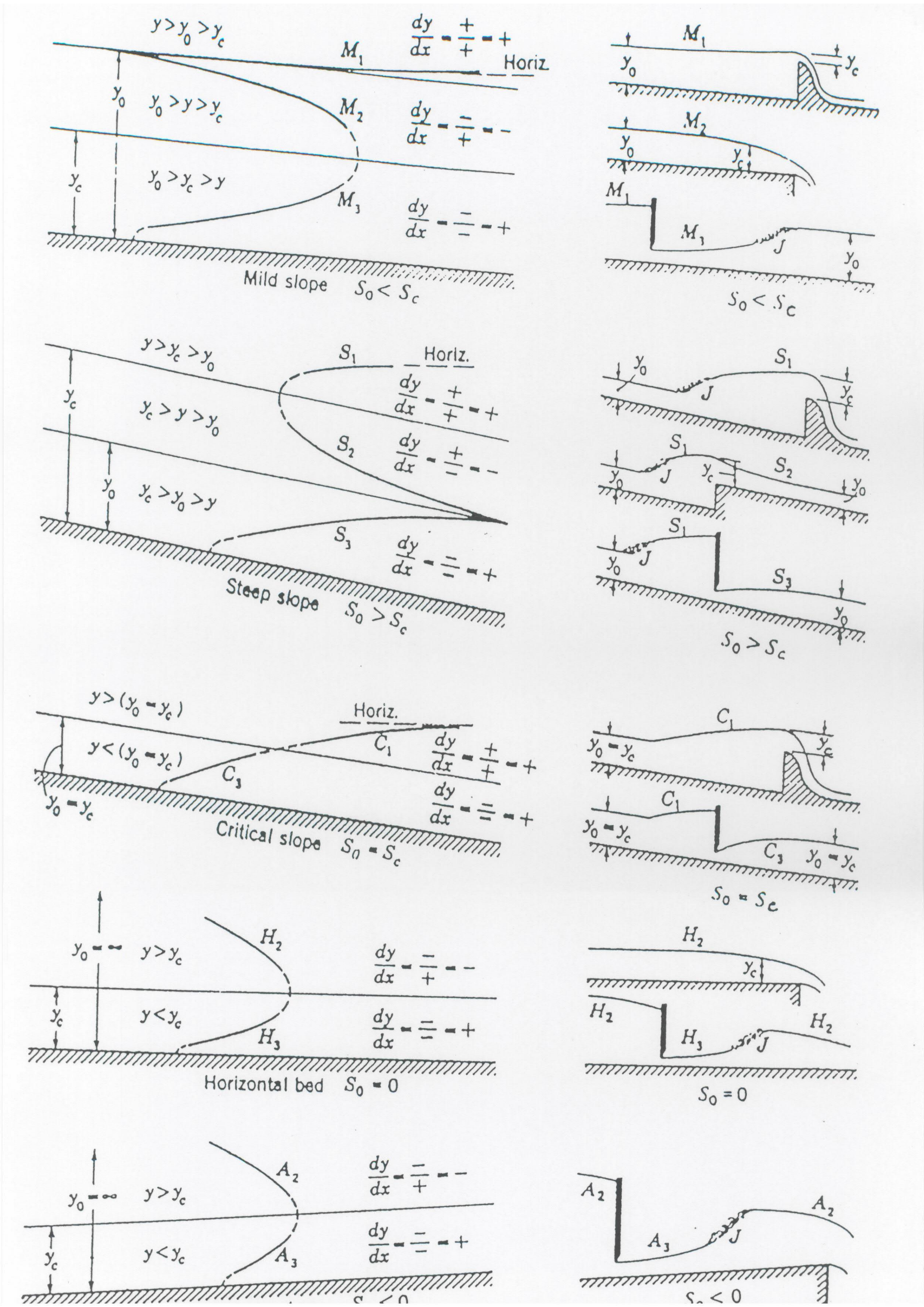


Figure 6.2.19. Drawdown Water Surfaces

2.7. Methods for Calculating Steady State Backwater and Drawdown Curves

Irrespective of whether steady-state flows are subcritical or supercritical, there are two well known methods for calculating the water surface profiles, be they (i) backwater curves or (ii) drawdown curves. In backwater profiles, the water surface deepens in the downstream direction and such flows are decelerating flows. In drawdown profiles, the water depths become shallower in the downstream direction and the flows are accelerating flows.

The two main numerical techniques for calculating steady water surface profiles are:

- the direct step method, and
- the standard step method.

A brief comparison of the two methods can be seen in Table 6.2.5.

Table 6.2.5. Comparison of the direct step and standard step methods

	Direct Step Method	Standard Step Method
Governing Equation	$\frac{\delta E}{\delta x} = S_o - S_f$	$\frac{\partial y}{\partial x} = \frac{S_o - S_f}{1 - Fr^2}$
Unknowns	Find location x for a specified depth y	Find depth y at a specified location x
Solution	Explicit equation - no iteration needed	Implicit equation - iteration needed
Restrictions	• prismatic channels • hydrostatic pressures • calculate upstream for Fr < 1 • calculate downstream for Fr < 1	• channels of general cross-section • hydrostatic pressures • calculate upstream for Fr < 1 • calculate downstream for Fr < 1

Since the direct step and other direct integration methods are based on a first order differential equation, a single boundary condition is required to initiate the computations. Suitable boundary conditions could be near (but not at) a control where the water level is known or can be approximated, or any location, reasonably well removed from any regions of rapidly varying flow, where the water level is known.

If the flow is subcritical, the flow is controlled from the downstream end and computations should advance in the upstream direction. On the other hand, for supercritical flows, the flow is controlled from the upstream end and computations should proceed in the downstream direction (McBean and Perkins, 1975; McBean and Perkins, 1970). If these guidelines regarding the direction of the solution procedure are not observed, it has been stated (McBean and Perkins, 1975; McBean and Perkins, 1970) that the calculations will eventually depart from the true solution. This rule-of-thumb is not without contention. There is some evidence which suggests that if an implicit finite difference method is employed to solve the governing equations, then the direction of computation is immaterial (Samuels and Chawdhary, 1992).

2.7.1. Direct Step Method for Calculating Backwater and Drawdown Curves

In the direct step method for calculating steady state, gradually varying flow profiles, the distance between two sections with specified depths is calculated.

The direct step method of calculating gradually varied flow profiles is only applicable to prismatic channels (which are more commonly manmade than natural). In this method, the flow depths at those sections where computations are to be carried out along the waterway are known (or specified) in advance. The (specified) depth increments or decrements between these sections need not be constant. With the depths (y) along the waterway known, the direct step method enables the distances between these sections (Δx) to be calculated directly without the need for iteration or trial and error. The governing equation is the first order ordinary differential equation:

$$\frac{\partial E}{\partial x} = S_o - S_f \qquad (6.2.36)$$

$$E = y + \frac{v^2}{2g} \qquad (6.2.37)$$

= specific head or specific energy (m) x = co-ordinate in downstream direction (m)

It is conventional to choose the x−coordinate such that it increases in the downstream direction, irrespective of whether the flow is subcritical or supercritical. When Equation (6.2.36) is recast in a finite difference form, Equation (6.2.36) becomes:

$$\frac{E_2 - E_1}{\Delta x} = S_o - S_f \qquad (6.2.38)$$

$$\Rightarrow \Delta x = \frac{E_2 - E_1}{S_o - S_f} \qquad (6.2.39)$$

where. . . 1, 2 = spatial indices which increase in the downstream direction

$$\Delta x = x_2 - x_1 \qquad (6.2.40)$$

$$\text{(bedslope) } S_0 = -\frac{\partial z}{\partial x} \qquad (6.2.41)$$

$$\text{(friction slope) } S_f = \frac{(nv)^2}{R_h^{4/3}} \qquad (6.2.42)$$

n is the Manning roughness parameter

S_f is the average friction slope over the elemental reach Δx

$$= \frac{\left(S_{f1} + S_{f2}\right)}{2} \qquad (6.2.43)$$

$S_{f\,1}$ = friction slope at Section 1

$S_{f\,2}$ = friction slope at Section 2

The reason that the direct step method is only applicable to prismatic channels is that the unknown in Equation (6.2.39) is Δx. In the case of a subcritical flow, the computations proceed in the upstream direction; while x_2 will be known (or specified), x_1 will be unknown. Unless the section properties at x_1 are known, the computation of the specific energy and friction slope at this location cannot (in principle) proceed. The requirement imposed by the direct step method is therefore that the channel be prismatic.

Since the direct step method is explicit, no iteration is needed. The solution of Equation (6.2.39) is straightforward and easily executed in tabular form on a spreadsheet.

2.7.2. Standard Step Method for Calculating Backwater and Drawdown Curves

The standard step method is more versatile than the direct step method in that it can be applied to irregular (usually natural) channels i.e. channels with cross-sections which are changing along the length of the waterway. The relevant equations for the standard step method are based on the definition for the total head, the difference in head between two sections separated by a horizontal distance Δx, and an expression for the friction slope based on the Manning (or Chezy or Colebrook-White) equation.

Figure 6.2.20 contains a definition diagram for calculating the gradually varying flow water surface profile for a subcritical flow.

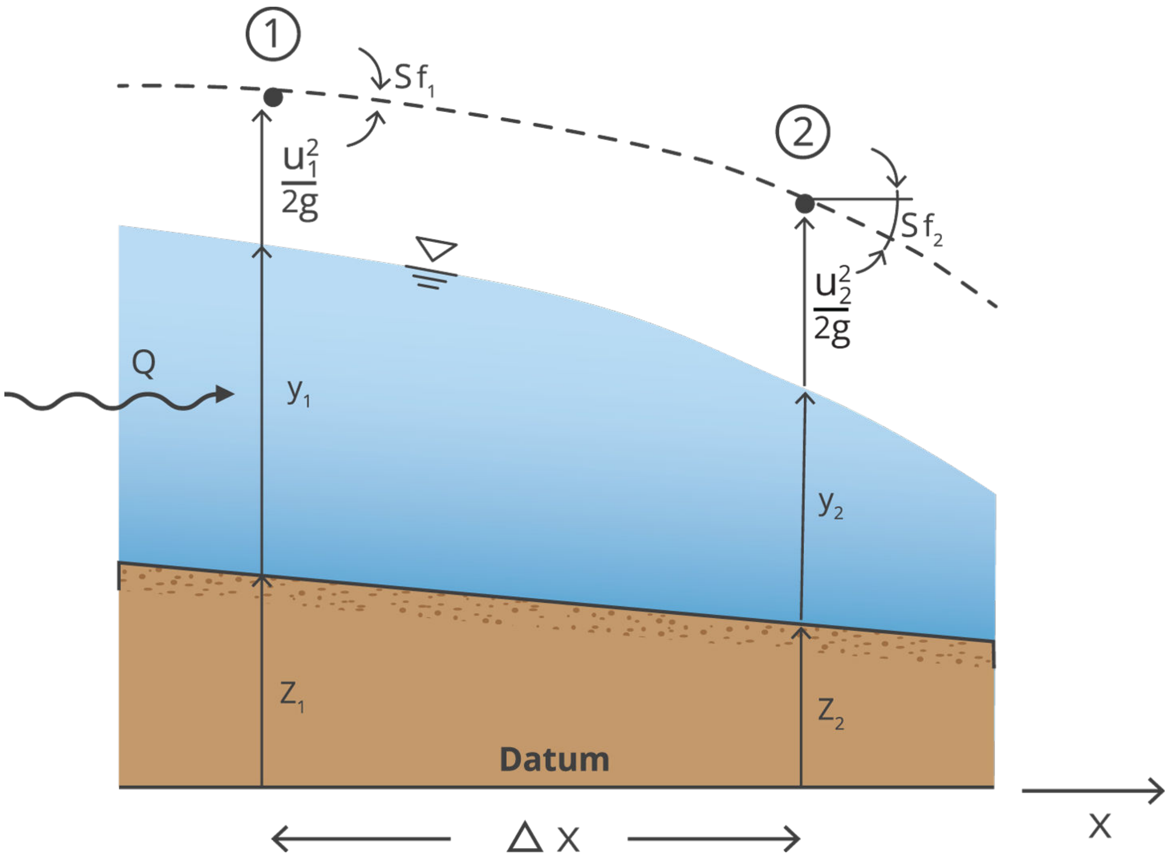


Figure 6.2.20. Lateral inflow

In the standard step method, the aim is to satisfy the two expressions below (Equation (6.2.44) and Equation (6.2.45)), and this will only happen when the correct unknown depth

y_1 has been determined. The depth y_2 is known for sub-critical flow (reversed for sueprcritical flow). Because the unknown depth y_1 is needed for the flow area A_1 and the friction slope S_{f1}, a trial and error process or some other iterative technique is needed to arrive at a solution.

$$H_1 = y_1 + z_1 + \frac{Q^2}{2gA_1^2} \tag{6.2.44}$$

$$H_1 = H_2 + \Delta x \frac{\left(S_{f1} + S_{f2}\right)^2}{2} \tag{6.2.45}$$

$$\text{where... } S_f = \frac{(nQ)^2}{A^2 R_h^{4/3}} \tag{6.2.46}$$

If the Newton-Raphson technique is applied to H_E = the difference in H_1 as given by Equation (6.2.44) less that from Equation (6.2.45), the solution process can be speeded up:

$$(y_1)^{now} = (y_1)^{old} - \frac{H_E}{1 - IF_1^2 + \frac{\Delta x}{y_1}\left(1 + \frac{2Rh_1}{3y_1}\right) S_{f1}} \tag{6.2.47}$$

$$(\text{for a wide channel with } \mathrm{Rh}_1 \approx y_1) \approx (y_1)^{old} - \frac{H_E}{1 - IF_1^2 + \frac{5}{3}\left(1 + \frac{\Delta x}{y_1}\right) S_{f1}} \tag{6.2.48}$$

where. . . H_E = error or difference between the two values of H_1 in Equation (6.2.44) and Equation (6.2.45)

$$= \left[y_1 + z_1 + \frac{Q^2}{2gA_1^2}\right] - \left[H_2 + \Delta x \frac{\left(S_{f1} + S_{f2}\right)}{2}\right] \tag{6.2.49}$$

$(y_1)^{old}$ = previous value of y_1

$(y_1)^{new}$ = new value of y_1 after iteration

In the case of a supercritical flow, the equations above would need to be modified with the unknown being y_2.

2.7.3. Averaging Required in Water Surface Profile Calculations

Because Equation (6.2.36) is an ordinary differential equation, they are a point relation which holds at all points in the 1D continuum. When these equations are discretised using (say) a finite difference method, the resulting equations span a small elemental reach of length Δx as shown in Equation (6.2.38) which contains non-linear terms that are discretised using a finite difference method, that in both cases, there are some non-linear terms which require representation over the length of the elemental reach Δx. Various methods have been adopted to provide these approximations, and four versions are identified below with respect to the friction slope S_f at sections j and (j + 1).

$$(\text{average conveyance}) \left(\overline{S_f}\right)_K = \left[\frac{Q}{K}\right]^2 \tag{6.2.50}$$

$$\text{(arithmetic mean)}\ \left(\overline{S_f}\right)_a = \frac{1}{2}\left[\left(S_f\right)_1 + \left(S_f\right)_2\right] \qquad (6.2.51)$$

$$\text{(geometric mean)}\ \left(\overline{S_f}\right)_g = \sqrt{\left(S_f\right)_1\left(S_f\right)_2} \qquad (6.2.52)$$

$$\text{(harmonic mean)}\ \left(\overline{S_f}\right)_h = \frac{1}{\frac{1}{2}\left[\left(S_f\right)_1 + \left(S_f\right)_2\right]} \qquad (6.2.53)$$

$$\text{where...}\ \bar{v} = \frac{1}{2}\left(v_1 + v_2\right) \qquad (6.2.54)$$

$Q = K.S^{1/2}$

K = conveyance

$K = 1/2\ (K_1 + K_2)$

The four averages listed above vary systematically (Laurenson (1986)) so that:

$\left(\overline{S_f}\right)_a > \left(\overline{S_f}\right)_g > \left(\overline{S_f}\right)_K > \left(\overline{S_f}\right)_h$

In addition to the above averages, there are many other equations which average reach-end parameters A, P, Rh by arithmetic, geometric or harmonic methods, but the averaged friction slope values have all been found to lie between the two extremes given by $\left(\overline{S_f}\right)_a$ and $\left(\overline{S_f}\right)_h$ (Cahdderton and Miller, 1980).

The effect of using the various estimates for the average friction slope have been explored by various investigators. Laurenson's conclusion (Laurenson, 1986) was that the best single method of averaging appears to be the arithmetic average of reach-end friction slopes, especially if this method is used in concert with the selection of representative cross-sections of the channel.

2.8. One Dimensional Unsteady Flow Equations

The governing equations for free surface flow are based on considerations of mass, momentum and energy. There are various combinations of dependent variables which are used, e.g. stage and flow; depth and flow; flow area and flow; depth and (cross-sectional averaged) velocity; and stage and velocity. Moreover, the equations can be expressed in conservation form or nonconservation form. The factors above give rise to different forms of the governing equations. Several of these forms are listed below.

It should be noted that the various forms of the equations below are not equivalent and some forms may be preferred over others due to:

- conservation vs non-conservation (i.e. divergent) form,
- choice of variables may be more accurate than others (Cunge et al., 1980), or
- if there are discontinuous solutions (Cunge et al., 1980).

By considering unsteady flow in an open channel through the following control volume shown in Figure 6.2.21 below:

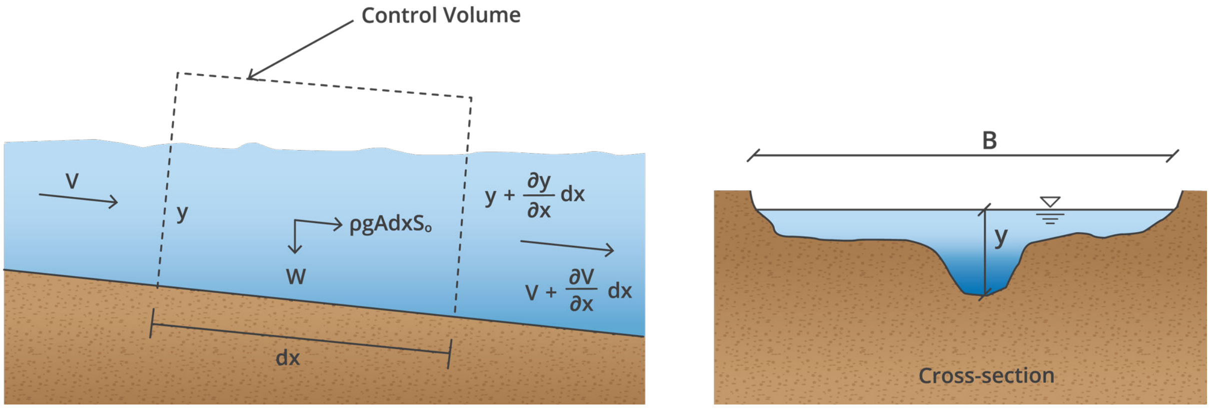


Figure 6.2.21. Control volume used to derive the gradually varying unsteady flow equations

In dealing with the control volume it has been assumed that the flow is incompressible, one dimensional and that the streamlines are straight and parallel. It has also been assumed that the slope of the channel is small, so that $\sin\theta \approx S_0$ and that there is no lateral inflow. Additionally, it has been assumed that the geometry of the channel does not change with time.

2.8.1. Derivation of the Continuity Equation for Gradually Varied Unsteady Flow in an Open Channel

For the control volume shown in Figure 6.2.21 the continuity equation can be derived using Equation (6.2.55) where *M* is the mass of the system of particles instantaneously occupying the control volume.

$$\frac{dM}{dt} = \frac{\partial}{\partial t}\int_{c\vartheta} \rho d\vartheta + \int_{c\vartheta} \rho \tilde{v} \tilde{d}A \quad (6.2.55)$$

where $\tilde{v}$ is the velocity vector, $\tilde{d}A$ is a vector with a magnitude equal to dA in a direction normal to the elemental area, ρ is the fluid density, t is time. Applying the above equation to the control volume shown in Figure 6.2.21 and evaluating the integrals yields:

$$\frac{dM}{dt} = \frac{\partial}{\partial t}\rho dx + \rho\left(V + \frac{\partial V}{\partial x}\partial x\right)\left(A + \frac{\partial A}{\partial x}\partial x\right) - \rho VA \quad (6.2.56)$$

Expanding Equation (6.2.56), ignoring second order terms and dividing by ρdx produces the Unsteady Continuity Equation.

$$\frac{\partial A}{\partial t} + u\frac{\partial A}{\partial x} + A\frac{\partial u}{\partial x} = 0 \quad (6.2.57)$$

where *A* is the cross-sectional area, *U* is the average velocity, *t* is time and *x* is the longitudinal distance along the channel.

2.8.2. Derivation of the Momentum Equation for Gradually Varied Unsteady Flow in an Open Channel

Application of Reynolds Transport Theorem to the control volume shown in Figure 6.2.21 for the momentum in the x-direction yields the one-dimensional unsteady momentum equation for open channel flow:

$$\sum F_x = \frac{\partial}{\partial t}\int_{c\vartheta}\rho v_x d\vartheta + \int_{c\vartheta}\rho v_x \tilde{v}\tilde{d}A = 0 \tag{6.2.58}$$

where $\sum F_x$ is the sum of the external forces acting in the x-direction on the control volume in Figure 6.2.21 and v_x is the velocity in the x-direction. By considering all the external forces acting on the system and evaluating the volume and surface integral terms on the right side of Equation (6.2.58) yields:

$$\rho g A dx S_0 - \rho g \frac{\partial y}{\partial x} dx A - \tau_o P dx = \frac{\partial}{\partial t}(AV)\rho dx + \frac{\partial}{\partial t}\left(\beta \rho A V^2\right) dx \tag{6.2.59}$$

where β is the momentum correction coefficient. By dividing by $\rho g A dx$ and expanding the derivative terms on the right-hand side, Equation (6.2.59) produces:

$$S_0 - \frac{\partial y}{\partial x} - \frac{\tau_o P}{\rho g A} = \frac{1}{gA}\left(A\frac{\partial V}{\partial t} + V\frac{\partial A}{\partial t} + \beta\frac{\partial}{\partial t}\left(AV^2\right) + AV^2\frac{\partial \beta}{\partial t}\right) \tag{6.2.60}$$

However, many of the individual terms in Equation (6.2.60) can be replaced by more convenient forms. For example:

$$\frac{\partial y}{\partial x} = \frac{1}{T}\frac{\partial A}{\partial x} \tag{6.2.61}$$

$$\frac{\partial \beta}{\partial x} = \frac{\partial \beta}{\partial y}\frac{\partial y}{\partial x} = \frac{1}{T}\frac{\partial \beta}{\partial y}\frac{\partial A}{\partial x} \tag{6.2.62}$$

and

$$\frac{\partial}{\partial x}\left(AV^2\right) = V^2\frac{\partial A}{\partial x} + 2AV\frac{\partial V}{\partial x} = V\left(V\frac{\partial A}{\partial x} + A\frac{\partial V}{\partial x}\right) + AV\frac{\partial V}{\partial x} \tag{6.2.63}$$

$$V\frac{\partial A}{\partial x} = \beta V\frac{\partial A}{\partial t} + (1-\beta)V\frac{\partial A}{\partial t} \tag{6.2.64}$$

Equation (6.2.60) can be rearranged after substitution of Equation (6.2.61) to Equation (6.2.64) and S_f for the friction slope to give Equation (6.2.65) below:

$$S_0 - \frac{1}{T}\frac{\partial A}{\partial t} - S_f = \frac{1}{gA}\left(A\frac{\partial V}{\partial t} + (1-\beta)V\frac{\partial A}{\partial t} + \beta V\left(\frac{\partial A}{\partial t} + V\frac{\partial A}{\partial x} + A\frac{\partial V}{\partial x}\right) + \beta AV\frac{\partial V}{\partial x} + \frac{AV^2}{T}\frac{\partial \beta}{\partial y}\frac{\partial A}{\partial x}\right) \tag{6.2.65}$$

The continuity equation (Equation (6.2.57)) can now be used to eliminate the third term on the right-hand side giving the Unsteady Momentum Equation.

$$S_0 - \frac{1}{T}\frac{\partial A}{\partial t} - S_f = \frac{1}{g}\frac{\partial V}{\partial t} + \frac{(1-\beta)}{gA}V\frac{\partial A}{\partial x} + \frac{\beta V}{g}\frac{\partial V}{\partial x} + \frac{V^2}{gT}\frac{\partial \beta}{\partial y}\frac{\partial A}{\partial x} \tag{6.2.66}$$

where S_f is the friction slope, S_o is the longitudinal bed slope, V is the mean cross-sectional velocity, A is the cross-sectional area, T is the channel top width, g is the gravitational acceleration, t is time and x is the distance in the direction of flow. This partial differential equation is the unsteady momentum equation for flow in open channels and includes the momentum correction factor (b) to account for a non-uniform distribution of velocity in the cross-section. Equation (6.2.66) is a more general form of the Saint-Venant equation. It was

Boussinesq in 1877 who first incorporated correction coefficients for the velocity distribution in the momentum equation. While he originally proposed three coefficients only one (b) is used in modern literature and is given by Chow (1959) as:

$$\beta = \frac{\int u^2 dA}{V^2 A} \tag{6.2.67}$$

where u is the local velocity through the elemental area *dA*, *A* is the cross-sectional area, and *V* is the mean velocity.

2.8.3. Why is the time step so important?

Often these equations are solved using finite difference, finite element or more commonly now finite volume methods. As these are a hyperbolic system of partial differential equations, very similar to the wave equations consideration must be given to the time step that is used in the computational scheme for both computational stability and more importantly for computational accuracy. While this is sometimes controlled automatically to maintain accuracy it is worth understanding the importance of the time step.

The unsteady momentum equation can be converted into a total differential equation using the method of characteristics. While the steady form of the momentum equation can be obtained from Equation (6.2.66) the transformation to a system of total differential equations will be carried out for later use.

The method of characteristics allows two partial differential equations to be combined using an unknown multiplier (λ_m) as shown below in Equation (6.2.68). For any two real and distinct values of λ_m two equations in *V* and *A* are obtained that contain the properties of the original two equations L_1 and L_2 and may replace them in any solution.

$$L = L_1 + \lambda_m L_2 = 0 \tag{6.2.68}$$

where L_1 and L_2 are equal to the unsteady momentum function (Equation (6.2.66)) and continuity relation (Equation (6.2.57)) respectively as shown below:

$$L_1 = \frac{1}{g}\frac{\partial V}{\partial t} + \frac{(1-\beta)}{gA}V\frac{\partial A}{\partial t} + \frac{\beta V}{g}\frac{\partial V}{\partial x} + \frac{1}{T}\left(1 + \frac{V^2}{g}\frac{\partial \beta}{\partial y}\right)\frac{\partial A}{\partial x} + S_f - S_0 \tag{6.2.69}$$

$$L_2 = \frac{\partial A}{\partial t} + V\frac{\partial A}{\partial x} + A\frac{\partial V}{\partial x} \tag{6.2.70}$$

Substituting Equation (6.2.69) and Equation (6.2.70) into Equation (6.2.68) yields:

$$\begin{aligned}&\frac{1}{g}\frac{\partial V}{\partial t} + \frac{(1-\beta)}{gA}V\frac{\partial A}{\partial t} + \frac{\beta V}{g}\frac{\partial V}{\partial x} + \frac{1}{T}\left(1 + \frac{V^2}{g}\frac{\partial \beta}{\partial y}\right)\frac{\partial A}{\partial x} + S_f - S_0 + \lambda_m\frac{\partial A}{\partial t} + \lambda_m V\frac{\partial A}{\partial x}\\ &+ \lambda_m A\frac{\partial V}{\partial x}\end{aligned} \tag{6.2.71}$$

If Equation (6.2.71) is rearranged by collecting separately the derivatives of velocity and area then:

$$\frac{1}{g}\left[(\beta V+\lambda_m gA)\frac{\partial V}{\partial x}+\frac{\partial V}{\partial t}\right]+\lambda_m\left[V+\frac{1}{\lambda_m T}\left(1+\frac{V^2}{g}\frac{\partial \beta}{\partial y}\right)\frac{\partial A}{\partial x}+\left(\frac{(1-\beta)V}{\lambda_m gA}\right)\frac{\partial A}{\partial t}\right]+S_f - S_0=0 \quad (6.2.72)$$

The total derivatives of *V*, *A* with respect to *t* are:

$$\frac{dV}{dt}=\frac{\partial V}{\partial t}+\frac{\partial V}{\partial x}\frac{dx}{dt} \ \ and \ \ \frac{dA}{dt}=\frac{\partial A}{\partial t}+\frac{\partial A}{\partial x}\frac{dx}{dt} \quad (6.2.73)$$

Equation (6.2.72) is therefore modified to:

$$\frac{1}{g}\left[(\beta V+\lambda_m gA)\frac{\partial V}{\partial x}+\frac{\partial V}{\partial t}\right]+\left(\lambda_m+\frac{(1-\beta)V}{gA}\right)$$
$$\left[\left(V+\frac{1}{\lambda_m T}\left(1+\frac{V^2}{g}\frac{\partial \beta}{\partial y}\right)\right)\left(\frac{\lambda_m gA}{\lambda_m gA+(1-\beta)V}\right)\frac{\partial A}{\partial x}+\frac{\partial A}{\partial t}\right]+S_f-S_0=0 \quad (6.2.74)$$

This leads to the following total differential equations by equating Equation (6.2.73) with Equation (6.2.74).

$$\frac{1}{g}\frac{dV}{dt}+\left(\lambda_m+\frac{(1-\beta)V}{gA}\right)\frac{dA}{t}+S_f-S_0=0 \quad (6.2.75)$$

$$\frac{dx}{dt}=(\beta V+\lambda_m gA)=\frac{\lambda_m gA\left(V+\frac{1}{\lambda_m T}\left(1+\frac{v^2}{g}\frac{\partial\beta}{\partial y}\right)\right)}{\lambda_m gA+(1-\beta)V} \quad (6.2.76)$$

Now solving for λ_m in Equation (6.2.76) to produce equal values of the term associated with both the velocity and area derivatives produces:

$$(\lambda_m gA+(1-\beta)V)(\beta V+\lambda_m gA)=\lambda_m gA\left(V+\frac{1}{\lambda_m T}+\frac{V^2}{g}\frac{\partial\beta}{\partial y}\right) \quad (6.2.77)$$

Expanding Equation (6.2.77) yields a quadratic equation in λ_m:

$$\lambda_m\beta A+\lambda_m^2 g^2A^2+(1-\beta)\beta V^2+\lambda_m gA(1-\beta)V=\lambda_m gAV+\frac{gA}{T}+V^2\frac{A}{T}\frac{\partial\beta}{\partial y} \quad (6.2.78)$$

Collecting and cancelling terms in Equation (6.2.78) results in the following solution for λ_m given in Equation (6.2.79):

$$(\lambda_m gA)^2=\frac{gA}{T}-\beta(1-\beta)V^2+\frac{V^2A}{T}\frac{\partial\beta}{\partial y} \quad (6.2.79)$$

$$\lambda_m gA=\pm\sqrt{\frac{gA}{T}-\beta(1-\beta)V^2+\frac{V^2A}{T}\frac{\partial\beta}{\partial y}} \quad (6.2.80)$$

$$\lambda_m gA=\pm\sqrt{\frac{gA}{T}+\beta V^2-V^2\beta+V^2\frac{A}{T}\frac{\partial\beta}{\partial y}} \quad (6.2.81)$$

$$\lambda_m gA=\pm\sqrt{\frac{gA}{T}+\beta^2V^2-V^2\beta+V^2\frac{A}{T}\frac{\partial\beta}{\partial y}} \quad (6.2.82)$$

The method of characteristics when applied to Equation (6.2.57) and Equation (6.2.66) transform these two partial differential equations into the following system of total differential equations.

$$\frac{1}{g}\frac{dV}{dt}+\frac{1}{gA}\left[(1-\beta)V\pm\sqrt{\frac{gA}{T}+\beta^2V^2-V^2\beta+V^2\frac{A}{T}\frac{\partial\beta}{\partial y}}\right]\frac{dA}{dt}+S_f-S_0 \tag{6.2.83}$$

and

$$\frac{dx}{dt}=\beta V\pm\sqrt{\frac{gA}{T}+\beta^2V^2-V^2\left(\beta-\frac{A}{T}\frac{\partial\beta}{\partial y}\right)}=\beta V\pm c_\beta \tag{6.2.84}$$

Or

$$\frac{1}{g}\frac{dV}{dt}+\frac{(1-\beta)V\pm c_\beta}{gA}\frac{dA}{dt}+S_f-S_0=0 \tag{6.2.85}$$

$$\frac{dx}{dt}=\beta V\pm c_\beta \tag{6.2.86}$$

where c_β is the celerity of a small disturbance. When β is set equal to unity (a common assumption) then $c_\beta=\sqrt{\frac{gA}{T}}$ for a non-rectangular section or for rectangular sections. The characteristic direction represents the direction along which a small disturbance travels is equal to the absolute wave velocity of a disturbance. The positive sign is used for the downstream direction, and the negative sign is used for the upstream direction. Referring to the Equation (6.2.84), *dx/dt* is positive for both alternatives of Equation (6.2.84) when the first term (βV) on the right hand side of Equation (6.2.84) is greater than the square root term. This represents supercritical flow and the disturbances can only travel in the downstream direction. Similarly, *dx/dt*is negative for the upstream direction and positive for the downstream direction when the first term (βV) on the right hand side of Equation (6.2.84) is smaller than the square root term. This case represents subcritical flow and the disturbances travel in both upstream and downstream directions. For critical flow, both terms of Equation (6.2.84) are equal. This physically based criterion may be used to determine the occurrence of critical flow, since it shows whether the flow control is located at an upstream or downstream location. The Froude number, which is so important in determining the flow type (subcritical, supercritical, and critical) is the ratio of the first term on the right hand side of Equation (6.2.84) to the second term on the right hand side. In a rectangular open channel flow (where β is usually set to unity) the Froude number becomes:

$$F=\frac{V}{\sqrt{gy}} \tag{6.2.87}$$

This relationship between space (dx) and time (dt) that results is termed the Courant number, and it seeks to ensure that the various disturbances or changes at one time level are captured appropriately at the next advanced time level. While some numerical schemes do not require this for stability, they will require it for accuracy. This is made even more important in 2D and 3D flow computations where the disturbances now need to be captured moving in the plane of the channel cross-section and not just along the channel.

2.8.4. Steady Flow Equations

The unsteady momentum equation (see Equation (6.2.66)) can be reduced to the steady form of the equation by eliminating the time derivative terms from Equation (6.2.88) as shown:

$$\frac{1}{g}\frac{V}{dt}+\frac{(1-\beta)}{gA}V\frac{\partial A}{\partial t}+\frac{\beta V}{g}\frac{\partial V}{\partial x}+\frac{V^2}{gT}\frac{d\beta}{dy}\frac{\partial A}{\partial x}+\frac{1}{T}\frac{\partial A}{\partial x}+S_f-S_0=0 \tag{6.2.88}$$

$$\frac{\beta V}{g}\frac{\partial V}{\partial x}\left[\frac{1}{T}+\frac{V^2}{gT}\frac{d\beta}{dy}\right]\frac{\partial A}{\partial x}+S_f-S_0=0 \tag{6.2.89}$$

Substituting $\frac{dV}{dx}$ for using the equation of continuity for steady flow:

$$\frac{dV}{dx}=\frac{-V}{A}\frac{dA}{dx} \tag{6.2.90}$$

and by letting:

$$\frac{dA}{dx}=\frac{dA}{dy}\frac{dy}{dx}=T\frac{dy}{dx} \tag{6.2.91}$$

an equation $\frac{dy}{dx}$ determining for steady gradually varied flow using the momentum approach (Equation (6.2.93)) is obtained by the substitution of Equation (6.2.90) and Equation (6.2.91) into Equation (6.2.88) and rearranging to give:

$$\frac{-\beta V^2 T}{gA}\frac{dy}{dx}+\left(\frac{1}{T}+\frac{V^2}{gT}\frac{d\beta}{dy}\right)T\frac{dy}{dx}+S_f-S_0=0 \tag{6.2.92}$$

$$\frac{dy}{dx}=\frac{S_0-S_f}{1-\frac{V^2}{gT}\left(\beta-\frac{A}{T}\frac{d\beta}{dy}\right)} \tag{6.2.93}$$

Taking the momentum correction coefficient as being equal to unity then:

$$\frac{dy}{dx}=\frac{S_0-S_f}{1-\frac{V^2}{gT}}=\frac{S_0-S_f}{1-F^2} \tag{6.2.94}$$

Note Equation (6.2.93) includes the derivative of the momentum flux correction coefficient. Similarly for the unsteady energy equation (Equation (6.2.95)):

$$\frac{\beta}{g}\frac{\partial V}{\partial t}=\frac{V}{2g}\left(3\alpha+\beta-\frac{\beta' A}{T}\right)\frac{\partial V}{\partial x}+\frac{V^2}{2gA}\left(\alpha-\beta-\frac{\beta A'}{T}-\frac{A\alpha'}{T}\right)\frac{\partial A}{\partial x}+\frac{1}{T}\frac{\partial A}{\partial x}-S_0+S_e=0 \tag{6.2.95}$$

Eliminating the time derivative terms and substituting Equation (6.2.90) and Equation (6.2.91) provides an equation determining $\frac{dy}{dx}$ for steady gradually varied flow (Equation (6.2.98)) after some rearrangement as shown in Equation (6.2.96) and Equation (6.2.97).

$$\frac{3}{2}\frac{\alpha V}{g}\frac{dV}{dx}+\frac{1}{T}\frac{dA}{dx}+\left[\frac{\alpha V^2}{gA}+\frac{V^2}{gT}\frac{\partial A}{\partial y}\right]\frac{dA}{dx}+S_e-S_0=0 \tag{6.2.96}$$

$$\frac{dy}{dx}\left[1-\frac{V^2 T}{gA}\left(\alpha-\frac{A}{2T}\frac{\partial A}{\partial y}\right)\right]=S_0-S_e \tag{6.2.97}$$

$$\frac{dy}{dx} = \frac{S_o - S_e}{1 - \frac{V^2 T}{gA}\left(\alpha - \frac{A}{2T}\frac{\partial \alpha}{\partial y}\right)} \tag{6.2.98}$$

Again taking the energy correction factor as unity :

$$\frac{dy}{dx} = \frac{S_0 - S_e}{1 - \frac{V^2 T}{gA}} = \frac{S_0 - S_e}{1 - F^2} \tag{6.2.99}$$

It should be noted that Equation (6.2.98) includes an additional term for the derivative of the kinetic energy correction coefficient, which is commonly neglected.

2.8.5. Simplifying from Gradually Varied to Steady Uniform Flow

In the case of steady uniform flow the depth and velocity are not changing with distance along the channel hence, $\frac{dy}{dx} = 0$. From Equation (6.2.94) and Equation (6.2.98) it can be seen that this only occurs when $S_0 = S_f = S_e$.

2.9. Numerical Modelling - Two Dimensional Models of Flood Flows

Fully 2D hydrodynamic models are based on the numerical solution of depth-averaged equations describing the conservation of mass and momentum in two horizontal dimensions *x* and *y*. In a form used by many of the commonly used 2D models, these equations can expressed in terms of three main dependent variables; ς, *u and v*, as shown in Figure 6.2.22.

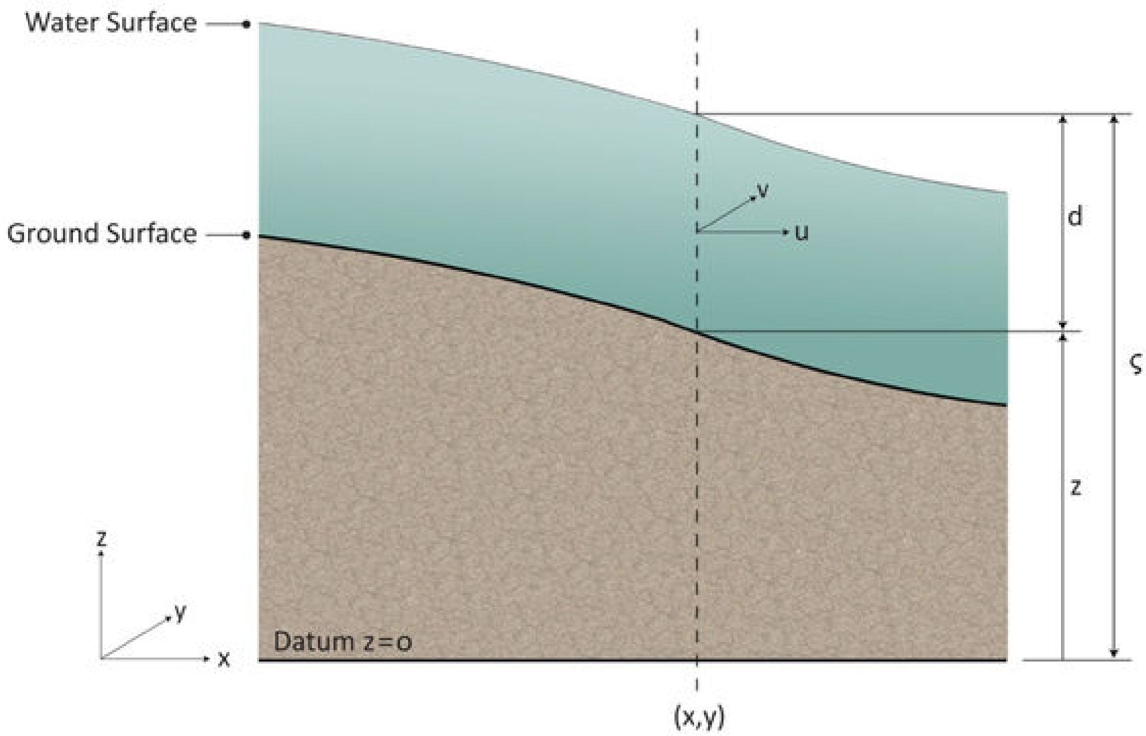


Figure 6.2.22. Definition of symbols

Where:

ς: is the water surface elevation relative to a fixed datum *(m).*

u: is the depth-averaged velocity in the *x* direction *(m/s)*

v: is the depth-averaged velocity in the *y* direction *(m/s)*

These are described as a function of the three main independent variables:

x: the horizontal distance in the x direction (*m*)

y: the horizontal distance in the y direction (*m*)

t: the time (*s*)

Additionally, the time varying water depth at any location *d(x,y),* can be expressed as:

where:

$$d = \varsigma - z \tag{6.2.100}$$

z: is the bed surface elevation relative to a fixed datum *(m).*

2.9.1. The Mass Equation

For flooding applications, water can be considered to be incompressible. As such, water volume can be used to represent the water mass. In terms of the variables described above, the depth-averaged equation describing the conservation of volume (and therefore mass) in two horizontal directions can be expressed as:

$$\frac{\partial \varsigma}{\partial t} + \frac{\partial (d.u)}{\partial x} + \frac{\partial (d.v)}{\partial y} = 0 \tag{6.2.101}$$

Where:

$\frac{\partial \varsigma}{\partial t}$ is the rate of increase (or decrease) in water level, which for a fixed cell size is representative of the rate of change of volume of water contained in the cell, and

$\frac{\partial (d.u)}{\partial x} + \frac{\partial (d.v)}{\partial y}$ is the spatial variation in inflow (or outflow) across the cell in the *x* and *y* directions.

Simply put, any increase (or decrease) in volume, must be balanced by a net inflow (or outflow) of water.

2.9.2. The Momentum Equations

In a similar form, the equations for describing the conservation of momentum in the *x* and *y* directions can be expressed as:

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + g\frac{\partial \varsigma}{\partial x} = 0 \tag{6.2.102}$$

$$\frac{\partial v}{\partial t} + v\frac{\partial v}{\partial y} + u\frac{\partial v}{\partial x} + g\frac{\partial \varsigma}{\partial y} = 0 \qquad (6.2.103)$$

where:

g: is the acceleration due to gravity (m/s^2)

The equations presented above are in the primitive, Eulerian form. The same equations can exist in other forms; e.g. the conservation law form (Abbott, 1979) and the conservative-integral form (LeVeque, 2002).

Due to the symmetry between the two *x* and *y* momentum equations, further discussion will be focused on the x-momentum equation only.

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + g\frac{\partial \varsigma}{\partial x} = 0 \qquad (6.2.104)$$

where:

$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y}$ is the partial differential form of the flow acceleration *du/dt*

$g\frac{\partial \varsigma}{\partial x}$ is the hydrostatic pressure gradient

It can be shown that the momentum equation is effectively an impulse/momentum equation, where the flow acceleration, that is, the rate of increase (or decrease) in momentum is balanced by the impulse of the hydrostatic pressure gradient.

2.9.3. Assumptions

In the derivation of these equations, it has been assumed that:

- The flow is incompressible,
- The pressure is hydrostatic (i.e. vertical accelerations can be neglected and the local pressure is dependent only on the local depth),
- The flow can be described by continuous (differentiable) functions of ς, *u* and *v* (that is, it does not include step changes in ς, *u* and *v),*The flow is two-dimensional (that is, the effects of vertical variations in the flow velocity can be neglected),
- The flow is nearly horizontal (that is, the average channel bed slope is small), and
- The effects of bed friction can be included through resistance laws (e.g., Manning equation) that have been derived for steady flow conditions.

Problems associated with accurate modelling of transport equations have been highlighted by Leonard (1979a). Simple first order schemes are inaccurate and diffusive, while second order schemes (that are good for solving wave propagation) tend to be oscillatory and unstable. This has lead to the use of more innovative approaches to modelling the convective momentum terms (Abbott and Rasmussen, 1977), and the use of higher third order solution schemes (Leonard, 1979b; Stelling, 1984).

2.10. Extension of the Equations for Modelling Applications

The 2D mass and momentum equations described in Book 6, Chapter 2, Section 9 and Book 6, Chapter 2, Section 9 are sometimes referred to as the 2D “long wave” equations. These equations can be used to describe the behaviour of waves, including flood waves, which are long relative to the water depth.

For practical modelling applications, these equations need to be expanded to include the additional effects of other phenomena of interest. The most important of these is probably the inclusion of the dissipative effects of bed-friction in the momentum equation. The inclusion of additional terms to form extended modelling equations is considered below.

2.10.1. Extension of the Mass Equation

For modelling applications, the mass equation can be expanded to include additional source and/or sink terms to allow for localised and/or distributed inflows and outflows, as follows:

$$\frac{\partial \varsigma}{\partial t}+\frac{\partial (d.u)}{\partial x}+\frac{\partial (d.v)}{\partial y}= \text{Sources-Sinks} \qquad (6.2.105)$$

Where the Source terms can represent localised inflows such as may occur at stormwater or pump outlets, or distributed inflows associated with rainfall, and the Sink terms can represent localised outflows at drainage pits or pump intakes or distributed losses due to infiltration or, in long-term simulations the evaporation.

2.10.2. Extension of the Momentum Equations

For modelling applications, the extension of the momentum equations to include the effects of bed-friction, eddy viscosity and other source and sink terms is discussed below.

Bed Friction

For flood modelling applications, the momentum equation must be coupled with a suitable friction formulation. This is typically achieved by adding a Chezy-type friction term to the momentum equation, which then becomes:

$$\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}+v\frac{\partial u}{\partial y}+g\frac{\partial \varsigma}{\partial x}= -\frac{gu\sqrt{u^2+v^2}}{C^2 d} \qquad (6.2.106)$$

where:

C: is a Chezy roughness coefficient ($m^{1/2}s^{-1}$)

For practical modelling applications, the Chezy coefficient can be related to the more usual (for Australian applications) Manning ‘n’ *roughness coefficient* by the Strickler relation, where:

$$\mathrm{n}=\frac{d^{\frac{1}{6}}}{C} \qquad (6.2.107)$$

In some European models, the friction coefficient is sometimes specified in terms of Manning ‘M’, where:

$$n = \frac{1}{M} \tag{6.2.108}$$

Eddy Viscosity

Most commercially available 2D models also include an “eddy viscosity” type term to allow for the effects of sub-grid scale mixing processes. This can be important when modelling flow separations and eddies, or in situations where it is necessary to model channel/ overbank interactions.

Introducing a typical eddy viscosity formulation, the *x* momentum equation becomes:

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + g\frac{\partial \varsigma}{\partial x} = -\frac{gu\sqrt{u^2+v^2}}{C^2 d} + E\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2}\right) \tag{6.2.109}$$

where:

E: is an “eddy viscosity” coefficient (m^2s^{-1})

If, for illustration purposes only, the hydrostatic pressure and friction terms are neglected, the *x* momentum equation can be rearranged to the form:

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} - E\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2}\right) = 0 \tag{6.2.110}$$

This is analogous to a two-dimensional advection-diffusion equation describing the transport and diffusion of *u*, the *x* velocity component. Continuing the analogy, the eddy viscosity coefficient *E* becomes equivalent to the diffusion coefficient used in advection-diffusion modelling. Thus, as well as having wave propagation and transport properties the momentum equation can also have diffusion properties.

Eddy viscosity and its application to 2D flood models is discussed in more detail in Book 6, Chapter 4. It is noted, however, that for eddy viscosity calculations to be meaningful, the $u\frac{\partial u}{\partial x}$ and $v\frac{\partial u}{\partial y}$convective momentum terms must be modelled with sufficient accuracy.

Other Terms

The early 2D flood models were originally derived from 2D coastal and estuarine models. These models typically included additional terms to represent wind shear and Coriolis effects. When these terms are included, along with additional source/sink terms to allow for the addition or loss of momentum associated with any sources or sinks of mass, discussed above, the *x* momentum equation becomes:

$$\begin{aligned}\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + g\frac{\partial \varsigma}{\partial x} = &-\frac{gu\sqrt{u^2+v^2}}{C^2 d} + E\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2}\right) + fVV_x - \Omega u \\ &+ \mathrm{Source/Sink}\end{aligned} \tag{6.2.111}$$

where:

f is a wind shear stress coefficient

V is the wind speed (m/s)

Vx is the component of the wind speed in the *x* direction (m/s)

Ω is a latitude dependent Coriolis parameter

The wind and Coriolis terms are only likely to become important in wide open floodplains or in lake or estuarine systems, and are not considered further in the present discussion.

2.10.3. Final Forms of the Equations

As developed above, the final forms of the mass and momentum equations used in many 2D flood models can be expressed as:

Mass

$$\frac{\partial \varsigma}{\partial t}+\frac{\partial (du)}{\partial x}+\frac{\partial (dy)}{\partial y}=\text{Sources - Sinks} \tag{6.2.112}$$

x-Momentum

$$\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}+v\frac{\partial u}{\partial y}+g\frac{\partial \varsigma}{\partial x}=-\frac{gu\sqrt{u^2+v^2}}{C^2 d}+E\left(\frac{\partial^2 u}{\partial x^2}+\frac{\partial^2 u}{\partial y^2}\right)+\text{Source/Sink} \tag{6.2.113}$$

y-Momentum

$$\frac{\partial v}{\partial t}+V\frac{\partial v}{\partial y}+u\frac{\partial v}{\partial x}+g\frac{\partial \varsigma}{\partial y}=-\frac{gv\sqrt{u^2+v^2}}{C^2 d}+E\left(\frac{\partial^2 u}{\partial x^2}+\frac{\partial^2 u}{\partial y^2}\right)+\text{Source/Sink} \tag{6.2.114}$$

This coupled system of equations provides the three equations necessary to solve for the three dependent variables; ς the free surface elevation, *u* the velocity in the *x* direction and *v* the velocity in the *y* direction.

2.10.4. Modelling Requirements and Simplifications

The previous sections show that the combination of the mass and momentum equations can describe the wave propagation properties associated with a flood, and how the momentum equations include terms for describing the effects that advection and dispersion of momentum can have on the flow. The relative importance of these properties can vary significantly depending on the flow conditions. This has little impact on how the mass equation is treated, but in some cases can allow simplifying assumptions to be made in the treatment of the momentum equations.

2.10.4.1. The Mass Equation

For flood modelling applications, it is important that the solution procedure used in the model does not generate or destroy mass numerically. It is therefore essential that all the terms in the mass equation are described accurately in the numerical solution procedure.

With the staggered grids used by most finite difference models, there can be issues with achieving time and space centring of the non-linear spatial derivative terms. In this respect, it is noted that Stelling et al. (1998) presented a numerical scheme that conserves mass and maintains non-negative water levels. Nevertheless, modellers should be aware that any errors in the mass equation, however small, can accumulate with time as the computation progresses. If the mass equation is not modelled correctly, the error accumulation can continue to the extent that the final solution may be compromised.

2.10.4.2. The Momentum Equation

For the momentum equations, the relative importance of the different terms can vary quite significantly depending on the flow conditions. In some conditions it may be possible for simplifying assumptions to be made either to the equations themselves, or to the way in which individual terms are treated numerically. The types of simplifications used tend to be made for numerical expediency, or to avoid numerical problems with particular types of flow (e.g., supercritical flow). The extent to which they can be used is dependent upon the level of detail and/or accuracy required.

Ignoring wind, Coriolis and source/sink terms, the x-momentum equation developed above can be expressed as:

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + g\frac{\partial \varsigma}{\partial x} = -\frac{gu\sqrt{u^2+v^2}}{C^2 d} + E\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2}\right) \tag{6.2.115}$$

Using this as a base, some of the more commonly used approximations to the momentum equation are discussed below.

The Linearised Momentum Equation

With this approximation, the convective momentum (momentum transport) terms are neglected. When these terms are neglected, the eddy viscosity (momentum dispersion) terms have little physical meaning and can also be neglected. With this approach, the *x*-momentum equation reduces to:

$$\frac{\partial u}{\partial t} + g\frac{\partial \varsigma}{\partial x} = -\frac{gu\sqrt{u^2+v^2}}{C^2 d} \tag{6.2.116}$$

This approach should only be used in areas where the velocities are small, and the wave propagation properties of the flow are dominant. This rarely happens in most practical flood flow simulations. However, it is noted that the linearised momentum equation is sometimes used for numerical expediency in order to maintain stability in high velocity flow areas, including regions of supercritical flow. Although this approximation maintains the wave propagation properties of the full momentum equation, it cannot model momentum dominated effects, including flow separations and eddies, and main channel/overbank momentum transfers.

The Steady State Momentum Equation

With this approximation, the local acceleration term *δu/δt* is neglected and the *x*-momentum equation reduces to:

$$u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + g\frac{\partial \varsigma}{\partial x} = -\frac{gu\sqrt{u^2+v^2}}{C^2 d} + E\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2}\right) \tag{6.2.117}$$

This approximation neglects the wave propagation properties of the momentum equation. It can be used in reaches with moderate to steep slopes, where the flow is dominated by friction. However, it should not be used for rapidly varying flows, such as in dam-breaks, or in reaches with flat slopes and/or deep water where the local acceleration term (and wave propagation properties of the equation) becomes more important.

The Diffusive Wave Approximation

With this approximation, the convective momentum and eddy viscosity terms are also neglected and the *x*-momentum equation reduces to:

$$\frac{\partial \varsigma}{\partial x} = -\frac{u\sqrt{u^2 + v^2}}{C^2 d} \tag{6.2.118}$$

That is, the water surface slope is balanced by the friction slope.

As for the steady state momentum equation, this approximation can be used to describe gradually varied flows in reaches with moderate to steep slopes. It includes backwater effects, but has the added limitation that it cannot be used to simulate flow separations and eddies, or main channel/overbank momentum transfers.

The Kinematic Wave Approximation

With this approximation, the surface slope of the water is assumed to be the same as the bed slope the *x*-momentum equation further reduces to:

$$\frac{\partial z}{\partial x} = -\frac{u\sqrt{u^2 + v^2}}{C^2 d} \tag{6.2.119}$$

That is, the friction slope is equal to the bed slope.

This approximation is effectively the same as solving for the flow properties using a steady state friction law (such as the Manning equation). Backwater effects are not included, and water can only flow downstream. As such, the kinematic wave approximation can only be used to describe gradually varied flows in reaches with moderate to steep slopes where backwater effects can be neglected.

2.11. Three Dimensional Flow Equations

Regardless of the nature of flow, all flow situations must satisfy the following relationships:

1. The continuity equation (law of conservation of mass).

2. Newton's law of motion, which must hold for every particle at every instant.

3. Boundary conditions, for example a real fluid has zero velocity relative to an adjacent boundary.

4. The first and second laws of thermodynamics.

Other relations such as Newton's law of viscosity or the Boussinesq eddy viscosity concept are also necessary so that solutions can be obtained for the equations developed from these relations.

An approach to 3D modelling of flows is achieved by integrating the point form (or more precisely, infinitesimal unit volume) equations of continuity, momentum and energy over the cross-section. This leads to the continuity equation and Navier-Stokes equations expressed in tensor notation below (Rodi, 1980).

Mass Conservation: Continuity equation

$$\frac{\partial u_i}{\partial x_i} = 0 \tag{6.2.120}$$

Momentum conservation: Navier-Stokes equations

$$\frac{\partial u_i}{\partial t} + u_j \frac{\partial u_i}{\partial x_j} = \frac{-1}{\rho} \frac{\partial P}{\partial x_j} + \upsilon \frac{\partial^2 u_i}{\partial x_j \partial x_j} \tag{6.2.121}$$

where u_i is the instantaneous velocity component in the direction x_i, P is the instantaneous static pressure and u is the molecular kinematic viscosity. The Navier-Stokes equations are exact equations describing the turbulent motion, and numerical procedures are available to solve these equations. However, the storage capacity and speed of present-day computers are still not sufficient to allow a solution for practically relevant turbulent flow. The reason for this is that turbulent motion contains elements which are much smaller than the extent of the flow domain (typically of the order of 10^{-3} times smaller). Thus to resolve the motion of these elements in a numerical procedure at least 10^9 grid points would be necessary to cover the flow domain in three dimensions.

A statistical approach to turbulence suggested by Reynolds (1894) may be used to solve the Navier-Stokes equations for mean values of velocity and pressure when the turbulence correlations $u'v'$ which result can be determined in some way. The determination of these correlations is the main problem in calculating turbulent flows and a turbulence model must be introduced which approximates the correlations and simulates the average character of real turbulence. The instantaneous values of velocity u_i and pressure P are separated into mean and fluctuating components, as shown in Equation (6.2.123).

$$u_i = \overline{u_i} + u_i' \text{ and } P = \bar{P} + P' \tag{6.2.122}$$

where the mean quantities are defined as:

$$u_i = \frac{1}{t_2 - t_1} \int_{t_1}^{t_2} u_i dt \text{ and } \bar{P} = \frac{1}{t_2 - t_1} \int_{t_1}^{t_2} P dt \tag{6.2.123}$$

and the averaging time t_2-t_1 is long compared with the time scale of the turbulent fluctuations. This results in the following equations:

Continuity equation:

$$\frac{\partial \overline{u_i}}{\partial x_i} = 0 \tag{6.2.124}$$

Momentum equation written in terms of the cartesian coordinates for the *x*-direction, Equation (6.2.121) becomes:

$$\begin{aligned} &\frac{\partial \bar{u}}{\partial t} + \bar{u}\frac{\partial \bar{u}}{\partial x} + \bar{v}\frac{\partial \bar{u}}{\partial y} + \bar{w}\frac{\partial \bar{u}}{\partial z} = \frac{-1}{\rho}\frac{\partial \bar{P}}{\partial x} - \left(\frac{\partial \overline{u'v'}}{\partial y} + \frac{\partial \overline{u'u'}}{\partial x} + \frac{\partial \overline{u'w'}}{\partial z}\right) \\ &+ \upsilon\left(\frac{\partial^2 \bar{u}}{\partial x^2} + \frac{\partial^2 \bar{u}}{\partial y^2} + \frac{\partial^2 \bar{u}}{\partial z^2}\right) \end{aligned} \tag{6.2.125}$$

where $\bar{u}$,$\bar{v}$and $\bar{w}$,and are the local time-averaged velocity values in the *x,y* and *z* directions and *u', v'* and *w'* are their fluctuating components. The terms on the left-hand side of the equation represent the momentum flux through an element dx, dy, dz. The three terms on the right-hand side are the external forces acting on the element.

The three components of the external forces are:

1. The body force of the element (due to its weight).

2. The resulting turbulent shear forces on all surfaces (resulting from Reynolds stress distributions).

3. The viscous forces causing shear stresses at the molecular level.

Physically, the correlations when multiplied by the density represent the transport of momentum due to the fluctuating motion as shown below:

$$\tau_{xy} = -\rho\overline{u'v'} \tag{6.2.126}$$

The equation above represents the transport of x-direction momentum in the y-direction and may be considered as a shear stress on the fluid called the turbulent or Reynolds stress.

Strelkoff (1969) integrated the equation of continuity and the Navier-Stokes equation to obtain the one-dimensional open channel flow equations for an incompressible homogeneous fluid. Further work was presented by Yen (1973) providing a more detailed and unified view of the general open channel flow equations.

Often 3D models are applied to steady flows. One approach was described by Olsen (2003) and Olsen (2004). He solves the three-dimensional Reynolds Averaged Navier-Stokes equations (RANS) for each cell. The equations can be written in Cartesian form as:

$$\frac{\partial V_i}{\partial x_i} = 0 \tag{6.2.127}$$

for continuity, and

$$\frac{\partial V_i}{\partial t} + V_j\frac{\partial V_i}{\partial x_j} = \frac{1}{\rho}\frac{\partial}{\partial x_j}\left(-p\delta_{ij} - \rho\overline{u_i u_j}\right) \tag{6.2.128}$$

for momentum, where *i* and *j* represent standard tensor notation indicating the *x, y* and *z* coordinate directions, V_i is the mean velocity component in the x_i direction, *p* is the pressure, *ρ* is the fluid density, δ_{ij} is the Kronecker delta and is the turbulent Reynolds stress, where u_i and u_j are fluctuating velocities, and is Reynolds averaged value of $u_i u_j$. The first term is the transient term which is neglected and the second term is the convective term. The third term is the pressure term and the final term is the Reynolds stress term which requires a turbulence model to be evaluated. The standard *k-ε* model was used for turbulence closure. The model calculates the eddy-viscosity as:

$$v_T = c_\mu\frac{k^2}{\varepsilon} \tag{6.2.129}$$

where *cμ* is a constant, $k = \frac{1}{2}\overline{u_i u_j}$ and is the turbulent kinetic energy and *ε* is the dissipation rate of turbulent kinetic energy. The turbulent kinetic energy *k* is modelled as:

$$\frac{\partial k}{\partial t} + V_j \frac{\partial k}{\partial x_j} = \frac{\partial}{\partial x_j}\left(\frac{V_T}{\sigma_k}\frac{\partial k}{\partial x_j}\right) + P_k - \varepsilon \tag{6.2.130}$$

where σk is a constant and $P_k = v_T \frac{\partial V_j}{\partial x_i}\left(\frac{\partial V_j}{\partial x_i} + \frac{\partial V_i}{\partial x_j}\right)$ which is a term for the production of turbulence. The dissipation of turbulent kinetic energy ε is modelled as:

$$\frac{\partial k}{\partial t} + V_j \frac{\partial k}{\partial x_j} = \frac{\partial}{\partial x_j}\left(\frac{V_T}{\sigma_k}\frac{\partial k}{\partial x_j}\right) + C_{\varepsilon 1}\frac{\varepsilon}{k}P_k - C_{\varepsilon 2}\frac{\varepsilon^2}{k} \tag{6.2.131}$$

where $C_{\varepsilon 1}$, $C_{\varepsilon 2}$ and $\sigma\varepsilon$ are constants. Recommended values for the five constants in the *k*-*ε* model given by Rodi (1980). The SIMPLE method (Patankar, 1980) can be used for the pressure and velocity coupling, and an implicit solver was used to produce the velocity field across the geometry. Model convergence was assumed when all residuals of the RANS and turbulence equations between consecutive iterations were of the order of 10^{-4}. An approach for the application of these equations in compound open channels is given in Conway et al. (2013).

2.12. Physical Modelling

The first designed physical model pre-dates the routine use of numerical models in hydraulic engineering practice by approximately a century. Prior to the advent of digital computers, physical models offered the most practical means for the investigation of problems involving complex bathymetry, sediment transport, unsteady flow and 2D or 3D flow.

Since the advent of numerical models, the domain of application of physical models has been shrinking. However, certain problems remain which are still more appropriately investigated through the use of physical models, and this is likely to remain the case for some time yet.

Physical models have been around for hundreds of years, however, it was only in 1885 when the first physical model study based on scientific principles, was undertaken. This model study was conducted by Osborne Reynolds for investigating the tidal currents in the Mersey Estuary near Liverpool, England. Reynolds is reputedly the first person to introduce the time scale into physical modelling (Lawson and O'Neill, 1980; Allen, 1970). His first model was distorted with the vertical and horizontal length scales differing by a factor of 33.1. The model sides were vertical and initially, the bed of sand was flat. After a period of model operation however, Reynolds observed that the bed was reshaped with the principal features of the natural estuary. This early success provided the impetus for Reynolds to follow up on this work with another bigger model, again of the Mersey Estuary.

2.12.1. The Basis for Physical Model Design

Physical models are scaled (usually reduced) representations of the real life or prototype flows and their boundaries. The flow boundaries may be:

- *fixed bed* - often made out of cement mortar. Such models yield information about the flow patterns and velocity field in, for example, estuaries, river channels or tidal inlets.
- *mobile bed* - with the model bed typically consisting of one of the following: sand, particles of coal or a granulated plastic. Mobile bed models yield (qualitative) information about the sediment movement as well as the water motion; they are of interest for investigating scour holes, or regions of sediment accretion or erosion.

The flow boundaries of a modelled region of interest can be:

- *natural* - where the floodplains and river channels of fixed bed models are constructed by first locating a series of templates made out of metal or plywood into position. Vertically, these sections are positioned using a theodolite. The channel bed and land form between the templates is interpolated. Sand is used as a fill material between the templates, and then a cement mortar capping is applied and frequently painted.

- *manmade* - in which case, physical models often incorporate a hydraulic structure such as a culvert, pipe, drainage channel, basin, levee, spillway or outlet works. In the model, hydraulic structures could be made out of (painted) timber, marine plywood, PVC pipes and also Perspex when visibility is a consideration e.g. for tracing the flow patterns through a structure with the use of a dye.

Accurate simulation of the flows in a physical model requires three kinds of similitude (or similarity) and it will be noticed that the word geometry enters the description of each kind of similitude:

- *Geometric similitude*- requires the geometry or shapes of the flow boundaries to be similar in model and prototype. Lengths in the model are scaled versions of the corresponding prototype lengths. Geometric similitude is secured by ensuring that the model is a scaled reproduction of the prototype.

- *dynamic similitude*- requires that all forces (be they pressure forces, weight, boundary friction forces, drag forces, surface tension forces, centripetal forces) at each point in the flow domain are each scaled by the same factor between model and prototype. If this were to be achieved, the force polygon acting on each elemental fluid parcel in the flow field, would have the same geometry in model and prototype; this is referred to as complete similitude. Complete similitude is the ideal situation, but in practice, is impossible to achieve. The reasons for this are twofold.

Firstly, the scaling requirements of the various forces are incompatible because some forces act through volumes (e.g. gravity and centripetal forces), other forces act over areas (e.g. pressure, drag and viscous forces) and another force acts over lengths (the surface tension force). The scales associated with volumes, areas and lengths are different. Consequently, the forces associated with volumes, areas and lengths will, in general, scale differently. If a model was built full scale, all forces would scale correctly. However, the smaller the model compared to the prototype, the larger the length scale and the greater the discrepancy between the scalings of the various forces which act through volumes or over areas or lengths.

Secondly, the limited fluids available for use in models (in terms of their fluid properties of density and viscosity, and cost and safety) restrict the range of length scales which can be used in physical models. In nearly all cases in engineering practice, the fluids used in hydraulic models are water (and occasionally air). Consequently, a compromise has to be reached in which only the dominant forces are correctly scaled by the same factor, and the incorrect scalings of the smaller forces are of negligible consequence in a well designed model. This is termed incomplete similitude.

The task of the modeller is to identify the dominant forces, ensure that these forces are scaled correctly, and disregard any insignificant forces. The effects (i.e. errors) due to those incorrectly scaled, insignificant forces are known as scale effects. In current engineering practice, most hydraulic investigations involve free surface flows and to a much lesser extent, pressurised or closed conduit flows. These two types of problems require different scaling criteria:

In free surface flows, the dominant forces are usually gravity, the associated pressure force, and boundary friction. By asserting equality of the Froude number between model and prototype at all points in the flow field, the gravity and pressure forces are scaled by the same (desired) factor. By adjusting the model boundary roughness, consistent scaling of the boundary friction force is then achieved. Dynamic similitude is achieved provided the scale effects are negligible. A model which is based on point-to-point equality of the Froude number between model and prototype is known as a Froude model. Effectively, what the modeller is doing here is to ensure that gravity, pressure and boundary friction forces are scaled consistently. All other forces are insignificant and so the momentum equation (or equations if in 2D or 3D) for any elemental parcel of fluid in the model mimics (i.e. is a scaled version of) the momentum equation(s) for the corresponding elemental parcel of fluid in the prototype. Dynamic similitude means that the corresponding terms in the model and prototype momentum equations for each of the dominant forces are all related by a constant factor. In models of closed conduit or pressurised flows, such as flows through pipes, the dynamic scaling requirement is that there is point-to-point equality of the Reynolds number in model and prototype. The dominant forces are the viscous and pressure forces which are correctly scaled in a Reynolds model.

Kinematic similitude requires the flow patterns in the model and prototype to be geometrically similar. In other words, the velocities at all points in the model, bear the same ratio between model and prototype. If this is achieved, then the model is in similitude with the prototype.

If any two of the above three kinds of similitude are satisfied, then the remaining similitude is inferred. In normal modelling practice, models are designed to satisfy both geometric and dynamic similitude. It then follows that kinematic similitude is also satisfied. The usual approach to model design is to satisfy geometric similitude through careful model construction and dynamic similitude by adopting the appropriate modelling criterion. In general, there are various modelling criteria involving various dimensionless numbers, but in practice, the most common criterion which has been mentioned above, is based on the Froude number (for free surface flows). The next most common criterion is based on the Reynolds number (for closed conduit or pressurised flows).

The Froude number may be regarded as the ratio of the inertial (or resultant) force to the gravity force, and the Reynolds number as the ratio of the inertial force to the viscous force. In models based on the Froude criterion or the Reynolds criterion, the correct scaling of the ubiquitous pressure forces is also achieved and the question arises as to how this comes about. The reason is that if the dominant forces (including the resultant force) are all correctly scaled, then the force polygon in model and prototype will also be correctly scaled. Consequently, one of these forces is a dependent force and this is taken to be the pressure force in Froude and Reynolds type models (Warnock, 1949).

Considering any small parcel of fluid in the prototype, what is required of the model is that the corresponding parcel of fluid in the model moves along the corresponding path at a scaled velocity of the prototype velocity. For this to happen, all forces (weight, pressure, drag, viscous, surface tension, elastic) acting on that parcel of fluid in the prototype must be in the same proportions to each other in the model. If this is the case, then there would be complete similitude between model and prototype. However, the various forces scale in different ways: some forces scale as the length cubed (e.g. weight, centripetal forces), other forces scale as length squared (e.g. pressure) and one other scales as length (i.e. surface tension). Therefore, as soon as the length scale departs from unity (i.e. full scale model), it is impossible or very difficult for all these forces to scale together in the same proportion.

Fortunately, however, it is frequently the case, that there are one or two dominant forces present in the flow field and it is therefore of no consequence if the minor forces are not scaled correctly, so long as the dominant forces are. This is termed incomplete similitude.

2.12.2. Model Scales

The design of a physical model requires the selection of a modelling criterion and following on from this, the scales to be used in the model.

In general, it is desirable that the model be as large as possible because:

- scale effects will be reduced,
- the accuracy of model construction will be less critical, and
- the results derived from the model will be less sensitive to errors in measurements.

On the other hand, there are a number of factors which tend to limit the size of the physical model:

- model construction costs. Construction costs depend on the complexity of the bathymetry and topography, and the detail in any hydraulic structures.
- the extent of the available floor area to accommodate the model. Models are preferably housed under cover so that the model testing is weather independent i.e. free of wind and rain and the model itself is sheltered. Moreover, to fit a model into a given area, a river channel can be 'folded' into a more compact form. If additional bends are introduced into the model which do not exist in the prototype, it may be necessary to take the additional head losses into consideration. Also, channel areas which in the prototype provide storage, can have their effects simulated by having areas of different shape but equivalent plan area.
- flows available from the water supply. The bigger the model, the larger the flow needed.

The length scales for most (undistorted) physical models are generally in the range of 1: 5 to 1: 2000.

2.12.3. Distorted Models

It is often useful to be able to distort a physical model by asserting different length scales in the horizontal and vertical directions. While undistorted models should be used whenever there is significant vertical and horizontal fluid motion, distorted models can be used whenever the fluid motion is mainly in the horizontal plain. As a rule of thumb, care should be taken whenever the horizontal to vertical length scales differ by more than a factor of about 5. (Recall that the first model of Reynolds had a distortion of 33.1.)

The advantage of distorting a free surface flow model is that for the same plan area of model, the depths of flow will be deeper in the distorted model. In nearly all prototype free surface flows, the flow is turbulent; and prototype turbulent flows can only be simulated in a model with turbulent flows. For example, if a model is distorted by a factor of 5, for the same (available) plan area, the model depths would be deeper by a factor of 5 and the model velocities would be greater by a factor of $\sqrt{5}$ when compared to an undistorted model. Consequently, the Reynolds numbers in the distorted model will be greater by a factor of $5^{3/2}$ = 11.2 compared to the corresponding undistorted model (with the same plan area) and turbulent flow is more likely to be guaranteed.

2.12.4. Model Scales in Froude Models

In a Froude model, the Froude number in model and prototype at corresponding locations is unity i.e $Fr = Fr_p/Fr_m = 1$ where $(\ )_r$ stands for the ratio of prototype value divided by the corresponding model value.

There are various scales of interest in a physical model and the main ones in a distorted Froude model are:

- length scales in the horizontal (x_r) and vertical directions (z_r),
- velocity scale ($Fr = v_r /g_r y_r = 1$)$\Rightarrow$ $v_r = y_r^{1/2} = z_r^{1/2}$ since $g_r = 1$),
- time scale for essentially horizontal motion ($t_r = x_r/v_r = x_r/z_r^{1/2}$),
- discharge scale for flow through a vertical plane ($Q_r = A_r v_r = (x_r y_r) z_r^{1/2}$),
- pressure scale ($p_r = \rho_r v_r^2 = z_r$ if $\rho_r = 1$ implying that the fluid in the model is the same as the fluid in the prototype), and
- dynamic pressure force scale ($(F_{press})_r = p_r A_r = (\rho_r z_r)(x_r z_r) = x_r z_r^2$), and
- hydrostatic pressure force scale ($(F_{press})_r = \rho_r g_r y_r A_r = x_r z_r^2$)

If the model is undistorted, the various scales above can be determined by setting the length scale $L_r = x_r = z_r$ in the expressions above. For example, the flow scale would be $Q_r = L_R^{5/2}$ and the velocity and time scales both become $v_r = t_r = L_R^{1/2}$.

2.12.5. Model Roughness in Froude Models

One scale which has not been discussed above is the roughness scale, and to determine this, recourse is made to the Manning equation (refer to Book 6, Chapter 2, Section 5) to determine the scale for Manning's n (n_r).

$$v_r = \frac{(R_h) r^{\frac{2}{3}} S_r^{\frac{1}{2}}}{n_r} \tag{6.2.132}$$

$$z_r^{\frac{1}{2}} = \frac{(R_h) r^{\frac{2}{3}} \left(\frac{z_r}{x_r}\right)^{\frac{1}{2}}}{n_r} \tag{6.2.133}$$

$$n_r = \frac{(R_h)_r^{2/3}}{x_r^{1/2}} \tag{6.2.134}$$

It is evident from Equation (6.2.134) that n_r depends upon the hydraulic radius scale ($(R_h)_r$) and for a distorted model, this in turn depends upon both x_r and z_r. One consequence of distortion is that the friction forces at the model bed are under-represented. Equation (6.2.134) indicates that usually, the Manning n in the model should be greater than the Manning's n in the prototype. For a model made out of cement mortar, the model roughness will be too smooth and additional artificial roughness has to be applied to the surface of the model. This can take the form of coarse sand, pebbles or other roughness elements glued to the model surface; sometimes a light gauge wire mesh is attached to the model. Model roughness is adjusted empirically during the calibration phase of the model investigation.

Figure 16 is of a distorted physical flood model in which the model roughness was increased in two ways:

- the over-bank areas were artificially roughened with appropriately spaced and patterned, vertical dowels or roughness elements. The dowels exert a drag force on the water moving over the model floodplain; this additional force is designed to make up for the deficit of boundary friction at the overly smooth model bed.
- the in-bank areas were roughened with small pads of synthetic fibrous material. The spacing of these pads was adjusted by trial and error until the slope of the water surface matched the required slope.

If the roughness in the model is too smooth, not only will the magnitude of the velocities be too large, but also, the flow paths will tend to be too straight. The opposite trends tend to occur in a model which is too rough.

In the design of the distorted model depicted in Figure 16, it was found that while the model simulation of floods with a 50 year and 100 year Average Recurrence Interval (2% and 1% AEP) satisfied the requirement for turbulent flow in the model, the model flow corresponding to a smaller flood with a return period of 25 years was transitional between laminar and turbulent and therefore could not be tested in the model.

In an undistorted model with $n_r = L_r^{1/6}$, the Manning n for the model surface is too high. Model spillways are usually constructed of timber and marine plywood and the usual approach is to try and achieve as smooth a surface as possible through sanding and painting, and to tolerate the mismatch. As the boundary friction forces in this flow scenario are not as important as the dominant gravity force, any scale effect in the model roughness is not usually significant.

2.12.6. Mobile Bed Models

Physical models can be classified as (i) fixed bed or (ii) mobile bed. Most studies by physical models are of the fixed bed variety. In fixed bed models, the bottom flow boundaries remain the same throughout model testing. In mobile bed models, the water movement and the sediment movement have to be reproduced at the same time. The model bed consists of a sediment which may be natural or artificial. A diverse range of materials have been used in mobile bed models to simulate non-cohesive prototype sediments; amongst other materials, they include sand, grains of coal, crushed walnut shells, granulated plastics. Mobile bed models are used to investigate erosion, accretion and localised scour holes.

Mobile bed models are considerably more complex than fixed bed models, and only some general considerations will be included here. With the introduction of a sediment, the specifications for the sediment particles in the model need to be decided. If the model sediment grain size was simply scaled down according to the length scale, the resulting model sediment particles would be so fine that they may well exhibit cohesive behaviour, and such a sediment could not be used to simulate the behaviour of a non-cohesive, prototype sediment. This problem is circumvented by using sediment grains in the model (which are oversized compared to the grain size which would be obtained from the length scale), but made out of a material which is less dense than sand, for example coal or a granulated plastic. By balancing sediment density and particle size for the model sediment particles, their fall velocity can be scaled according to the velocity scale according to the Froude criterion.

Mobile bed modelling of non-cohesive sediments is far from routine. The bed roughness of a mobile bed is a combination of grain roughness and bedforms. The size and type of the bedforms (ripples or dunes) cannot be specified a priori, but rather are determined by interactions between the near-bed fluid dynamics and morphodynamics; this includes the turbulence intensity and its distribution, the permeability of the bed and the shape of the deformable bed.

With the introduction of sediment into a model, there is the need to establish a morphological time scale which is best based on comparisons between the model and prototype bed evolution. This may require distortion of the flow scale in the model to achieve.

The motion of the sediment in the prototype is a mixture of suspended sediment and bed load. While the suspended sediment moves at approximately the flow velocity, the bed load travels much slower. The simulation of sediment movement which consists of nearly all suspended load or nearly all bed load is easier than the much more difficult problem of modelling the total sediment load which consists of comparable proportions of both suspended and bed load. It is possible also, that the proportions of bed load and suspended load vary with time, such as during a flood hydrograph. When this happens, the sedimentation time scale will also vary with time.

In connection to physical modelling, the comment or question is often raised in which physical modelling is referred to as an art or a science?

Some flow scenarios may involve a combination of free surface flow (which required a Froude model) and pressurised flow (which requires a Reynolds model). The resulting scales from applying the Froude and Reynolds criteria are in conflict. Therefore, some other solution, probably involving some compromises, must be sought in such a situation.

2.12.7. Stages of an Investigation by Physical Models

A summary of the main stages in a hydraulic investigation by physical modelling follow:

- *Model design* - the modeling criterion is selected ($IF_r = 1$), a decision on the model type is made (distorted or undistorted model, fixed bed or mobile bed), the length scale/s is/are selected taking cognisance of available space, flows and funding, drainage system and model boundaries, which should be well removed from the study area.

- *Model calibration* - the model is adjusted so as to reproduce a known event, such as a flood, to within an accepted tolerance. If the model fails to achieve this, the validity of the model geometry should be checked as a first response. Next, consideration should be given to the possibility of changes having taken place between the time of the recorded event and the time of the bathymetric survey. Following on from this, the roughness of the flow boundaries should be adjusted until the event has been successfully simulated in the model (Jenkins, 1987).

- *Model verification* - the performance of the model is further checked on another event, which is independent of the data used for calibration. While verification is a desirable stage of model testing to perform, it is not often carried out due to a lack of appropriate data or inadequate funding. In selecting the data for both the calibration and verification phases, it good practice to use the data from events which are of comparable magnitude to the scenarios to be tested. For example, it is not sound modelling practice to calibrate a model on a small and frequent event when the purpose of building the model is to undertake tests corresponding to rare events. The reason for this is that the reliability of extrapolating the model performance cannot be taken for granted and should be questioned.

- *Model testing* - the model is tested under one or more scenarios which may correspond to real or synthesised events.
- *Reporting of model results* - a technical report is prepared which includes the investigation methodology and the findings of the investigation. All results should be in terms of prototype values rather than model values.

2.12.8. Physical Models Development And Non-dimensional Analysis

In most cases of fluid motion in hydraulics, the complexity is such that the strict application of basic equations is possible in only relatively simple geometries. Analytical treatment requires the situation to be idealised to some extent and the effect of the consequent simplifications can only be tested by experiment.

As a result, the science of hydraulics has been marked by intense development of experimental methods. Experimental observation and measurements, and consequent conceptual deductions, have been at the heart of many of the great discoveries in fluid mechanics and hydraulics. Along with the experimental study of basic fluid phenomena, the science and art of physical hydraulic modelling have developed.

With the advent of widespread, powerful, and cheap computing facilities, numerical modelling has advanced significantly. Physical modelling, however, is by no means obsolete. Indeed, as discussed by Martins (1989)Martins (1989), the development of physical modelling has kept pace with numerical modelling. Often the two are intertwined through the concept of “hybrid modelling” where a physical model of a complex flow region provides the boundary conditions for a numerical model covering a much larger area.

This section examines the particular application of physical modelling to the design of hydraulic structures and identifies outstanding issues that remain to be solved. The field of physical modelling is vast, both with regard to the range of problems tackled and the breadth of literature in the field. Excellent reviews and texts include those of Martins (1989), Kobus (1980), and Novak and Cábelka (1981).

Firstly general modelling criteria are reviewed. In particular, the need to supplement pure dimensional analysis with process functions, based on sound analytical concepts, is emphasised. Attention is then focussed on the modelling of hydraulic structures and the potential implications of scale effects. Actual model studies are used to illustrate these issues. Finally some outstanding issues for further development are then identified.

2.12.9. Model Criteria - Dimensional Analysis and Process Functions

As an example of the use of dimensional analysis, and to illustrate its insufficiency on its own, the simple case of flow in a fixed-bed open channel is considered. Adopting the mantle of dimensional analysis, the controlling parameters and their dimensional units are identified as follows:

Flow velocity, V, L/T

Channel width, W, L

Channel depth, y, L

Fluid density, ρ, M/L^3

Fluid viscosity, u, M/(LT)

Fluid surface tension, σ, M/T^2

Surface roughness, ε, L

Gravitational acceleration, g, L/T^2

Dimensional analysis enables the grouping of these parameters in a number of ways. Adopting V, y, and ρ as the repeating variables, we can develop a legitimate set of dimensionless variables as follows:

$$f\left(\frac{V^2}{gy}, \frac{V^2}{\frac{\sigma}{y\rho}}, \frac{Vy\rho}{\mu}, \frac{W}{y}, \frac{\varepsilon}{y}\right) = 0 \tag{6.2.135}$$

Strict similitude would only be possible if all five groups are identical in model and prototype. It can quickly be established, however, that this is not possible, especially if the same fluid is used in model and prototype. Using physical understanding and a process function the first term of Equation (6.2.135) represents the ratio of inertial forces to gravitational forces. Since open channel flow phenomena in general, and most hydraulic structure flows in particular, are gravity driven, this parameter must be retained. Requiring equality of the first term at homologous points in the model and the prototype leads to the well-known Froude law of modelling, appropriate to open channel flows, of:

$$\lambda V = \sqrt{\lambda y} \tag{6.2.136}$$

where λ means “the scale of” (model to prototype).

The second term of Equation (6.2.135) is a Weber number, representing the ratio of inertial forces to surface tension forces. This ratio increases with model size because the inertial forces act on a volume whereas the surface tension forces act on an area. Thus the surface tension forces become negligible, provided the model is reasonably large, and the second term can be disregarded.

Turning now to the third term of Equation (6.2.135), we identify a Reynolds number, Re, representing the ratio of inertia forces to viscous forces. In the context of an open channel flow, viscous forces affect the surface resistance, apparently requiring Reynolds number equality between model and prototype for full similarity.

If the same fluid is used in model and prototype, Reynolds number equality at homologous points would require that:

$$\lambda V = \frac{1}{\lambda y} \tag{6.2.137}$$

and this condition is clearly incompatible with Equation (6.2.136). Indeed, it is readily shown that, if the velocity scale is based on Equation (6.2.136):

$$\lambda \mathrm{Re} = \lambda y^{\frac{3}{2}} \tag{6.2.138}$$

This can be resolved by making use of a process function for flow resistance which links the friction factor, Reynolds number, and relative roughness through the well-known Colebrook-

White equation. This function is conveniently plotted as a Moody diagram and is reproduced in Figure 6.2.23.

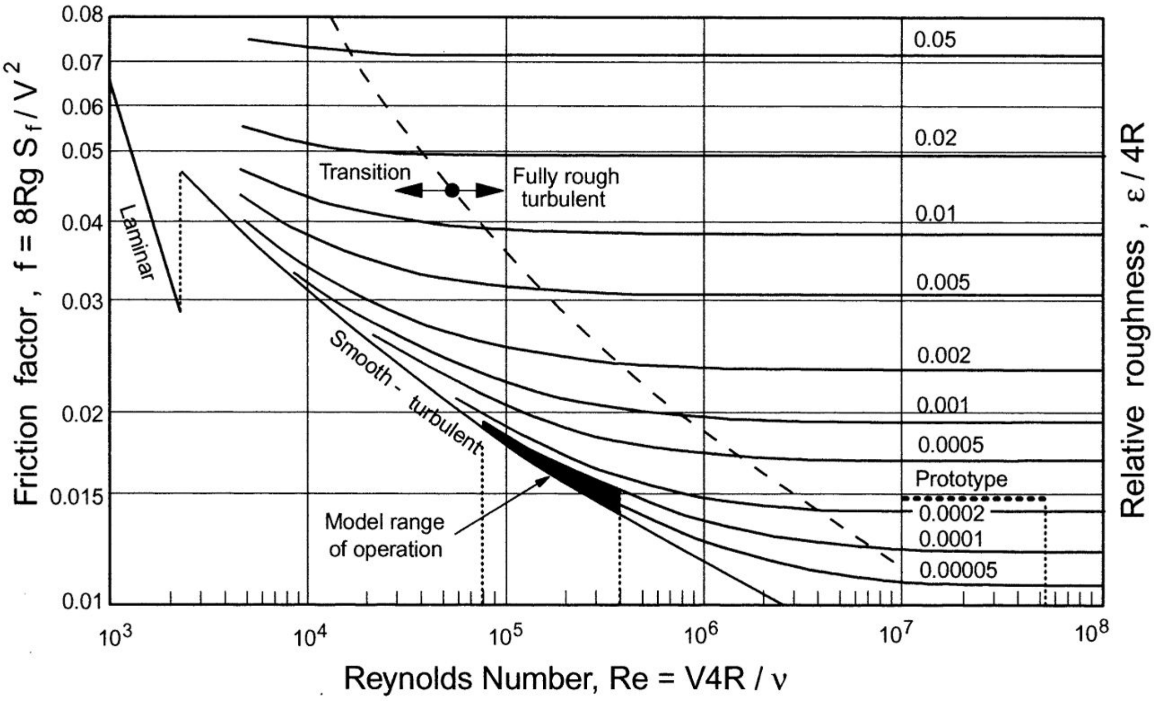


Figure 6.2.23. Process Function Diagram for Friction

The equation for friction factor, the ordinate of Figure 6.2.23, shows that the Froude criterion of Equation (6.2.136) can only be satisfied if the friction factor is the same in model and prototype. Superimposed on Figure 6.2.23 is a hypothetical range of prototype Reynolds numbers and a corresponding range of model operation, assuming a model scale of 1:25. For the example given, it is evident that equality of friction factor between model and prototype can only be obtained if the model is relatively smoother than the prototype. It is noted, further, that this equality is only possible for one particular operating condition (characterised by Reynolds number). For other operating conditions, the model friction factor will be different from that in the prototype, introducing a friction scale effect. The scale effect can be calculated, however, and model results adjusted when scaling up to prototype values.

Figure 6.2.23 also demonstrates that, if the prototype is relatively smooth, it may not be possible to build a model with a low enough friction factor to match that of the prototype. In this situation, the higher model friction factor may be accepted as at least conservative with respect to predicted flow depths, or, again, the scale effect can be calculated and used to adjust the predicted prototype values.

The discussion above has demonstrated that dimensional analysis is insufficient on its own to provide a basis for the modelling of open channel flow. Indeed, relying solely on dimensional analysis, it would be concluded that accurate modelling is not possible. It is only by using knowledge of flow resistance and its corresponding process function to dimensional analysis that an appropriate modelling procedure is possible.

Other examples of the necessity for process functions, in addition to dimensional analysis, for physical modelling of weir flows and vortex drop shafts have been discussed by Ackers (1987).

The discussion above applies to undistorted models only - ie those for which the horizontal and vertical scales are identical. Undistorted models are common in hydraulic structure investigations, but are often impractical for large rivers because of their typically large width:depth ratios. A typical river may have a width of 500 m and a depth of perhaps 2 m. The corresponding undistorted model of a scale of, say, 1:250 would be 2 m wide and 8 mm deep. The model flow is then likely to be totally different in character to the prototype flow due to surface tension effects and the likelihood of laminar model flow.

This situation is resolved by utilising a vertical scale that is larger than the horizontal. The Froude relationship is still expressed in the form of Equation (6.2.136), where, however, *ly* represents the vertical scale because it is vertical, rather than horizontal distances which measure the effect of gravity on velocity.

Given Figure 6.2.23 and the expression for head loss:

$$h_L = f\frac{L}{4R}\frac{V^2}{2g} \tag{6.2.139}$$

Rearrangement and expression in terms of scale ratio yields:

$$\lambda S = \frac{\lambda y}{\lambda x} = \lambda f\frac{\lambda R}{\lambda x} \tag{6.2.140}$$

or

$$\lambda f = \frac{\lambda R}{\lambda x} = \frac{\lambda y}{\lambda x} \tag{6.2.141}$$

if the channel is wide.

Because λy is always greater than λx, Equation (6.2.141) shows that the model must be rougher than the prototype. Indeed, in direct contrast to undistorted models, significant effort is often required to make the model rough enough!

Mobile bed models introduce an additional degree of complexity because the roughness of the bed is largely dependent on the form losses associated with the bed features. Because the bed features are formed by the flow conditions, and hence cannot be directly established by the modeller, it is important that the model flow conditions are such that the model bed simulates closely the bed of the prototype.

Resolution of these complexities is beyond the scope of this chapter. Further details are provided in Keller (1998).

2.12.10. Scale Effects

The whole issue of scale effects is too broad to be effectively covered in this chapter. There is a large body of literature on the topic and several specialist symposia have been held eg Kobus (1984). Herein, attention is confined to scale effects in hydraulic structures, specifically flow measurement structures where scale effects can significantly affect the model determination of prototype rating curves.

An undistorted geometric scale model is normally built and operated under conditions of Froudian similarity. The model head (stage)-discharge data are then simply scaled up to prototype values, utilizing the equations:

$$\lambda h = \lambda L \tag{6.2.142}$$

$$\lambda Q = \lambda L^{\frac{5}{2}} \tag{6.2.143}$$

The characteristics of boundary layer growth are such that model scaling for reproduction of free surface effects (Froudian scale) will introduce dissimilarities. We illustrate this by considering the equations for turbulent boundary layer growth on a flat plate Streeter and Wylie (1979):

$$\delta = \frac{0.38x}{\mathrm{Re}^{0.2}} \tag{6.2.144}$$

$$\delta^* = \frac{1}{8}\delta \tag{6.2.145}$$

Equation (6.2.145) is developed from the one-seventh power law velocity distribution:

$$\frac{u}{U} = \left(\frac{y}{\delta}\right)^{\frac{1}{7}} \tag{6.2.146}$$

In the above equations, δ and δ^* are boundary layer thickness and boundary layer displacement thickness respectively, Re is the flow Reynolds number defined with respect to the length *x* of boundary layer development, *u* is the velocity at elevation *y* above the bed, and *U* is the free stream velocity.

Equation (6.2.144) and Equation (6.2.145) indicate that similarity of boundary layer growth will only be possible if the Reynolds numbers are the same in model and prototype. This criterion is not met in a Froudian model, for which $\lambda\mathrm{Re} = \lambda L^{1.5}$.

The measured water surface elevation upstream of the flume control is affected by the boundary layer growth on the flume floor in that the water surface is displaced by a distance equal to the boundary layer displacement thickness. Accordingly, any dissimilarities in the modelling of the boundary layer displacement thickness will reflect as a dissimilarity in the position of the water surface, and a consequent scale effect in the measurement of pressure head.

Keller (1984a) has developed a procedure for determining the magnitude of the scale effect and for adjusting the model data to correctly predict the prototype behaviour. For details, the reader is referred to the original paper. The procedure relies on the use of the process function for boundary layer growth embodied in Equation (6.2.144) to Equation (6.2.146).

Keller (1984b) has applied the procedure to undrowned cut-throat flumes and typical results are presented in Figure 6.2.24. These data were obtained from a study involving three geometrically similar flumes to scale ratios of 1:2:4. Flume 1 (x) is the smallest, Flume 2 (⨀) is twice as large as Flume 1, and Flume 3 (+) is four times as large as Flume 1. The ordinate is the piezometric head, *ha*, normalised with the throat width, *BT*. The abscissa is the non-dimensional discharge parameter.

The data in Figure 6.2.24(a) are uncorrected and show a tendency at values of *ha*/*BT* below about 0.8 to plot progressively to the right with flume size - ie for *ha* to be slightly less for the large flume than would be predicted from tests on the small flumes. Expressed in terms of more relevance to the practising engineer, an uncorrected model rating would result in an under-prediction in the discharge through a four times larger prototype structure by up to

10%. Figure 6.2.24(b) shows the data adjusted for dissimilar boundary layer growth. It is clear that the small trend with flume size has been completely eliminated.

The point of this example has been to demonstrate that scale effects arise in many model studies. However, with an understanding of the physical processes which govern the phenomena and with a knowledge of the appropriate process functions, the scale effects can be assessed and, in many cases, explicitly determined.

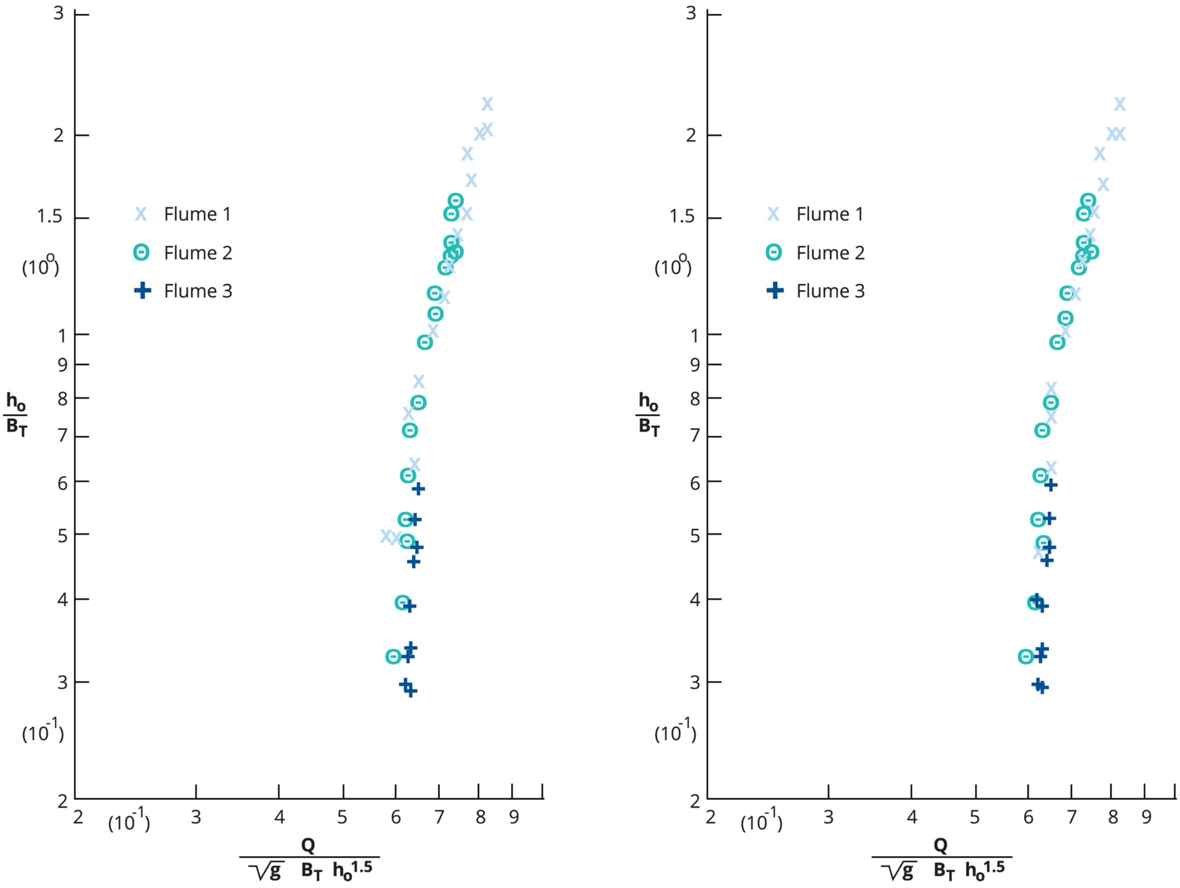


Figure 6.2.24. Data for Cut-throat flumes (a) Uncorrected, (b) Corrected for Scale Effects (after Keller (1984b))

2.12.11. Some Issues for the Future

It is fashionable in some quarters to predict the eventual demise of physical modelling as computing power becomes ever cheaper and more readily available, and as our ability to translate knowledge of the physics of a phenomenon into process equations for its quantitative solution develops. It is indeed true that some problems that were once routinely solved using physical models may now be solved using numerical models. Among these are simple spillway layouts, flow measurement structures, and far-field dispersion problems in rivers. It is equally true, however, that many problems remain the exclusive province of the physical modeller. Problems with complex boundary conditions and/or strongly three-dimensional characteristics remain extremely difficult to solve by numerical means. The same problems, however, are amenable to study by physical models because the model represents, within generally understood limits, an exact replica of the prototype.

Developments over the next few years are likely to concentrate on hybrid models – ie model approaches where physical modelling and numerical modelling are applied in tandem to the solution of complex hydraulic problems. There is evidence of this already, and two examples are given in the following.

The mixing processes downstream of a pollutant outfall are often classified as “near field” and “far field”. Sometimes an additional “mid-field” may be introduced. Rutherford (1994) provides an excellent review of mixing processes.

In all but the most simple of cases, near-field mixing is extremely difficult to model numerically. The flow field is very strongly three-dimensional and the mixing processes may be dominated by vertical mixing, transverse mixing, or both. In the far-field region, full vertical mixing may well have been achieved, and a numerical two dimensional mixing model may be adequate to describe the continuing diffusion of the pollutant. In this situation, the most efficient modelling framework may be to build an undistorted physical model to simulate the near-field region and to use its measured downstream parameters as the upstream boundary conditions for the numerical model.

The second example has been described by Ackers (1987) and concerns prototype phenomena where air entrainment is a primary parameter. On spillways, self-aeration through floor slots is commonly permitted in order to control cavitation damage on the spillway. The amount of air that is entrained depends on the length of the trajectory of the nappe which springs from the upstream edge of the spillway slot. However, the length of the trajectory depends on the air pressure beneath the nappe. This depends on the rate of air entrainment, which, in turn, is a function of the resistance of the air supply ducts. Neither of these features can be properly simulated in an undistorted scale model. The trick is, in fact, to link the general spillway model with a separate computational study of the performance of the ducts with a design value of air demand. The aerodynamic resistance of the prototype supply duct determines the pressure to be expected beneath the nappe and this (sub-atmospheric) pressure must then be reproduced artificially in the model.

There are many other areas that will keep the physical modeller busy for many years to come. Some examples are:

- Interactions between hydrodynamic loads and structural loads and vibrations,
- Self-aeration in free surface spillway flows,
- Modelling of the scour potential of cohesive sediments and clays,
- Modelling of the influence of turbulence and flocculation on the performance of settling basins, and
- Modelling of the scour resistance of bank and bed vegetation.

2.13. References

AS Committee (2006), PL-045. Australian Standard AS 2200, Design Charts for Water Supply and Sewerage. Standards Australia, Sydney, January 2006. ISBN 0 7337 7084 3.

ASCE Task Force (1963), Friction Factors in Open Channels. Journal of Hydraulics Division, ASCE, 89(2), 97-143.

Abbott, M.B. (1979), Computational Hydraulics - 's of the Theory of Free Surface Flows, Pitman, London.

Abbott, M.B. and Rasmussen, C.H. (1977), On the Numerical Modelling of Rapid Contraction and Expansion in Models that are Two-dimensional in Plan, Proc. XV11 Congress, IAHR, Baden-Baden.

Ackers, P. (1991), Hydraulic Design of Straight Compound Channels. Report SR281, HR Wallingford, October 1991 (Volumes 1 and 2).

Ackers, P. (1992), Hydraulic design of two stage channels. Proc. of the Institution of Civil Engineers, Water, Maritime and Energy, Dec. 1992, 96: 247-257.

Ackers, P. (1987), 'Scale Models. Examples of how, why, and when - with some ifs and buts', Proceedings of XXII Congress, International Association for Hydraulic Research, Technical Session B, Lausanne, Switzerland, pp: 1-15

Allen, J. (1968), The life and work of Osborne Reynolds, chapter 1, pages 1-82. Manchester University Press and Barnes and Noble, Inc., Manchester and New York, 1970. Osborne Reynolds and Engineering Science Today, edited by McDowell, D.M. and Jackson, J.D. Papers presented at the Osborne Reynolds Centenary Symposium, University of Manchester, September 1968, ISBN 0 7190 0376 8.

Barnes, H.H.Jr. (1967), Roughness characteristics of natural channels. Geological Survey Water- Supply Paper 1849, U.S. Department of the Interior, Geological Survey, pages 1-213, 1967. Electronic full text available as 'Roughness characteristics of normal supply channels', available at web.utk.edu/btschant/manning%20n/two.htm.

Bhowmik, N.G. and Demissie, M. (1982), Carrying capacity of flood plains. Proc. ASCE, Journal of the Hydraulics Division, Mar. 1982, 108(3), 443-452.

Blalock, M.E. and Sturm, T.W. (1981), Minimum Specific Energy in Compound Open Channel. Journal of the Hydraulics Division, ASCE, Jun. 1981, 107(6), 699-717.

Cahdderton, R.A. and Miller, A.C. Friction slope models for M2 profiles. Water Resources Bulletin, American Water Resources Association, 16(2), 235-242, April 1980. Paper No. 79037.

Chow, V.T. (1959), Open-Channel Hydraulics. McGraw-Hill Book Company, New York.

Chow, V., Maidment, D. and Mays, L. (1988), Applied hydrology. New York: McGraw-Hill.

Conway, P., O'Sullivan, J.J., Lambert M.F. (2013), Stage-discharge prediction in straight compound channels using 3D numerical models, Proceedings of the Institution of Civil Engineers: Water Management, Vol.166, No.1, January, 3-15.

Cunge, J.A., Holly, F.M. and Verwey, A. (1980), Practical Aspects of Computational River Hydraulics, Pitman, London (Reprinted by University of Iowa).

Einstein, H.A. (1934), Der hydraulische oder profile-radius [The hydraulic or cross-section radius]. Schweizerische Bauzeitung, Zurich, 103(8), 89-91 (in German).

Elliot, S.C.A. and Sellin, R.H.J. (1990), SERC flood channel facility: skewed flow experiments. IAHR, Journal of Hydraulic Research, Feb. 1990, 28(2), 197-214.

Ervine, D.A. and Ellis, J. (1987), Experimental and computational aspects of overbank flood plain flow. Transactions of the Royal Society of Edinburgh: Earth Sciences, 1987, 78: 315-325.

Fenton, J. (2007), Lecture Notes on Open Channel Hydraulics. pp59, University of Karlsruhe, accessed on the web, 2007, April 2007.

Greenhill, R.K. (1992), An Investigation into the Mechanisms of Compound Meandering Channel Flow. PhD Thesis, Department of Civil Engineering, University of Bristol, U.K.

Henderson, F.M. (1966), Open Channel Flow, Macmillan Publishing Co., New York.

Horton, R.E. (1933), Separate roughness coefficients for channel bottom and sides. Engineering News-Record, Nov. 1933, III(22), 652-653.

Hydraulics Research Ltd, Wallingford, Thomas Telford, London. (1990), Charts for the Hydraulic Design of Channels and Pipes, sixth edition, 1990. ISBN 0 946466 02 5.

James, M. and Brown, R.J. (1977), Geometric parameters that influence flood plain flow. U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Jun. 1977, Research Report H-77-1.

James, C.S. and Wark, J.B. (1992), Conveyance Estimation for Meandering Channels, Report SR329, HR Wallingford, U.K., Dec. 1992.

Jenkins, B.S. (1987), Aspects of Hydraulic Calculations, volume 1 of Australian Rainfall and Runoff, A Guide to Estimation, D. H. Pilgrim (editor-in-chief), chapter 4, pp. 53-91. The Institution of Engineers, Australia, Barton, ACT, revised edition, 1987. ISBN 085825 434 4.

Keller, R.J. (1998), The Continuing Application of Physical Models to Hydraulic Engineering, Keynote Address to 6th International Conference on Hydraulics in Civil Engineering, Proceedings of HYDRASTORM '98 combining 3rd International Symposium on Stormwater Management and 6th International Conference on Hydraulics in Civil Engineering, Adelaide, Australia, 27-30 September 1998, Institution of Engineers, Australia, pp: 11-21.

Keller, R.J. (1984a),'Boundary Layer Scale Effects in Hydraulic Model Studies of Discharge Measuring Flumes', in KOBUS, H. (ed),'Symposium on Scale Effects in Modelling Hydraulic Structures', International Association for Hydraulic Research, Esslingen, Germany, 1984, pp: 2.10-1 to 2.10-4.

Keller, R.J. (1984b)'Cut-throat Flume Characteristics', Journal of Hydraulic Engineering, American Society of Civil Engineers, 110(9), 1248-1263.

Knight, D.W. and Sellin, R.H.J. (1987), The SERC Flood Channel Facility. J. Inst. Water and Environmental Mgmt, 1(2), 198-204.

Kobus, H. (1980) (ed.),'Hydraulic Modelling', German Association for Water Resources and Land Improvement, Bonn.

Kobus, H. (1984) (ed),'Symposium on Scale Effects in Modelling Hydraulic Structures', International Association for Hydraulic Research, Esslingen, Germany.

Lambert, M.F. and Myers, W.R.C. (1998),'Estimating the discharge capacity in compound channels.'Water, Energy and Maritime, Journal of Institution of Civil Engineers, London, 130: 84-94.

Lambert, M.F. and Sellin, R.H.J. (1996a),'Velocity distribution in a doubly-sinuous compound channel.'Water, Energy and Maritime, Journal of Institution of Civil Engineers, London, 118: 10-20.

Lambert, M.F. and Sellin, R.H.J. (1996b),'Discharge prediction in compound channels using the mixing length concept.'Journal of Hydraulic Research, International Association of Hydraulic Research, 34(3), 381-394.

Laurenson, E. M. (1986), Friction slope averaging in backwater calculations. Journal of Hydraulic Engineering, ASCE, 112(12):1151-1163, December 1986. ISSN 0733 9429/86/0012 1151, Paper No.21083.

Lawson, J. D. and O'Neill, I. C. (1980), The role of physical and mathematical models in hydraulic engineering, June 1981. ACADS Publication no. U217, Presentation to the Association for Computer Aided Design Limited (ACADS), Water Engineering Technical (WET) Committee, Wednesday 12 November, 1980.

LeVeque, R.J (2002), Finite Volume Methods for Hyperbolic Problems, Cambridge University Press, Cambridge.

Lee P., Lambert M.F. and Simpson A.R. (2002),'Critical depth prediction in straight compound channels.'Water and Maritime Engineering, Journal of Institution of Civil Engineers, London, 154(4), 317 - 332.

Leonard, B.P. (1979a), A Survey of Finite Differences of Opinion on Numerical Muddling of the Incomprehensible Defective Confusion Equation, In: T.J.R. Hughes (ed.) Finite Element Methods for Convection Dominated Flows, AMSE, New York.

Leonard, B.P. (1979b), A Stable and Accurate Convective Modelling Procedure Based on Quadratic Upstream Interpolation. Computer Methods in Applied Mechanics and Engineering, 19(1), 59-98.

Lipscomb, E.B. (1956), Hydraulic capacity of Meandering Channels in Straight Floodways, Report T.M. No. 2-2429, Waterways Experiment Station, Vicksburg, Miss, U.S.A.

Lotter, G.K. (1933), Soobrazheniia k gidravlicheskomu raschetu rusel s razlichnoli sherokhovatostiiu stenok (Considerations on hydraulic design of channels with different roughness of walls). Izvestiia Vsesoiuznogo Nauchno-Issledovatel skogo Instituta Gidrotekhniki (Trans., All Union Scientific Research, Institute of Hydraulic Engineering, Leningrad), 9: 238-241. Cited from Chow (1959).

Martin, L.A. and Myers, W.R.C. (1991), Measurement of overbank flow in a compound river channel. Proc. of the Institution of Civil Engineers, London, Dec. 1991, Part 2, 91(9729), 645-657.

Martins, R. (1989),'Preface to Recent Advances in Hydraulic Physical Modelling', R. Martens (ed), Kluwer Academic Publishers, Dordrecht, The Netherlands.

McBean, E. and Perkins, F. (1975), Numerical errors in water profile computation. Journal of the Hydraulics Division, American Society of Civil Engineers, 101(HY11):1389-1403.

McBean, E. A. and Perkins, F. E. (1970), Error criteria in water surface profile computations. Technical Report 124, Massachusetts Institute of Technology Hydrodynamics Laboratory, Cambridge, Masachusetts, June 1970.

Myers, W.R.C. (1987). Velocity and discharge in compound channels. Proc. ASCE, Journal of Hydraulic Engineering, Jun. 1987, 113(6), 753-766.

Myers, W.R.C. and Lyness, J.F. (1989), Flow resistance in rivers with flood plains. Final Report on Research Grant GR5/D/45437, University of Ulster.

Nikuradse, J. (1933), Strömungsgestze in rauhen Rohren (Laws of Flow in Rough Pipes). Ver. Dtsh. Ing. Forschungsh, Vol.361.

Novak, P. and Cábelka, J. (1981),'Models in Hydraulic Engineering - Physical Principles and Design Applications', Pitman Publishing Limited, London.

Olsen, N.R.B. (2003), 3D CFD Modeling of a Self-Forming Meandering Channel, Journal of Hydraulic Engineering, No.5, May, pp:366-372, doi:10.1061/(ASCE)0733-9429(2003)129:5(366).

Olsen, N.R.B. (2004), Closure to 'Three-dimensional CFD modeling of self-forming meandering channel', Journal of Hydraulic Engineering, 130(8), 838-839, doi:10.1061/(ASCE)0733-9429(2004)130:8(838).

Patankar, Suhas. (1980), Numerical heat transfer and fluid flow. CRC Press.

Petryk, S. and Grant, E.U. (1978), Critical Flow in Rivers with Flood Plains, Journal of the Hydraulics Division, May, 1978, 104(5), 583-594.

Phillips, J.T. and Ingersoll, T.L. (1998) Verification of roughness coefficients for selected natural and constructed stream channels in Arizona. U.S. Geological Survey Professional Paper, (1584).

Rajaratnam, N. and Ahmadi, R.M. (1983), Meandering Channels with Flood plains. Unpublished Paper.

Reynolds, O. (1894), On the Dynamic Theory of Incompressible fluids and the Determination of Criterion. Phil. Trans., Roy. Soc., 186: 123-164.

Rodi, W. (1980), Turbulence models and their applications in hydraulics - a state of the art review. Delft: IAHR book publications.

Rutherford, J.C. (1994),'River Mixing', John Wiley and Sons, Inc., New York.

Samuels, P.G. (1989), Backwater lengths in rivers. Proceedings of the Institution of Civil Engineers, Part 2, 87: 571-582, December 1989. Paper 9479, Water Engineering Group.

Samuels, P.G. and Chawdhary, K.S. (1992), A backwater method for trans-critical flows. In R. A. Falconer, K. Shiono, and R. G. S. Matthew, editors, Proceedings of the Second International Conference on Hydraulic and Environmental Modelling of Coastal, Estuarine and River Waters, volume two, pp.79-89, Aldershot, U.K., 1992. Ashgate Publishing Limited. Conference organised and sponsored by Hydraulics Research Limited, ISBN 1 85742 085 3.

Sellin, R.H.J. (1964), A laboratory investigation into the interaction between flow in the channel of a river and that over its flood plain. La Houille Blanche, Nov. 1964, 19(7), 793-801.

Sellin, R.H.J. and Giles, A. (1988), Two Stage Channel Flow, Final Report for the Thames Water Authority, Department of Civil Engineering, University of Bristol.

Sellin, R.H.J., Ervine, D.A. and Willets, B.B. (1993), Behaviour of meandering two-stage channels. Proc. of the Institution of Civil Engineers, Water, Maritime and Energy, June 1993, 101(2), 99-111.

Smith, C.D. (1978), Effect of channel meanders on flood stage in valley. Proc. ASCE, Journal of the Hydraulics Division, Jan. 1978, 104(1), 49-58.

Stelling, G.S. (1984), On the Construction of Computational Methods for Shallow Water Flow Problems, Rijkswaterstaat Communications, No.35/1984, The Hague.

Stelling, G.S., Kernkamp, H.W.J and Laguzzi, M.M. (1998), Delft Flooding System: A Powerful Tool for Inundation Assessment based upon a Positive Flow Simulation, in V. Babovic and L.C. Larsen (eds) Hydroinformatics '98, A.A. Balkema, Brookfield.

Streeter, V.L. and Wylie E.B. (1981), Fluid Mechanics. McGraw-Hill Ryerson Ltd., Singapore.

Streeter, V. L. and Wylie, E. B. (1979),'Fluid Mechanics', (7th ed), McGraw-Hill Kogakusha Ltd., Japan.

Strelkoff, T. (1969), One dimensional Equations of Open-Channel Flow. Journal of the Hydraulics Division, ASCE, May 1969, 95(3), 861-876.

Toebes, G.H. and Sooky, A.A. (1966), Hydraulics of meandering rivers with flood plains. ASCE Water Resource Engineering Conference, Denver, Colorado.

U.S. Department of Agriculture. (1963), Guide for selecting roughness coefficient (n) values for channels, Soil Conservation Service, Washington D.C.

Wallingford, H.R. (1992), SERC Flood Channel Facility Experimental Data - Phase A, Report SR 314, Wallingford, United Kingdom, May 1992, Vol.1-14.

Warnock, J.E. (1950), Hydraulic Similitude, chapter II, pages 136-176. John Wiley and Sons, Inc. and Chapman and Hall, Ltd., New York, London, 1950. Engineering Hydraulics, H. Rouse (editor), Proceedings of the Fourth Hydraulics Conference, Iowa Institute of Hydraulic Research, June 12-15, 1949.

Yen, B.C. (1973), Open Channel Flow Equation Revisited. Journal of the Engineering Mechanics Division, ASCE, Oct. 1973, 99(EM5), 979-1009.

Zheleznyakov, G.V. (1965), Relative deficit of mean velocity of unstable river flow, kinematic effect in river beds with flood plains. Proc. of the 11th International Congress IAHR, Leningrad, USSR.

Chapter 3. Hydraulic Structures

Robert Keller, William Weeks

Chapter Status	Final
Date last updated	14/5/2019

3.1. Introduction

Hydraulic structures are used to guide and control water flow velocities, directions and depths, elevation and slope of the streambed, general configuration of the waterway, and its stability and maintenance characteristics.

Careful and thorough hydraulic engineering is justified for hydraulic structures and consideration of environmental, ecological and public safety objectives should be integrated with hydraulic engineering design. The correct application of hydraulic structures can reduce maintenance costs by managing the character of the flow to fit the environmental and project needs.

Examples of hydraulic structures include flow measurement structures, transitions, constrictions, channel drops, low-flow checks, energy dissipators, bridges, bends, and confluences. Their shape, size, and other features vary widely for different projects, depending on discharge and the function to be accomplished. Hydraulic design procedures must govern the final design of all structures, including model testing for larger structures when the proposed design requires a configuration that differs significantly from known documented guidelines or when questions arise over the character of the structure being considered.

This review deliberately focusses on few the most important structures in urban and rural setting, as the general field of hydraulic structures is extremely vast. . Therefore, structures such as large dams are not considered. Hydraulic structures covered include flood bypass channels, control structures (gates, weirs and flumes, and spillways), levees, culverts, bridge waterways, floodways on roads, scour, rock chutes, rock riprap, and flow measurement structures.

3.2. Flood Bypass Channels

Diversion channels are used to divert waters from the main channel for many purposes including flood control, municipal water supply, and irrigation. The term *flood bypass channel* is typically used to describe a separate channel into which floodwaters are diverted to lessen the impact of flooding on the main river system. Such channels may bypass the flood flows into an adjacent waterway or return the flows back into the same stream a distance downstream from the point of the diversion.

On large river systems, flood bypass channels may simply comprise of adjacent low-lying areas or old river courses. Typically, control structures may be located at the head of the diversion channel to divert flows during periods of high water and return flows after the flood has passed. Flood bypass channels are often used in urban areas where it is not possible to widen the existing channel due to development.

Design considerations for flood bypass channels include:

- Determination of the percentage of the flood flow that should be carried by the bypass channel;
- Design of appropriate controls;
- Determination of the size of the channel to convey the design discharge; and
- Design the channel to reduce maintenance.

For effective reduction in the flood stage, the distance between the point of diversion and point of return to the main channel must be of sufficient length to prevent backwater effects. Additionally, it is essential to consider potential morphologic effects on both the main channel and receiving channel.

Flood bypass channels generally have steeper slopes than the main channel and this may lead to stability problems such as erosion of the channel bed and banks. The bed of tributary channels may be higher than that of the floodway channel, and bed degradation may migrate upstream of the tributary, resulting in excessive sediment transport and deposition in the floodway. Methods to mitigate channel instability such as grade control, channel lining and bank stabilisation may be required on diversion projects.

Additionally, diversion flows can adversely impact the main channel. If the flow rate is reduced in the main channel due to a diversion then, noting that the main channel slope and particle size remain constant, the sediment transport capacity of the main channel will decrease. This, in turn, could lead to aggradation in the main channel between the point of diversion and the point of re-entry. However, if excess bed material is diverted, the sediment transport capability of the stream may increase with the resultant rise in channel instability. Flow returning to the main channel from a diversion can also result in accelerated erosion of the channel and banks around the point of re-entry. Therefore, it is essential to conduct a detailed geomorphic and sediment transport analysis at the design stage of a diversion project, accounting for potential problems.

There are many environmental benefits of using a flood bypass channel as an alternative to modifying the main channel to convey flood flows. The original stream substrates and meanders are maintained, as well as in-stream cover and riparian vegetation. If designed only for occasional flood flows, the bypass channel can have multiple social and lifestyle benefits such as an urban greenbelt or sports and recreation areas.

One major application of flood bypass channels lies in reinstating meandering channels. Many previously meandering rivers in Australia are artificially straightened, thereby increasing the gradient of the river channel. The effect of this is to increase the conveyance of the river channel, thereby improving the drainage of the land and reducing the frequency and duration of overbank flooding. The consequences include deepening and widening and consequent instability of the main channel and a major decline of ecological function.

Where the original meanders are still available, there is a significant focus on redirecting the river channel flow back into the meanders, thereby *renaturalising* the river. Normally, however, the meanders have a significantly reduced flow capacity since they were filled up significantly during the years that the river has been straightened. For this reason, the straight alignment of the river can be treated as a flood bypass channel during the passage of large floods. This is achieved by introducing a weir into the straight bypass channel that overtops when the flow exceeds a predetermined value.

Figure 6.3.1 shows a schematic of the arrangement of a meander, floodway and weir.

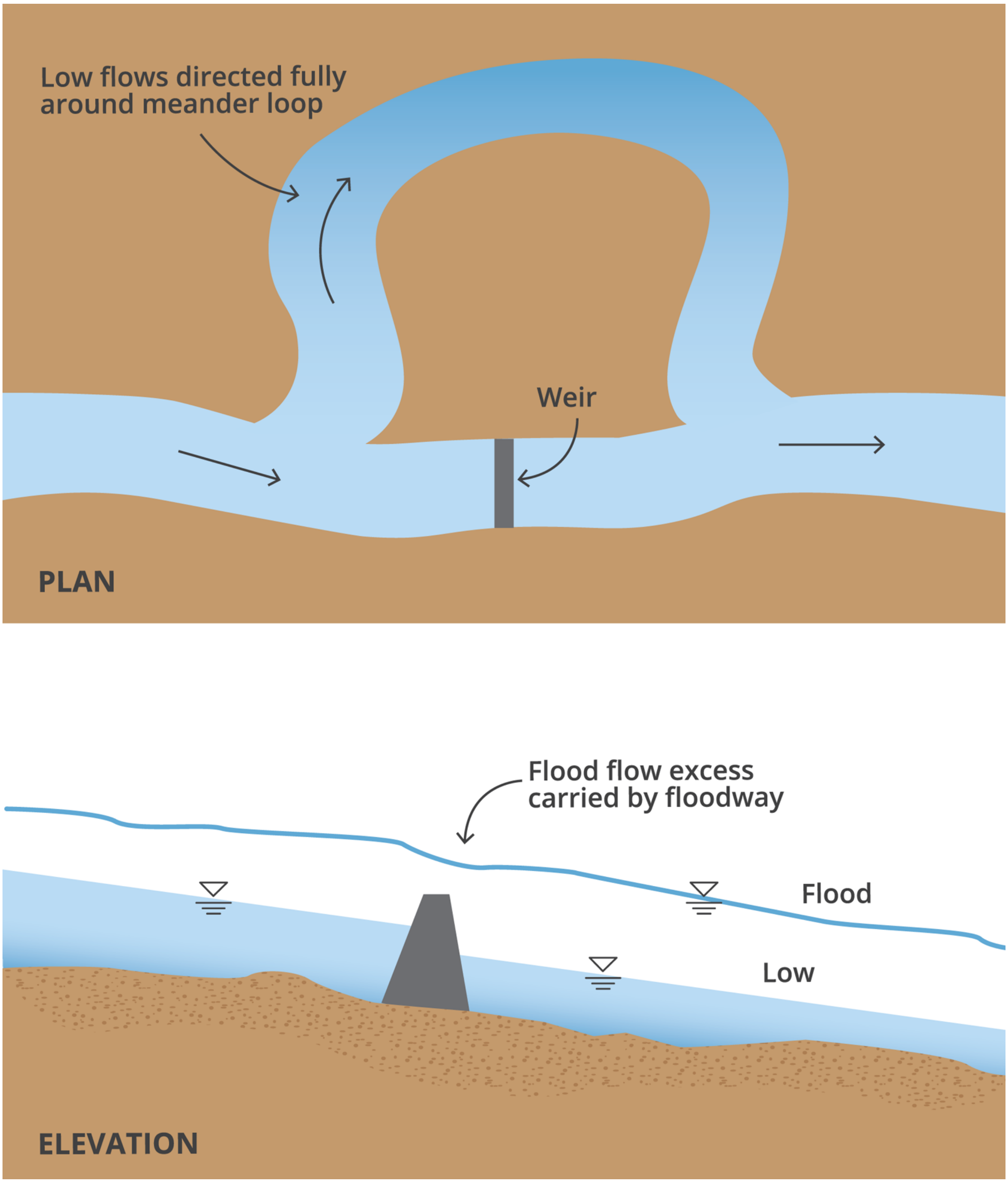


Figure 6.3.1. Schematic of Meander, Floodway, and weir

The design of this system requires careful consideration of the weir under drowned conditions. Under low flow conditions the weir directs all river flow around the meander. However, for a given design flood, the floodway and weir must pass all of the flow in excess of the capacity of the meander.

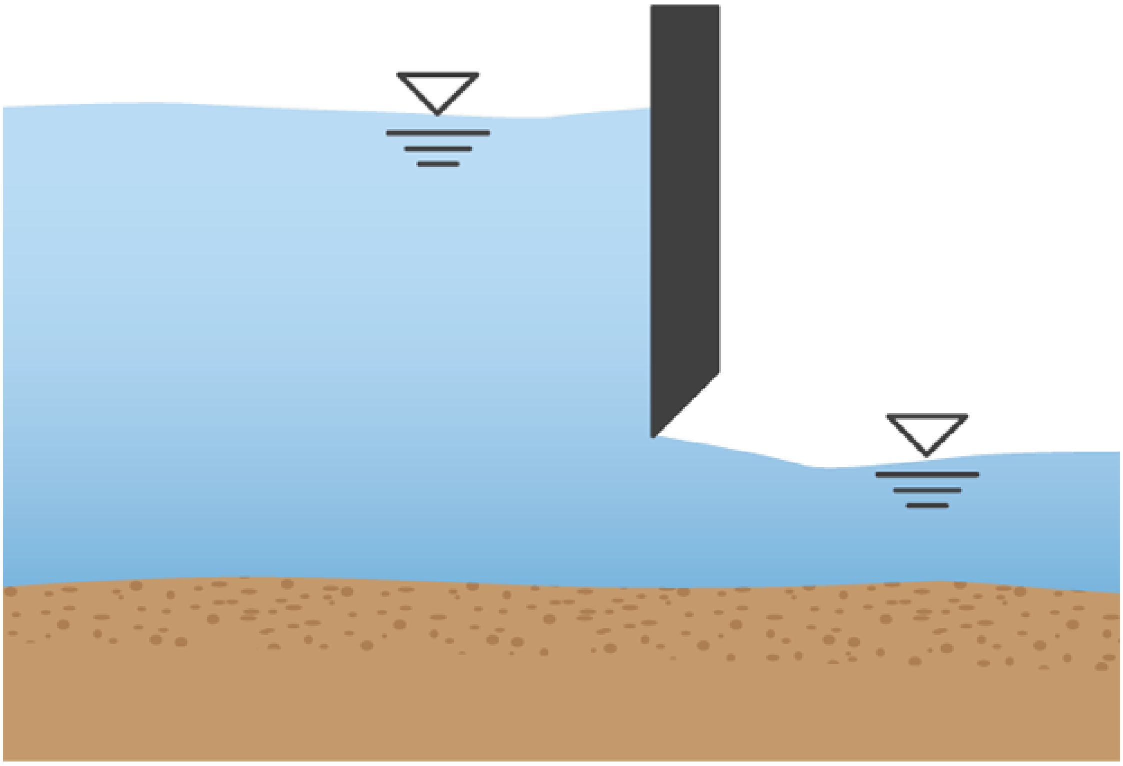
Keller (1995) has developed the theoretical analysis of the drowned weir. The analysis has been verified by experimental studies in the laboratory and with limited field data by Keller et al. (2012). The design process is assisted by the use of a spreadsheet based program described by Keller et al. (2012).

In summary, the design of a flood bypass channel must be aimed at preventing channel instability in the main channel and the diversion channel. Channel design must take into account the design flows and sediment transport to ensure bed and bank stability. The hydraulic design of flood bypass channels can be accomplished with standard hydrology and hydraulics analysis techniques, while determinations of sediment transport through the diversion are much more difficult.

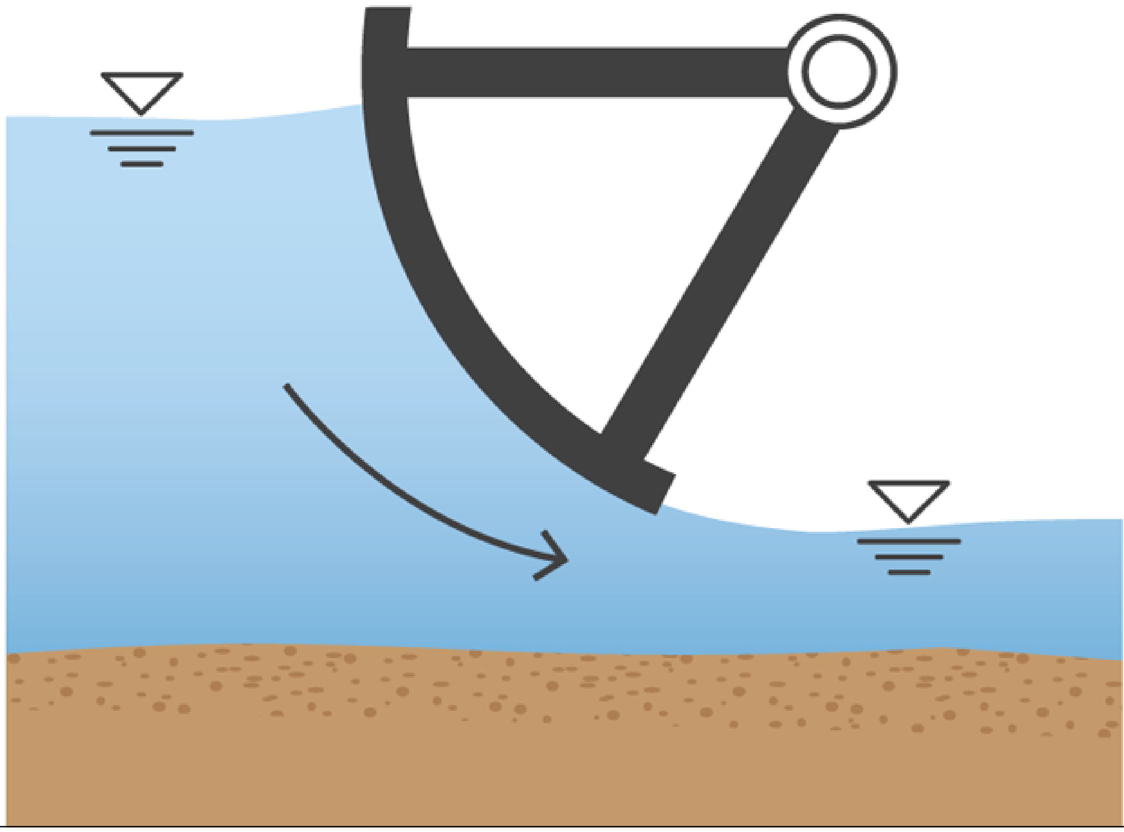
3.3. Control structures – Gates, weirs and flumes and spillways

3.3.1. Sluice gates and other control gates

Sluice gates are used to control the river flow, artificial channels and are sometimes referred to as underflow gates because the flow passes under the gate and control is exercised by lowering the gate. In addition to controlling the flow rate, gates can be used for flow measurement if they are calibrated by field measurement or by model testing. Figure 6.3.2 shows three different types of underflow gates – the vertical sluice gate, the radial (or Tainter) gate, and the drum gate.



(a)



(b)

The choice of gate in a particular situation depends on a number of factors. The vertical sluice gate is the simplest to construct, but has the disadvantage of requiring an expensive guide system to transmit the hydraulic thrust to the side-walls. The radial gate is better in this case because the thrust is carried through the radial arms upto the hinge. Drum gates are hollow gate sections that float on water and are pinned to rotate up or down. Water is allowed into or out of the flotation chamber to allow the gate to, respectively, fall or rise.

Flow through an underflow gate may be classified as free outflow or submerged outflow and the analysis for each is different.

3.3.2. Free Outflow

Free outflow conditions occur when the issuing jet of supercritical flow is open to the atmosphere and this is shown schematically in Figure 6.3.3.

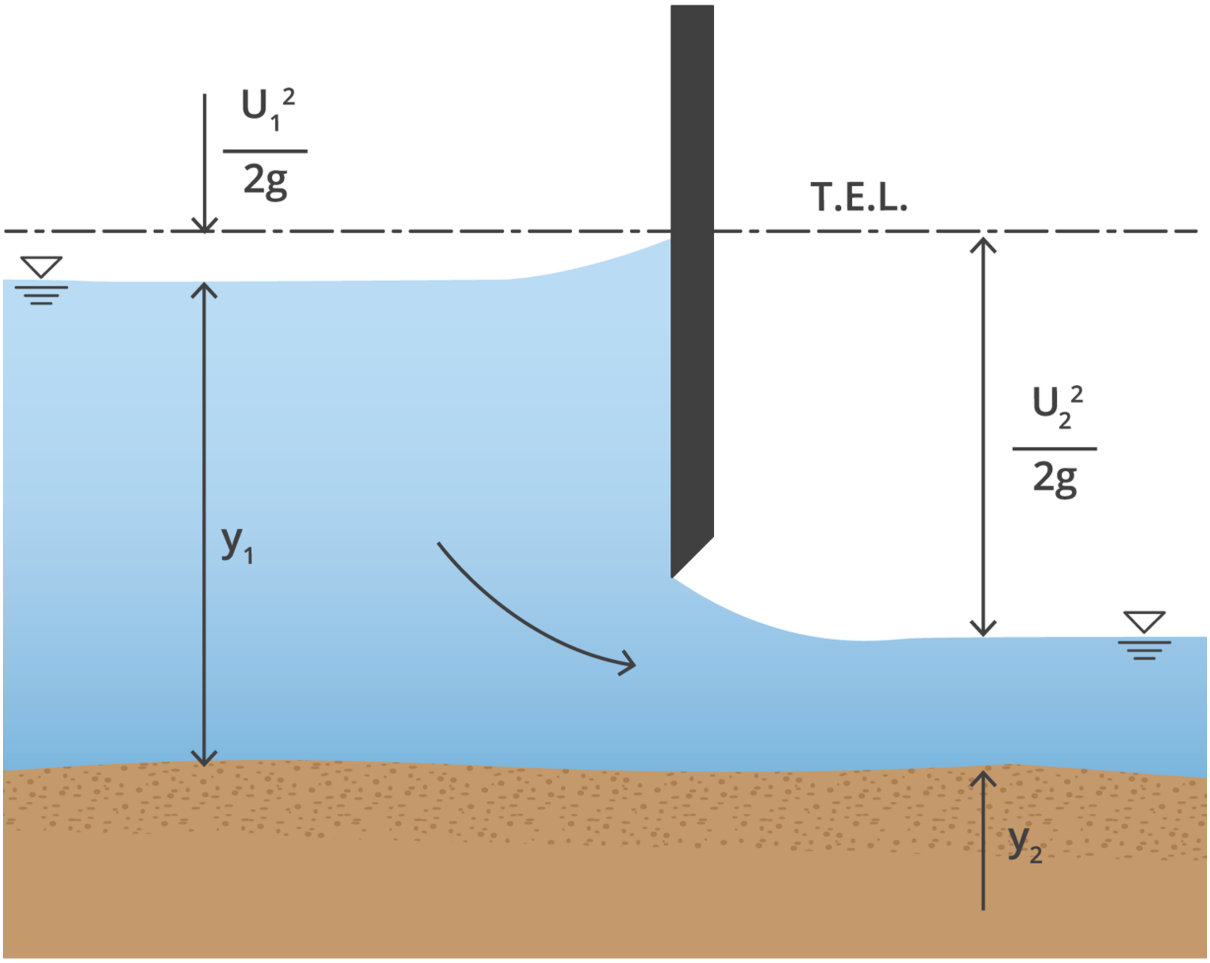


Figure 6.3.3. Schematic for flow under a Sluice Gate

The sluice gate is a case of rapidly varied flow in which large variations in depth and velocity occur over a short length of channel. Furthermore, because the flow contracts smoothly under the gate with a minimum of turbulence, energy losses are negligible and the energy level can be assumed to be the same on both sides of the gate.

For a rectangular channel with a horizontal bed and of uniform width, it can be shown – Henderson (1966) that:

$$Q = C_c wB\sqrt{2gy\frac{y_1}{y_1 + y_2}} \qquad (6.3.1)$$

where $C_c = \frac{y_2}{w}$

Equation (6.3.1) is then written as:

$$Q = C_d wB\sqrt{2gy_1} \qquad (6.3.2)$$

where $C_d = \sqrt{\frac{C_c}{1 + \frac{C_c w}{y_1}}}$

The contraction coefficient, C_c typically has the value of 0.611. However, for increased values of the ratio w/y_1, the value decreases slightly.

The same analysis can be undertaken for the radial gate. In this case, however, the contraction coefficient, C_c, varies significantly depending on the angle that the gate lip makes with the horizontal. To an accuracy of 5%, the contraction coefficient is given by:

$$C_c = 1 - 0.75\theta + 0.36\theta^2 \qquad (6.3.3)$$

where the unit of θ is 900.

Equation (6.3.3) shows that C_c has the value of 0.61 for θ = 1 (90^0), which is the value used for the vertical sluice gate.

3.3.3. Drowned Outflow

A schematic of a drowned vertical sluice gate is shown in Figure 6.3.4. The depth y_2 is produced by the gate, and the depth y_3 is produced by some downstream control. It is clear that if y_3 is greater than the subcritical depth required to form a hydraulic jump with y_2, then the gate outlet must be drowned; a condition where subcritical flow impinges on the downstream side of the gate. The effect is that the jet of water issuing from beneath the gate is overlaid by a mass of water which, although strongly turbulent, has no net motion in the longitudinal direction.

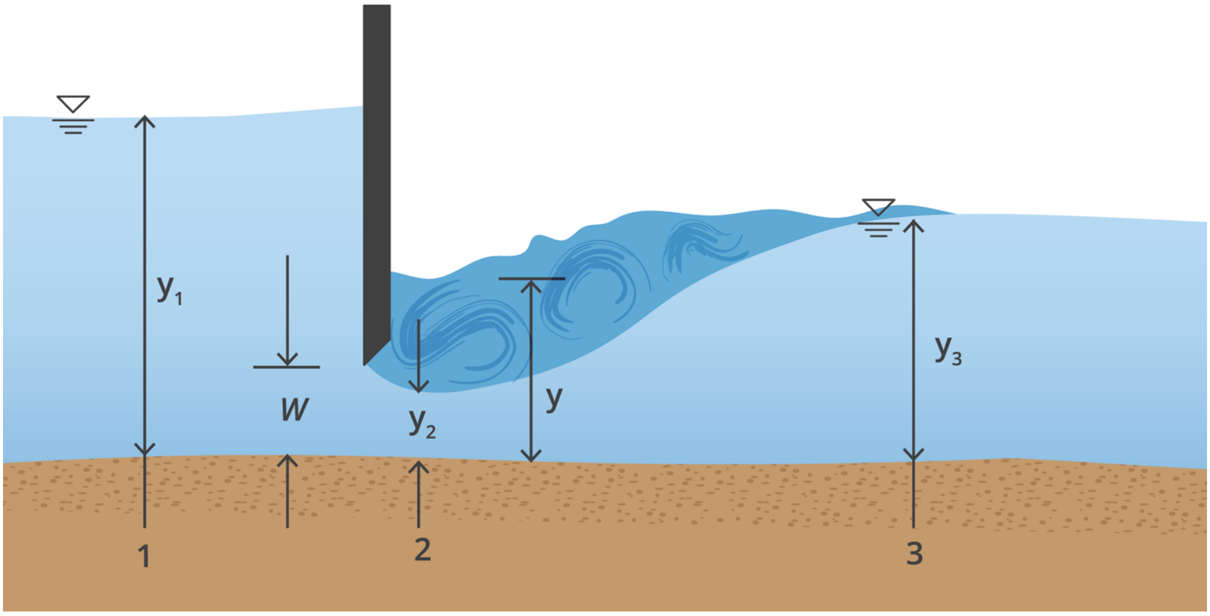


Figure 6.3.4. Schematic of Drowned Vertical Sluice Gate

An approximate analysis can be made by treating the case as one of *divided flow*, in which part of the flow section is occupied by moving water, and part by stagnant water. While there will be some energy loss between Sections 1 and 2, a much greater proportion of the total loss will occur in the expanding flow between Sections 2 and 3. The approximation enters when it is assumed that all of the energy loss occurs between Sections 2 and 3, where the momentum equation is utilised.

This procedure, developed in Henderson (1966), leads to two independent equations with two unknowns – q and y.

The solution for drowned gates in non-rectangular channels follows the same basic methodology although the computations are more complex.

The methodology has been tested experimentally and shown to predict the flow rate within 5%.

3.3.4. Weirs and flumes

Weirs and flumes are often used for flow measurement, as noted in the section of this chapter on Flow Measurement Structures. However, since the design of these structures involves application of a known relationship between flow rate and water surface elevation, they can also be employed as control structures – used to control the water level or water level range, for a given flow rate or flow rate range.

In this context, the structure design follows the same procedure as developed earlier. In particular, broad-crested weirs are robust structures that span the full width of the channel and are normally constructed of reinforced concrete. Especially for flow control in relatively large rivers, they are preferred over sharp-crested weirs, which can be easily damaged. Furthermore, because it is a critical depth meter, the broad-crested weir has the advantage that it operates effectively with higher downstream water levels than a sharp-crested weir.

For relatively small open channels, the long-throated flume is a better alternative than a weir and is capable of measuring relatively large flows. It is basically a width constriction that, in plan, a rounded converging section, a parallel throat section, and a diverging downstream section. It is typically constructed of concrete.

The design of a long-throated flume requires some compromise between ensuring that the throat is narrow enough to control the flow without submergence, but not so narrow that it creates unacceptable afflux.

As noted in this chapter on Flow Measurement Structures, the analysis of both the broad-crested weir and the long-throated flume is identical as both rely on the relationship between the upstream water level (which may be measured) and critical depth within the constricted section, which is a known function of the flow rate. Thus, a unique relationship between flow rate and upstream water surface elevation can be determined.

3.3.5. Spillways

Flow behaviour on spillways has been investigated extensively by the US Army Corps of Engineers Waterways Experiment Station since the early 1950s (USACE: Water Ways Experiment Station, 1952). Hydraulic design charts and a Manual of Practice (USACE, 1995) have been prepared, enabling the design of a spillway profile and a water surface profile for a given design flood condition. However, the design charts are only applicable for certain types of spillway profiles and pier configurations and cover a limited range of flood levels.

In the past, this limitation was overcome by building scaled physical models to investigate the flow behaviour. These models tended to include both the spillway and any associated energy dissipation structure. Physical models are considered later in the chapter. More advanced mathematical models may also be appropriate, which are also discussed later in this Book.

A spillway is ideally designed so that, when operating at its design head, the pressure at the spillway surface is atmospheric. Consequentially, when the reservoir level is below or above the design flood level, the pressure over the spillway will be above or below atmospheric respectively. In the latter case, the negative pressures may create unstable conditions on the spillway surface and damage due to cavitation.

Existing dams and spillways in Australia were designed and constructed to handle estimated design floods. Since their construction, the increase to and reanalysis of, hydrological data have led in many cases to a revision upwards of the design floods, requiring major upgrades to spillway capacity.

To select optimum upgrade design, many dam owners have needed to consider the most cost-effective way to analyse the behaviour of the spillway flow under conditions of increased maximum flood. In many cases, as in the original design, use has been made of physical scale models.

With appropriate recognition of scale effects, physical models have been the upgrade design method of choice. However, the use of numerical methods is attractive in terms of lower cost and substantially reduced preparation time. Additionally, results can be obtained throughout the flow domain rather than at selected monitoring locations.

3.3.5.1. Design

A spillway is sized to provide the required capacity, usually the entire spillway design flood, at a specific reservoir elevation. This elevation is normally at the maximum operating level or at a surcharge elevation greater than the maximum operating level. Hydraulic design of a spillway usually involves four conditions of flow, each occurring at a different location as follows:

1. Subcritical flow in the spillway approach, initially at a low velocity, accelerating, however, as it approaches the crest.
2. Critical flow as the water passes over the spillway crest.
3. Supercritical flow in the chute below the crest.
4. Transitional flow at or near the downstream end of the chute where the flow must transition back to subcritical, typically with the dissipation of large amounts of energy.

When a relatively large storage capacity can be obtained above the normal maximum reservoir elevation by increasing the dam height, a portion of the flood volume can be stored in this reservoir surcharge space and the size of the spillway can be reduced. The use of a surcharge pool for passing the spillway design flood involves an economic analysis that considers the added cost of a dam height compared to the cost of a wider and/or deeper spillway. When a gated spillway is considered, the added cost of higher and/or additional gates and piers must be compared to the cost of additional dam height.

When an un-gated spillway is considered, the cost of reduced flood-control benefits due to a reduction in reservoir storage must be compared to the cost of additional dam height.

Chute design and stilling basin design are considered in particular in the following sections.

3.3.5.2. Chute Design

The basic principle used to analyse steady incompressible flow on a chute spillway is the law of conservation of energy expressed by the Bernoulli equation. This equation has been developed elsewhere in this chapter. Herein, the issues consequent to the (generally) steep spillway slope are considered.

The elevation of the hydraulic grade line is typically given by:

$$HGL = z + y_1 \tag{6.3.4}$$

where z is the elevation of the bed above datum

y_1 is the depth of flow, normal to the channel bottom

Strictly speaking the second term on the right hand side should be replaced by $y_1\cos\theta$ where θ is the slope of the channel bottom. Additionally, the form of Equation (6.3.4) assumes that the pressure distribution at the point under consideration must be hydrostatic. This is a valid assumption if vertical accelerations are small and the bed slope is mild. A non-hydrostatic pressure distribution will occur whenever the value of $\cos 2\theta$ departs materially from unity, such as on a steep spillway slope. This does not mean that the energy equation cannot be used on a steep slope. It does mean, however, that the designer must recognise that the values derived from the energy equation become increasingly inaccurate as the value of $\cos 2\theta$ departs further from unity. This conditions describes one of the basic reasons that physical model studies may be required when designing a spillway.

When applying Equation (6.3.4) to spillway design, correct account should be taken of energy loss on the spillway surface. This has three components - boundary roughness (friction), turbulence resulting from boundary alignment changes (form loss), and boundary layer development. Boundary roughness is normally dealt with using a standard friction loss equation such as Manning's equation. For information on the other two loss terms, reference should be made to (USACE, 1995).

3.3.5.3. Stilling Basin Design

The transition of flow from supercritical on the chute to subcritical usually involves considerable energy dissipation. Dissipation of hydraulic energy is accomplished by various methods such as the hydraulic jump, impact, dispersion, etc. The type of energy dissipator used is dependent upon factors that include site geology, the type of dam structure, and the magnitude of the energy to be dissipated. The design discharge for effective energy dissipation is frequently set at the standard project flood rate; however, each facility must be evaluated, and the design discharge used should be dependent upon the damage consequences when the design discharge is exceeded.

Hydraulic jump stilling basins are structures located downstream of chutes, gates and spillways to dissipate excess kinetic energy. The dimensions of these structures depend on the length of the hydraulic jump and the conjugate depth of the jump.

Peterka (1978) classified the hydraulic jump into five categories based on the value of the upstream Froude number. On the basis of extensive experimental studies, he developed four types of hydraulic jump stilling basins. These are now known as the USBR Type 1, 2, 3, and 4 Basins. A major focus of the development of these basins was to reduce the size of the structure by forcing the jump to occur using blocks and end sills within the basin. In addition to localising the hydraulic jump, the Type 4 basin is designed for the special purpose of wave suppression and is not considered herein.

The Type 1 Basin is a classic hydraulic jump basin without baffle blocks or an end sill. It is a relatively large structure and is suitable only for small upstream Froude numbers.

When the Froude number is greater than 4.5, Type 2 or Type 3 Stilling Basins are recommended. The Type 2 basin incorporates a series of chute blocks at the upstream end of the basin to stabilise the start of the jump and to feather the incoming jet into several jets. At the downstream end, a continuous or dentated sill is present, designed to force the jump to occur within the basin and to prevent it from moving downstream. Figure 6.3.5 shows a schematic of the Type 2 Basin. In this figure, y_2 is the required conjugate downstream depth.

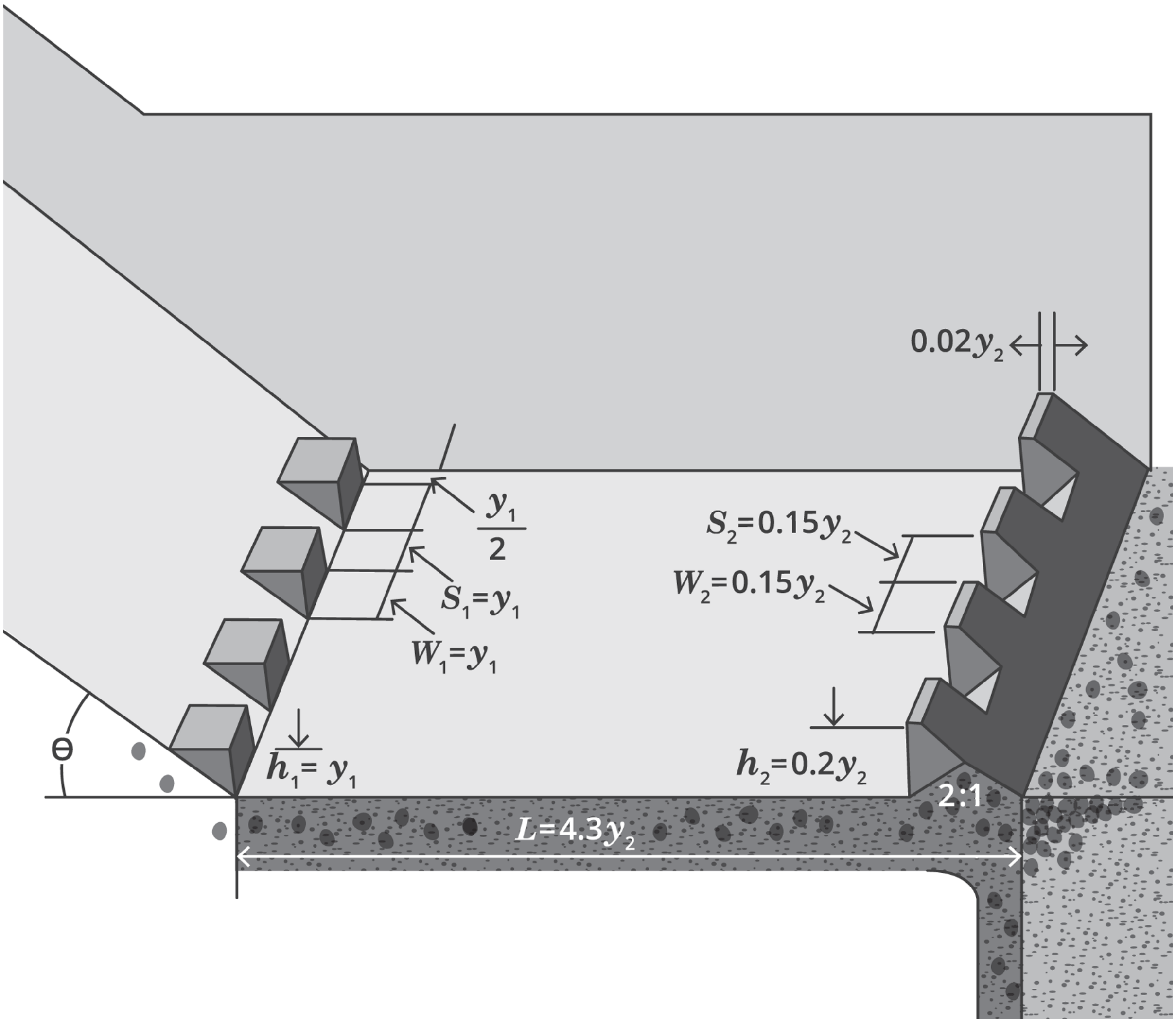


Figure 6.3.5. Schematic of USBR Type 2 Stilling Basin

The length of the Type 2 basin is less than the length for the Type 1 basin.

The Type 3 stilling basin is similar to the Type 2 basin but baffle blocks are included to provide additional energy dissipation by direct impact, increased turbulence and consequent mixing of the high velocity incoming jets into the water body of the basin. This results in a required basin length that is up to 60% shorter than a Type 1 basin for the same flow conditions. However, it should be noted that the presence of the baffle blocks can create conditions of cavitation in their vicinity with consequent severe structural damage. For this reason the Type 3 basin should not be used in conditions where the incoming velocity exceeds 16m/s. Figure 6.3.6 shows a schematic of a Type 3 Basin.

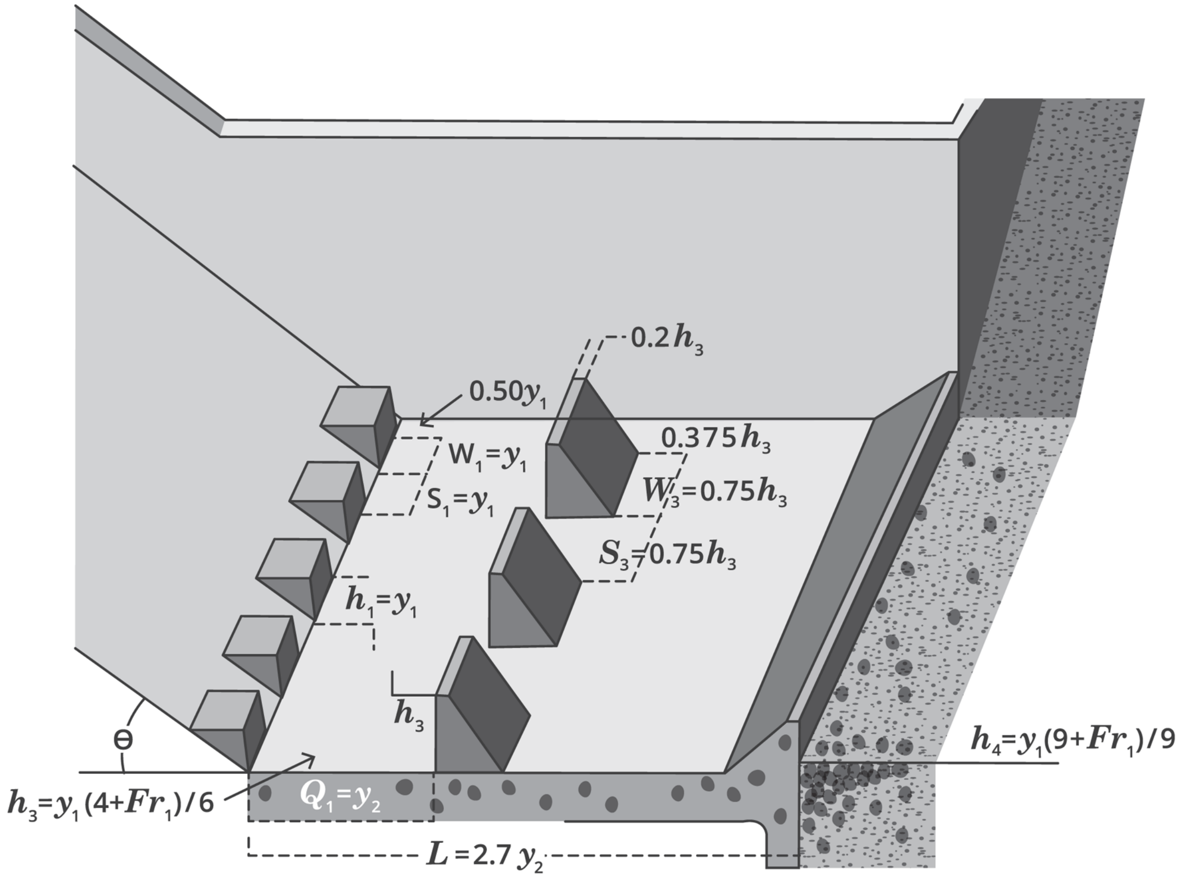


Figure 6.3.6. Schematic of USBR Type 3 Stilling Basin

3.4. Levees

Levees are embankments that are constructed to artificially increase the capacity of a channel, confining high flows that otherwise would overtop the banks and spread over the floodplain. Levees are key components of a flood control plan to protect communities and agricultural areas within the floodplain. Levees are used in conjunction with reservoirs, floodways, control structures and various channel modification activities to reduce and control the extent and duration of flooding.

The design elevation of levees is based on containing a design discharge, generally for a short period of time. The levee cross section is generally designed as a trapezoid, with an access road running along the levee crown. To control seepage, a long, tapering berm may be extended on the landside of the levee as subsequently discussed. Fill material for levees is generally obtained locally from borrow areas adjacent to the riverside of the embankment. Although the local materials may not be ideally suitable for construction, economic necessity normally dictates its use.

On streams without levees, flood flows spread out over the floodplain. The floodplain acts as storage for the additional flows, lowering the peak of the flood hydrograph. The construction of levees decreases the floodplain storage, resulting in an increase of the peak of the hydrograph. Furthermore, because levees typically confine river flows to a narrower cross section, water elevations are higher during flood flows. If levees are not set back from the main channel, the hydraulic connectivity between the river and the floodplain is lost, thus confining flows and putting more energy into the flow. The required levee height can be determined using industry-standard analysis programs such as HEC-RAS. Frequently it is useful to compare the water surface elevations for a river without levees and with levees to evaluate the cost-benefit of levee construction.

Channel instabilities may arise from streams with levees because degradation of the bed and banks may occur. Sometimes, aggradation may occur due to the increased sediment load in the main channel and the lack of available floodplain sediment storage. The precise response is complex and is a function of the width of levees, the effects on duration of flows, and other factors.

Seepage is a major problem with levees during time of high water. When water is contained on one side with the other side being dry, a head differential exists across the levee. This tends to force water through the porous soil, eventually seeping out to the landward side of the levee. This seepage carries both fine and coarse particles through the levee. This internal erosion of the levees can lead to piping through the levee and catastrophic failure.

To prevent excessive seepage, impervious barrier materials such as clay can be built into the levee. Flows from tributaries that are cut off from the river system due to levees must be carefully assessed to prevent flooding on the landward side of the levee. Pumping stations can be applied to divert tributary flows.

On streams without levees, flows periodically flow onto the floodplain depositing sediment, flushing riparian aquatic environments, and generally providing valuable habitat for aquatic organisms and waterfowl. The flora and fauna are adapted to periodic flooding and the unique environment that it creates. As noted above, levees act as a barrier for overbank flows. Confining stream flows within a levee system creates a dryer environment on the landside of the levee system and a wetter environment on the streamside.

The dryer environment results in changes in both flora and fauna that occupy the floodplain. After a levee system is constructed, upland trees and vegetation colonise the floodplain. The lands between the levee and the stream bank will experience more prolonged flooding with more extreme fluctuations in water level. This may inhibit the growth of ground cover, thus reducing the available habitat for ground-dwelling mammals (Fredrickson, 1979). Frequently, for reasons of economy, material used to construct the levees is sourced from areas within the floodplain, resulting in vegetation removal and loss of habitat. The flat slopes used for levees in rural areas require large land requirements for the embankments and berms.

To offset changes in riparian habitat, consideration can be given to the habitat provided by the levees themselves and the adjacent borrow pits. Traditionally, the vegetation on levees is kept to a minimum. However, with proper maintenance, certain species of shrubs and plants can be allowed to grow without affecting the integrity of the levee. However, this vegetation may provide habitat for burrowing animals that must be controlled.

Borrow pits remaining from levee construction can serve as valuable aquatic habitat. Normally, the pits will fill with rainwater or groundwater after construction. Riverside borrow pits will exchange water with the river system, thus recharging the pit with fish and other aquatic organisms. In this way, borrow pits partially compensate for the loss of aquatic habitat in the floodplain. Additionally, siting levees further from the channel will conserve wetland environments between the levee and the river.

Levees must be periodically inspected and maintained to provide the designed degree of flood protection. Conditions affecting the integrity of the levee include erosion of the banks, seepage, and damage from burrowing animals. Vegetation planted on the levees for aesthetic reasons should be well maintained. Vegetation that may affect the integrity of the levee should be removed.

Seepage beneath the levee foundations is one of the principal causes of levee failure. Without control, this seepage may result in excessive hydrostatic pressures beneath an impervious top stratum on the landside, sand boils, and/or piping beneath the levee itself. Seepage problems tend to be most acute in situations where the levee is built above a pervious substratum, which extends both landward, and riverward of the levee and where a relatively thin top stratum exists on the landside of the levee.

Among seepage control measures are cutoffs, riverside blankets, and landside seepage berms.

3.4.1. Cutoffs

A cutoff beneath a levee to block seepage through pervious foundation strata is the most positive means of eliminating seepage problems. A cutoff may consist of an excavated trench backfilled with compacted earth or slurry. Trenches are usually located near the riverside toe.

To be effective, a cutoff must penetrate at least 95 percent of the thickness of the pervious strata to be effective. For this reason cutoffs are rarely economical where they must penetrate more than about 12 m. Steel sheet piling can significantly reduce the possibility of piping of sand strata in the foundation, but is not always entirely watertight due to leakage at the interlocks between individual sheet piles.

3.4.2. Riverside Blankets

Levees are frequently situated on foundations having natural covers of relatively fine-grained soils overlying pervious sands and gravels. These surface strata constitute impervious or semi-pervious blankets when considered in connection with seepage control. If these blankets are continuous and extend riverward for a considerable distance, they can effectively reduce seepage flow and seepage pressures on the landside of the levee.

Where seepage beneath the levee is expected to be a problem, riverside borrow operations should be limited in depth to prevent breaching the impervious blanket. If there are limited areas where the blanket becomes thin or disappears entirely, the blanket can be remediated by placing impervious materials in these areas. The effectiveness of the blanket depends on its thickness, length, distance to the levee riverside toe, and permeability and can be evaluated by flow-net or approximate mathematical solutions (USACE, 2000). Protection of the riverside blanket against erosion is important.

3.4.3. Landside Seepage Berms

If uplift pressures in pervious deposits underlying an impervious top stratum landward of a levee become greater than the effective weight of the top stratum, heaving and rupturing of the top stratum may occur, resulting in sand boils. The construction of landside berms (where space is available) can eliminate this hazard by providing the additional weight needed to counteract these upward seepage forces. Furthermore, the berm can provide the additional length required to reduce uplift pressures at the toe of the berm to acceptable values. Seepage berms may reinforce an existing impervious or semi-pervious top stratum, or, if none exists, be placed directly on pervious deposits. A berm also affords some protection against degradation of the landside levee slope.

Berms are relatively simple to construct and require very little maintenance. They frequently improve and reclaim land as areas requiring remediation treatment for seepage are often low and wet. Because they require additional fill material and space, they are used primarily with agricultural levees where land use pressures are less severe than in urban areas.

Subsurface profiles must be carefully studied in selecting berm widths. For example, where a levee is founded on a thin top stratum and thicker clay deposits lie a short distance landward, as shown in Figure 6.3.7, the berm should extend far enough landward to lap the thick clay deposit, regardless of the computed required length. Otherwise, a concentration of seepage and high exit gradients may occur between the berm toe and the landward edge of the thick clay deposit.

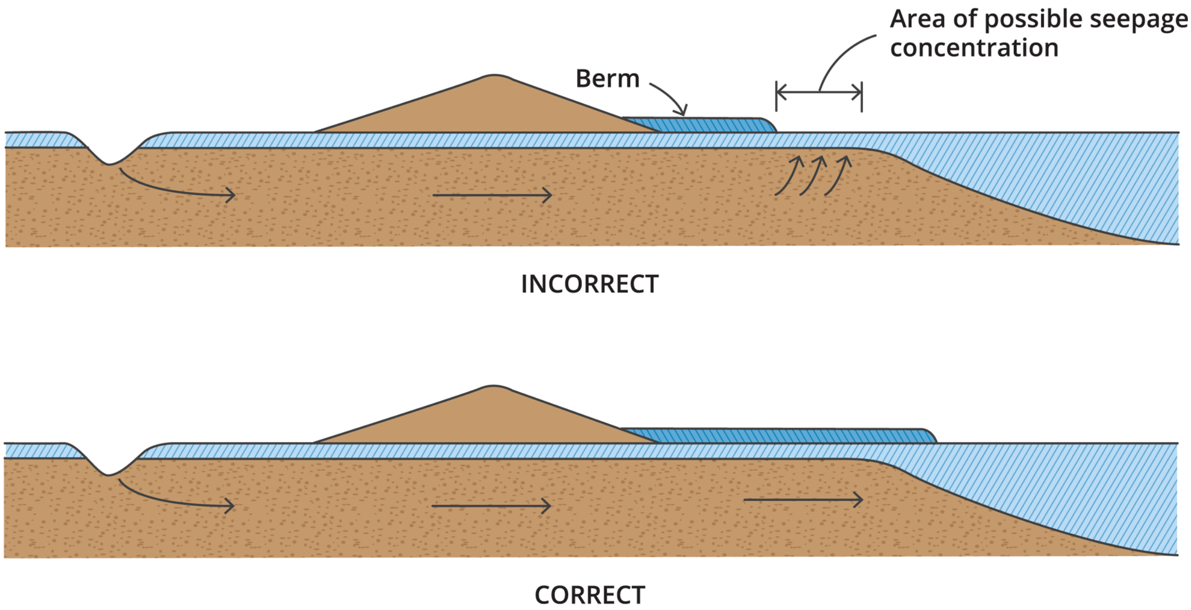


Figure 6.3.7. Example of incorrect and correct berm length according to existing foundation conditions (USACE, 2000)

In summary, levees are embankments that artificially increase the capacity of a channel, confining high flows that otherwise would overtop the banks and spread over the floodplain. They are key components of a flood control plan to protect communities and agricultural areas within the floodplain.

Seepage is a major problem with levees during high water and is one of the principal causes of levee failure. When water is contained on one side with the other side being dry, a head differential exists across the levee. Without control, this seepage may result in excessive hydrostatic pressures beneath an impervious top stratum on the landside, sand boils, and/or piping beneath the levee itself.

Among seepage control measures are cutoffs, riverside blankets, and landside seepage berms.

3.5. Culverts

3.5.1. Culvert Flow Principles

The term *culvert* is normally applied in engineering practice to any large underground pipe especially where used in relatively short lengths to convey streams or flood water under an embankment. The design of culverts has been the subject of considerable research and considerable misunderstanding. Despite this, the culvert is such a common structure that analysis and design have become quite standardised. The hydraulic analysis and subsequent selection of the proper culvert size is aided by charts and nomographs prepared for the specific shape and type of culvert. These design procedures incorporate directly such factors as the entrance loss coefficient for a particular pipe shape and inlet configuration.

The emphasis herein is on the basic analysis of culverts and is, thus, more general than direct recourse to design charts. It is accepted, of course, that engineers will continue to use the available charts and nomographs. However, the material presented herein is aimed at giving a better understanding of culvert flow principles and will make the use of standard charts and nomographs clearer and more reliable.

Because a culvert is a closed conduit, it has a larger wetted perimeter than a channel. Accordingly, the average energy gradient through the culvert will be steeper than in the equivalent length of channel. In general, the only way that the steepening of the hydraulic gradient through the culvert can occur is by raising the water surface elevation at the upstream side of the embankment.

However, the headwater level cannot be increased indefinitely without severe consequences. The consequent backwater effect may cause water to overflow the channel banks and cause flooding of the surrounding land. This may have severe social and economic repercussions. There are, however other less obvious consequences of an increase in the upstream water level such as bank stability, scour of the earth embankment, and erosion of downstream channel.

It is apparent, then, that the culvert cross-sectional area and hydraulic properties are of great importance. It may be possible to evaluate economically the consequences of a headwater rise against the cost of the culvert and embankment height. Under these circumstances the system with the least total cost of structure and flooding should be selected.

The factor subject to most misunderstanding in culvert design is that arising from the determination of the point of control – either inlet or outlet control. In some cases, the operating control is not clear and careful calculations are necessary to determine both the type of control and the various hydraulic characteristics.

The hydraulic operation of culverts is complex and often difficult to predict. However, once the type of operation is established, the analysis may proceed according to well-defined principles. The factors affecting the discharge in a culvert are the following:

a. The geometry of the inlet.

b. The combined effect of entrance, length, slope and roughness of the culvert barrel.

c. The elevation of the outlet tailwater.

The flow characteristics and, hence, the discharge capacity of a culvert are determined by the location of the control section. In general, the discharge is controlled either at the culvert entrance or at the outlet and is designated inlet control and outlet control respectively. Inlet control will exist as long as the ability of the culvert barrel to carry the flow exceeds the ability of water to enter the culvert through the inlet. Outlet control will exist when the ability of the culvert barrel to carry water away from the entrance is less than the flow than can enter the inlet. The location of the control section may shift as the relative capacities of the entrance and barrel sections change with increasing or decreasing discharge.

Inlet control: With the inlet control operation, the discharge is independent of the pipe length, slope and roughness of the pipe wall. The discharge depends only upon the headwater elevation above the invert at the entrance, the inlet size, and the inlet geometry. Although a variation in factors affecting the culvert barrel will affect flow characteristics within a barrel, they will normally have no effect on the total discharge. The only exception occurs if the variations in barrel design are sufficiently severe to cause the control section to shift to the outlet.

A culvert operating under inlet control will always flow part full for at least part of the culvert length. In many cases, particularly at high discharges, the headwater will submerge the entrance of the culvert. In these cases, flow contraction occurring at the entrance will limit the discharge. It should be noted that roughness, slope and length are not influential in determining the discharge capacity of a culvert operating with inlet control, but are important in determining outlet velocities and the discharge at which the operation mode changes from inlet control to outlet control.

Outlet control: Under outlet control, the total discharge is affected by all hydraulic factors upstream of the outlet. These factors include the headwater elevation, entrance geometry, barrel size, wall roughness, barrel length and slope. The tailwater elevation is a factor as long as it is above the pipe outlet.

Culverts flowing full throughout their length are always under outlet control. However, as will be shown, a culvert flowing part full may operate under either inlet control or outlet control.

Hydraulic Analysis: For computational convenience, flow through culverts is divided into size categories based on the relative heights of the head and tailwater and, for three of the categories, on barrel slope. The six types of flow are shown schematically in Figure 6.3.8 and their respective characteristics are summarised in Table 6.3.1. In the table, D is the maximum vertical dimension of the culvert, y_1 is the depth of flow in the approach section, d_c is the critical depth of flow, and y_4 is the tailwater depth of flow.

The limit for $\frac{y_1}{D}$ of 1.5 recommended in Table 6.3.1 is not universally accepted and several texts suggest a limiting value of 1.2. This lower limit is probably more appropriate since it allows for effects of factors such as wave motion or transitory debris blockage for example.

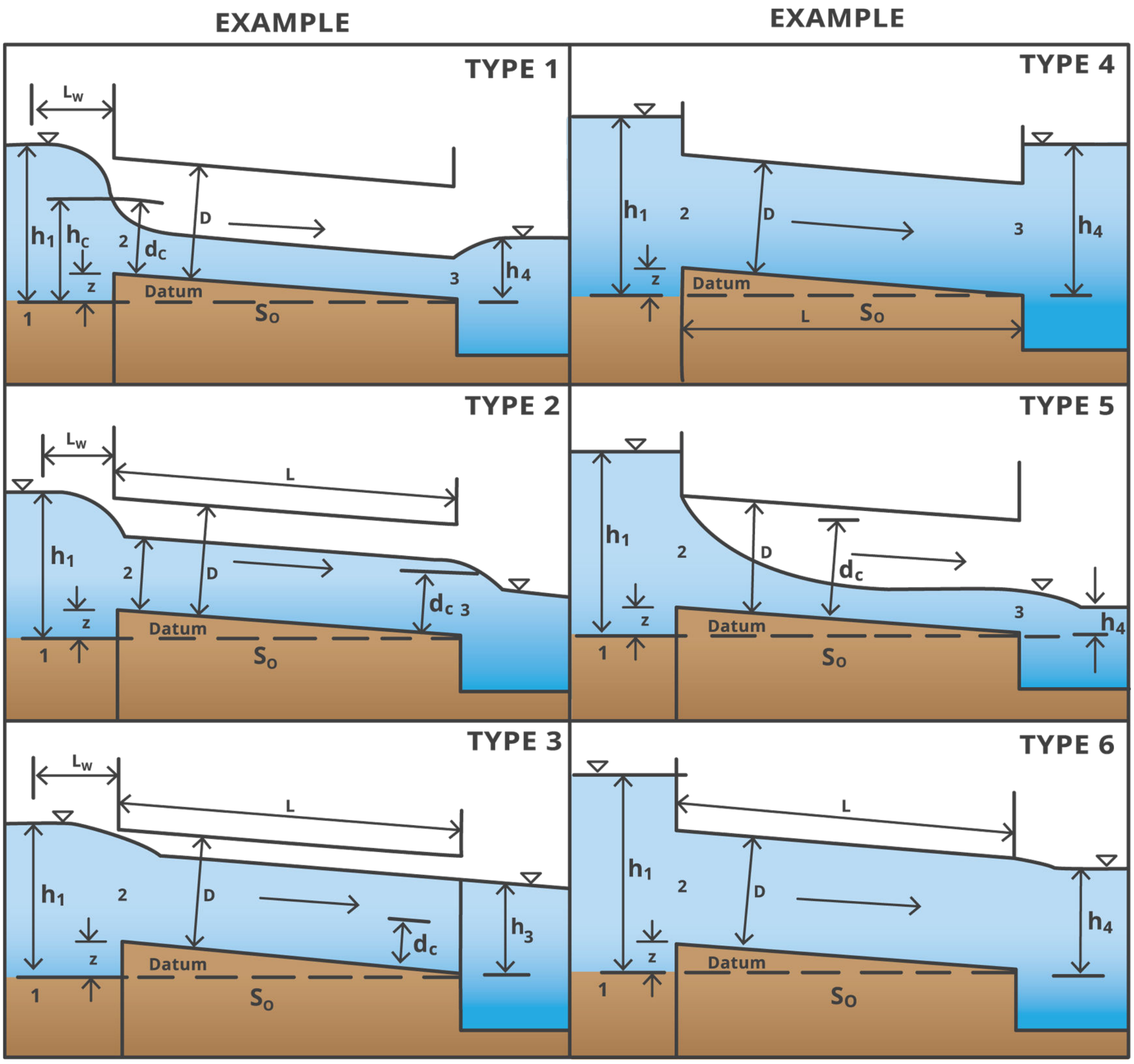


Figure 6.3.8. Culvert Flow Types

Table 6.3.1. Culvert Flow Characteristics

Flow Type	**Culvert barrel flow**	**Location of downstream section**	**Control type**	**Culvert slope**	$\frac{y_1}{D}$	$\frac{y_1}{d_c}$	$\frac{y_4}{D}$
1	Partly full	Inlet	Critical depth	Steep	<1.5	<1.0	≤1.0
2	Partly full	Outlet	Critical depth	Mild	<1.5	<1.0	≤1.0
3	Partly full	Outlet	Backwater	Mild	<1.5	<1.0	≤1.0
4	Full	Outlet	Backwater	Any			> 1.0

5	Partly full	Inlet	Entrance geometry	Any			≤1.0
6	Full	Outlet	Entrance and barrel geometry	Any			≤1.0

Full details of the analysis of each flow type are presented by French (1985).

There are several practical factors associated with culvert design which may be of equal importance as the hydraulic analysis. Some of these aspects are discussed in the following.

3.5.2. Inlet Design

Full utilisation of the culvert cross-sectional area requires that it should run full or nearly so. This may not be possible especially in the case of low tailwater levels or steep gradients, leading to flow Type 5. Careful attention is then necessary in the inlet design to ensure minimum contraction of the flow and, hence, a maximum discharge coefficient. The objective is to ensure that the flow traverses the inlet section with a minimum of separation.

A variety of methods are available for improving the inlet conditions and these include a steep throat, a drop inlet, wingwalls, a hood and bevelled edges. The shape of the soffit is the most important and of the invert, the least important, because the flow at the invert is horizontal.

Some of these inlet improvements are illustrated schematically in Figure 6.3.9. The simplest improvement is a vertical headwall above the culvert entrance, thereby eliminating the re-entrant angle in the case of a battered embankment. The soffit of the inlet can be bevelled as shown in Figure 6.3.9(a). It is recommended that the bevel be at least 10% of the culvert height and at between 33° and 45° to the culvert axis (Portland Cement Association, 1964). This can increase the flow by up to 20%.

Full details and design charts and tables for improved culverts inlets may be found in a number of publications (e.g. Portland Cement Association (1964), U.S. Department of Transportation (1972)).

3.5.3. Outlet Design

In practice, culvert outlets have little significance in efficient culvert operation.

However, outlet structures have two important practical purposes:

a. to retain the embankment and support the end of the culvert.

b. to prevent damage by scour to the culvert, embankment, stream bed, or adjacent property.

Despite the common practice of making inlet and outlet structures identical, it should be noted that the two structures serve different purposes and, therefore, logically should be treated separately.

A number of different types of culvert outlets are shown in Figure 6.3.10.

The simple projecting outlet is sufficient when flow velocities are low and the filldoes not require special protection. The endwall structure alone acts to support the end of the culvert and as a retaining wall for the embankment. The wingwall helps to transition the culvert flow smoothly into the downstream channel and protects the endwall so that it may continue to function in its original capacity. A concrete apron serves to provide protection to the endwall structure by removing the point of potential erosion well away from the endwall foundation, thereby ensuring the stability of the structure.

Where wingwalls are used for bank protection and not merely as retaining walls, a concrete apron should always be provided. The absence of such an apron may encourage channelling and undercutting along the wingwall.

Where outlet velocities are particularly high, special energy dissipation structures may be required.

Scour at culvert outlets is not necessarily only due to concentrated flow issuing from the culvert barrel. Recirculating eddies, associated with a downstream channel which is significantly wider than the culvert, can cause potentially serious scour damage to the embankment fill.A further form of scour at culvert outlets is channel degradation which may occur if the culvert does not permit the passage of sediment from upstream.

Whatever the cause, the process of erosion may be associated with the excavated material being redeposited in the channel some distance downstream from the point of scour. It is entirely possible that with time, a shoal will form capable of causing excessively high tailwater depths during periods of high flow. Such a process should always be considered because high tailwater depths may not necessarily work to the advantage of culvert operation.

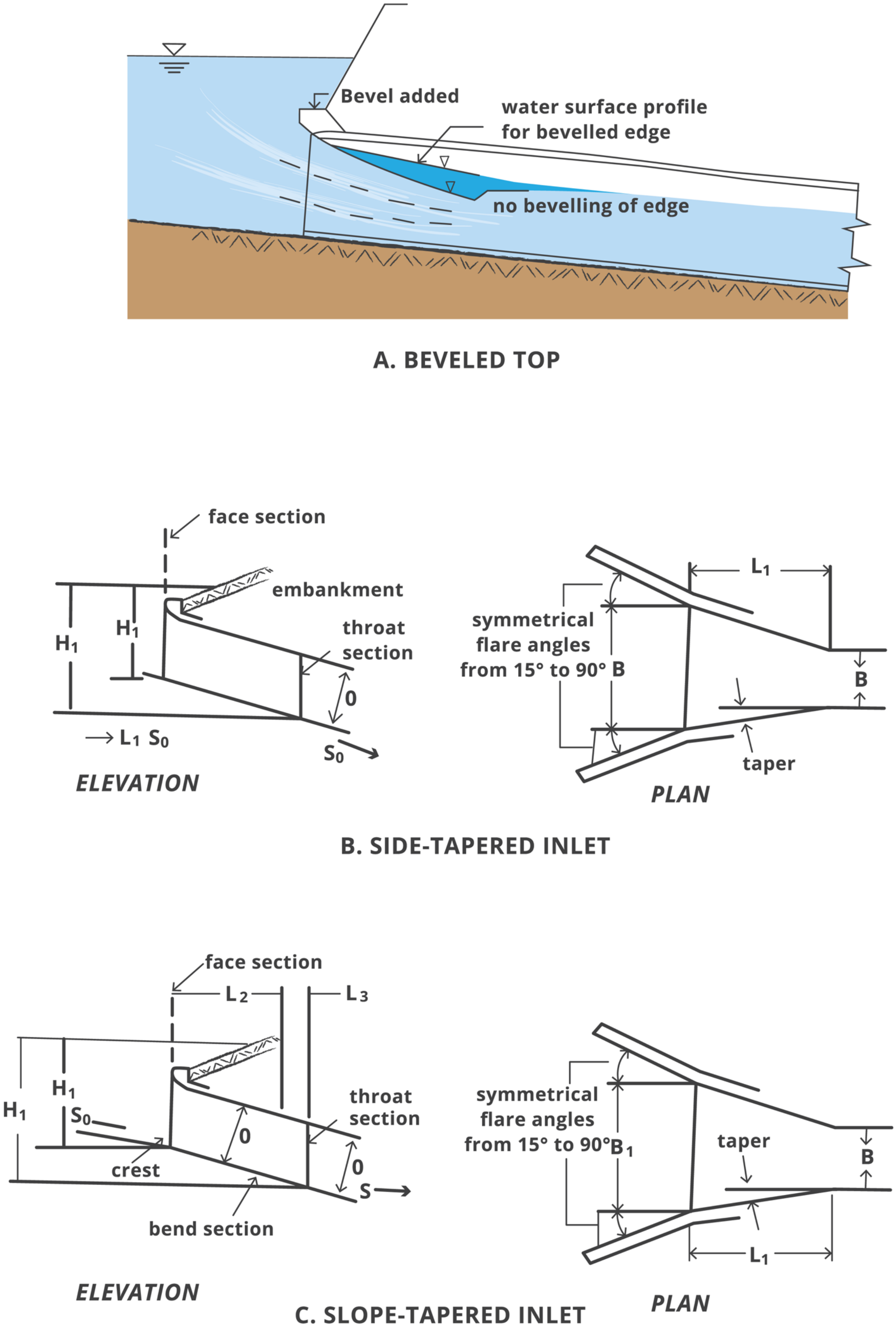


Figure 6.3.9. Schematics of Improved Inlets

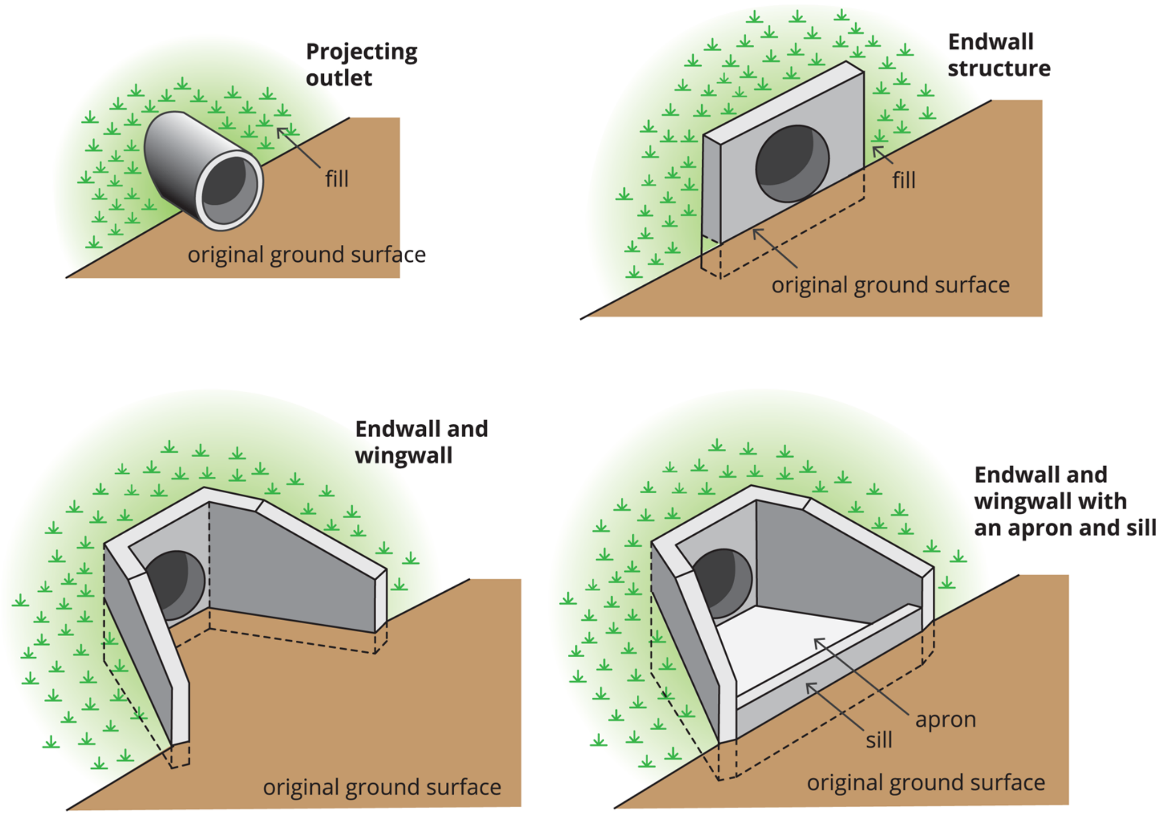


Figure 6.3.10. Examples of Culvert Outlets

Erosive velocities vary widely, depending upon the characteristics of the channel material, the depth of flow in the channel, and the velocity distribution. Erosive velocity limits for various types of soils are published in a number of texts (e.g. Portland Cement Association (1964)). Such published values should, however, be treated with caution because of the vast variations in naturally occurring materials.

Special problems occur in situations where relief culverts discharge directly on to an unchannelled flood plain. In this case, the tailwater level is likely to be significantly lower that the water level of the emerging supercritical jet. The erosive potential in this case is high and a schematic of the flow situation is shown in Figure 6.3.11.

It is apparent that the discharging jet will spread beyond the culvert and energy dissipation additional to bed friction will result from the interaction between the jet and the tailwater. The latter phenomenon is manifested in the zones of recirculation shown in Figure 6.3.11.

One method of analysis for this case has been proposed by Keller (1986). He drew a comparison between this phenomenon and that due to the interaction between shallow flood plain and deep main channel flows where the turbulent shear stresses are of the same order of magnitude as an equivalent wall shear stress if the interaction region were replaced by a solid wall. The present case can then be solved by assuming that the culvert outflow is contained within diverging vertical walls of roughness equal to that of the bed. A divergence angle of about 20° is indicated by the work of Rouse et al. (1951) and List and Imberger (1973). The modification of the outlet velocity with distance from the outlet can then be calculated using a water surface profile program such as HEC-RAS and appropriate invert protection determined.

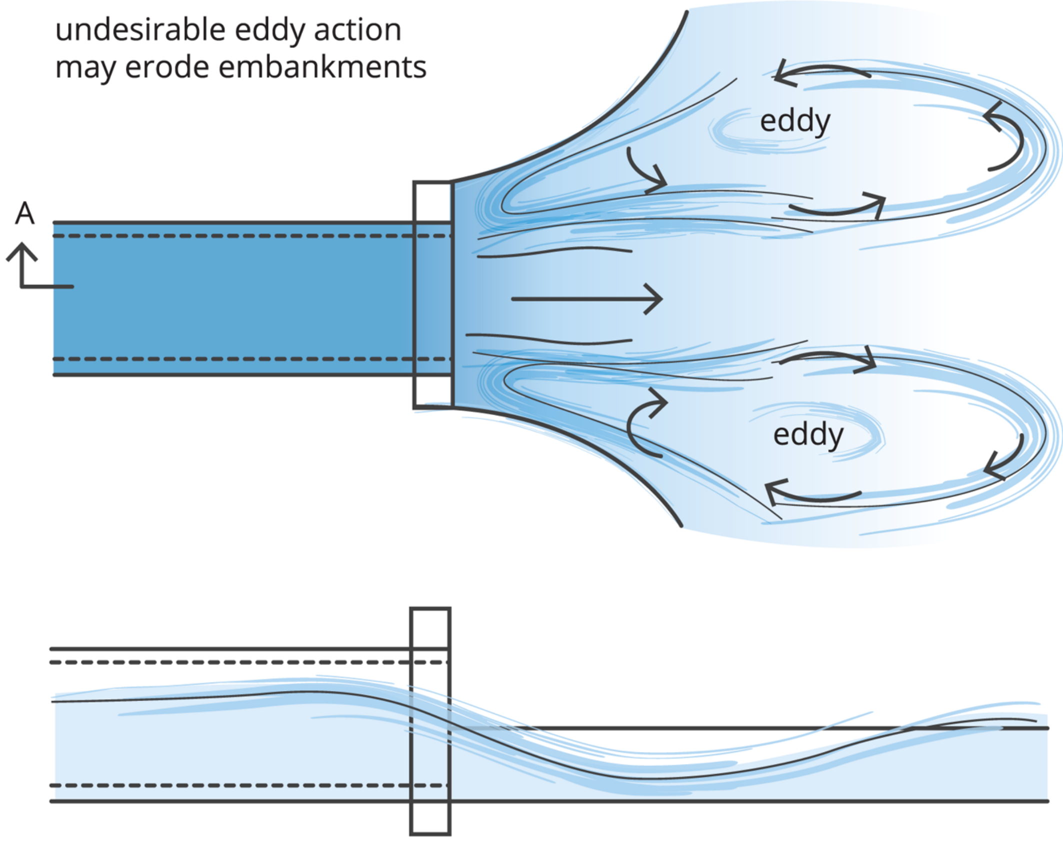


Figure 6.3.11. Schematic of Flow at Unsubmerged Outlet

Culvert Modeling Using HEC-RAS

HEC-RAS incorporates a module for the accurate design of culverts. Although the detail is outside the scope of this section, some comments are provided in the following.

Data for the culvert structure is simply entered on two templates in HEC-RAS – the Deck/ Roadway Editor for roadway information and the Culvert Data Editor for the physical data defining the culvert.

Although not required for culvert computations, the modeller may choose to enter embankment side slopes for the upstream and downstream embankment faces in the Deck/ Roadway Data Editor. The sloping embankment is used for graphical purposes only on the cross-section plots.

The primary information for inlet and outlet control analyses is entered in the Culvert Data Editor. For inlet control, these data are the inlet geometry with the corresponding chart and scale numbers. For outlet control computations, the entrance loss coefficient is required along with the Manning's n values for different portions of the culvert cross-section. A table is available to assist the modeller in choosing an appropriate entrance loss coefficient.

The exit loss coefficient defaults to the value 1, but the modeller has the option to adjust this parameter. A tail water elevation is not required because it is computed by HEC-RAS as part of the downstream water surface profile calculations.

3.6. Bridge Waterways

3.6.1. Introduction

Bridges are a necessary component of waterways that are crossed by roads and other embankment structures. For reasons of economy, bridges do not span the full width of a river, especially when it is in flood. The flow through bridges inevitably, then, involves energy losses that reflect in a higher water surface elevation upstream than would be the case if the waterway could flow freely.

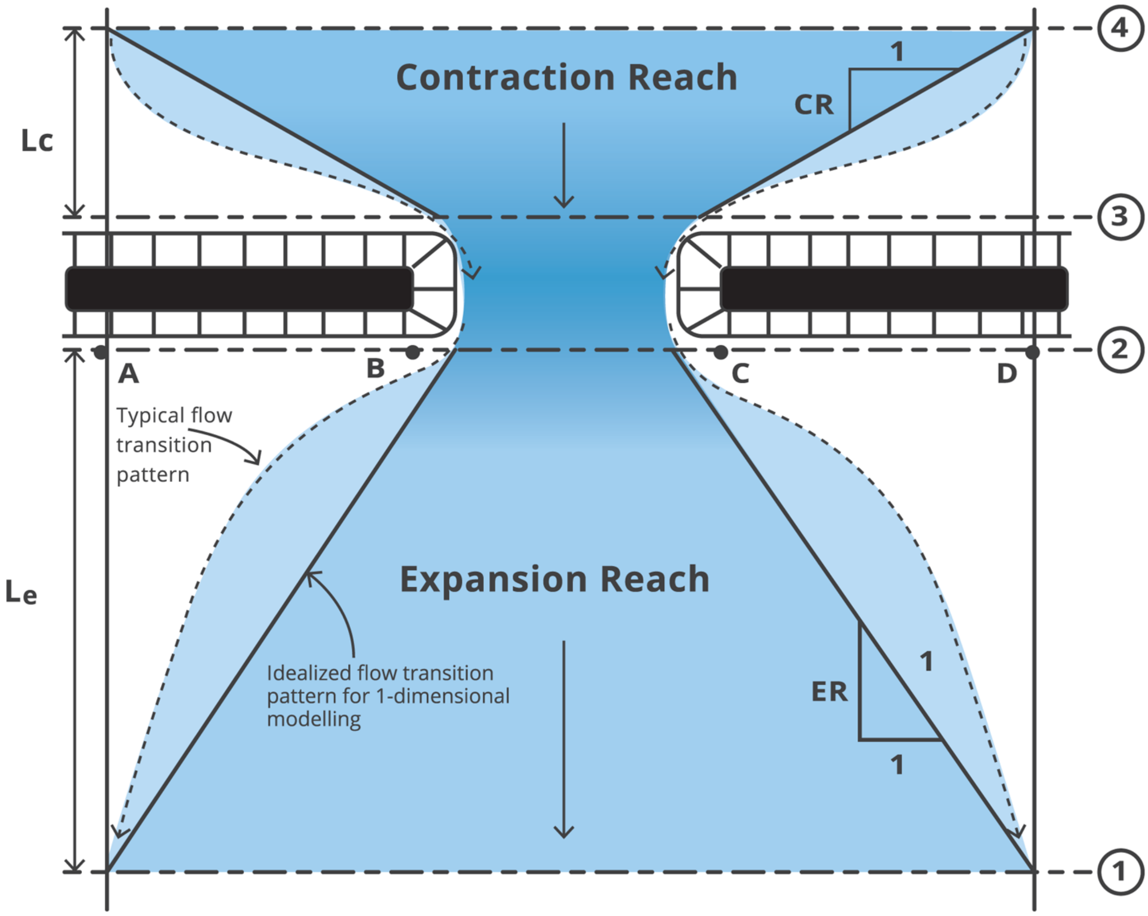
Despite the simple appearance of a bridge, its hydraulics is by no means simple. In addition to the potential for the constricted flow through a bridge site to cause flooding, there is a second issue of importance in the assessment of bridges and this is the issue of scour. Bridges continue to fail through scour of piers and/or abutments and it is vital to be able to determine the magnitude of this scour. Both the energy loss at a bridge site and the potential for scour are complex topics and special techniques have been developed to cope with their difficulties.

It should be noted that conservatism in the design of bridge waterways for their flooding potential requires an under-estimate of the magnitude of scour since this will minimise the size of the bridge opening, maximise the velocity through the bridge, and maximise the energy loss across the bridge. Conversely, conservatism in assessing the structural integrity of the bridge requires an over-estimate of the magnitude of scour. Thus, it is important to keep in mind the reason for undertaking the hydraulic analysis of the bridge site when assessing the results of such an analysis.

In this chapter, the hydraulics of flow through bridges is discussed, with special emphasis on energy losses and scour. A brief discussion is also presented on the bridge analysis routines contained within the HEC-RAS computer program.

3.6.2. Energy Losses at Bridges

Energy losses at bridge sites have three components. The first consists of losses that occur in the reach immediately downstream from the structure where an expansion of flow takes place. The second component comprises the losses that occur at the structure itself. The third component comprises the losses that occur in the reach immediately upstream of the structure where the flow is contracting to pass through the bridge opening. These three components are illustrated in Figure 6.3.12.



Bridge losses within the expansion reach downstream (sections 1 to 2) can be relatively large. The region is characterised by recirculation zones on either side that are maintained by extracting energy from the mean flow. Energy losses through the contraction (sections 3 to 4) are relatively smaller because the channel length over which the change in cross-section occurs is less and the recirculation zones are smaller.

Within each of these regions the energy loss is normally calculated as the sum of friction losses and expansion or contraction losses. Friction and contraction losses between sections 3 and 4 are calculated the same as friction and expansion losses between sections 1 and 2. Friction losses are typically determined using standard step profile equations. Contraction and expansion losses are described in terms of a coefficient times the absolute value of the change in velocity head between adjacent cross sections. For a detailed discussion on selecting contraction and expansion coefficients at bridges, the user is referred to Chapter 5 of the HEC-RAS Hydraulic Reference Manual.

Within the bridge structure itself (between sections 2 and 3) the computation of the energy loss can be simple or complex, depending on the flow characteristics. A low flow, where the water surface does not interact with the bottom chord of the bridge, may be analysed using a simple standard step procedure. On the other hand, interaction of the water surface with the bridge deck structure may lead to a combination of low flow and weir flow or pressure flow and weir flow. These cases require more complex modelling techniques that are outside the scope of this section. Modelling details may be found in the HEC-RAS Hydraulic Reference Manual.

In most cases of flow through bridge sites, the river flow downstream and upstream is sub-critical. Under these circumstances, the hydraulic effect of the bridge is to increase the water level upstream of the bridge, with no effect downstream. On the other hand, in the rare cases where the natural flow is characterised by super-critical conditions, the effect of the bridge manifests downstream. In such cases, upstream water levels are affected only if the bridge constriction is sufficiently severe to transform the supercritical flow to subcritical.

3.6.3. Modelling Approaches for Non-Standard Bridge Crossings

Non-standard bridge crossings include perched bridges, submerged bridges, skewed bridges, parallel bridges and multiple opening bridges. Notes on the modelling of each are presented in the following.

A perched bridge is one for which the road approaching the bridge is at the flood plain ground level, and only in the immediate area of the bridge does the road rise above ground level to span the water course. This condition is shown schematically in Figure 6.3.13.

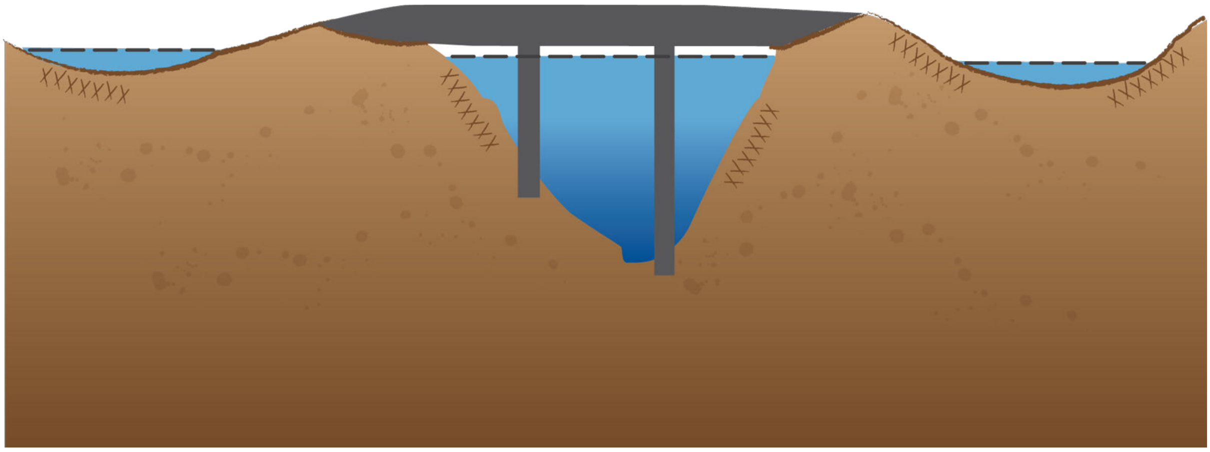


Figure 6.3.13. Schematic of a Perched Bridge

A typical flow situation with this type of bridge is low flow under the bridge and overbank flow around the bridge. Because the road approaching the bridge is usually not much higher than the surrounding ground, the assumption of weir flow is usually not justified.

For this reason, perched bridges should generally be modelled using the energy-based method, especially when a large percentage of the total flow rate is carried in the overbank areas.

A submerged bridge (or low water bridge) is designed to accommodate only low flows under the bridge. Flood flows are carried over the bridge and road. A typical example of a submerged bridge is shown in Figure 6.3.14.

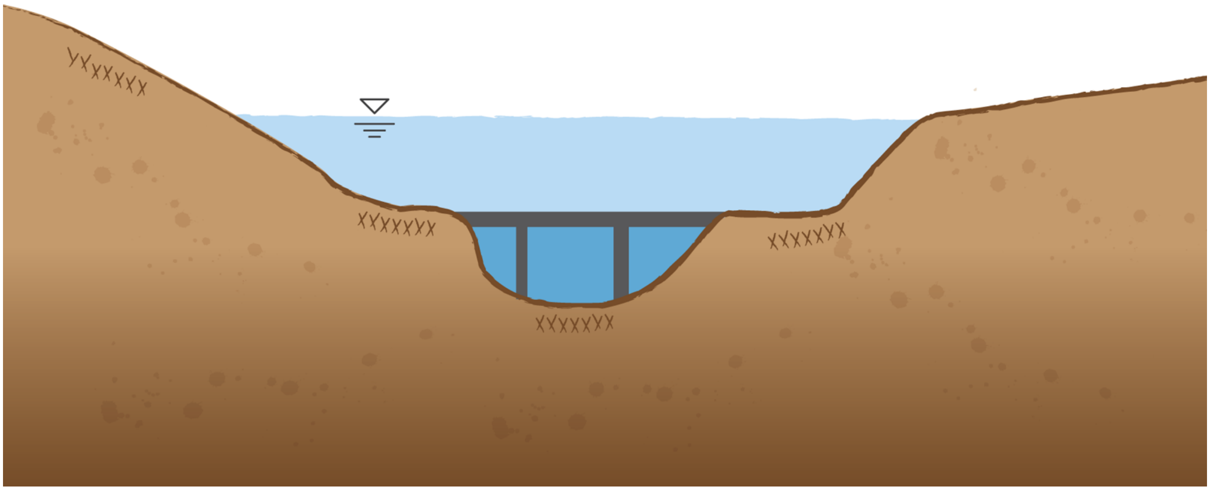


Figure 6.3.14. Schematic of a Submerged Bridge

When modelling this bridge for flood flows, the anticipated solution would be a combination of pressure and weir flow. However, with most of the flow passing over the top of the bridge, the correction for submergence can introduce considerable error. For this reason, if the tailwater level is likely to be relatively high, the energy based method of analysis is recommended.

A schematic of a skewed bridge is shown in Figure 6.3.15.

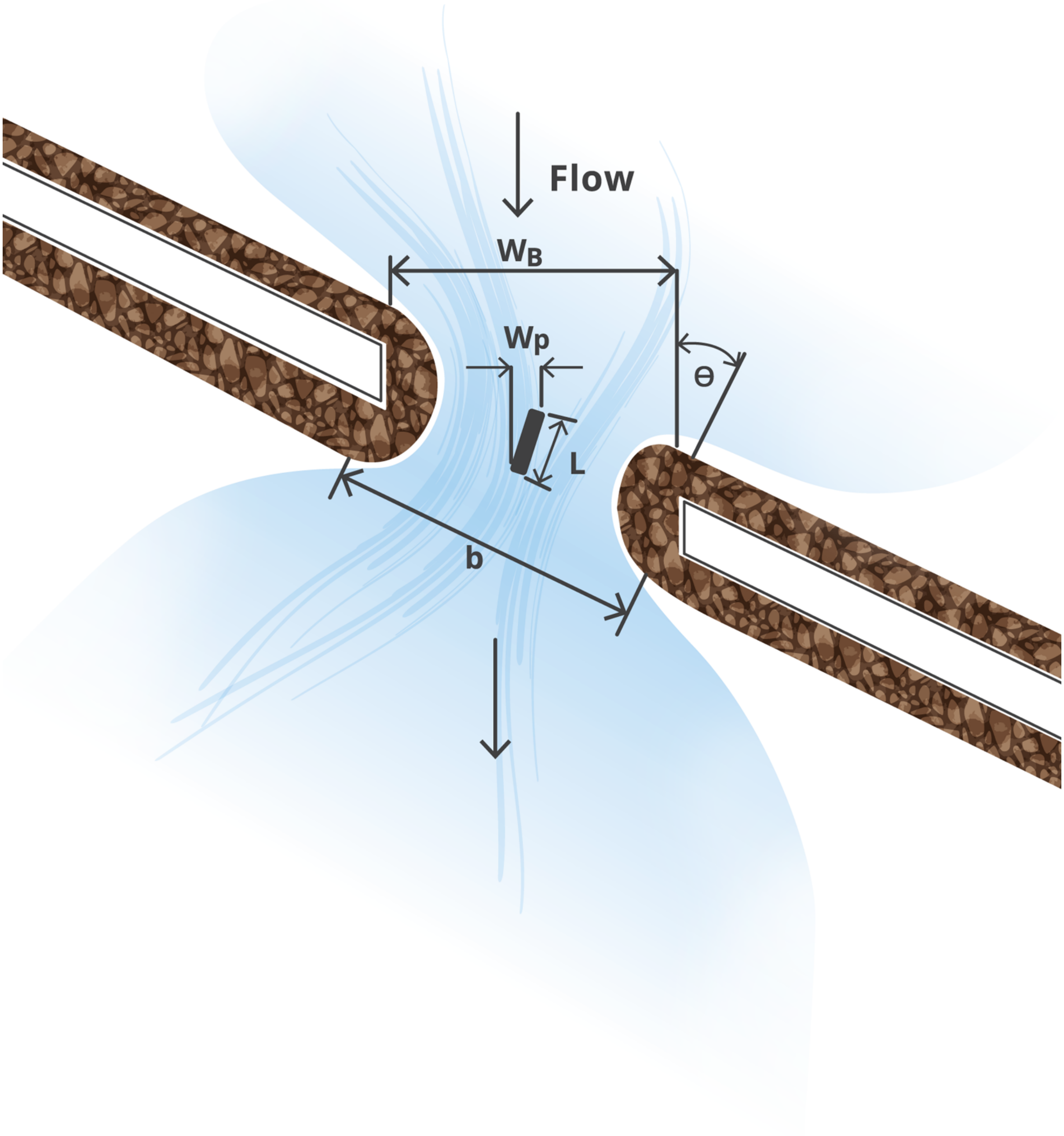


Figure 6.3.15. Schematic of a Skewed Bridge

A skewed bridge crossing is generally handled by making adjustments to the bridge dimensions to define an equivalent cross-section perpendicular to the flow lines.

For low flow, skewed crossings with angles up to 20 degrees show no objectionable flow patterns. However, for larger angles of skew, the flow efficiency decreases. For reasonably small flow contractions, the projected length is adequate for assessing the impact of skew up to skew angles of 30 degrees.

With reference to Figure 6.3.15, the projected width of the bridge opening, perpendicular to the flow lines, is given by:

$$W_B = b\cos\theta \tag{6.3.5}$$

The pier information must also be adjusted to account for the skew of the bridge. The program HEC-RAS assumes that the piers are continuous, as shown in Figure 6.3.15.

Thus, the projected width of the piers, perpendicular to the flow lines, is given by:

$$W_P = L\sin\theta + w_p\cos\theta \tag{6.3.6}$$

The construction of divided highways often leads to the common modelling problem of parallel bridges. The situation is shown schematically in Figure 6.3.16.

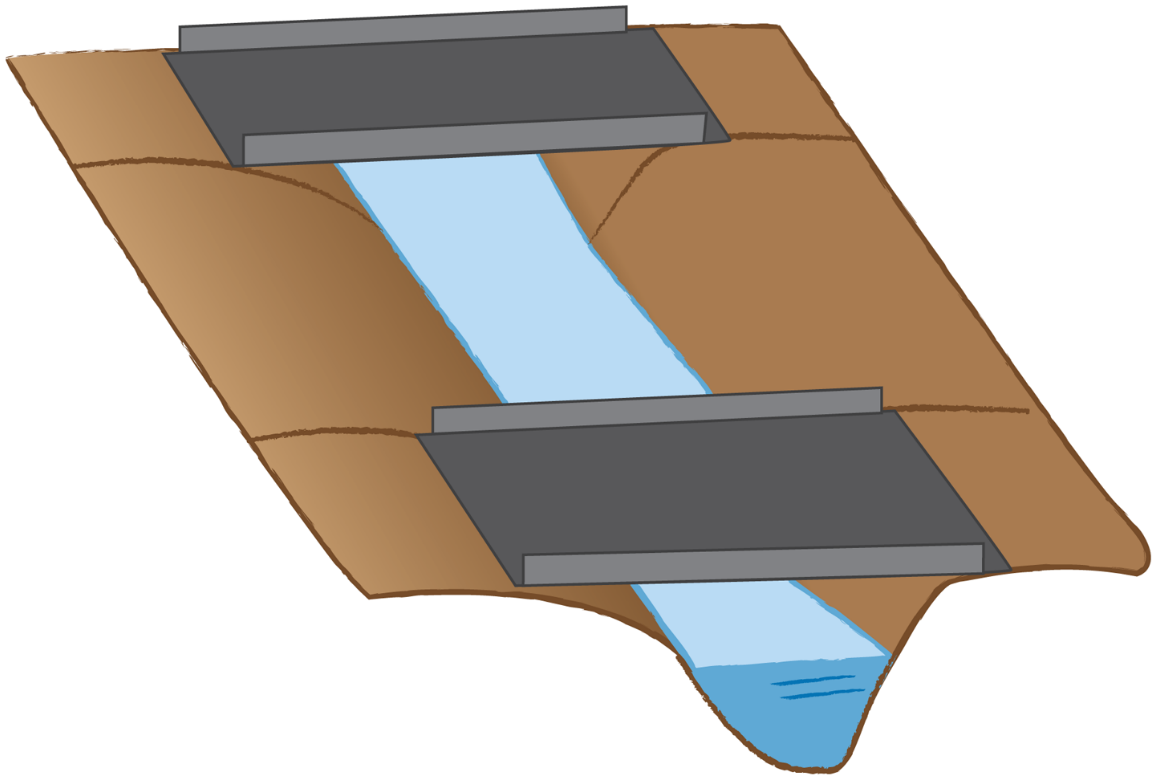


Figure 6.3.16. Schematic of Parallel Bridges

For new highways, these bridges are often identical structures.

Depending on the spacing between the two bridges, the loss may be between 1.3 and 2 times the loss for a single bridge. If the bridges are very close to each other and the flow cannot expand between the bridges, the system can be dealt with as a single structure. If both bridges are modelled, care should be taken in depicting the expansion and contraction of flow between the bridges. Expansion and contraction rates should be based on the same procedures as single bridges.

Some bridges are characterised by more than one opening for flood flow, especially over a very wide flood plain. Figure 6.3.17 shows the situation schematically and illustrates the nomenclature used in the following discussion.

It is necessary to ensure compatibility between the determination of individual flow rates through each opening and the equality of the head loss along each flow path.

With reference to Figure 6.3.17, this requires that:

$$Q_T = \sum Q_i \tag{6.3.7}$$

and

$$H_4 - H_1 = \Delta H_{Q_1} = \Delta H_{Q_2} = \Delta H_{Q_3} \tag{6.3.8}$$

Mutual satisfaction of Equation (6.3.7) and Equation (6.3.8) ensures that the computed energies at the upstream point where flow separates or stagnation point are equal – this is defined by the correct apportioning of flow QI through each opening.

The downstream stagnation point defines where flow merges and the flow path of all the openings are assumed to have equal energy level at this point, ie. downstream boundary.

In HEC-RAS, Up to seven openings (of combinations of open conveyance area, bridges and culverts) can be defined at any one river crossing. The program automatically locates the stagnation points within the range defined by the user unless there are physical stagnation points such as bridge abutments or islands for example.

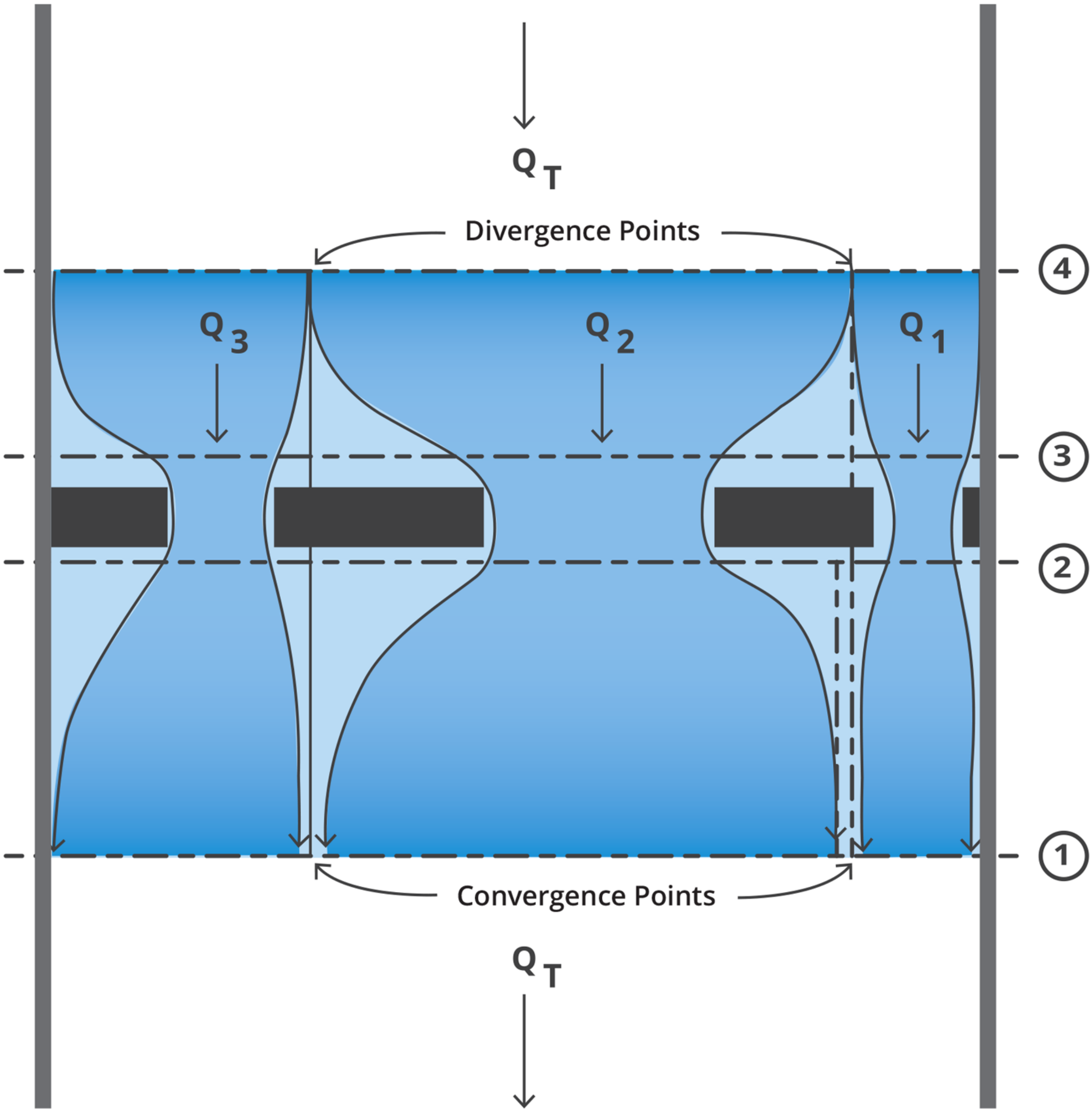


Figure 6.3.17. Schematic of Bridge with Multiple Openings

Alternatively, the divided flow approach can be used whereby the flow paths of each opening are modelled separately with manual adjustment of flow distribution (similar to flow split modelling at upstream of opening and flow combining modelling at downstream of opening).

With this method, the cross section will need to be physically split along assumed stagnation points at each cross sections using the ineffective area option. This method is most suited to anabranches and breakaway flow paths in wide floodplain with multiple road crossings. A typical example is shown in Figure 6.3.18.

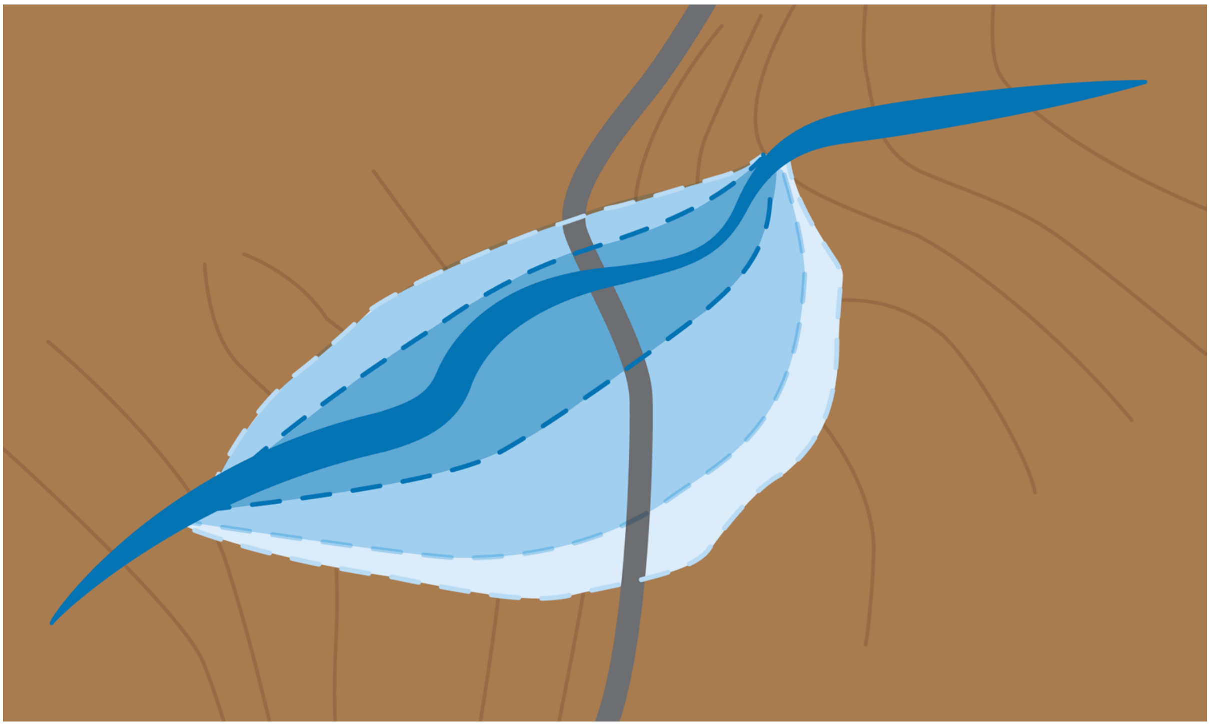


Figure 6.3.18. Illustration of Divided Flow Approach

Scour at Bridge Sites

Scour occurs at bridges because of changes to the natural flow conditions and is a serious concern. It can be defined simply as the excavation and removal of material from the bed and banks of streams as a result of the erosive action of flowing water. In the context of this book, it is assumed that this erosive action may potentially expose the foundations of a bridge. Scour is usually considered to be a local phenomenon, but includes degradation that can cause erosion over a considerable length of a river.

Scour at bridges is described in Book 6, Chapter 3, Section 8.

3.7. Floodways on Roads

3.7.1. Introduction

Floodways are sections of roads that have been designed to be overtopped by floodwater during relatively high annual exceedance probability flood events. These events could be as high as AEP 10% to 5%, but are often designed for overtopping in small floods.

Floodways are therefore planned for locations where flood immunity is not a serious concern or the duration of road closure is low, and are suitable for locations where extensive floodplain width and shallow flows make bridge or culvert construction difficult or expensive. Floodways are often a preferred approach for locations where a relatively cheap floodplain crossing is needed and where flood immunity or flood closures are not a significant concern. They are therefore often preferred for roads with low traffic volumes in arid regions where flood events are infrequent and short duration.

The Queensland Department of Transport and Main Roads Road Drainage Manual (QDTMR, 1986) and the Austroads Guide to Road Design Part 5 (Austroads, 2013) has detailed guidance on floodway design.

Floodways may require costly batter protection and therefore a higher level road together with a larger culvert or bridge option may be more cost effective. Floodways also have smaller waterway (under road) requirements and may be more prone to blockage by debris. These cost related performance factors should be considered as well as trafficability and other requirements in the selection of final road level. Floodways may offer environmental advantages over culverts or bridges, since they will tend to spread flows more widely. This means that the risk of scour to waterway and surrounding land is generally reduced because flow is less concentrated. It is also important that a floodway be designed so that it is not covered by water from ponding or backwater for any significant period of time after a flood event.

The advantages of floodways are as follows:

- Generally, simple to design;
- May offer environmental advantages over culverts and bridges, since they will tend to spread flows more widely, reducing the risk of scour when flow is concentrated in culverts or bridges;
- Typically have low embankments; and
- Risk of scour to waterway and surrounding land is reduced.

There are however some disadvantages, as follows, which mean that design needs careful consideration.

- Allow water flow over road which leads to flood immunity and safety issues;
- Increased disruption to traffic due to overtopping;
- Can have higher construction costs than culverts;
- Batter slopes can be affected by erosion or scour (particularly for higher embankments);
- Generally have costly batter protection requirements;
- Susceptible to stream / channel migration;
- Can have environmental impacts (fauna / fish passage); and
- Potential for failure of embankment (depending on provided protection).

Geometric and Safety Issues with Floodways

It is important that adequate approach sight distance be provided to allow drivers time to recognise water over the road and to stop. It is also important that the length of a floodway be limited at about 300 m so that drivers do not become disorientated when confronted with wide open stretches of water. Where a proposed floodway would be longer than 300 m, it is recommended that the proposed floodway be broken into shorter lengths by providing sections of road that are raised above the maximum flood level. As a general principle, floodways should be designed so that the depth of water over the road should be as uniform as possible over the flooded section. Building a floodway on a level grade avoids the possibility of a driver unexpectedly encountering deeper water and possibly stalling or being swept downstream.

Exceptions to the level grading may occur where bridges have been built significantly higher than the flooded approaches on both sides. The bridges have been built on the basis that the approaches will be raised sometime in the future. Floodways should not be placed on horizontal curves as:

- there are problems in defining the edge of the pavement for motorists;
- any superelevation may change the normal flow distribution ie. push more water to the non-superelevated sections of road; and
- the water depth will be deeper on one side of the road than the other in a superelevated section of road and there is the possibility of the high side being trafficable but not the other, thus creating a safety problem.

Floodways should also not be located on vertical curves to avoid variations in flow depth.

Hydraulic Design

A floodway consists not only of the roadway embankment to accommodate flow over the road but also waterway openings to provide for flow under the road. These openings may be required for one or more of the following functions:

- reduce the afflux or rise in water level upstream due to the obstruction (embankment);
- raise the tailwater level so that less batter protection is required on the downstream side e.g. grass instead of concrete; and
- act as anti-ponding structures for low flow stream conditions.

Flow over roadways may be:

- free flow; and
- submerged flow.

In the initial stages of overtopping a low tailwater usually exists and free flow occurs. Under these circumstances flow passes through critical depth over the road and the discharge is determined by flood levels upstream.

Free flowmay be either:

- plunging flow which flows over the shoulder and down the downstream face of the embankment. The flow then penetrates the tailwater surface producing a submerged hydraulic jump on the downstream slope. Velocities are likely to be high and erosive; and
- surface flow which separates from the surface of the road embankment and rides over the surface of the tailwater. This flow will have less erosion potential downstream.

Submerged flowoccurs when the discharge is controlled by the tailwater level as well as the headwater levels. This occurs when the depth of flow over the road is everywhere greater than the critical depth. Typical velocities of flow over a floodway are shown in the Figure 6.3.19 as sourced from *Waterway Design* (Austroads 1994) after *Cameron and McNamara* (1966)Cameron and McNamara (1966).

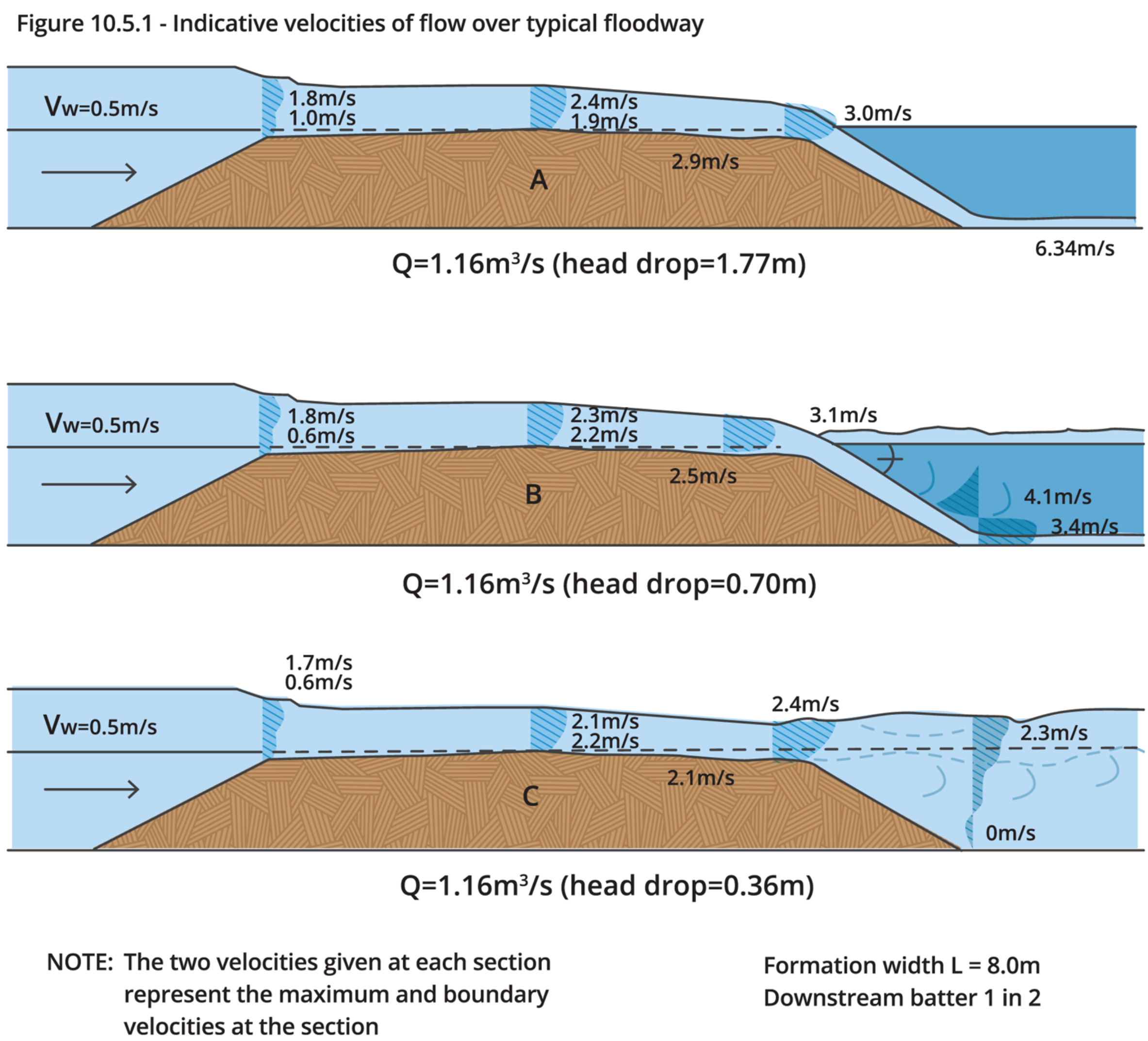


Figure 6.3.19. Floodway flows (DTMR)

Flow over the road

Hydraulic calculation for flow over the road embankment is based on the broad crested weir formula as described elsewhere in this Chapter.

With free-flow conditions on the embankment – that is, in the absence of submergence of the control – analysis indicates that the discharge across the embankment may be expressed in terms of the other relevant parameters by an expression of the form:

$$Q = CLH^{\frac{3}{2}} \qquad (6.3.9)$$

where Q = the discharge

C = a coefficient

L = the length of the embankment (that is, the width of the flow)

H = the head of the approach flow, determined as shown in Figure 6.3.19 with the elevation of the embankment crown as datum.

The coefficient C embodies numerical coefficients and the gravitational acceleration. For an ideal fluid, the value of C would be 1.70 in SI units. Values of C for real fluids would generally be expected to be somewhat less than this value, incorporating empirically the effects of differences between real and ideal fluid behaviour.

3.8. Scour

3.8.1. Scour at Bridges

Scour at bridges is an important risk for these structures and design must incorporate mitigation measures. In the case of existing bridges, where scour becomes apparent, measures must be provided to protect the bridge asset and prevent further damage. Scour is a very serious problem. Floods that result in scour are the principal cause of bridge failure.

Some of the observable effects of scour are shown in Figure 6.3.20.

Figure 6.3.20(a) shows the pier caps and pile caps exposed. Figure 6.3.20(b) shows pier and abutment riprap moved downstream. Figure 6.3.20(c) shows a downstream scour hole and bank erosion. Figure 6.3.20(d) shows a downstream scour hole arising from submergence of the opening. Figure 6.3.20(e) shows slumped material at the toe of the bank arising from failure of the riprap or bank. Figure 6.3.20(f) shows erosion and failure of a highway embankment with flow on both sides of the abutment.

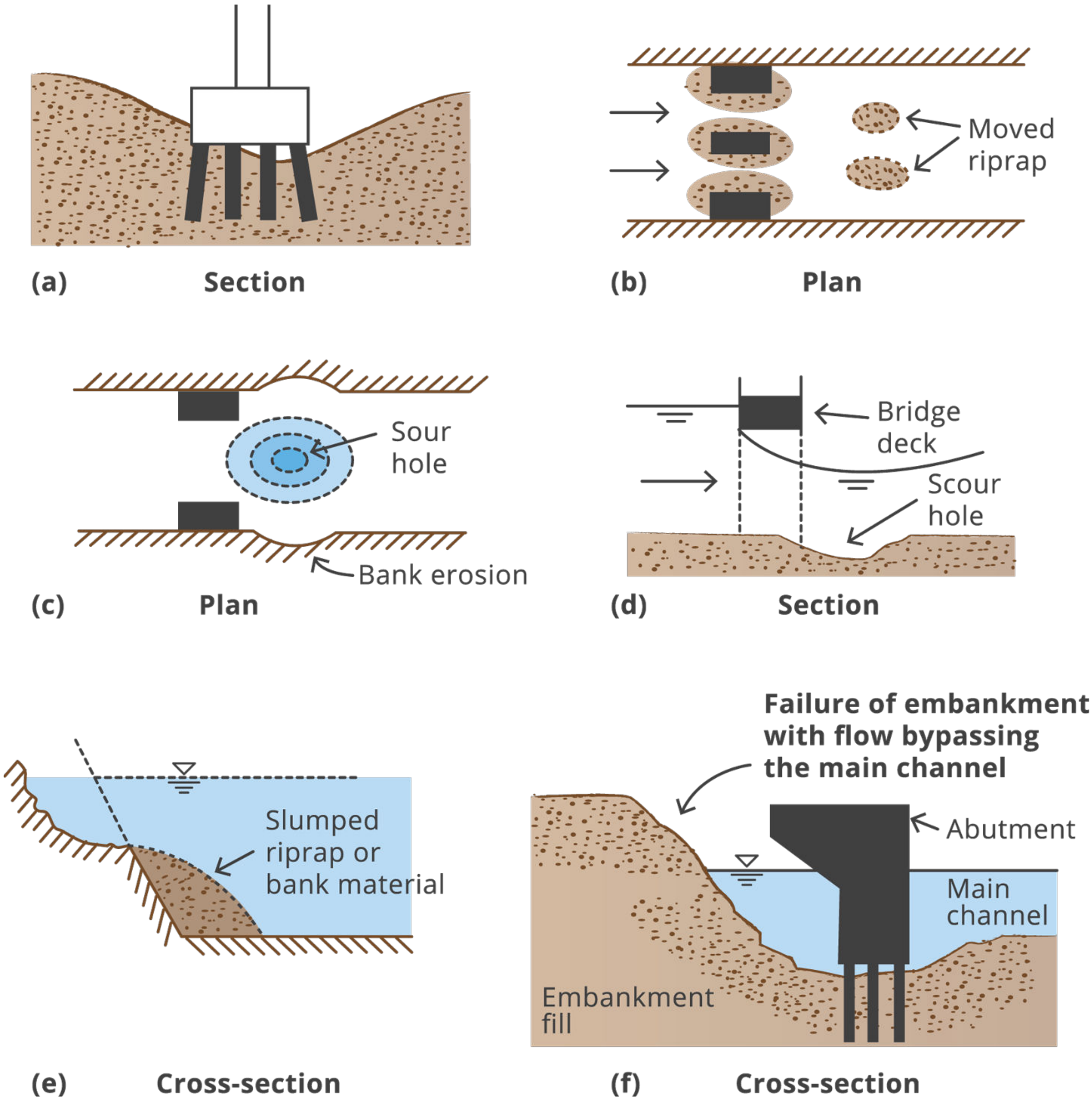


Figure 6.3.20. Some Effects of Scour

The biggest and most frequently encountered scour-related problems usually concern loose sediments that are easily eroded. It is not true, however, to assume that the scour depth in cohesive or cemented soils cannot be as large – it merely takes longer for the scour hole to develop.

Many of the equations for scour were derived from laboratory studies, for which the range of validity is unknown. Some were verified using very limited field data, which itself may be of doubtful accuracy. In the field, the scour hole that develops on the rising stage of a flood, or at the peak, may be filled in again on the falling stage. For this reason, the maximum depth of scour cannot be easily assessed after the event.

Scour can also cause problems with the hydraulic analysis of a bridge. Scour may considerably deepen the channel through a bridge and effectively reduce or even eliminate the backwater. This reduction in backwater should not be relied on, however, because of the unpredictable nature of the processes involved.

When considering scour it is normal to distinguish between non-cohesive or cohesionless (alluvial) sediments and cohesive material. The former are usually of most interest and are considered further in this section. Cohesive materials require special techniques and are outside the scope of this chapter.

The first major issue when considering scour is the distinction between *clear-water* scour and *live-bed* scour. The critical issue here is whether or not the mean bed shear stress of the flow upstream of the bridge is less than or larger than the threshold value needed to move the bed material.

If the upstream shear stress is less than the threshold value, the bed material upstream of the bridge is at rest. This is referred to as the clear-water condition because the approach flow is clear and does not contain sediment. Thus, any bed material that is removed from a local scour hole is not replaced by sediment being transported by the approach flow. The maximum local scour depth is achieved when the size of the scour hole results in a local reduction in shear stress to the critical value such that the flow can no longer remove bed material from the scoured area.

Live-bed scour occurs where the upstream shear stress is greater than the threshold value and the bed material upstream of the crossing is moving. This means that the approach flow continuously transports sediment into a local scour hole. By itself, a live bed in a uniform channel will not cause a scour hole - for this to be created some additional increase in shear stress is needed, such as that caused by a contraction (natural or artificial, such as a bridge) or a local obstruction (e.g. a bridge pier). The equilibrium scour depth is achieved when material is transported into the scour hole at the same rate at which it is transported out.

These concepts are illustrated by Figure 6.3.21.

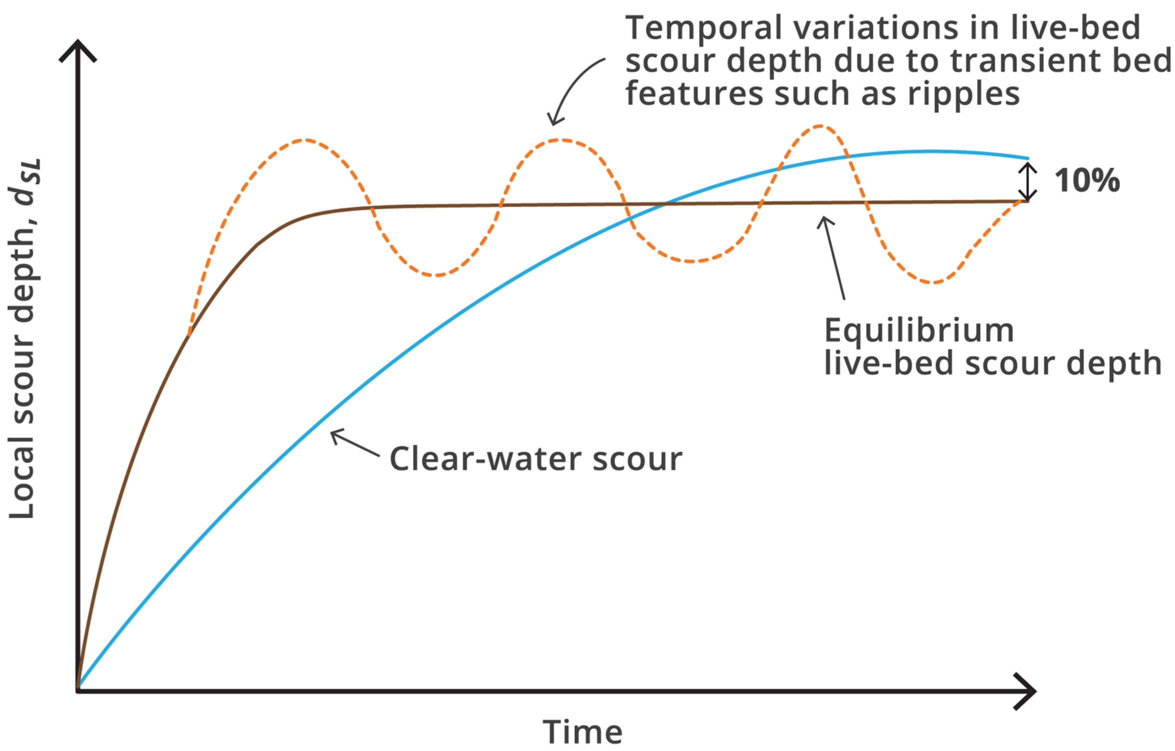


Figure 6.3.21. Development of Clear-water and Live-bed scour with Time

It is noted from Figure 6.3.21 that typically the maximum equilibrium clear-water scour is about 10% larger than the equilibrium live-bed scour. Conditions that favour clear water scour are:

- Channels with flat bed slopes during low flows;
- A coarse bed material that is too large to be transported (e.g. riprap);
- Channels with natural vegetation or artificial reinforcement where velocities are only high enough to cause scour near piers and abutments; and
- Flow over grassed floodplains.

At any particular location both clear-water and live-bed scour may be experienced. During a single flood the bed shear stress will increase and decrease as the discharge rises and falls. Thus, it is possible to have clear-water conditions initially, then a live bed, then finally clear water again. The maximum scour depth may occur under clear-water conditions, not at the flood peak when live-bed scour is experienced. Similarly, relatively high velocities can be experienced when the flow is just contained within the banks, rather than spread over the floodplains at the peak discharge.

It is also possible to have the clear-water and live-bed conditions occurring at the same time. For example, if the floodplains are grassed or composed of material that is larger in diameter than that in the main channel, clear-water conditions may occur on the floodplain with live-bed conditions in the main channel.

It is evident from this discussion that the problem may not always be as simple or as well defined as would be desirable. If there are any uncertainties or if the consequences of failure are large, prompting a conservative approach, it is recommended that clear-water conditions be assumed at the peak flow condition.

Urbanisation has the effect of increasing flood magnitudes and causing hydrographs to peak earlier, resulting in higher stream velocities and degradation. Channel improvements or the extraction of gravel (above or below the site in question) can alter water levels, flow velocities, bed slopes and sediment transport characteristics and consequently affect scour. For instance, if an alluvial channel is straightened, widened or altered in any other way that results in an increased flow-energy condition, the channel will tend back towards a lower energy state by degrading upstream, widening and aggrading downstream.

The significance of degradation scour to bridge design is that the engineer has to decide whether the existing channel elevation is likely to be constant over the 100 year life of the bridge, or whether it will change. If change is probable then it must be allowed for when designing the waterway and foundations.

The lateral stability of a river channel may also affect scour depths, because movement of the channel may result in the bridge being incorrectly positioned or aligned with respect to the approach flow. This problem can be significant under any circumstances but is potentially very serious in arid or semi-arid regions and with ephemeral (intermittent) streams. Lateral migration rates are largely unpredictable. Sometimes a channel that has been stable for many years may suddenly start to move, but significant influences are floods, bank material, vegetation of the banks and floodplains, and land use.

Scour at bridge sites is typically classified as contraction (or constriction) scour and local scour. Contraction scour occurs over a whole cross-section as a result of the increased velocities and bed shear stresses arising from a narrowing of the channel by a constriction such as a bridge. In general, the smaller the opening ratio ($M = q/Q$ or b/B) the larger the waterway velocity and the greater the potential for scour. If the flow contracts from a wide floodplain, considerable scour and bank failure can occur. Relatively severe constrictions may require regular maintenance for decades to combat erosion. It is evident that one way to reduce contraction scour is to make the opening wider.

Contraction scour is caused by a constriction in the floodplain either by a bridge or when overbank flow is confined by road embankments. A decrease in flow area results in an increase in average velocity and bed shear stress. Contraction scour is different from long-term degradation in that contraction scour occurs in the vicinity of the constriction (bridge), it may be intermittent, and/or related to the passing of a particular flood event.

Contraction scour at a bridge is illustrated in Figure 6.3.22.

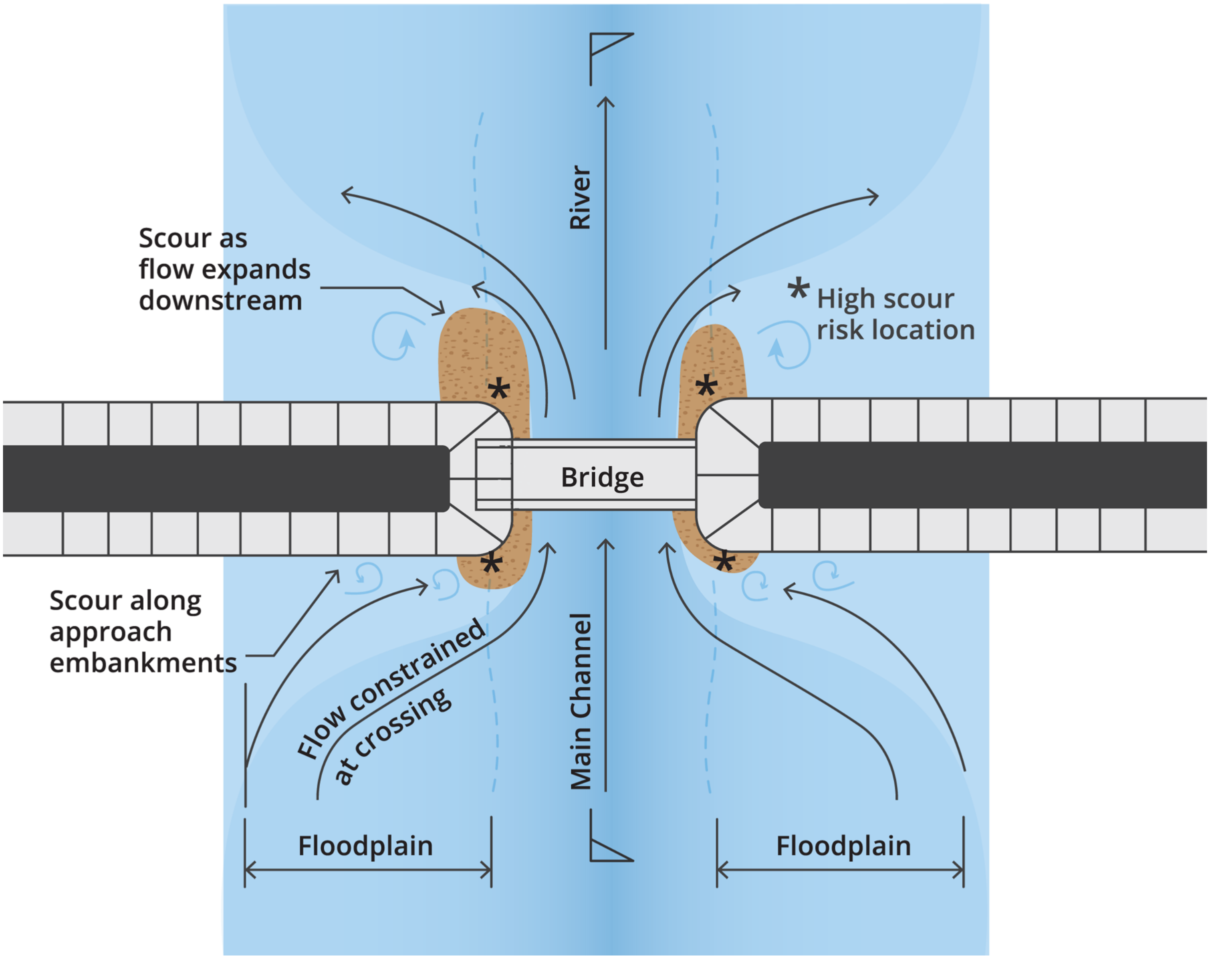


Figure 6.3.22. Contraction scour (QDTMR, 2013)

Contraction scour also occurs in the vertical where flow is contracted vertically as water flows under the bridge and velocity increases, potentially causing a scour hole to develop under the bridge, as shown in Figure 6.3.23.

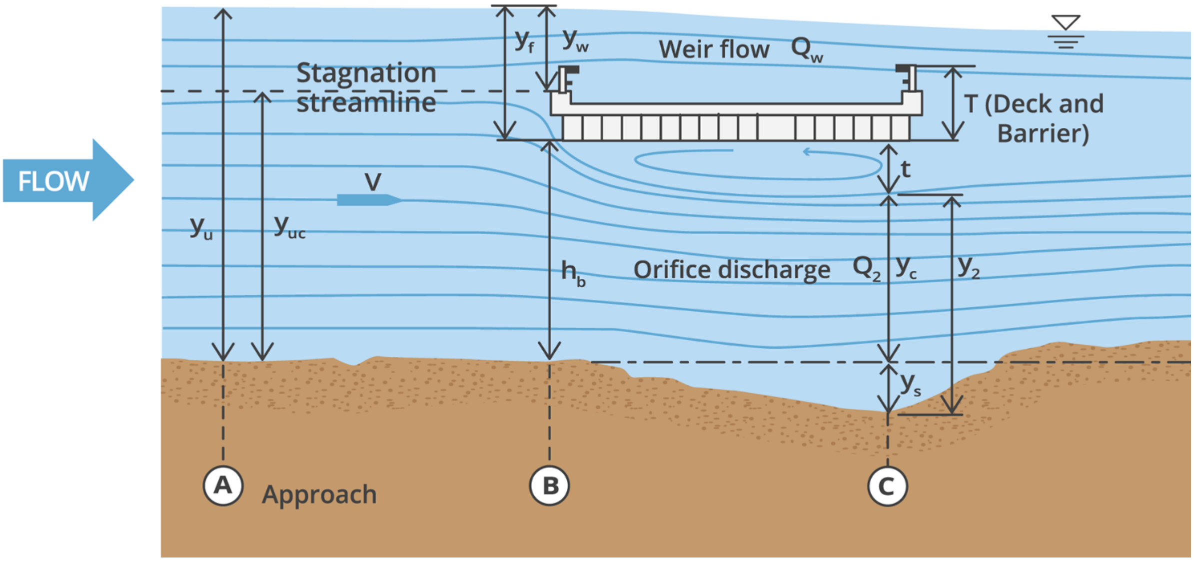


Figure 6.3.23. Vertical contraction scour (QDTMR, 2013)

Local scour arises from the increased velocities and associated vortices as water accelerates around the corners of abutments, piers and spur dykes. The flow pattern around a cylindrical pier is shown in Figure 6.3.24. The approaching flow decelerates as it nears the cylinder, coming to rest at the centre of the pier. The resulting stagnation pressure is highest near the water surface where the approach velocity is greatest, and smaller lower down. The downward pressure gradient at the pier face directs the flow downwards. Local pier scour begins when the downflow velocity near the stagnation point is strong enough to overcome the resistance to motion of the bed particles.

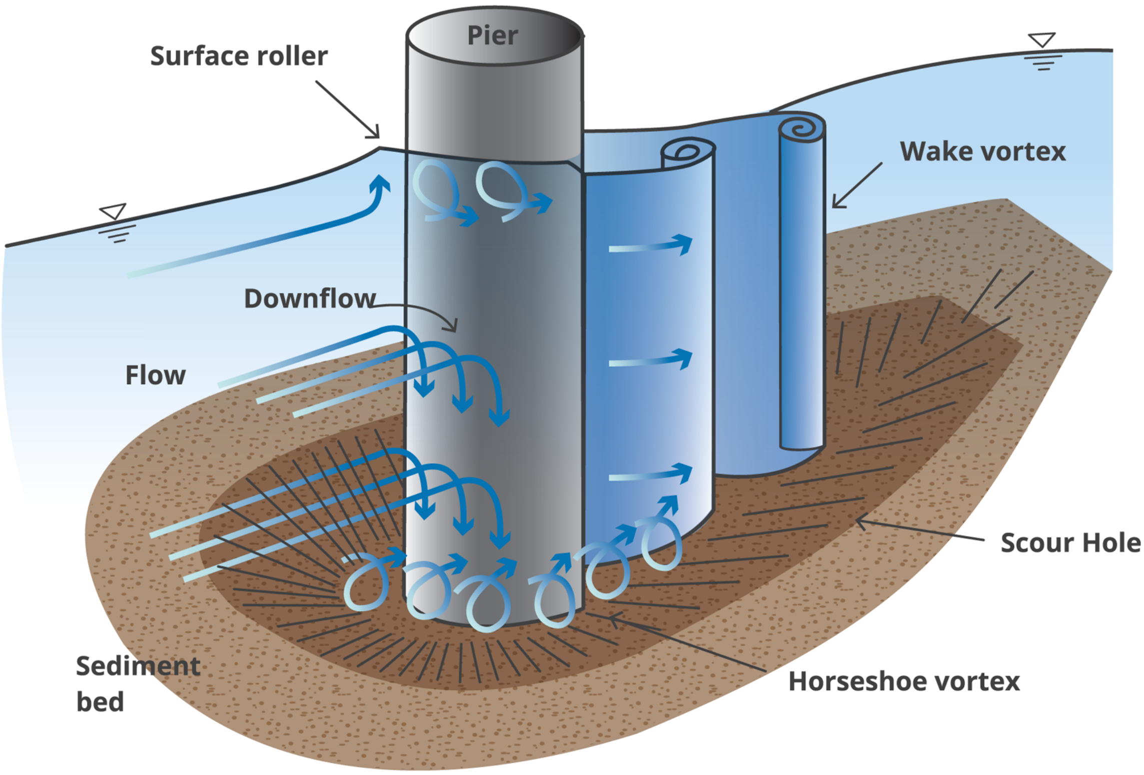


Figure 6.3.24. Schematic of Local Scour at a Bridge Pier

When scour occurs the maximum downflow velocity is about 80% of the mean approach velocity. The impact of the downflow on the bed is the principal factor leading to the creation of a scour hole. As the hole grows the flow dives down and around the pier producing a horseshoe vortex, which carries the scoured bed material downstream.

The combination of the downflow with the horseshoe vortex is the dominant scour mechanism. As the scour hole becomes progressively deeper the downflow near the bottom of the scour hole decreases until at some point in time equilibrium is reached and the depth remains constant.

At the sides of the pier flow separation occurs, resulting in a wake vortex whose whirlpool action sucks up sediment from the bed. As the vortices diminish and velocities reduce, the scoured material is deposited some distance downstream of the pier.

For piers that are essentially rectangular in plan and aligned to the flow the basic scour mechanism is similar to that just described, although rather more severe because of the square corners. However, as the angle of attack to a rectangular pier increases, so does its effective width, so the scour depth increases and the point of maximum scour moves downstream of the nose to a point on the exposed side.

With a large degree of skew the maximum scour may occur at the downstream end of the pier. If the flow direction is likely to change there is merit in using cylindrical piers to avoid these complications.

The scour mechanism at a bridge abutment is similar to that at a pier, although the boundary layer at the abutment or channel wall may result in an additional deceleration of the flow compared with a central pier. The approach flow can be considered as separating into an upper layer, which forms an upflow surface roller on hitting the abutment, and a lower layer, which becomes the bottom or principal vortex. This is shown schematically in Figure 6.3.25.

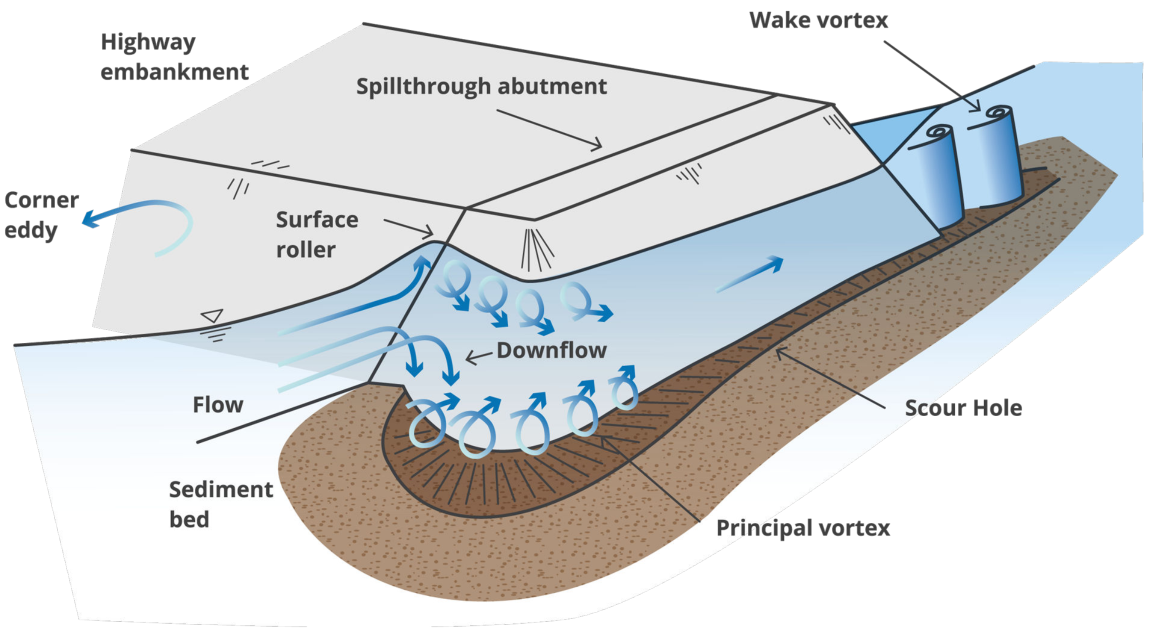


Figure 6.3.25. Schematic of Abutment Scour

Viewed in plan, the upper layer divides or separates, with part of the flow accelerating around the upstream corner of the abutment into the bridge waterway while the remainder slowly rotates in an almost stationary pool trapped against the face of the abutment and the river bank.

In the bottom layer, the flow near the bank forms an almost vertical downflow, while that nearer to the end of the abutment accelerates down and into the waterway, forming the principal vortex. Usually scouring starts in this region of accelerating flow and grows along the faces of the abutment. Wake vortices form downstream of the abutment.

The basic scouring process is the same for most types of abutment, although with wingwall and vertical wall types the stagnation region is larger, and scour is most severe near the end of the abutment where the principal vortex is concentrated.

The total scour depth is obtained by summing degradation, contraction and local scour. This procedure is, strictly, only valid where the scour holes overlap. For instance, contraction scour may have to be added to pier or abutment scour to get the total scour depth. However, pier scour and abutment scour would not be added unless the two scour holes overlap.

This usually has to be determined by drawing a cross-section through the waterway and superimposing the scour depths. If the holes do overlap the resultant scour depth is often larger than the two components, but difficult to predict. Nevertheless, as a general and conservative rule, the total scour depth is the sum of the three components.

The scour computation capability in the HEC-RAS software allows the user to compute contraction scour and local scour at piers and abutments. The details may be found in the HEC-RAS Hydraulic Reference Manual.

3.8.2. Design for scour at bridges

The best scour protection measure is to minimise the risk of scour in the bridge design.

Hydraulic modelling of a bridge site is an integral part of any bridge design and these studies should address the sizing of the bridge waterway helping to ensure that the foundations can be designed to minimise scour.

It must be recognised that damage to bridge approaches from rare floods can be repaired relatively quickly to restore traffic service. On the other hand, a bridge which collapses or suffers major structural damage from scour can create safety hazards as well as significant social impacts and economic losses for a prolonged period of time. Therefore, scour resistant bridge foundations should be designed to a higher hydraulic standard. These concepts must be reflected in bridge design procedures.

There are many methods for estimating scour as part of bridge design and these include equations by Holmes, Neill, Faraday and Charlton, Melville and Coleman, the CSU equation, FHWA HEC-18 equation, Froehlich equations and HIRE equations, with details for all provided in QDTMR (2013).

Encroachment in the stream channel by abutments and piers reduces the channel section and may cause significant contraction scour. Severe constriction of floodplain flow may cause approach embankment failures and serious contraction scour in the bridge waterway, where auxiliary (relief) openings can be considered but must be carefully designed. On wide floodplains the design should seek to avoid excessive diversion of floodplain flows towards the main bridge opening and skewed crossings of floodplains should also be minimised as much as possible.

The increase in the velocity through the bridge waterway opening occurs as a result of the increase in the energy head. The restriction in the waterway results in water banking upstream to a level sufficient to develop the additional head to increase the velocity to maintain equilibrium flow.

3.8.2.1. Length

In most cases it is not economical to bridge the full width of flood flow and the problem reduces to what is an acceptable length of bridge. As a consequence, the road embankment in the approaches to the bridge causes a restriction on the flow occurring under natural conditions. Consideration of the increase in velocity and hence scour potential and afflux would be the main determining factors for the length of a bridge. Longer bridges increase the cost but reduce the extent of constriction and therefore the risk of scour.

3.8.2.2. Height of abutments

The height of abutments should be considered in determining the length of a bridge. High abutments result in large retaining structures and embankments with inherent stability issues both in terms of the surcharge load to underlying material and long term structural issues including rotations and horizontal deflections. Instances have occurred where vertical and horizontal displacements at high abutments in soft soils has resulted in structural distress to the abutment and jamming of expansion joints.

3.8.2.3. Bridge height

The bridge height will be influenced by a number of factors being flood height, navigation clearance and span lengths.

3.8.2.4. Flood height

For high level bridges the deck level adopted will be above the design flood level. The clearance from the underside of the superstructure to the flood level (including freeboard) should be a minimum of 0.6 – 1.00 m. However the type, amount and size of debris likely may require an increased freeboard depending on local conditions.

3.8.2.5. Span lengths

In some cases the minimum span lengths may be determined by the size of the debris carried by the stream. The potential exists for a debris dam to be built up by log lengths greater than the spans.

The total bridge length is an important design feature, but others also need consideration, since this influences the flow velocity through the bridge and therefore the risk of scour. The important design consideration is to minimise the bridge length (and cost of the bridge) while keeping the risk of scour to an acceptable level.

A detailed approach to assessing scour as part of bridge design is given in QDTMR (2013).

3.8.3. Countermeasures for existing scour susceptible bridges

The greatest damage to bridges during floods is normally observed between the bridge approach and the abutment. Historically, this is the intersection between the road and bridge designer's responsibility. Protection of the bridge should consider the impacts of overtopping flows at the roadway.

Scour countermeasures are incorporated at a bridge site to monitor, control, inhibit or minimise stream stability problems and bridge scour. In many cases, the best countermeasure is appropriate design that avoids causing stream instability but scour protection is needed for existing bridges that have experienced scour problems.

Over the last several decades, a wide variety of countermeasure structures, armouring materials and monitoring devices have been used at existing bridges to mitigate scour and stream stability problems.

Since scour susceptible bridges are already in place, options for structural or physical modifications such as replacement or foundation strengthening are limited and expensive. Unless these bridges are programmed for replacement, their continued operation will ultimately require the design and installation of a scour countermeasure.

Riprap is one of the primary scour countermeasures to resist local scour forces at abutments of typical bridges. Riprap is generally abundant, inexpensive and requires no special equipment. However, proper design and placement is essential. Guidelines for proper grading and placement methods are included in QDTMR (2013). When designing riprap countermeasures, maintaining an adequate hydraulic opening through the bridge must be considered. Improperly placed riprap may reduce the hydraulic opening significantly and create contraction scour problems. If placed improperly, riprap can increase local scour forces. Although riprap is widely used, the following countermeasures can be considered as alternatives to riprap, but are not all covered here:

Armouring countermeasures:

- Rock riprap;
- Gabion boxes/ rock mattresses;
- Sack gabions;
- Grouted riprap;
- Grout-filled mats; and
- Articulating concrete blocks.

River training countermeasures

River training structures alter stream hydraulics to mitigate undesirable erosional and/or depositional conditions. They are commonly used on unstable stream channels to redirect stream flows to a more desirable location through the bridge, and require specialist design:

- Spurs (both permeable and impermeable);
- Bendway weirs;
- Guide banks; and
- Drop structures and check dams.

Scour protection design for bridges

Typical scour repair methods at bridges include:

- Dumped rock over a geofabric layer at piers, abutments and channel banks;
- Gabion mattresses over a geofabric layer at piers, abutments and channel banks; and
- Concrete (shotcrete) at bridge abutments.

Normally the scour protection is used to fill any scour holes that have formed to the original bed levels. Rigid measures such as concrete slabs are not as desirable due to potential for catastrophic failure. Flexible scour protections have an ability to self heal once a failure mode commences.

If shotcrete (concrete) is used at the bridge abutments for scour repair it must be tied into the abutment slope. If it is not properly tied into the slope it can be undermined and result in further damage to the abutment. This method is often not effective, particularly where the scour is being caused by a geotechnical failure of the embankment slopes.

Detailed descriptions of scour repair and protection for existing bridges is included in QDTMR (2013).

3.8.4. Scour protection for culverts

Culverts concentrate flow and also allow an increase in flood level upstream of the embankment. These factors increase the flow velocity at the culvert outlet compared to the natural velocity in the channel. If this increase produces a velocity where scour could be introduced, protection measures are required, though good practice would lead to a design solution where the culvert design maintains a flow velocity at the outlet that is below the rate that causes scour.

Outlet protection is required in situations where:

- outlet velocity exceeds the scour velocity of the bed or bank material;
- an unprotected channel bend exists within a short distance of the culvert outlet;
- the outlet channel and banks are actively eroding; and
- if an erodible channel bank exists less than 10 to 13 times the pipe diameter downstream of the outlet, and this bank is in-line with the outlet jet (ie. likely to be eroded by the outlet jet) the bank should adequately protected to control any undesirable damage as a result of the outlet jetting.

The most appropriate outlet protection is determined by considering the hydraulic performance of the outlet in the prevailing stream environment. At outlet structures, the best hydraulic performance is obtained when the confining sidewalls are parallel and the distribution of flow across the channel is uniform.

Culverts, however, are generally narrower than the natural waterway and a transition section is required to return the flow to the natural channel. When culvert outlet velocities are high, additional measures at the outlet may prove to be necessary for energy dissipation.

To check whether standard inlet and outlet structures with headwalls, wingwalls, aprons and cut-off walls are adequate, the outlet velocity for the culvert requires examination with respect to:

- natural environment (soil and vegetation cover);
- size of peak flow; and
- duration of large flows.

If outlet velocities exceed the acceptable limits, it may be necessary to check for potential bed scour problems. Where the outlet flows have a Froude Number (*Fr*) less or equal to 1.7 and outlet velocities less than 5.0 m/s, an extended concrete apron or rock pad (commonly used) protection is recommended.

Design details are provided by Austroads (2013).

3.9. Flow Measurement Structures

3.9.1. Introduction

Sharp-crested weirs are often used to measure flow in open channels. Among their advantages, are that they are easy to install, accurate, and relatively inexpensive.

They do, however, have a major disadvantage in flows containing substantial amounts of sediment, in that they trap sediment and other solids behind them, leading to putrescible deposits in sewer applications. For this reason, sharp-crested weirs are most commonly used with relatively clean effluent, or in temporary flow monitoring locations.

The emphasis in this sub-section is on the analytical techniques, weir properties, and choice for particular purposes, and submergence characteristics. In particular, complete details on variations in discharge coefficient are not given, and the reader is referred to specialist texts for such information.

3.9.2. Rectangular Sharp-Crested Weir

The analysis of the sharp-crested weir is best illustrated by reference to a weir comprising a vertical plate mounted at right-angles to the flow. This represents the so-called “rectangular sharp-crested weir”.

The flow over such a weir is illustrated schematically in Figure 6.3.26. This figure includes also the necessary nomenclature for the analysis. Before proceeding with the analysis, some comments are provided on the flow situation.

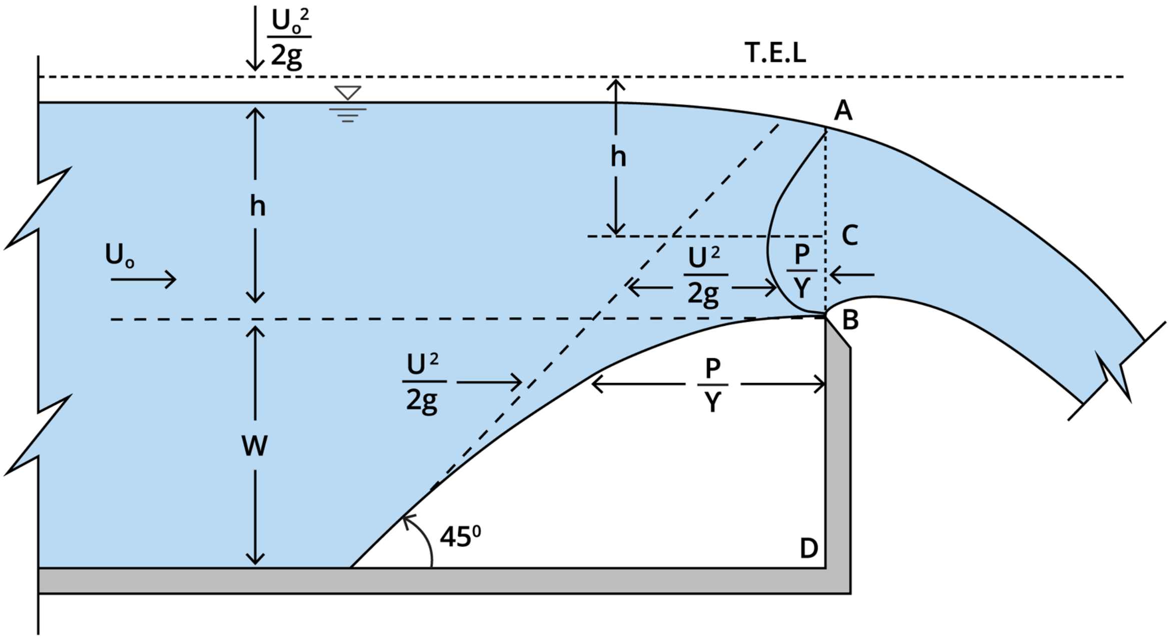


Figure 6.3.26. Schematic of Flow Over Rectangular Sharp-Crested Weir

It is noted first that the pressure distribution at the weir crest is non-hydrostatic. This situation arises because the pressure at both Points A and B is atmospheric by definition, and there are significant vertical components in the velocity as the flow contracts to pass over the weir crest.

Secondly, it is noted that, as expected, the total energy line (TEL) is situated an elevation $\frac{v_0^2}{2g}$ above the free surface, where v_0 is the approach velocity. In many cases, the magnitude of the approach velocity head may be considered to be negligible. For simplicity, this is assumed in the following analysis, although the influence of the approach velocity head will be included later.

Two further assumptions are utilised for simplicity:

1. The flow does not contract as it passes over the weir – ie. the elevation of A is the same as that of the upstream water surface; and

2. The pressure is atmospheric across the whole section AB.

With reference toFigure 6.3.26, these assumptions lead to an expression for the velocity at C of:

$$v = \sqrt{2gy} \tag{6.3.10}$$

The flow rate per unit width through an elemental strip of height dy at C, is then given by:

$$d_q = \sqrt{2gy}dy \tag{6.3.11}$$

The integral of Equation (6.3.11) may then be expressed as:

$$q = \int_0^h \sqrt{2gy}dy \tag{6.3.12}$$

where q is the flow rate per unit width.

Equation (6.3.12) is simply integrated and a contraction coefficient, C_c, introduced to allow for flow contraction over the crest, to yield:

$$q = \frac{2}{3}C_c\sqrt{2g}h^{\frac{3}{2}} \tag{6.3.13}$$

If the magnitude of the approach velocity head cannot be ignored, the integral form of Equation (6.3.11) is expressed as:

$$q = \int_{\frac{v_0^2}{2g}}^{h+\frac{v_0^2}{2g}} \sqrt{2gy}dy \tag{6.3.14}$$

Evaluation of Equation (6.3.14) yields:

$$q = \frac{2}{3}\sqrt{2g}\left[\left(\frac{v_0^2}{2g}+h\right)^{\frac{3}{2}} - \left(\frac{v_0^2}{2g}\right)^{\frac{3}{2}}\right] \tag{6.3.15}$$

Manipulation and introduction of the contraction coefficient leads finally to the result:

$$q = \frac{2}{3}C_c\sqrt{2g}h^{\frac{3}{2}}\left[\left(1+\frac{v_0^2}{2gh}\right)^{\frac{3}{2}} - \left(\frac{v_0^2}{2gh}\right)^{\frac{3}{2}}\right] \tag{6.3.16}$$

The equation is made more compact by introducing a discharge coefficient, C_d, leading to:

$$q = \frac{2}{3}C_d\sqrt{2g}h^{\frac{3}{2}} \tag{6.3.17}$$

in which:

$$C_d = C_c\left[\left(1+\frac{v_0^2}{2gh}\right)^{\frac{3}{2}} - \left(\frac{v_0^2}{2gh}\right)^{\frac{3}{2}}\right] \tag{6.3.18}$$

It is evident from the form of Equation (6.3.18) that, if the velocity head is negligible compared with h, $C_d = C_c$ and Equation (6.3.17) is then identical to Equation (6.3.13).

Early work indicated that the value of C_d is given by:

$$C_d = 0.611 + 0.08\frac{h}{W} \tag{6.3.19}$$

A small value of h relative to W is equivalent to a negligibly small approach velocity head. Under these circumstances, C_d = 0.611 and Equation (6.3.17) becomes:

$$q = 0.407\sqrt{2g}h^{\frac{3}{2}} \tag{6.3.20}$$

The total flow rate over the weir is then given by the product of Equation (6.3.20) and the transverse crest length.

3.9.3. V-Notch Sharp-Crested Weir

The triangular sharp-crested weir is analysed under the same assumptions as the rectangular weir. The structure is shown schematically in Figure 6.3.27. The following analysis is again simplified by the assumption that the approach velocity head is negligible.

It needs to be recognised, however, that the concept of "flow rate per unit width" cannot be used because the width varies over the height of the weir. Accordingly, the elemental flow rate through the element of width b is given by:

$$dQ = b\sqrt{2gy}dy \tag{6.3.21}$$

Integration of Equation (6.3.21) requires the expression of b as a function of y. From Figure 6.3.27, and using similar triangles:

$$b = 2\tan\frac{\theta}{2}(h-y) \tag{6.3.22}$$

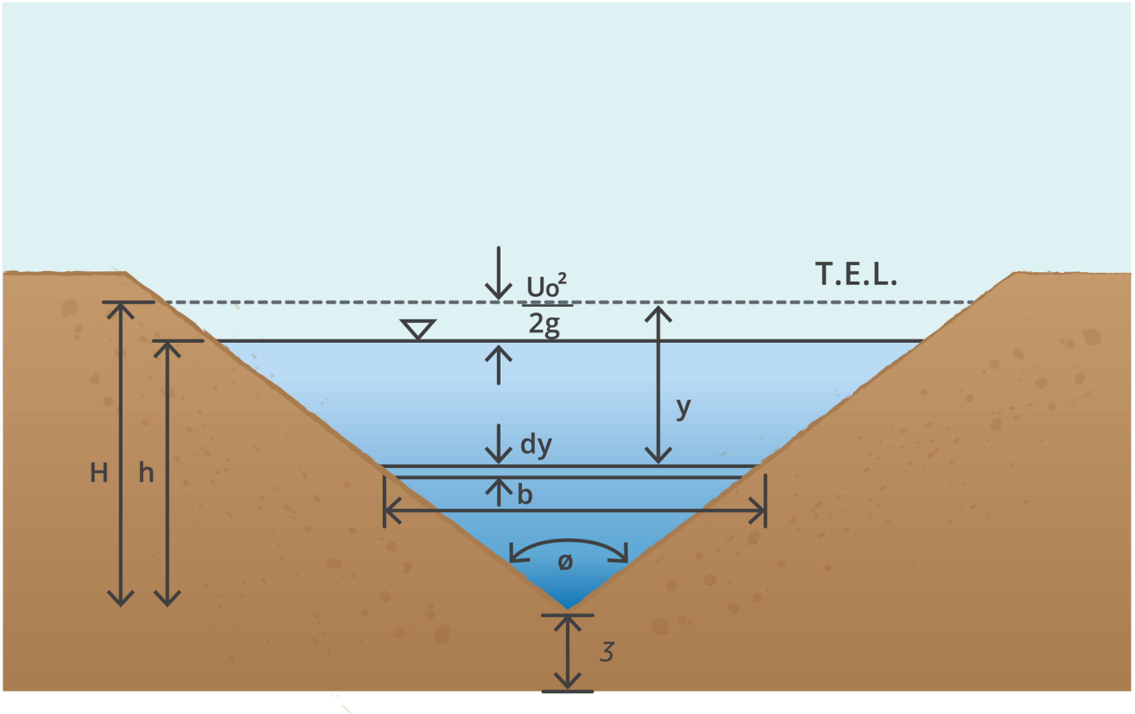


Figure 6.3.27. Schematic of Flow Over V-notch Sharp-Crested Weir

Substitution of Equation (6.3.22) into Equation (6.3.21), integration between the limits of $y = h$ and $y = 0$, and inclusion of the discharge coefficient yields:

$$Q = C_d \frac{8}{15}\sqrt{2g}\tan\frac{\theta}{2}h^{\frac{5}{2}} \tag{6.3.23}$$

The value of C_d is dependent on the ratio of $\frac{h}{w}$, but more particularly, on the vertex angle, θ. For the commonly used value for θ of $90°$, a value for C_d of 0.58 is commonly assumed. For other situations, values for C_d may be obtained from standard texts on flow measurement.

The analytical techniques discussed above can be applied to any weir crest shape.

3.9.4. Submerged Weirs

When the tailwater level is higher than the crest of the weir, the weir is termed *drowned* or *submerged*. This state is not desirable because measurements of flow are more uncertain. Nevertheless, if submerged conditions cannot be avoided, a procedure is required to effect flow measurements.

A submerged weir is shown schematically in Figure 6.3.28.

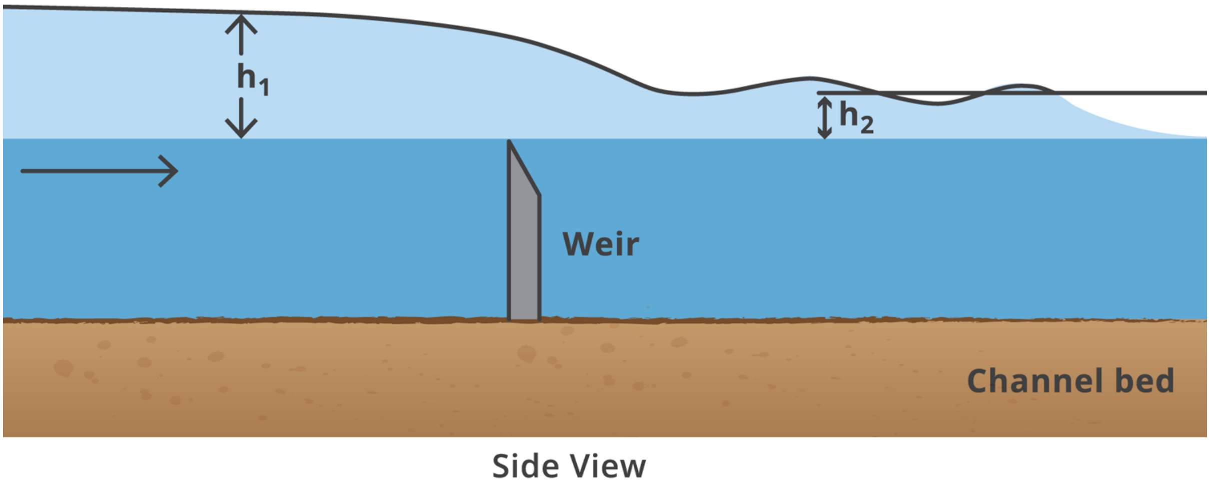


Figure 6.3.28. Schematic of Submerged Weir

A number of experiments were carried out on rectangular, triangular, parabolic, and Sutro submerged weirs and determined that the flow rate could be determined from the following equation:

$$\frac{Q}{Q_1} = \left[1 - \left(\frac{h_2}{h_1}\right)^n\right]^{0.385} \tag{6.3.24}$$

where Q is the flow rate under submerged

conditions Q_1 is the flow rate assuming unsubmerged conditions

h_1 is the upstream head

h_2 is the downstream head

n is the exponent in the unsubmerged flow equation $Q_1 = Ch_1^n$

3.9.5. Broad-crested Weirs and Long-Throated Flumes

Because critical flow represents a unique relationship between depth and flow rate, devices which induce critical flow are often used to measure flow in open channels. Most of these devices, however, require calibration in the laboratory because the flow characteristics are not in accordance with the usual theoretical assumptions.

Chief among these is the assumption that the pressure distribution is hydrostatic. In many devices, the strongly curved stream lines negate this assumption, resulting in the necessity for empirical coefficients.

The broad-crested weir and the long-throated flume are devices for which the flow rate can be predicted theoretically without the need for such coefficients. The broadness of the crest and the length of the throat are such that the stream lines are close to horizontal in the region of critical flow, permitting the assumption that the pressure distribution is hydrostatic.

The analysis of both the broad-crested weir and the long-throated flume is identical in that both rely on the determination of the relationship between the upstream water level (which may be measured) and critical depth within the constricted section – which is a known function of the flow rate. Thus, a unique relationship between flow rate and the upstream water surface elevation can be determined.

Broad-crested weirs are more prone to sediment buildup than long-throated flumes and are, consequently, less common in sewerage systems. For this reason, the emphasis in the following is on long-throated flumes, while recognising that broad-crested weirs are analysed in the same manner.

The long-throated flume is widely used in sewerage systems, within the pipe system and to monitor open channel inflows and outflows at sewage treatment plants.

In this section, the advantages of this type of structure are first discussed. The derivation of the theoretical rating curve is then presented for the rectangular flume and it is shown how this can be generalised for arbitrary cross-section shape. Their use in practice is then illustrated.

3.9.6. Advantages

The primary advantages of these devices are listed in the following:

- Provided that critical flow occurs in the throat, a rating table can be calculated with an error of less than 2% in the listed discharge. This can be done for any combination of a prismatic throat and an arbitrarily-shaped approach channel.
- The throat, perpendicular to the direction of flow, can be shaped in such a way that the complete range of discharges can be measured accurately, without creating an excessive backwater effect.
- The head loss across the structure required to obtain undrowned flow conditions is minimal, and can be estimated with sufficient accuracy for any of the structures placed in any arbitrary channel.
- Because of their gradually converging transitions, these structures have few problems with floating debris.
- Field and laboratory observations indicate that the structures can be designed to pass sediment transported by channels with subcritical flow. It should be noted, however, that excessively high sediment loads or significant reductions in the velocity of the approach flow may create sedimentation problems.
- Provided that its throat is horizontal in the direction of flow, a rating table based upon post-construction dimensions can be produced, even if errors were made in construction to the designed dimensions.
- Under similar hydraulic and other boundary conditions, these structures are usually found to be the most economical for the accurate measurement of flow.

3.9.7. Disadvantages

The major property of a long-throated flume is that it is designed to create a constriction in the flow area sufficient to produce critical flow over the full range of expected flow rates. In addition, the head loss across the structure should not be excessive and afflux should be kept to a minimum.

A typical long-throated flume is shown schematically in **Figure 6.3.29**. With regard to the hydraulic characteristics of the flume itself, five components may be recognised as follows:

1. The approach channel, where the flow should be stable so that the water level and the energy level can be accurately determined.

2. A converging transition region into the throat, which is designed to provide a smooth acceleration of the flow with no discontinuities or flow separation. The transition may be rounded or consist of plane surfaces.

3. The throat, where the flow is accelerated to the critical condition. The throat must be horizontal in the flow direction, but can, in principle, be of any shape transverse to the flow. The invert of the throat may be higher than the invert of the upstream and downstream channels.

4. A diverging transition to reduce the flow velocity to an acceptable level and to recover head. If there is ample available head, an abrupt transition may be used.

5. The tailwater channel in which a known hydraulic control is exercised by the downstream conditions and the hydraulic properties of the channel.

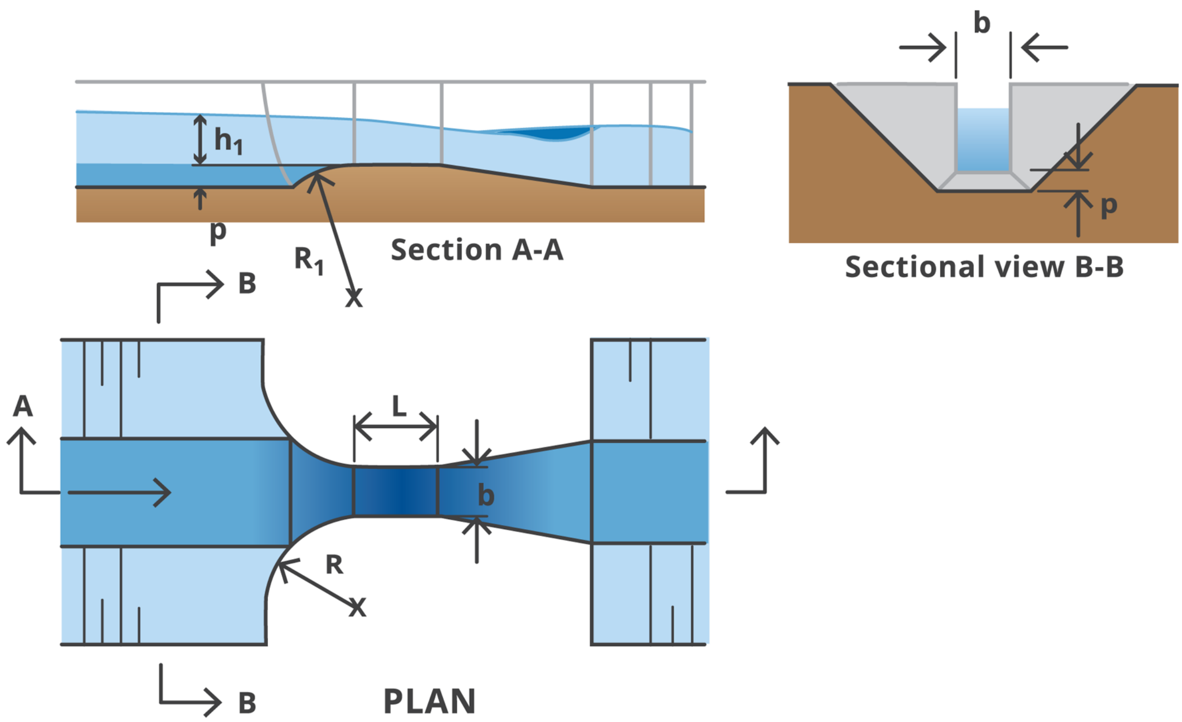


Figure 6.3.29. Schematic of Long-Throated Flume

The general profile of flow through a long-throated flume is shown schematically in Figure 6.3.30. The figure also shows the nomenclature for the theoretical analysis of the flume. In particular, we note that the energy level, H, and the stage height, h, are referenced to the invert level in the throat. As noted in 3, above, this is not necessarily the same as the channel invert level.

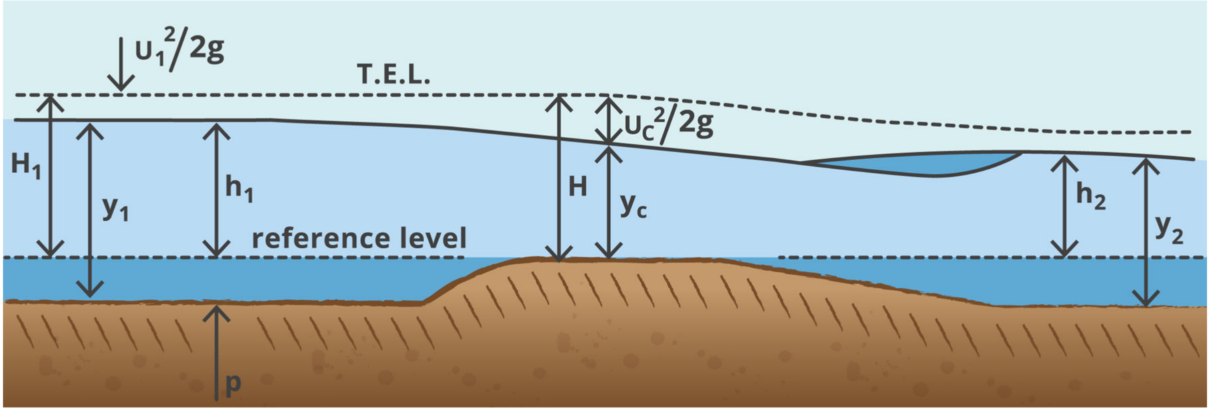


Figure 6.3.30. Flow Profile Through a Long-Throated Flume

The control section is the approximate location of critical flow within the throat of the flume. It is not necessary to know precisely where this occurs because the developed head-flow rate relationship is expressed in terms of the head upstream.

3.9.8. Analysis

With reference to Figure 6.3.30, application of the energy equation yields:

$$H_1 = y_c + \frac{v_c^2}{2g} \tag{6.3.25}$$

where subscript c refers to critical conditions.

To proceed further, the shape of the control section must be known. For a rectangular cross-section, the properties of critical flow are such that:

$$y_c + \frac{v_c^2}{2g} = \frac{3}{2}y_c = \frac{3}{2}\sqrt[3]{\frac{q^2}{g}} \tag{6.3.26}$$

where q is the flow rate per unit width within the control section.

Substitution of Equation (6.3.26) into Equation (6.3.25) and expanding yields:

$$H_1^3 = \left(\frac{3}{2}\right)^3 \frac{q^2}{g} \tag{6.3.27}$$

from which:

$$q = \frac{2}{3}\sqrt{\left(\frac{2}{3}g\right)}H_1^{\frac{3}{2}} \tag{6.3.28}$$

In terms of the width of the control section, b_c,Equation (6.3.28) is written as:

$$q = \frac{2}{3}\sqrt{\left(\frac{2}{3}g\right)}b_c H_1^{\frac{3}{2}} \tag{6.3.29}$$

where Q is the total flow rate.

The development of Equation (6.3.29) has assumed ideal flow conditions – in particular, that there is no energy loss between the location of the upstream head, H_1, and the critical control. This is taken into account by introducing a discharge coefficient, C_d, such that:

$$Q = C_d \frac{2}{3} \sqrt{\left(\frac{2}{3} g\right)} b_c H_1^{\frac{3}{2}} \tag{6.3.30}$$

C_d may be determined by an analysis of the boundary layer between the upstream head measurement point and the control section, but the complex procedure is rarely justified. High accuracy can be obtained by using the simpler equation:

$$C_d = \left(1 - \frac{0.006L}{b_c}\right)\left(1 - \frac{0.003L}{h}\right)^{\frac{3}{2}} \tag{6.3.31}$$

However, at the design stage, it is normally sufficient to assume a value for the discharge coefficient of 0.95.

Now,

$$H_1 = h_1 + \frac{v_1^2}{2g} = h_1 + \frac{Q^2}{2gA_1^2} \tag{6.3.32}$$

where A_1 is the cross-sectional area at the upstream location.

Equation (6.3.32) demonstrates that Equation (6.3.30) is difficult to use in practice because the head term, H_1, contains the unknown flow rate, Q, in addition to the measured head, h_1. An iteration method can be followed, using the following steps:

1. Assume, as a first approximation, that $h_1 = H_2$ and compute the discharge.

2. Use this approximate discharge to determine the velocity head and then use these data to calculate an improved value of the total head at the gauging section.

3. Compute a more refined discharge value using this total head value.

4. Repeat steps (2) and (3) until the difference between successive discharge values is an order of magnitude less than the required tolerance.

This process, although tedious, will lead to high accuracy.

A much more convenient approach is developed by defining a velocity coefficient, C_v, from the equation:

$$Q = C_d C_v \frac{2}{3} \sqrt{\left(\frac{2}{3} g\right)} b_c h_1^{\frac{3}{2}} \tag{6.3.33}$$

Comparison of Equation (6.3.33) and Equation (6.3.32) then shows that:

$$C_v = \left(\frac{H_1}{h_1}\right)^{\frac{3}{2}} = \left(\frac{h_1 + \frac{v_1^2}{2g}}{h_1}\right)^{\frac{3}{2}} = \left(1 + \frac{v_1^2}{2gh_1}\right)^{\frac{3}{2}} \tag{6.3.34}$$

Noting that $v_1 = \frac{Q}{A_1}$, Equation (6.3.34) is expressed as:

$$C_v = \left(1 + \frac{Q^2}{2gh_1A_1^2}\right)^{\frac{3}{2}} \tag{6.3.35}$$

Substitution of Equation (6.3.33) for Q and simplification yields:

$$C_v = \left[1 + \frac{4C_v^2}{27}\frac{C_d^2(b_c h_1)^2}{A_1^2}\right]^{\frac{3}{2}} \tag{6.3.36}$$

We now replace $b_c h_1$ by A^*, the imaginary cross-sectional area of the control section if the water depth there was equal to h_1, and further simplify to give:

$$C_d\frac{A^*}{A_1} = 2.60\sqrt{\frac{C_v^{\frac{2}{3}} - 1}{C_v^2}} \tag{6.3.37}$$

A plot of C_v against the area ratio, $C_d\frac{A^*}{A_1}$, can then be drawn and is presented in Figure 6.3.31. In this figure, the upper curve is a continuation of the lower curve beyond the right hand limit of the figure.

Because A^* and A_1 can be expressed in terms of the measured water surface elevation, h_1, the velocity coefficient, C_v, can be directly determined.

Corresponding graphs for C_v for non-rectangular cross-sections may be obtained in a similar manner. Such graphs are available in standard texts.

The rating equations for non-rectangular cross-sections are easily determined once the relationship between the critical depth, y_c, and the upstream energy level, H_1 is known. For example, application of the specific energy principles to the triangular cross-section yields:

$$y_c = \frac{4}{5}H_1 \tag{6.3.38}$$

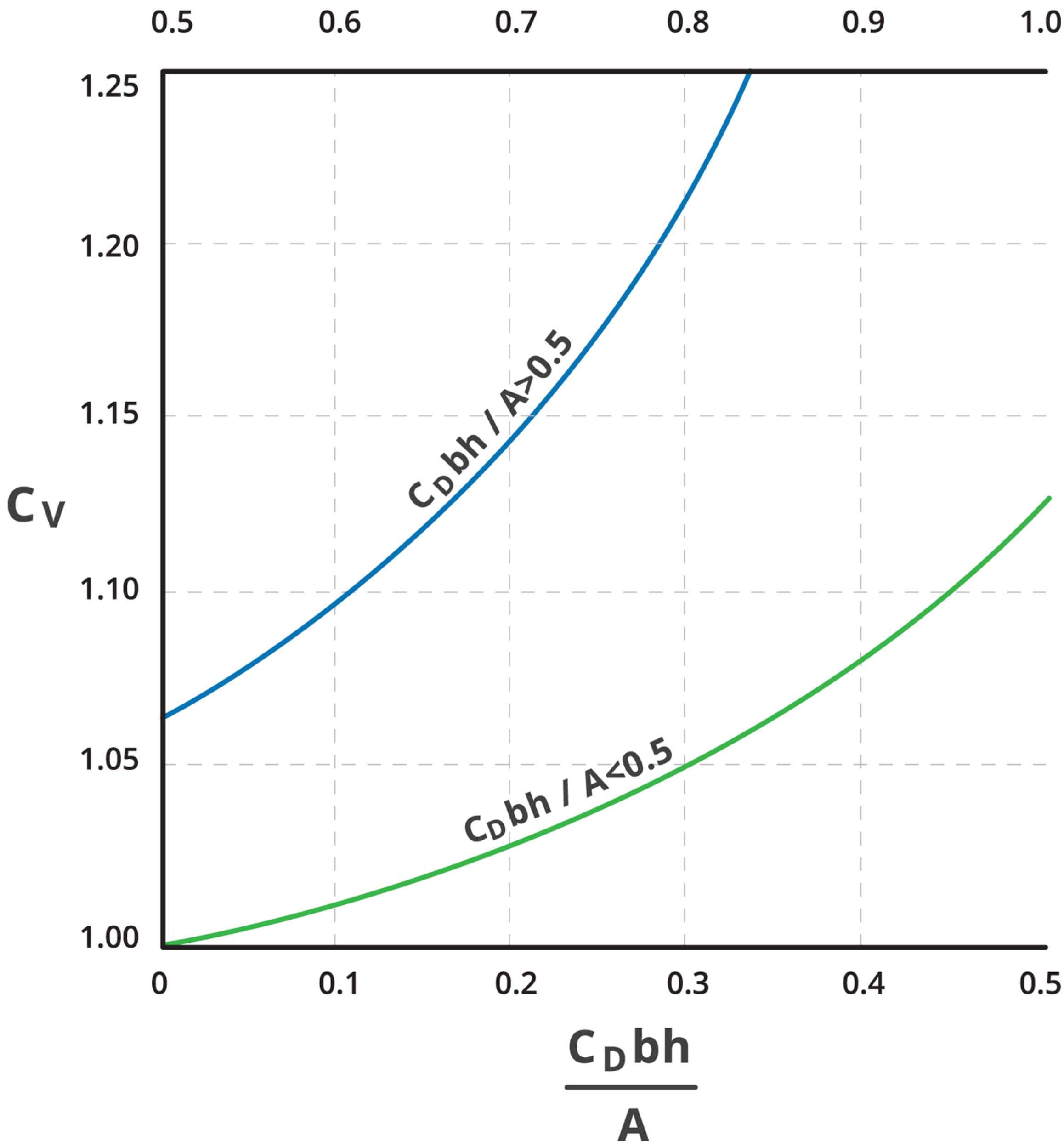


Figure 6.3.31. C_v Relationship for Rectangular Cross-Sections

Substitution of Equation (6.3.38) into Equation (6.3.23) and subsequent manipulation yields:

$$Q = C_d C_v \frac{16}{25}\sqrt{\frac{2}{5}g}\tan\frac{\theta}{2}h_1^{\frac{5}{2}} \tag{6.3.39}$$

where θ is the vertex angle of the control section.

3.9.9. Uncertainty in Flow Measurement Structures

No measurement is perfectly accurate or exact. Many instrumental, physical and human limitations cause measurements to deviate from the true values of the quantities being measured. We refer to these deviations as uncertainties, although, more commonly, the shorter word error is used. In this review, we will continue to use the word *uncertainty* since it is a more accurate descriptor

3.9.9.1. Systematic and Random Uncertainty

Generally, uncertainties can be divided into two broad and rough but useful classes: systematic (or determinate) and random (or indeterminate).

Systematic uncertainties tend to shift all measurements in a systematic way so their mean value is displaced. *Systematic* means that when the measurement of a quantity is repeated several times, the uncertainty has the same size and algebraic sign for every measurement.

Systematic uncertainties may be due to such things as incorrect calibration of an instrument, consistently improper use of equipment or failure to properly account for some effect. A systematic uncertainty is a true error and large systematic errors can and must be eliminated in a good experiment. Every effort should be made to minimise the possibility of these errors, by careful calibration of the apparatus and by use of the best possible measurement techniques. However, small systematic errors will always be present. For instance, no instrument can ever be calibrated perfectly.

Other sources of systematic errors are external effects which can change the results of the experiment, but for which the corrections are not well known.

Systematic errors can be more serious than random uncertainties for three reasons as follows:

1. There is no sure method for discovering and identifying them just by looking at the experimental data;

2. Their effects cannot be reduced by averaging repeated measurements; and

3. A systematic error has the same size and sign for each measurement in a set of repeated measurements, so there is no opportunity for positive and negative errors to offset each other.

Random uncertainties fluctuate from one measurement to the next and are present in all experimental measurements. As such, they cause a measuring process to give different values when a measurement is repeated many times (assuming all other conditions are held constant to the best of the operator's ability). Random uncertainties can have many causes, including operator errors or biases, fluctuating physical conditions, varying environmental conditions and inherent variability of measuring instruments.

The effect that random uncertainties have on results can be somewhat reduced by taking repeated measurements then calculating their average. The average is generally considered to be a better representation of the true value than any single measurement, because uncertainties of positive and negative sign tend to compensate each other in the averaging process. They yield results distributed about some mean value.

A measurement with relatively small random uncertainty is said to have high precision. A measurement with small random uncertainty and small systematic error is said to have high accuracy. Precision does not necessarily imply accuracy. A precise measurement may be inaccurate if it has a systematic error.

3.9.9.2. Determination of Flow Rate Uncertainty from Uncertainty in Measured Head

Frequently, the assessed flow rate will not be measured directly. Rather, it will be determined through a functional relationship from a measurement of head with its own uncertainty. The question is: What is the resulting uncertainty in the assessed flow rate?

The answer depends on the equation linking the flow rate with the directly measured parameters. We first return to the functional relationship derived for the rectangular sharp-crested weir:

$$Q = 0.407B\sqrt{2g}h^{\frac{3}{2}} \tag{6.3.40}$$

Using the rules of differentiation we can determine the derivative of this equation in the form:

$$Q = 0.407B\sqrt{2g}\frac{3}{2}h^{\frac{1}{2}}dh \tag{6.3.41}$$

Now we divide Equation (6.3.41) by Equation (6.3.40). Because of the equality, we can divide the left side of Equation (6.3.41) by the left side of Equation (6.3.40) and the right side of Equation (6.3.41) by the right side of Equation (6.3.40).

Thus:

$$\frac{dQ}{Q} = \frac{3}{2}\frac{dh}{h} \tag{6.3.42}$$

In words, Equation (6.3.42) indicates that the percentage uncertainty in flow rate (Q) is equal to (3/2) x the percentage uncertainty in measured head (h). We note that the fraction 3/2 is the exponent of the function in Equation (6.3.40).

Indeed, we can now state the completely general equation for uncertainty for a functional relationship of the form $y = Ax^n$ as follows:

If $y = Ax^n$, and there is an uncertainty in the independent variable of dx the consequent uncertainty in the dependent variable is given by:

$$\frac{dy}{y} = n\frac{dx}{x} \tag{6.3.43}$$

Equation (6.3.43) is a very simple equation to apply.

3.10. References

Austroads (2013) Guide to Road Design - Part 5: Drainage, Austroads Ltd, Sydney.

Cameron and McNamara, (1966), Model investigations of causeway designs - Report to Commonwealth Department of Works, Cameron and McNamara Consulting Engineers, Brisbane.

Fredrickson, L.H. (1979), Floral and faunal changes in lowland hardwood forests in Missouri resulting from channelization, drainage, and improvement. Report No. FWS/OBS-78/91, Fish and Wildlife Service, U.S. Department of the Interior, Washington, DC.

French, R.H. (1985), Open Channel Hydraulics, McGraw-Hill Book Company, New York.

Henderson, F.M. (1966), Open Channel Flow, Macmillan Publishing Co., New York

Keller, R.J. (1995), Meander re-instatement using submerged weirs. Department of Civil Engineering, Monash University, Final Report, Land and Water Resources Research and Development Corporation Partnership Project P85/05.

Keller, R.J. (1986), Hydraulic Investigation of Proposed Culvert Structures, Report for Road Construction Authority.

Keller, R.J., Peterken, C.J, and Berghuis, A.P. (2012), Design and assessment of weirs for fish passage under drowned conditions, Ecological Engineering, 48: 61-69.

List, E.J and Imberger, J. (1973), Turbulent Entrainment in Buoyant Jets and Plumes, Journal of the Hydraulics Division, ASCE, 99(9), 1461-1474.

Peterka, A.J. (1978), Hydraulic design of stilling basins and energy dissipaters. Engineering Monograph No.25, USER, Denver, Colorado, USA.

Portland Cement Association (1964), Handbook of Concrete Culvert Pipe Hydraulics, Portland Cement Association, Illinois.

QDTMR (Queensland Department of Transport and Main Roads) (2015), 'Road Drainage Manual', Available at http://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Road-drainage-manual.

QDTMR (Queensland Department of Transport and Main Roads) (2013),'Bridge Scour Manual', available at http://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Bridge-scour-manual.aspx.

Rouse, J., Bhocta, B.V., Hsu, E.Y (1951), Design of Channel Expansions, Trans. ASCE, 1(116), 347.

U.S. Department of Transportation (1972), Hydraulic Design of Improved Inlets for Culverts, Hydraulic Engineering Circular No.13.

United States Army Corps of Engineers (USACE). (1995), Hydraulic Design of Spillway, Technical Engineering and Design Guides as adapted from the US Army Corps of Engineers, No.12 ASCE.

United States Army Corps of Engineers (USACE) (2000), Design and Construction of Levees, Report #EM 1110-2-1913, April.

US Army Corps of Engineers Waterways Experiment Station (1952), revised in subsequent years. Corps of Engineers Hydraulic Design Criteria.

Chapter 4. Numerical Models

Andrew McCowan

Chapter Status	Final
Date last updated	14/5/2019

4.1. Introduction

Hydrology and hydrologic modelling are generally related to the determination of discharge characteristics of flood flows. By comparison, the main aim of hydraulic modelling is to describe the details of the main water level and velocity characteristics of the hydrologically derived flood flows. An appropriately set-up and calibrated hydraulic model can be used to not only describe the details of flood flows and their distribution throughout a river and floodplain system, but also to predict the likely impacts that any changes to that system may have on these flows. Typical applications for hydraulic models may include:

- Prediction of the behaviour of floods, including extreme flood events;
- Evaluation of the effects of proposed changes that may affect flood flows; and
- Assessment of a range of flood mitigation works

Prior to the advent of computers, hydraulic modelling of river flows could only be carried out in physical models. Although geometrically similar to the physical systems they represent physical models are subject to scaling constraints as described in Book 6, Chapter 2, Section 2. Due to the time and costs involved, physical modelling could only be justified for major projects. Today, the use of physical models is typically limited to modelling complex flows in relatively small reaches of river, and for modelling the behaviour of flows in hydraulic structures.

Although the basic equations governing river flow were derived in the 19th Century, it was not until the development of computers in the 1960s and 1970s that numerical modelling of river flows became practical. With the rapid on-going development of computers and computing power, there has been a continual evolution of numerical modelling and modelling techniques. This has resulted in the availability of a wide range of numerical models with increasing capability, and increased complexity.

The first numerical hydraulic models were little more than computerised backwater calculators for steady flows in one (along stream) dimension. These one-dimensional (1D) models gradually increased in sophistication to include hydraulic structures, unsteady flows, simply connected (dendritic) branched channel networks and, ultimately, multiply connected branched channel networks. With the multiply connected channel models it became possible to separate floodplain flows from main channel flows through the introduction of separate floodplain flow paths or systems of overbank floodplain cells. These models were sometimes called quasi-2D models.

In the early to mid-1990s, numerical modellers began to apply fully two-dimensional (2D) hydraulic models to river and floodplain systems. Many of these models had been originally developed for modelling flows in bays and coastal seas and required modifications to make them more suitable to simulating river and floodplain flows. 2D flood models use square or curvilinear grids, or flexible meshes, to provide much greater resolution of the flows in floodplains. As a result, 2D modelling was rapidly embraced by the modelling community and, by the early 2000s, had become almost a de facto “standard” for flood modelling in Australia.

Further development saw coupling of 1D channel models with full 2D models to provide a better description of in-bank flows and flows in sub-grid (or mesh) scale channels. These coupled 1D/2D models also allowed the introduction of model structures in localised 1D model branches to provide a better representation of hydraulic structures such as culverts, weirs and bridges. Full 2D models have also been applied extensively to simulating the hydraulics of urban stormwater flows. For these applications 2D models have also been coupled with pipe network models to provide a better description of the flows in underground stormwater drainage systems.

In the early to mid-2000s, the use of full 2D models was extended to also include the effects of rainfall over the model domain. With this “direct rainfall” or “rainfall on grid” approach it is possible to simulate the rainfall/runoff (hydrologic) processes throughout the model area and integrate them with the hydraulic routing of the resulting overland flows. This is particularly useful in urban applications or other situations where the model area includes a significant proportion (or even all) of the catchment contributing to the flow within the model. In these cases, the approach can reduce (and in some cases eliminate) the need for separate hydrologic modelling, but is still the subject of on-going research (Engineers Australia, 2012).

In general, it can be said that the more realistic the modelling approach, the greater the probability of achieving a successful outcome. However, the use of the most sophisticated modelling approach available will not, in itself, guarantee success. This is because the skill of the modeller adapting a generic modelling system to a specific application, and the quality of the data used as model input can be equally (or even more) important in determining the success of a modelling exercise. This can especially true for direct rainfall modelling. Indeed, there will be applications where simplified approaches, suitably applied, may be more appropriate than the use of more sophisticated models.

This chapter introduces the basis of numerical hydraulic models of river and floodplain flows. The differences between steady and unsteady flow models are discussed, and the range of modelling approaches that are currently available are described. More details are then provided on the application of 2D models for describing flood flows.

4.2. Development Stages for Numerical Hydraulic Models

The aim of a numerical hydraulic model is to provide a discrete numerical representation of flood flows in what is a physically continuous river and floodplain system. In this respect, the development of a site specific numerical model can be thought of as comprising a sequence of four main steps, as follows:

1. Review and define the physical system (the river and floodplain system to be modelled)

2. Select an appropriate mathematical model (the set of equations or combination of sets of equations to be used to describe the physical system)

3. Select a generic numerical model (the modelling software to be used to solve the equations of the mathematical model)

4. Develop the site-specific numerical model (through the application of site specific inputs to the generic modelling software. These inputs will typically include topographic data, bed-friction coefficients, flow boundary conditions and other parameters such as structure information, as appropriate)

At each step different types of assumptions, approximations and/or simplifications must be made. The steps are discussed briefly below, with further supporting information provided in the following sections. The conceptualisation process is shown schematically in Figure 6.4.1.

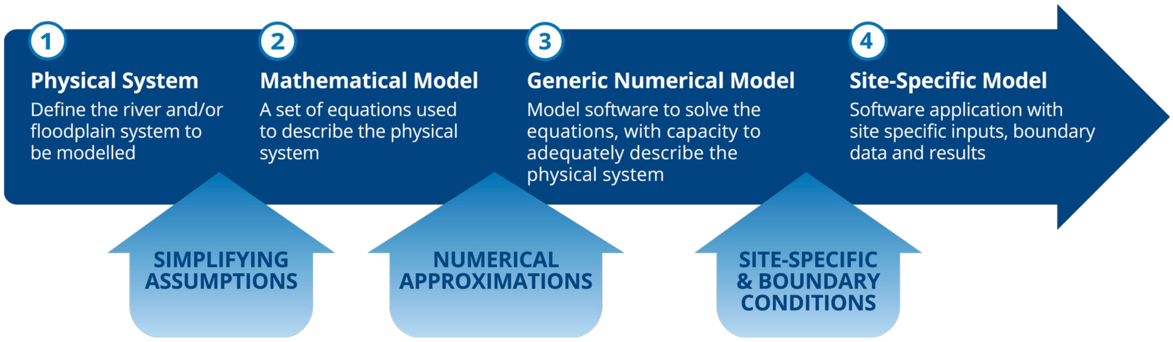


Figure 6.4.1. Stages in Numerical Hydraulic Model Conceptualisation and Development

4.2.1. Review of the Physical System

It is essential to have a broad understanding of the hydraulic behaviour of the physical system to be modelled in order to select the most appropriate mathematical model or models to be used. Whilst it is common that detailed hydraulic behaviour of the system is unknown, a good working knowledge of the study area and contributing catchment is required. Aspects such as the study area, shape and slope are important. (Is the subject land low-lying or close to the sea?) The location and dimensions of the main flow paths as well as the effects of any flow controls need to be understood. This can include the effects of flood protection works, such as levees, as well as the effects of formal drainage infrastructure, such channels, hydraulic structures and pipe networks. Land-use is also an important element of the physical system to consider. The distribution of roads, number and type of buildings and the extent and composition of vegetation can all affect the conceptualisation and selections made in following steps.

4.2.2. Selection of the Mathematical Model(s)

Selection of the mathematical model (or models) to be used to describe the physical system is the most important decision to be made in this four step process. In this step, the description of the flow in the physical system, which can be complex and highly turbulent, must be reduced to an equation, or set of equations describing the main characteristics of the flow. Here assumptions have to be made as to whether the flow can be considered as being one dimensional (1D), two dimensional (2D), a combination of 1D and 2D, or even three dimensional (3D). Further, a decision must be made as to whether the flow can be described as being steady (i.e., constant with time), or unsteady (time varying).

In 1D models, the flow is described in terms of cross-sectionally averaged discharges and water levels along pre-determined flow paths. In 2D models the flow is described by depth averaged velocity variables in two horizontal directions and water levels across a regular grid or flexible mesh. In all cases (1D, 2D and 3D), the flow equations generally assume there are no vertical accelerations and that the pressure distribution is hydrostatic. As a result, empirical structure equations are frequently used to describe the flow across structures (e.g., weirs, bridges and culverts) and at other locations where vertical accelerations may become important (e.g., across levee banks, or road embankments). It is important that the modeller has an understanding of the limitations of the selected mathematical modelling approach.

4.2.3. Selection of the Numerical Model

In going from the mathematical model to the numerical model, continuous mathematical equations have to be approximated by discrete arithmetic equations that can be solved by a computer. This process is generally carried out within a generic numerical model or modelling software package. The errors introduced by the discretization process are called “truncation” errors and reduce with the grid size and/or time step. That is, the smaller the grid size and time step, the smaller the truncation errors. In some models, low-order dissipative numerical truncation errors are deliberately introduced, or can be introduced as an option, to help stabilize the numerical computation.

Numerical solution procedures can involve finite difference, finite element and finite volume techniques. Finite differences are generally used in 1D models, while finite volume techniques are becoming increasingly popular in 2D and 3D applications. The introduction of “parallel processing” through multiple core computers and graphics processing units (GPUs) has dramatically increased the computing power available allowing much larger model domains and/or finer grid/mesh sizes to be used in 2D and 3D modelling applications. Additionally, the various commercially available generic modelling packages can have quite different approaches to modelling different flow characteristics such as flooding and drying, super-critical flows and sub-grid scale processes. It is therefore important that the modeller has an understanding of the capability and any potential limitations that the generic numerical model may have for simulating the flow conditions in any particular application.

4.2.4. Development of the Site-Specific Model

The Site-Specific model is developed from the generic Numerical Model (software package) through:

- Selection of a modelling domain,
- Selection of the grid or mesh size and time step,
- The input of site specific data including: topographic data (cross sections and/or topography) and bed-friction data, and
- The application of flow and /or water level boundary conditions.

Once a site-specific model has been developed it must be calibrated and, where possible, validated to ensure that it is capable of providing a reliable description of the flow characteristics within the area of interest. This is described in the following section.

4.3. Model Reliability

Assuming that the most appropriate mathematical model has been selected to describe a particular physical system, there are two main steps for determining the accuracy and reliability of the model results. These are model verification and model calibration and validation. These are described briefly below.

4.3.1. Model Verification

Model verification is the process whereby checks are made to ensure that the generic numerical model (i.e., the modelling software package) is actually solving the equations of the mathematical model. Most hydraulic modelling work will generally involve the use of welltried and proven software packages. In these cases, it can be assumed that the modelling software has been validated and that it is capable of solving the equations of motion correctly. Nevertheless, it can be useful for inexperienced modellers to carry out their own verification runs using standard test cases to provide confidence that they are operating the model correctly. These verification runs could include reproduction of uniform channel flow and/or reproduction of standard backwater or drawdown cases.

Modellers should be aware that all software packages have “bugs”, which can lead to spurious results and/or instability, particularly when used in modelling high velocity flows and in “non-standard” applications. Modellers should also be aware that different modelling packages can use different forms of the under-lying equations, different numerical solution procedure, and different approaches and assumptions to modelling special flow cases such as super-critical and flows at structures. Here it is important for a modeller to be aware of the assumptions that are used in the selected modelling package, and to be confident that they are appropriate for the physical system to be modelled.

Finally, there is a tendency for modellers to be innovative and to use models in situations well beyond the range of conditions for which they were originally developed. Typical examples might include the use of a 2D model to simulate flows where there may be significant vertical accelerations, or where there may be a significant three-dimensional component to the flow. Caution should always be used when using a package in these types of applications. Wherever possible, model results should be compared with analytical results, physical model results, or more appropriate (e.g., 3D or CFD) model results to obtain an estimate of the likely magnitude of the errors involved.

4.3.2. Model Calibration and Validation

Model calibration and validation provides an overall check of the reliability of a model. That is, how well the final site-specific model is representing the flow conditions in the physical system to be modelled. Ideally, calibration and validation is a two stage process, as follows.

Model calibration is the process of comparing model results against measured flood levels and extents and adjusting model parameters to obtain a “best-fit”. For flood studies, model calibration is typically carried out on the largest flood for which reliable water level data is available. In studies where more frequent flooding may be important, the model should also be calibrated against measurements taken from a more frequent flood event. During the calibration process, model parameters (typically bed-friction coefficients) are adjusted and the model re-run until the results give the best reproduction of the measured data.

In the first instance, the calibration process is also used to identify any inconsistencies in the model terrain data and boundary conditions. If after repeated efforts, it is not possible to obtain a reasonable representation of the measured data or, if this can only be achieved by the use of physically unrealistic input parameters, then it will be necessary to look more closely at: the assumptions made in the selection of the generic mathematical model, the appropriateness of the selected modelling package for reproducing the flow conditions under consideration, and the reliability of the boundary conditions that have been applied to the model.

Model Validation is the process whereby the calibrated model is used to simulate an independent flood event to provide a check on the reliability of the calibration process. The flood event will typically be somewhat lower than the calibration case and, in some cases, the results may be used to further refine the calibration process.

4.4. Steady vs Unsteady Hydraulic Models

Numerical hydraulic models can be described as being either steady flow models, where the depth of flow remains constant with time, or unsteady flow models, where the depth of flow can vary with time. These models typically assume the flow to be gradually varying in space. That is, variations in flow depth are small in relation to the distance over which they occur. Under these conditions the pressure distribution can be assumed to be hydrostatic.

In the early days of numerical hydraulic modelling, the models were limited to 1D and the distinction between steady flow and unsteady flow models was quite marked. This was because Hydrolic Engineering Center of the US Army Corps of Engineers made their River System, HEC-RAS, a relatively sophisticated 1D steady flow model, freely available to anyone who wanted to use it. By comparison, unsteady flow modelling required modellers to either have specialist modelling skills to develop or adapt research-based models, or to purchase what was then quite expensive specialised modelling software packages. As a result there was a tendency for modellers to “push” the use of HEC-RAS well beyond the range of application for which it was originally designed.

With the advent of more readily available and relatively sophisticated 1D and 2D unsteady flow models, the distinction between steady and unsteady flow models has become less marked. This is because there is less reliance on specialised steady flow models and, where required, much of the steady flow modelling is carried out by unsteady flow models using steady flow boundary conditions.

4.4.1. Steady Flow Models

In steady flow modelling, the flow velocity *u* and discharge *Q* can be assumed to be locally constant and do not vary in time. Under these conditions, the acceleration terms in the equations of motion, described in Book 6, Chapter 4, Section 6, can be assumed to be negligible. This means that the flow is assumed to be in equilibrium with any potential increase in momentum or energy as water flows downstream being balanced by bed-friction and other friction losses such as may occur at structures.

1D steady flow models such as 1D steady flow version of HEC-RAS (USACE, 2010) can be used to compute flood profiles in a wide range of situations. This version of HEC-RAS is based on the numerical solution of the profile equation presented in Book 6, Chapter 2, Section 8. This in turn is based on the 1D energy equation. Manning’s equation is used to compute energy losses due to bed-friction, and other contraction and expansion losses are calculated using a coefficient times the change in velocity head. The momentum equation is used in situations where there are rapid variations in the water surface profile, such as at hydraulic jumps. The computations can also include empirical structure equations to describe rapidly varying flows at a range of structures including bridges, culverts, weirs and spillways.

When using steady flow models, such as HEC-RAS, or using unsteady flow models with steady flow boundary conditions, it is noted that the main underlying assumptions of steady flow are effectively that:

- The discharge is constant (usually calculated by a hydrologic model);
- The peak flood levels coincide with the peak discharge; and
- The peak discharge and corresponding flood levels occur simultaneously over the full length of the reach of channel under consideration.

For these reasons, and for the reasons described in the following section, steady flow models are best suited to modelling flows along relatively short reaches of river with well-defined flow paths, and/or for modelling flows at structures. Steady flow models should not be used to describe flows where there are:

- Rapidly changing hydrographs;
- Flat channel slopes where wave propagation effects can become important;
- Wide floodplains and/or other features where storage effects may affect the flow; and
- Channel networks where the flow splits are not well defined.

The Manning Equation

The Manning equation for uniform flow can be considered as the simplest form of steady flow model. Here the average velocity u across a cross-section can be related to a Manning roughness coefficient *n*, the hydraulic radius *Rh*, and the bed slope *S*:

$$u = \frac{1}{n} R_h^{\frac{2}{3}} S^{\frac{1}{2}} \tag{6.4.1}$$

For a wide open floodplain, the hydraulic radius can, to a first approximation, be replaced by the flow depth y:

$$u = \frac{1}{n} y^{\frac{2}{3}} S^{\frac{1}{2}} \tag{6.4.2}$$

These equations can be used by modellers as a sanity check to ensure that their models are providing results with flow velocities of the correct order of magnitude. For example, a relatively smooth floodplain with n = 0.04, a moderate slope of S = 0.001, and a depth of y = 1.0m would be expected to have a flow velocity of approximately u = 0.8 m/s.

4.4.2. Unsteady Flow Models

Unsteady flow models are not restricted by the steady flow assumption, and can be applied to a much wider range of flow conditions. Referring to the limitations of steady flow models, described above, situations in which an unsteady flow modelling should be used are discussed below.

Rapidly Changing Hydrographs

With rapid changes in flow, the inertial (acceleration) terms in the equations of motion become important. (These terms are not included in steady flow models). Examples include modelling of dam-breaks and structure operations, as well as modelling of reaches where the time of propagation of a flood wave through the reach cannot be considered insignificant relative to the duration of the flood hydrograph.

Flat Channel Slopes

For flat channel slopes, flow velocities and, correspondingly, the effects of bed friction can become small relative to the main time-varying acceleration terms in the equations of motion. This is particularly true in lakes and estuaries. For this reason, the American Society of Engineers (1996) recommends that unsteady flow modelling should be carried out for channel slopes less than about 4x10^{-3} and, depending on the study objectives, for slopes up to about 1x10^{-3}.

Storage Effects

In reaches where there are significant storage effects, the rating curve on the rising limb of a flood hydrograph will be different to the rating curve on the falling limb. This results in a “looped” rating curve, as shown in Figure 6.4.2. Under these conditions the peak water level will not correspond to the peak discharge. As a result, unsteady models should be used to simulate these effects.

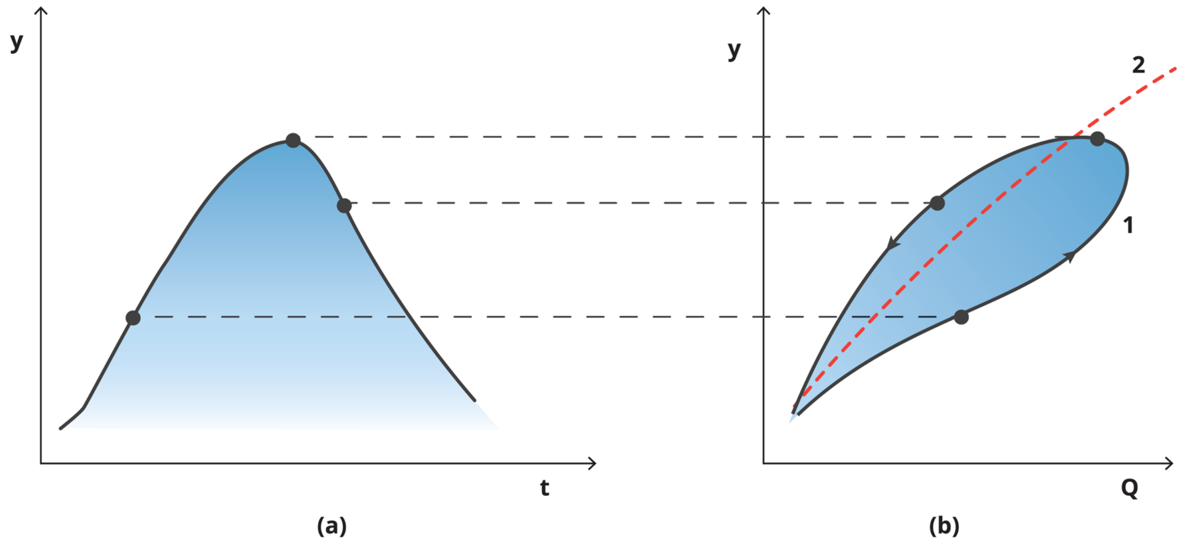


Figure 6.4.2. Storage Effects on the Rating Curve; (a) Flood Hydrograph, (b) Corresponding Rating Curve (Cunge et al., 1980)

Channel Networks

Unsteady flow models have the ability to compute the distribution of flows throughout a channel network or across a model domain, whereas steady flow models can only modelled on the basis of pre-determined flow splits. In real-life situations peak flows in tributaries rarely correspond with the peak flow in the main channel and, in some cases, backwater effects from one part of a network (e.g., a main channel) can cause time-varying flow reversals in another part (e.g., a tributary). Unsteady flow models are required to simulate these effects.

4.5. Types of Unsteady Flow Models

With the exception of the steady 1D form of HEC-RAS most models currently in use have unsteady modelling capability. In order of increasing level of sophistication, the main types of unsteady flow model that can be applied to flood investigations, include:

- 1D models
- 2D models
- Coupled 1D/2D models
- 3D models

- CFD, physical and other non-hydrostatic models

These are described briefly below. More details on the use of 1D, 2D and coupled 1D/2D models in flood modelling applications are then provided in the following sections.

4.5.1. 1D Models

1D flow models are based on the numerical solution of the Saint Venant equations for describing gradually varying unsteady flow in one horizontal dimension. Early 1D models required the main channel and flood plain of a river to be schematised as a single one-dimensional channel. The use of these early 1D models was generally restricted to modelling single river branches, or simply connected (dendritic) branched river systems. As part of the evolutionary process of model development these relatively simple models have been replaced by more sophisticated models that allow arbitrary connections of multiple channel systems. In these models, floodplains can be represented as separate flow paths and there can be multiple flow paths within a single floodplain. This makes it possible to provide a somewhat more realistic description of the flows in real river and floodplain systems.

1D models are computationally quick to run and are well suited to modelling flows along well-defined channel and floodplain flow paths. Their more general use in flood studies has been largely superseded by 2D models which can provide a much more detailed description of flood flows in overbank areas. 1D models are still used in applications where large numbers of multiple model runs are required and computational time requirements make 2D modelling impractical. 1D models have also been integrated with 2D models in order to make the most of the relative advantages of both types of model.

4.5.2. 2D Models

2D flow models are based on the numerical solution of the depth-averaged equations describing the conservation of mass and momentum in two horizontal dimensions. These equations assume that the flow velocity is uniform over the depth, both in magnitude and direction. This assumption is reasonable in most floodplain applications where the flow depth is relatively shallow with respect to the horizontal dimensions of the main physical features to be resolved in the model.

The 2D model equations are solved at each active water grid point or mesh element over a two-dimensional model grid or mesh. The computational domain may be a square, rectangular or curvilinear grid, or may be a flexible mesh comprising triangular and/or quadrilateral mesh elements. With these models survey information for the area of interest is digitised onto the two-dimensional model grid or mesh. This capable of providing a detailed description of the flow in floodplains and overbank areas.

2D models can have problems in providing adequate resolution of in-bank flows and, compared with 1D models, are heavy computationally. With respect to the latter, it is noted that the use of parallel processing coupled with multi-core computers and/or graphics processing units (GPUs) has significantly enhanced the computational capability of 2D models.

4.5.3. Coupled 1D/2D Models

Coupled 1D/2D models are aimed at making the most of the best features of both 1D and 2D models. Depending upon the particular software package being used the coupling can occur in two ways. In the first, an overall 1D model of an extended reach of river may be coupled dynamically to one or more detailed 2D model domains to provide a more detailed description of the flows in local areas of interest. In the second, one or more 1D model branches may be dynamically coupled within a 2D model domain to provide a better description of in-bank channel flows, and flows through hydraulic structures such as culverts, weirs and bridges. In some software packages, the 1D/2D coupling has also been extended to include 1D pipe network models. This has extended the range of application of coupled 1D/2D models to providing a more detailed description of the flooding associated with urban stormwater flows.

4.5.4. 3D Models

3D flow models are based on the numerical solution of the Reynolds-averaged Navier-Stokes equations describing the conservation of mass and momentum in three-dimensions. For most 3D river and estuarine modelling applications, these equations are simplified by assuming that the pressure distribution is hydrostatic. This assumption is consistent with the equations used in 1D and 2D modelling, described above. As for 2D models, the computational domain of a 3D model may be formed using a square, rectangular or curvilinear grid, or using a flexible mesh comprising triangular and/or quadrilateral mesh elements. With 3D models there are additional grid cells or mesh elements in the vertical dimensional for describing the variations in flow with depth.

The use of a full 3D model should be considered in cases where it is important to simulate three-dimensional flow effects. These can include: stratified flows and wind-driven over-turning circulations in lakes and estuaries, “helicoidal” flows around river bends, flows associated with hydraulic structures, and flows where the water depth is of the same order of magnitude as, or greater than, the horizontal dimensions of the main physical features to be resolved.

3D models require significantly more computing time (typically an order of magnitude or more) than equivalent 2D models, even with the use of parallel processing. Further, in most floodplain applications, many of the three-dimensional flow effects noted above are of secondary importance, relative to the main flood flows, or can be accounted for through relatively well defined additional loss parameters. In these cases, the additional complexity and computing time associated with using a 3D model is considered to be unnecessary and unwarranted. As such, 3D models are best suited to modelling the details of complex flows in relatively short reaches of rivers, around structures and in other flows cases where three-dimensional effects become important in determining localized flood effects.

4.5.5. 1D, 2D and 3D Model Limitations

The 1D, 2D and 3D models that have been discussed so far generally assume that the pressure distribution in the flow to be modelled is hydrostatic. This assumption is applicable for most flood flows but becomes invalid in flow situations where vertical accelerations become significant. The most common cases where this might occur include flows over weirs and levee banks, and flows through hydraulic structures. This is one of the reasons why most 1D, 2D and even 3D models incorporate structure equations for describing flows at structures such as weirs, levee banks and culverts.

Additionally, the numerical methods used to solve the equations of motion are generally based on the assumption that the flow is sub-critical. As such, super-critical flows can usually only be modelled through locally simplifying the momentum equation(s) and/or through the addition of significant amounts of numerical dissipation. These approaches make it possible to maintain the numerical computation through regions of super-critical flow. Modellers should, however, be aware that these approaches only provide approximate solutions to super-critical flow. Care should be taken when interpreting the results in these regions, and in the transition zones between super-critical and sub-critical flow (and, in particular, in the location and size of any resulting hydraulic jumps).

4.5.6. CFD and other Non-Hydrostatic Models

In cases where it becomes important to describe the details of non-hydrostatic flow and/or of super-critical flow, it will become necessary to use more sophisticated numerical modelling approaches such as Computational Fluid Dynamics (CFD) or Smoothed Particle Hydrodynamics (SPM), or to use a physical model.

Computational Fluid Dynamics (CFD) refers to a class of models that are based on the numerical simulation of the more generalised form of the Navier-Stokes equations. (See for example, Roache (1998)). CFD can be used to model the details of non-hydrostatic flows, including flows where there is no free-surface (e.g., internal flows within a structure). CFD can also be used to model supercritical flows, as well as the transitions between super-critical and sub-critical flows. CFD models require significantly more computing time than the 3D models discussed above and are more demanding with respect to model set-up and boundary condition requirements. As such, the use of CFD in flood modelling is generally limited to simulating the details of complex flows at or within hydraulic structures (e.g., flow over a dam spillway or flow through a complex system of culverts).

CFD and the other types of model considered above typically operate in what is known as an “Eulerian” reference frame. This involves the computation of the fluid flow relative to a fixed model grid or mesh. By comparison, Smoothed Particle Hydrodynamics (SPM) uses a “Lagrangian” reference frame (See for example, Violeau (2012)). Here the fluid itself is broken down into small individual “particles” and, rather than using a fixed grid or mesh, the computation follows the movements of these particles. This makes SPM particularly well suited to modelling the dynamics of interactions at the water-air interface. For reliable results SPM requires the use of very large numbers of particles. SPM has similar disadvantages to CFD in relation to computational time and the demands associated with model set-up and the application of appropriate boundary conditions. As for CFD, the use of SPM in flood modelling is generally limited to simulating the details of complex flows at or within hydraulic structures.

Physical models are geometrically scaled reproductions of the physical system to be modelled and are disscussed in more detail in Book 6, Chapter 2, Section 12. They typically use the same fluid (i.e., water) and operate under the normal laws of gravity. This makes physical models suitable for modelling non-hydrostatic flows and super-critical flows, including the transitions between super-critical and sub-critical flows. As a result, physical models can be used to model flow cases similar to those for 3D, CFD and SPM models discussed above. The main limitation on the more general use of the physical models is the restriction on the scales that can be used in order to avoid “scale effects”. This generally restricts the use of physical models to modelling the details of flows in relatively short reaches of rivers. Although “distorted scale” models can be used to reduce scale effects when modelling larger areas, distorted models should not be used when accurate reproduction of three-dimensional flow effects is required.

4.6. 1D Unsteady Hydraulic Models

Commercially available 1D unsteady hydraulic models typically solve the full one-dimensional unsteady Saint Venant equations. They can be applied to branched and looped channel and river and floodplain systems, and can be used to simulate flows in a wide range of physical systems from steep river reaches to tidal estuaries. The capabilities of these models typically include most, if not all of the following features:

- Hydraulic structures such as bridges, culverts, weirs, levees, etc., through the use of in-built structure routines.
- Options for including user-defined structures for describing structures such as flow regulation gates and pumps.
- The capability to approximate super-critical flows, including super-critical to sub-critical flow transitions.
- Options for specifying simplified diffusive wave or kinematic wave approximations to the equations of motion to improve computational speed, where appropriate

4.6.1. 1D Equations of Motion

The main properties of one-dimensional flow along a channel can be uniquely defined by two dependent variables, the water level or stage *h,* and the averaged discharge *Q*, as shown in Figure 6.4.3. These can be described as a function of the two main independent variables: the chainage or distance along centreline of the channel *x*, and the time *t*

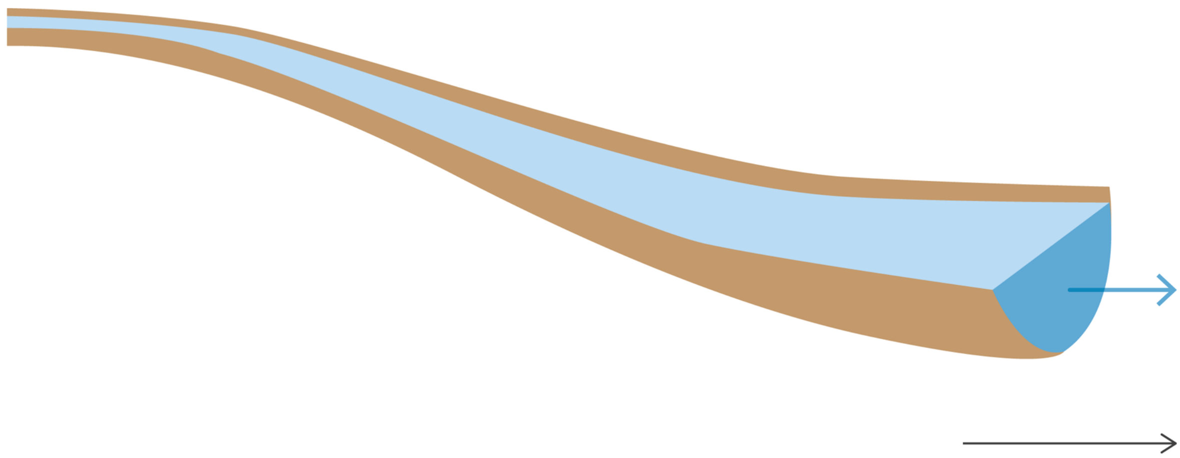


Figure 6.4.3. The main dependent variables for the 1D model equations

With two dependent variables it is necessary to have two equations to describe the flow in terms of the stage height *h* and discharge *Q* at any given point in *x* and *t*. The equations used are the Saint Venant equations. These equations describe the cross-sectionally averaged conservation of mass and conservation of momentum. The momentum equation is used in unsteady flow models in preference to the energy equation that is used in steady flow models. This is because momentum is a vector and introduces directionality into the computation. By comparison, energy is a scalar and cannot. Additionally, the momentum equation can be used to maintain the computation through discontinuities such as hydraulic jumps.

The Mass Equation:

The one-dimensional unsteady continuity equation as derived in Book 6, Chapter 2, Section 8 can be given as:

$$\frac{\partial A}{\partial t} + V\frac{\partial A}{\partial x} + A\frac{\partial V}{\partial x} = 0 \tag{6.4.3}$$

By combining the two spatial derivatives, the conservation law form of the cross-sectionally averaged mass equation can be given as:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \tag{6.4.4}$$

For modelling applications, this equation is transformed into terms of the required water level *h* through the introduction of a storage width b_{st} and, when a lateral inflow of q_l per unit length of channel is included, the mass equation becomes:

$$b_{st}\frac{\partial h}{\partial t} + \frac{\partial Q}{\partial x} = q_1 \tag{6.4.5}$$

The Momentum Equation:

In a similar form, the equation describing the one-dimensional cross-sectionally averaged conservation of momentum can be given as:

$$\frac{\partial Q}{\partial t} + \beta\frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + gA\left(\frac{\partial h}{\partial x} + S_f\right) = q_1 u_l \tag{6.4.6}$$

Where: *A* is the cross-sectional area, *g* is the gravitational constant, S_f is the friction slope, *β* is the momentum correction factor, and q_l is a lateral discharge with a downstream velocity component u_l relative to the velocity of the main stream.

With the introduction of a bed friction term based on Manning's equation, the momentum equation can be expressed as:

$$\frac{\partial Q}{\partial t} + \beta\frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + gA\left(\frac{\partial h}{\partial x} + \frac{n^2Q^2}{A^2R^{4/3}}\right) = q_1 u_l \tag{6.4.7}$$

Where: *n* is Manning's roughness coefficient and *R* is the hydraulic radius (defined as the cross-sectional area divided by the wetted perimeter)

Equation Assumptions:

The Saint Venant equations for unsteady flow, as described above, are based on the following assumptions:

- The flow is one-dimensional (i.e., the flow velocity is uniform and the water surface is horizontal across each cross-section).
- The pressure is hydrostatic (i.e., streamline curvature is small and vertical accelerations can be neglected).
- The effects of bed friction and turbulence can be included through resistance laws (e.g., Manning's equation) that have been derived for steady flow conditions.
- The flow is nearly horizontal (i.e., the average channel bed slope is small)

Unsteady flow models based on the numerical solution of the full de Saint Venant equations are sometimes called "dynamic wave" models.

4.6.2. Simplified Forms of the 1D Momentum Equation

The water surface slope in the hydrostatic pressure term of the momentum equation can be expressed as:

$$\frac{\partial h}{\partial x} = \frac{\partial y}{\partial x} - S_b \tag{6.4.8}$$

Where y is the water depth and S_b is the bed slope.

Ignoring the effects of lateral inflows for the purpose of this exercise, the momentum equation can then be rewritten as:

$$\frac{\partial Q}{\partial t} + \beta \frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + gA\left(\frac{\partial y}{\partial x} - S_b + S_f\right) = 0 \tag{6.4.9}$$

Diffusive Wave Approximation

In the diffusive wave approximation, the local acceleration term $\delta Q/\delta t$ and the convective acceleration term $\delta(Q^2/A)/\delta x$ are neglected, and the momentum equation reduces to:

$$\frac{\partial h}{\partial x} = \frac{\partial y}{\partial x} - S_b = \ -S_f \tag{6.4.10}$$

That is, the water surface slope is balanced by the friction slope.

This approximation includes backwater effects, but the “dynamic” or wave propagation effects associated with the “inertial” acceleration terms have been excluded. As a result, a model based on the diffusive wave equation does not have the same stability constraints as an equivalent full dynamic wave model, and can use much larger time steps.

The diffusive wave approximation is valid for describing gradually varying flows in reaches with moderate to steep slopes. It should not be used for rapidly varying flows such as in dam-breaks, or in reaches with flat-bed slopes, including lakes and estuaries, where the acceleration terms become important. Diffusive wave models are sometimes used to describe regional scale flows where the use of larger time steps can provide significant reductions in the amount of computation required.

Kinematic Wave Approximation

In the kinematic wave approximation, it is assumed that the momentum equation can be further reduced to:

$$S_b = S_f \tag{6.4.11}$$

That is, the friction slope is equal to the bed slope. Backwater effects are excluded from this approximation and water can only flow downstream.

The kinematic wave approximation is only valid for describing gradually varying flows in reaches with moderate to steep slopes where bed-friction dominates and backwater effects can be neglected. As such it is not well suited to general modelling applications. The kinematic wave approximation is very stable and is sometimes used to describe super-critical flows in localized regions of an otherwise dynamic wave model.

4.6.3. Model Set-up

Topographic input to 1D models is generally specified in terms of survey cross-sections of the channel and floodplain system to be modelled. These cross-sections are given as a series of x–z co-ordinates specified perpendicular to the direction of flow. At each cross-section key computational parameters may be pre-computed as a function of water level. These parameters include cross-sectional area, storage width and conveyance. The effects of varying roughness across a cross-section (e.g., between in-bank and over-bank areas) may also be included in the conveyance calculations.

The computation can be carried out on a “staggered” grid where water level *h* and discharge *Q* grid points are specified alternately along each model branch, or on a “non-staggered” grid where the water level *h* and the discharge *Q* are specified at each grid point, as shown in Figure 6.4.4. Grid points are typically allocated to each surveyed cross-section; water level *h* points for staggered grids, and combined water level *h* and discharge *Q* grid points for non-staggered grids. If necessary, additional intermediate grid points may be allocated between cross-sections. For the staggered approach, discharge *Q* grid points are located midway between adjacent water points.

Implicit finite difference procedures are typically used to solve the equations of motion along each model reach. The scheme first attributed to Abbot and Lonescu (1967) is commonly used with staggered grid models. With this scheme, numerical approximations to the mass equation are centred on each water level *h* grid point, while numerical approximations to the momentum equation are centred on each discharge *Q* grid point. The Preissmann (1961)Preissmann (1961) scheme is generally used in non-staggered grid models. With this scheme, the numerical approximations to both the mass and momentum equations are centred on the mid-point between each combined water level *h* and discharge *Q* grid point.

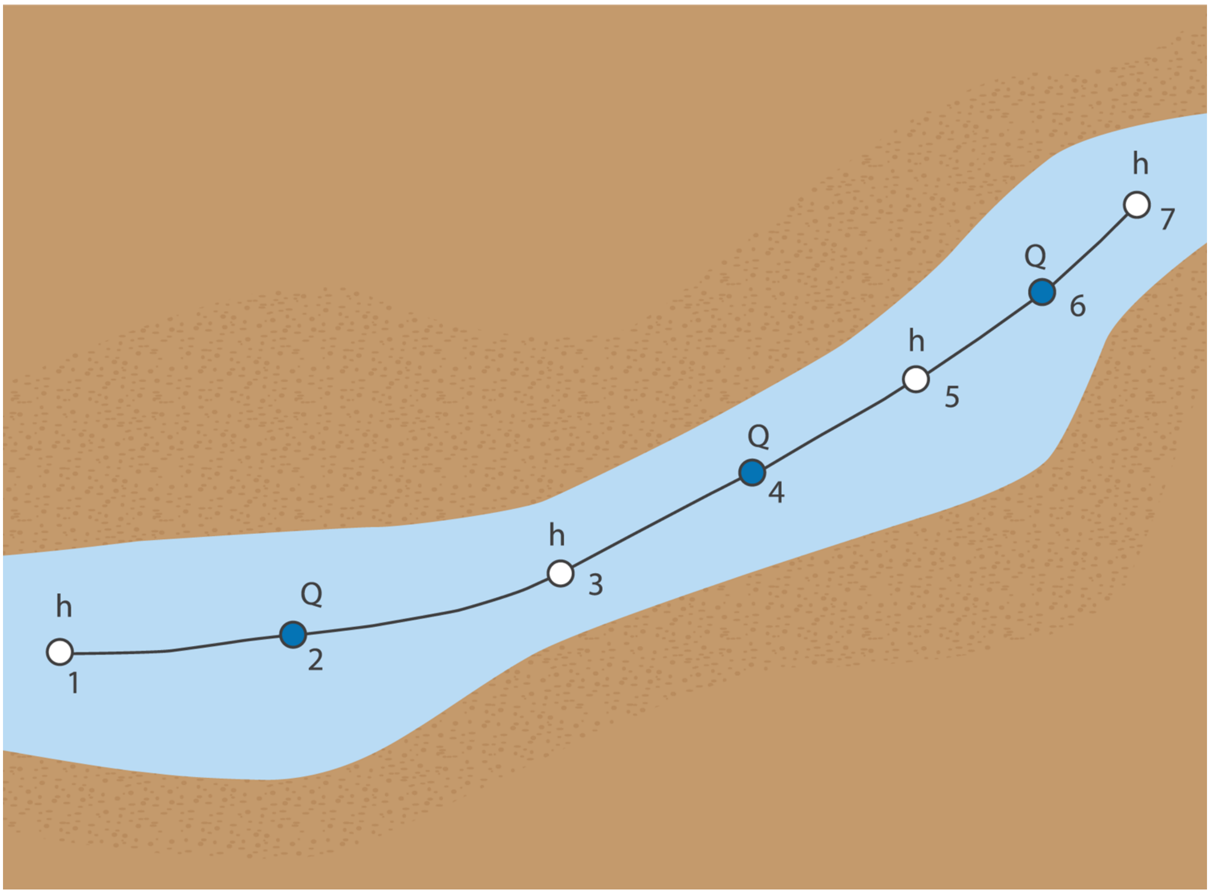


Figure 6.4.4. The Computational Grid

The staggered Abbot and Lonescu (1967) scheme has some advantages with the way in which structures can be incorporated. This is discussed briefly below. By comparison, the non-staggered Preissmann (1961) scheme has the advantage that the model grid size can be varied from one grid point to another with no loss of numerical accuracy.

Initial Conditions

Before starting a model simulation, initial water level *h* and discharge *Q* values must be specified at each water level and discharge point, irrespective of whether a staggered or non-staggered grid is being used. Depending upon the particular modelling package being used, these initial values can be obtained from:

- User specified values;
- Hot start conditions obtained from the results of an earlier model run;
- Internally generated “auto-start” conditions computed using an assumed initial steady state flow solution; and
- A combination of user defined and auto-start conditions (e.g., using user defined conditions in initially dry flood plains).

Boundary Conditions

The main input to the model is generally provided by the boundary conditions. These must be specified at each upstream or downstream open boundary. In most flood studies the upstream boundary conditions are generally specified as discharge hydrographs (typically computed by a hydrologic model). Corresponding downstream boundary conditions are generally specified as tail water levels (either constant or time varying), or through some means of relating the model discharge to a corresponding water level. This can be done by specifying a rating curve, where the discharge is explicitly linked to the water surface elevation, or through the use of the kinematic wave approximation at the boundary. The latter effectively links the discharge to the water surface elevation through the Manning equation. In tidal estuaries the downstream boundary will generally be specified in terms of a fixed or time-varying tidal elevation.

Model Structures

1D hydraulic models typically incorporate a range of structure formulations for including flow control structures within a model. These may include:

- Weirs: for describing flows over weirs, levees, road and rail embankments and overtopping of bridges, etc. A range of weir formulations may typically be available for describing different weir characteristics (e.g., broad-crested or sharp-crested) and different flow combinations (e.g., free overflow or drowned flow). Special weir formulations may also be included through which user-defined flow relationships can be specified.

- Culverts: for describing flows through culverts, bridges and pipes. Different formulations are used for a range of different upstream and downstream controlled flow conditions. User-defined culvert relationships can also be used.

- Regulating structures: where flows at structures such as gates or pumps can be specified as a level or discharge at another point in the model.

For simplicity, the remianing discussion has been limited to the use of staggered grid models. In these models, structures are located at discharge grid points, and water level grid points (and therefore cross-sections) must be specified immediately upstream and downstream of the structure, as shown in Figure 6.4.5. For these cases, the momentum equation at the discharge grid point is replaced by a structure equation. Flow through the structure then becomes a function of the upstream and/or downstream water levels, depending on the flow conditions (e.g., free overflow or drowned flow for weirs, or inlet or outlet control for culverts).

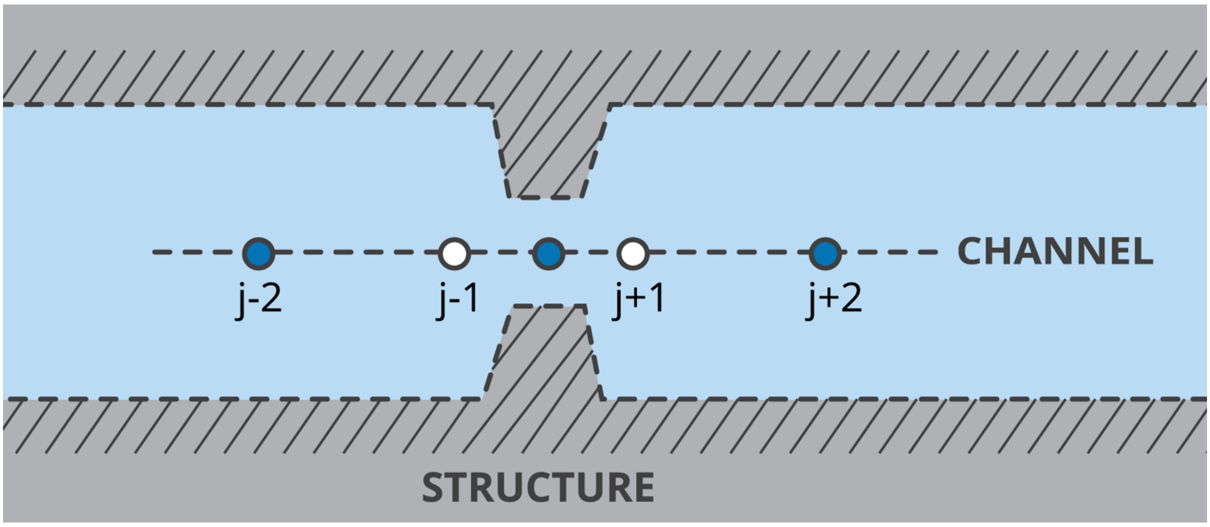


Figure 6.4.5. Schematisation of a Channel and Structure Grid

Multiple structures can be defined at a single discharge point. For example, a bridge which can be overtopped can be described by the combination of a culvert, for normal flow conditions, and a weir, for overtopping flows.

Floodplain Flows

The treatment of floodplain flows can be very different depending on the main characteristics of the river channel and floodplain system. For floodplains that drain naturally to the river channel, as shown in Figure 6.4.6(a), the water level can be the same in both the river and the floodplain. Under these conditions the effects of storage and flow along the floodplain can be included directly in a single combined river channel and floodplain branch.

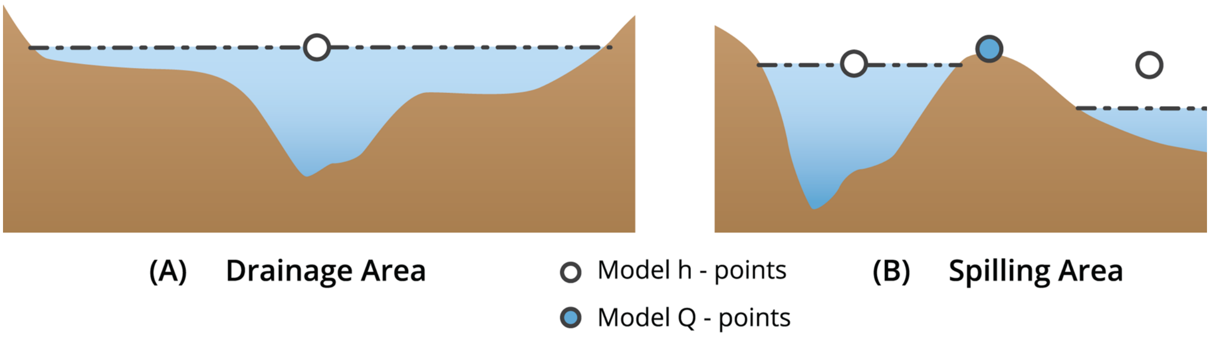


Figure 6.4.6. Different Types of Flood Plain Flow

For floodplains where the river flows spill out over a natural or man-made levees, as shown in Figure 6.4.6(b), the water level in the floodplain can be very different to the water level in the main channel. Under these conditions, the effects of storage and flow along the floodplain should be incorporated in a separate model branch. Flow exchange between the river channel and floodplain branches can then be controlled by a link branch with a broad-crested weir representing the levee bank.

A simple river channel/floodplain branch system is schematised in Figure 6.4.7. Combinations of multiple river channel and floodplain branches can be used to describe the flows in quite complex river and floodplain systems.

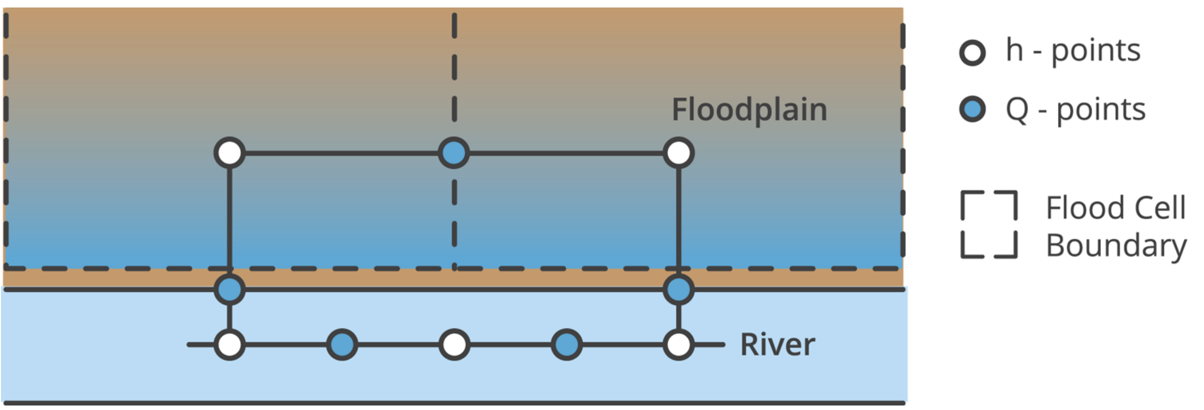


Figure 6.4.7. Schematisation of a Simple River Channel/Floodplain Branch System

4.6.4. Data Requirements

To construct a 1D model of a river and floodplain system, it is necessary to have the following data:

- A knowledge of the main flow paths throughout the system.
- Survey cross-sections at representative locations across the river channel and floodplain.
- Survey levels along flow controls such as levees, weirs and road embankments.
- Survey of control structures such as bridges and culverts.
- Survey cross-sections immediately upstream and downstream of branch junctions, and flow control structures.

Additionally, data on historic flood levels is required for model calibration. Ideally, a 1D model should be calibrated against water level hydrographs at various locations throughout the model area. This will provide a measure of how well the model can reproduce the timing of a flood and the shape of the hydrograph. In many cases, however, water level hydrographs of historic flood events do not exist. Consequently, most models are calibrated against peak water levels surveyed after a flood.

Figure 6.4.8 shows a plan view of a branched 1D model of the Lindsay and Murray River system in northwest Victoria and southwest New South Wales. The 1D model is able to provide a good description of the flows along the main channels of the Lindsay River in the south, the Murray River to the north, and a range of older flow paths across the flood plain, including Mullaroo Creek. Figure 6.4.9 shows a water surface profile along one particular branch of the model.

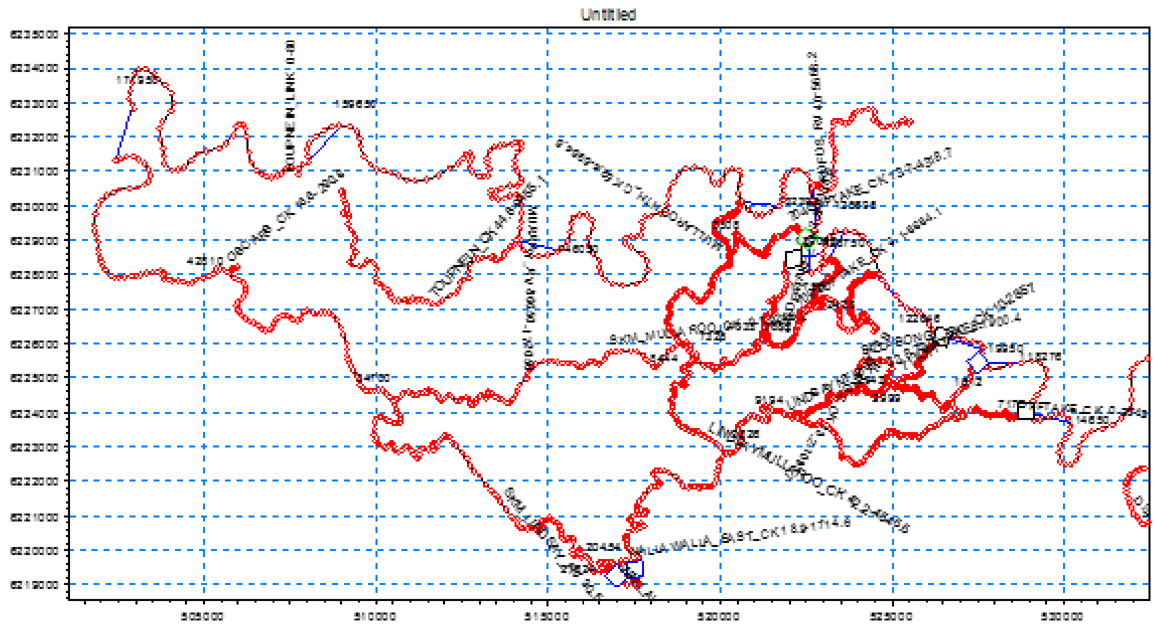


Figure 6.4.8. A Branched 1D Model Network of the Lindsay and Murray River Systems

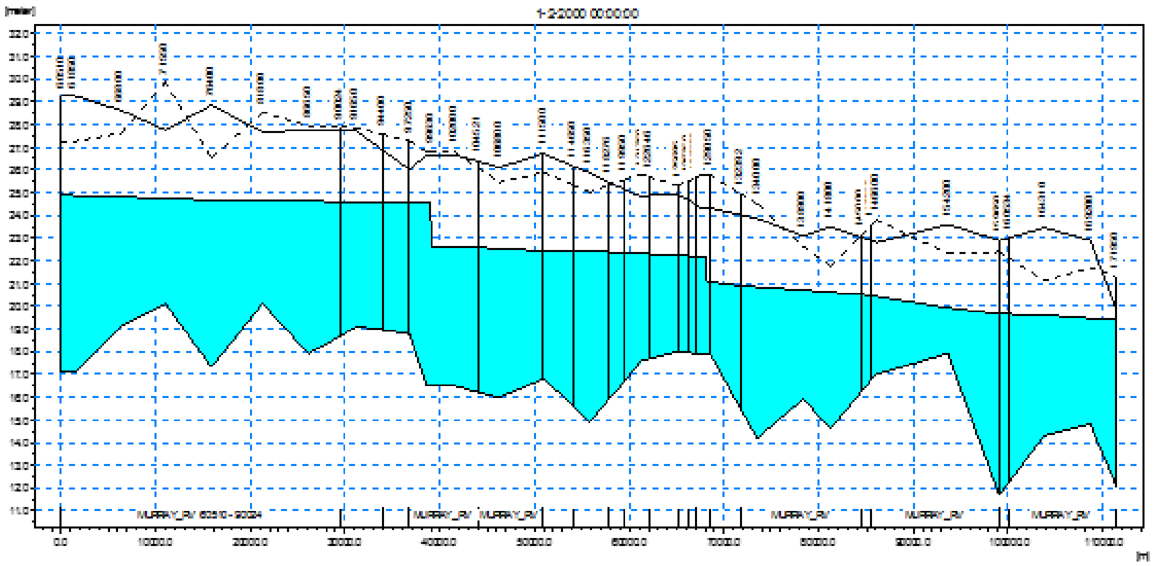


Figure 6.4.9. Lindsay River

4.6.5. Advantages and Limitations

The main advantages of a 1D model are that the model simulations are relatively quick to run computationally (i.e., relative to full 2D models), and that the main river channels can be well defined by survey cross-sections. As such, 1D models are well suited to modelling long reaches of river systems, and to modelling of river and floodplain systems where the flow paths are reasonably well defined.

The main disadvantages of 1D models are that they are based on cross-sectionally averaged one-dimensional equations of motion. As such:

- The floodplain flow paths must be pre-determined by the user.
- The flow paths are by definition 1D and no information is available on the distribution of flows within individual flow paths.
- Losses due to two-dimensional effects such as bends, flow separations, etc., must all be lumped into the bed-friction parameter (making detailed calibration essential).
- There can be problems in interpreting model results for mapping flood extents and depths of inundation.

As such, 1D models are not well suited to modelling the details of flood flows within the floodplain.

4.7. 2D Unsteady Hydraulic Models

Due to the limitations 1D models, there was a trend in the early 1990s towards the use of full 2D models in urban flood studies and other flood studies where the details of the flow distribution across the floodplain became important. (See for example Bishop et al. (1995)). This trend continued through to the early 2000s. By this time 2D modelling had become a virtually a standard for most rural and urban flood modelling studies.

4.7.1. Model Equations

2D unsteady hydrodynamic models are based on the numerical solution of the depth-averaged shallow water wave or long wave equations. These equations describe the conservation of mass and momentum in two horizontal dimensions *x* and *y*. In a form used in many 2D flood models, these equations can be expressed as:

Mass

$$\frac{\partial \varsigma}{\partial t} + \frac{\partial (d\,.u)}{\partial x} + \frac{\partial (d\,.v)}{\partial y} = \text{Sources} - \text{Sinks} \tag{6.4.12}$$

x-Momentum

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + g\frac{\partial \varsigma}{\partial x} = -\frac{gu\sqrt{u^2+v^2}}{C^2 d} + E\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2}\right) + \text{Source/Sink} \tag{6.4.13}$$

y-Momentum

$$\frac{\partial v}{\partial t} + v\frac{\partial v}{\partial y} + u\frac{\partial v}{\partial x} + g\frac{\partial \varsigma}{\partial y} = -\frac{gv\sqrt{u^2+v^2}}{C^2 d} + E\left(\frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2}\right) + \text{Source/Sink} \tag{6.4.14}$$

This coupled system of equations provides the three equations necessary to solve for the three dependent variables; ς the free surface elevation, *u* the velocity in the *x* direction and *v* the velocity in the *y* direction. It is noted that in this secton and the following sections, *d.u* and *d.v* refer to the depth *d* multiplied by the *x*-velocity component *u* and the *y*-velocity component *v*, respectively.

The mass and momentum equations include sources and sinks for describing the effects of localised inflows and outflows. The *x* and *y* momentum equations also include a quadratic Chezy-type friction formulation and a simple eddy viscosity formulation (with eddy viscosity coefficient *E*). For practical modelling applications, the Chezy coefficient *C* can be related to the more usual (for Australian applications) Manning's *"n"* by the Strickler relation:

$$n = d^{1/6} C^{-1} \tag{6.4.15}$$

In some European models, the friction coefficient is sometimes specified in terms of Manning's *"M"*, where:

$$n = M^{-1} \tag{6.4.16}$$

For modelling large expanses of open water, such as in lakes and estuaries, these 2D model equations can be extended to include the effects of wind shear and/or Coriolis forces.

4.7.2. Assumptions

In the derivation of the 2D model equations, it has been assumed that:

- The flow is incompressible
- The pressure is hydrostatic (that is, vertical accelerations can be neglected and the local pressure is dependent only on the local depth).

- The flow can be described by continuous (differentiable) functions of ς, *u* and*v* (that is, the flow does not include step changes in ς, *u* and *v).*
- The flow is two-dimensional (that is, the effects of vertical variations in the flow velocity can be neglected).
- The flow is nearly horizontal (that is, the average channel bed slope is small).
- The effects of bed friction can be included through resistance laws (e.g., Manning's equation) that have been derived for steady flow conditions.

4.7.3. Other Forms of the Equations

The equations presented in Book 6, Chapter 4, Section 7 form the basis of many of the 2D numerical models currently in use. Another form of the equations that is finding increasing popularity is the so-called "conservation law" form described for example by Abbott (1979). In this form of the equations, the depth average momentum "fluxes" (*d.u* and *d.v* in the *x* and *y* directions, respectively) are used as the time dependent velocity variables in the momentum equations. In a form similar to that considered above, , the resulting conservation law form of the shallow water equations can be expressed as:

Mass

$$\frac{\partial\varsigma}{\partial t}+\frac{\partial(d.u)}{\partial x}+\frac{\partial(d.v)}{\partial y}=\text{Sources}-\text{Sinks} \tag{6.4.17}$$

x-Momentum

$$\frac{\partial(d.u)}{\partial t}+\frac{\partial}{\partial x}\left(d.u^2+\frac{1}{2}g.d^2\right)+\frac{\partial(d.u.v)}{\partial y}=-\frac{gu\sqrt{u^2+v^2}}{C^2d}+E.d\left(\frac{\partial^2u}{\partial x^2}+\frac{\partial^2u}{\partial y^2}\right)+\text{Source/Sink} \tag{6.4.18}$$

y-Momentum

$$\frac{\partial(d.v)}{\partial t}+\frac{\partial(d.u.v)}{\partial x}+\frac{\partial}{\partial y}\left(d.v^2+\frac{1}{2}g.d^2\right)=-\frac{gv\sqrt{u^2+v^2}}{C^2d}+E.d\left(\frac{\partial^2v}{\partial x^2}+\frac{\partial^2v}{\partial y^2}\right)+\text{Source/Sink} \tag{6.4.19}$$

This is effectively the form of the equations that provides the basis of the finite difference solution procedure used by DHI (2005). A distinct advantage of the conservation law formulation of the equations is that, when the depth average momentum fluxes *d.u* and *d.v* are used as the dependent velocity variables, the mass equation becomes linear and, barring coding errors, the numerical solutions should remain mass conservative.

4.7.4. Numerical Solution Procedures

Development of numerical solution procedures for the shallow water wave equations is an active area of on-going research. There are three main approaches that have used; finite difference, finite element and finite volume techniques. The main characteristics of, and the differences between, these methods are discussed in detail by, for example, Sherwin and Peiro (2005). The main features of each of these approaches are outlined below.

Finite Difference Methods: With finite differences, the differential forms of the equations of motion described above are used directly. The dependent water level and velocity variables are defined at individual grid points on a structured rectilinear or curvilinear grid. Spatial derivatives are approximated by taking arithmetic differences between the dependent variables in adjacent grid points, while time derivatives are approximated by taking arithmetic differences between the variables at different time levels. The main advantages of finite difference methods are that they are relatively simple to implement and are easy to use. The main disadvantage is that complex geometries cannot be readily resolved without the use of fine scale grid resolution. Finite difference techniques for providing solutions to the shallow water wave equations have been described by, for example, Abbott (1979); Stelling (1984); and Abbott and Basco (1989).

Finite Element Methods: With finite elements, the equations of motion are transformed into integral formulations. Weighting or trial functions are introduced and the resulting equations are then solved numerically over an mesh of regular or irregularly shaped elements. The shapes of these elements are typically triangles and/or quadrilaterals, but can take other forms. Finite element methods provide solutions that are smooth and continuous over each element and which have matching values at the interfaces between elements. One of the main advantages of finite elements is that the integral formulation of the equations does not require a structured mesh. This makes it possible to use an unstructured flexible mesh which can be aligned with the local flow direction, or which can provide greater resolution in particular areas of interest. The main disadvantage with flood modelling with standard finite element techniques is that mass is not necessarily conserved. Finite element techniques for providing solutions to the shallow water wave equations have been described by, for example, Connor and Brebbia (1976); and Zienkiewicz et al. (2014).

Finite Volume Methods: Finite volume methods are similar to finite elements in that they use integral formulations of the equations of motion and can be solved over an unstructured flexible mesh of irregularly shaped (typically triangular and/or quadrilateral) mesh cells. The main differences are that finite volume methods use integral forms of the conservation law form of the equations of motion and each mesh cell is treated as a control volume represented by volume-averaged values of the conserved variables (mass and momentum). The rates of change of these conserved variables are derived by integrating the cell-interface fluxes. A key step in these methods involves calculating the numerical fluxes at the cell interfaces. As for finite elements, the integral formulation of the equations used in finite volumes makes it possible to use unstructured flexible meshes that can be aligned with the local flow directions and which can provide greater resolution in particular areas of interest. The main advantage of finite volumes over finite elements is that mass is conserved. Finite volume techniques for providing solutions to the shallow water wave equations have been described by, for example, LeVeque and Bale (2012).

Finite difference methods have been used extensively in the historical development of numerical 2D flood modelling practice (e.g., Bishop et al. (1995); Stelling et al. (1998); McCowan et al. (2001) and Syme (2001)) and are still used widely today. Early flexible mesh flood model development was mostly carried out using finite element techniques (e.g., King and Roig (1988)). However, due to potential mass conservation issues, more recent flexible mesh model development has focussed more on finite volume techniques. In this respect, it is noted there are also hybrid approaches that combine the finite element and finite volume schemes. Further, the conservation law formulation of the shallow water wave equations can be used to develop finite difference solution procedures that apply the finite volume approach to a structured rectilinear or curvilinear grid.

Explicit vs Implicit Solution Procedures

The way in which these different solution procedures move forward in time can be described as being either “explicit” or “implicit”. In this respect it is noted that the “Courant” number *Cr* is a key parameter for defining the differences between explicit and implicit solution procedures. The Courant number expresses the number of grid or flexible mesh cells that flow information can travel in one timestep. The Courant number *Cr* can be defined as:

$$C_r = \left[\frac{(u + \sqrt{gd})\,.\,\Delta t}{\Delta x}\right] \tag{6.4.20}$$

With an explicit solution procedure, the solutions to the water surface elevations and flow velocities at the new timestep are computed directly (explicitly) as a function of the known values at the old timestep. Explicit schemes tend to be computationally efficient, but have a stability constraint that information can only travel a maximum of one grid/mesh element in a single timestep. That is, that the Courant number must always be less than or equal to one (i.e., $Cr \leq 1$). This provides a stability constraint on the timestep Δt, that is commonly called the “Courant” stability criterion, where:

$$\Delta t_{\max} \leq \left[\frac{\Delta x}{(u + \sqrt{gd})}\right]_{\min} \tag{6.4.21}$$

By comparison, in an implicit solution procedure, the water surface elevations and flow velocities at the new timestep are expressed as a combination of both the known values at the old timestep and adjacent unknown values at the new timestep. As a result, the solutions at one grid/mesh element are linked to those in the neighbouring cells. These solutions are, in turn, linked to those in their neighbouring cells, and so on. In this way, the solutions to the discrete numerical approximations to the mass and *x* and *y* momentum equations for each grid/mesh cell are linked “implicitly” to those in every other cell over the entire model domain. This approach allows flow information to travel much further than one grid point per timestep. As a result, the Courant stability criterion does not apply to implicit solution procedures and model time steps can be determined more by accuracy requirements rather than by stability constraints.

ADI Solution Procedures

Most of the early 2D model developments were for coastal and marine applications where the use of high Courant numbers was found to provide significant computational advantages. Following Leendertse (1967) many of these models used finite difference schemes which used what is known as an “alternating direction implicit” or ADI algorithm. This approach involves the use of a series of implicit 1D sweeps alternating along x-grid lines and y-grid lines which is much simpler to implement than a fully two-dimensional implicit solution procedure.

The ADI approach can be shown to be independent of the Courant stability criterion and has been used extensively in two-dimensional flood modelling (e.g., Stelling et al. (1998); McCowan et al. (2001) and Syme (2001)). It should be noted, however, that the ADI approach is not directly equivalent to a fully two-dimensional implicit scheme. Although the timestep is not subject to the Courant stability criterion, it is subject to accuracy constraints, particularly when the solution involves flow in relatively narrow channels at an angle to the grid (Benque et al., 1982). In practice, the timestep should be selected such that the Courant number is less than the minimum number of grid cells used to describe the width of a channel. In many cases this tends to restrict the timestep such that the Courant number is of order $Cr \approx 1$ in narrow channels, although higher Courant numbers can occur in other parts of the model.

Parallel Processing and GPUs

With the advent of multiple-core processors and graphics processing units (GPUs) it became possible to carry out multiple computations in parallel. This has provided significant increases in computational speed for models with code that could be readily “parallelised”. The direct explicit relationships between water surface elevations and flow velocities at the new timestep and the corresponding values at the old time step make explicit solution procedures particularly well suited to parallel processing. As a result, much of the recent development in 2D flood modelling has focussed on explicit finite volume numerical solution procedures (need references here). These models have the capability to increase the effective computational speed by one to two orders of magnitude, depending upon what features are, or are not, included in the model.

4.7.5. Discretisation Errors

Irrespective of the numerical method being used, the modeller should be aware that the resulting Numerical Model is only a numerical approximation to the Equations of Motion (the Mathematical Model). Discretisation errors are introduced when the continuous mathematical equations have to be approximated by discrete (discontinuous) arithmetic expressions that can then be solved by the computer. These errors are called “truncation” errors. They are different to computer “round-off” errors, and can have significant implications on the accuracy of a model’s results.

To illustrate this effect, a simple first-order finite difference approximation to the $u\delta u/\delta x$ “convective momentum” term in the x momentum equation has been examined more closely. This term is to be discretised on a square “staggered” grid similar to that shown in Figure 6.4.10.

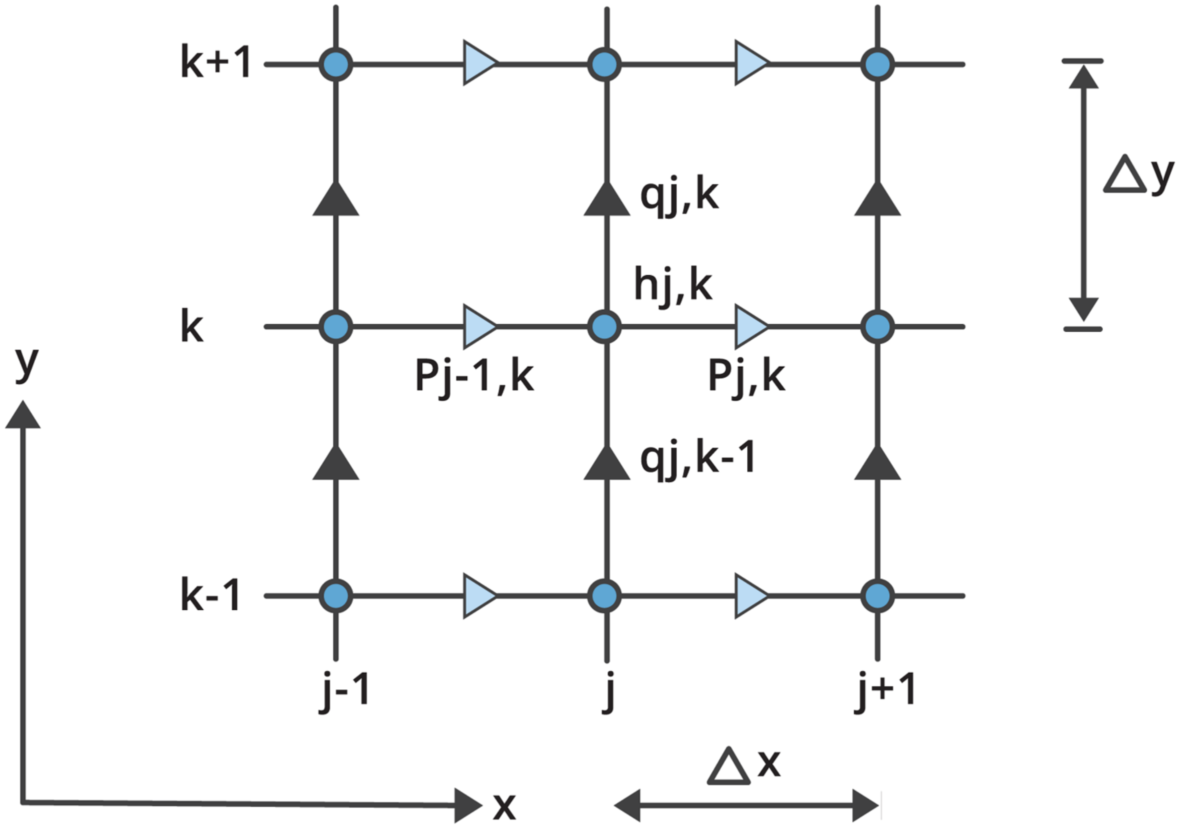


Figure 6.4.10. Example of a Computational Grid

For a given timestep *nΔt* and *x* grid line at *kΔy*, the term *uδu/δx* can be approximated by:

$$u\frac{\partial u}{\partial x} \approx u\frac{\Delta u}{\Delta x} = u_j\frac{(u_{j\Delta x} - u_{(j-1)\Delta x})}{\Delta x} \tag{6.4.22}$$

With this approximation, the continuous derivative of the velocity *u* with respect to *x* is approximated by the linear gradient of the velocity between the grid point at which the derivative is to be taken, and the grid point immediately upstream. As a result, this approach is often termed "upwind" differencing. The errors associated with this approximation can be determined by using Taylor's Series to expand the terms in the right hand side of this equation in terms of the velocity *u* velocity at the centre-point, *jΔx*. This results in the following expression:

$$u\frac{(u_{j\Delta x} - u_{(j-1)\Delta x})}{\Delta x} = u\frac{\partial u}{\partial x} - u\frac{\Delta x}{2}\frac{\partial^2 u}{\partial x^2} + u\frac{\Delta x^2}{6}\frac{\partial^3 u}{\partial x^3} - u\frac{\Delta x^3}{2}\frac{\partial^4 u}{\partial x^4} + \dots\dots\dots \tag{6.4.23}$$

This shows that, the discrete finite difference approximation is equal to the original continuous partial differential term *u.δu/δx*, but with additional second, third, fourth and higher order truncation error terms. Further, it can be seen that the truncation error includes a second order term which is of the same form as one of the eddy viscosity terms in the *x*-momentum equation. That is, upwinding of the convective momentum term *u.δu/δx* can be seen to be equivalent to introducing a numerical eddy viscosity term with, in this case, an eddy viscosity coefficient of:

$$E_N = u\frac{\Delta x}{2} \tag{6.4.24}$$

This numerical eddy viscosity coefficient is grid size and flow velocity dependent. For typical floodplain flow conditions and model dimensions, the numerical eddy viscosity introduced by first-order upwind differencing can be an order of magnitude or more greater than the corresponding physically realistic values. Issues associated with first-order upwind differencing of the convective momentum terms are discussed in detail by Leonard (1979).

Similar truncation error terms can be developed for numerical approximations to the other terms in the mass and momentum equations. The properties of these truncation errors can have a significant effect on the accuracy and stability of the numerical procedures being used.

Accuracy and Stability

From the above, it can be seen that first-order schemes have second-order truncation errors that are proportional to the grid or mesh size *Δx* (or timestep *Δt* for time derivatives). In a similar manner, it can be shown that second-order schemes have third-order errors that are proportional to the square of the grid or mesh size *Δx2* (or of timestep *Δt2*). If the grid or mesh size *Δx* and timestep *Δt* are treated as fractions of representative length and time scales of the flood under consideration, the truncation errors of a second-order scheme can be shown to reduce quadratically with decreasing grid size and timestep. That is, the finer the grid or mesh size or shorter the timestep, the smaller the numerical truncation errors.

In general, it can be said that first-order schemes are “diffusive”. That is, they tend to damp out sharp gradients in water levels and flow velocities. The artificially high levels of numerical eddy viscosity, discussed above, help to smooth out flow irregularities and make the model calculations very stable. However, the unrealistically high levels of smoothing results in the suppression of flow separations and eddy formation and makes it impractical to compute channel/overbank interactions, where the specification of appropriate eddy viscosity coefficients becomes important.

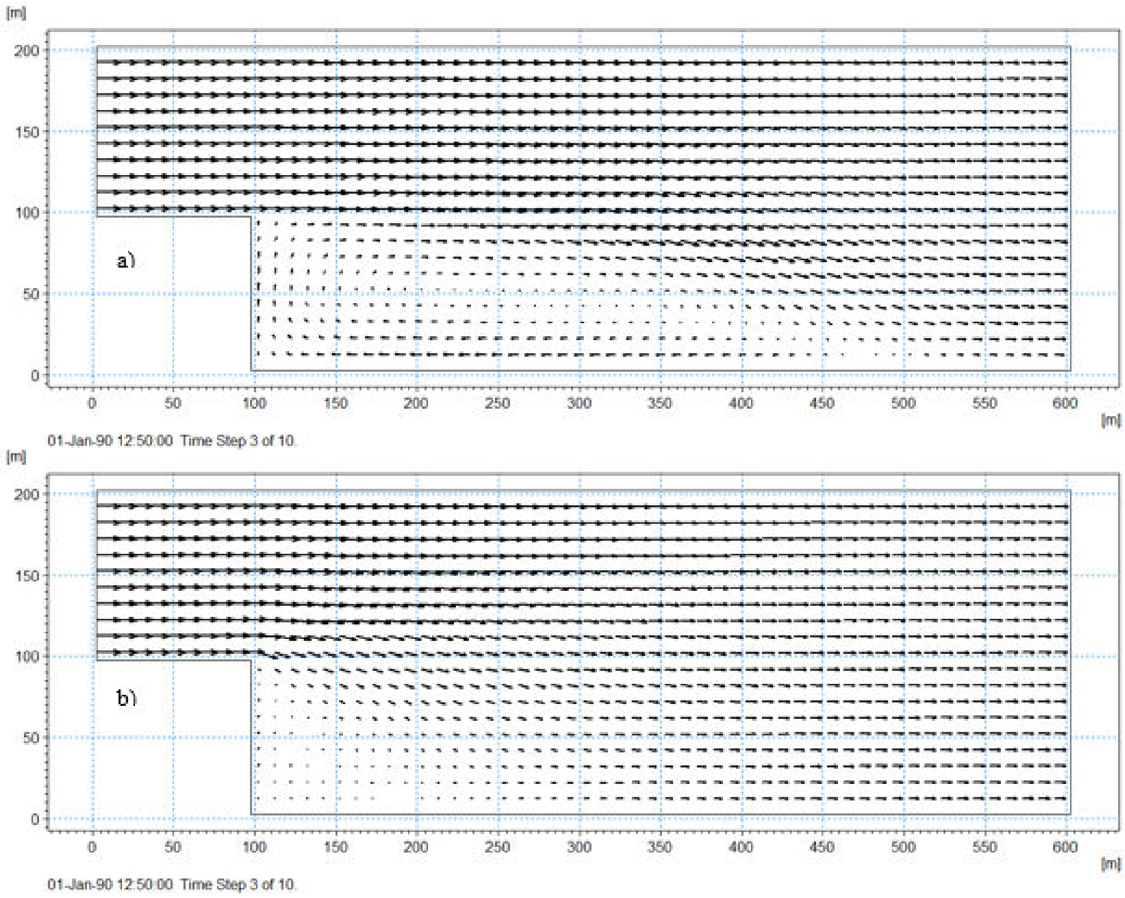


Figure 6.4.11. Figures showing (a) flow separation and eddy formation, and (b) suppression of flow separation with numerical eddy viscosity form a first order upwind scheme

By comparison, second-order schemes are “dispersive”. That is, high frequency components of flood flow can travel at different speeds. This is not normally a problem with the propagation of flood waves which tend to be very long (low frequency) with respect to the model grid or mesh size and timestep. As a result, second-order schemes are generally well suited to modelling the wave propagation properties of flood flow. They are, however, not as well suited to modelling high velocity flows and flows where there are strong velocity gradients. Under these conditions, the convective momentum terms in the momentum equations become more important and the use of second-order schemes can result in artificial “zig-zagging” of the flow and non-linear instabilities (Abbott and Rasmussen, 1977). As a result, Leonard (1979) advocates the use of higher third-order schemes for modelling transport dominated flows.

4.7.6. Model Applications

Most of the 2D flood models in common use are based on the numerical solution of the full mass and momentum equations, described above. This generally works well in straight-forward flow situations. However, many of the commonly used solution procedures can have stability issues when modelling high velocity flows, and most, if not all, become ill-conditioned when modelling super-critical flows. This is generally approached by either deliberately introducing numerical stabilising terms, or by using simplified forms of the momentum equation to describe the flow.

Stelling et al. (1998) describe a finite difference model based on first-order upwind differences. The model provides smooth stable solutions for high velocity flows. However, as discussed above, the high levels of numerical diffusion (numerical eddy viscosity) may result in reduced accuracy under flow conditions where the use of more physically realistic values of eddy viscosity may be required. To avoid this problem, McCowan et al. (2001) use a second order scheme for the main part of the computation, but gradually introduce first-order upwinding in localised areas where the Froude number exceeds Fr = 0.25. In both cases, the numerical diffusion introduced in this way is sufficient to maintain the numerical computation through regions of super-critical flows.

As an alternative, BMT WBM (2008) uses the kinematic wave approximation to describe flow conditions in regions with super-critical flow. This approach is reasonable for most flooding applications as super-critical flows are upstream controlled and are normally friction dominated. However, whenever simplified forms of the equations are used, it is important for the modeller to understand their limitations, and care may be required in interpreting the results, particularly in transition areas.

It is noted that the full set of the equations should always be used for describing flows in relatively flat river reaches, and in regions of relatively flat, deep water such as in estuaries and lakes

Clearly, there is a wide range of models with quite different solution procedures with varying orders of accuracy available to the numerical flood modeller. It is important for the modeller to be aware of the type of solution procedure being used, and of any constraints that that this may impose on the timestep, model accuracy, and on the way in which the computation is carried out.

4.7.7. Site Specific Model Development

To this point the discussion has focussed on the different types and properties of 2D Generic Numerical Models. The process of developing a Site Specific 2D Model is discussed at length by Engineers Australia (2012) and only an overview of the key stages of model development have been summarised here. They include:

- Selection of the Generic 2D Numerical Model to be used
- Model schematisation (model domain, cell size, time step)
- Key model inputs (topography, bed resistance, eddy viscosity)
- Inclusion of flow controls and hydraulic structures
- Initial Conditions
- Boundary Conditions
- Hydraulic Structures

Generic Numerical Model Selection

In practice, the selection of the Generic Numerical Model or modelling software package to be used can be limited to the software available within the modeller's organisation or, in the case of a consulting project, may even be specified externally by the client. Where a choice is available some of the key factors to be considered in selecting the particular software to be used include:

- The skill, experience and personal preferences of the modeller;
- The choice of finite difference or finite volume solution procedures;
- The choice of the use of a structured grid or an unstructured flexible mesh: and
- The availability of any particular features required in the modelling.

Modeller Experience: The skill level and experience of the modeller is an important factor in determining the success of any modelling exercise and should be taken into account when selecting the software package to be used.

Finite Difference vs Finite Volume: A big advantage with finite volume models and finite difference models based on finite volume techniques is that, barring coding errors, they conserve mass. This is a very important property in a flood model, as any loss or gain of mass caused by the approximations made in the numerical solution procedure can invalidate the model results. It should be noted that this does not preclude the use of finite difference models. It only means that care should be taken when using finite difference models to ensure that there is no significant loss or gain of mass during the model simulations.

Structured Grid vs Unstructured Flexible Mesh: Until relatively recently, most of the commercially available flood models operated with a structured rectilinear (square or rectangular) grid. The advantages of these structured grid models are that they are easy to set-up and can be very computationally efficient. A disadvantage is that they do not provide a good description of flows along relatively narrow channels aligned at an angle to the grid, as shown in Figure 6.4.12(a). To get adequate resolution of these flow paths may require reduction in the model grid size, as shown in Figure 6.4.12(b), with a corresponding significant increase in computational requirements.

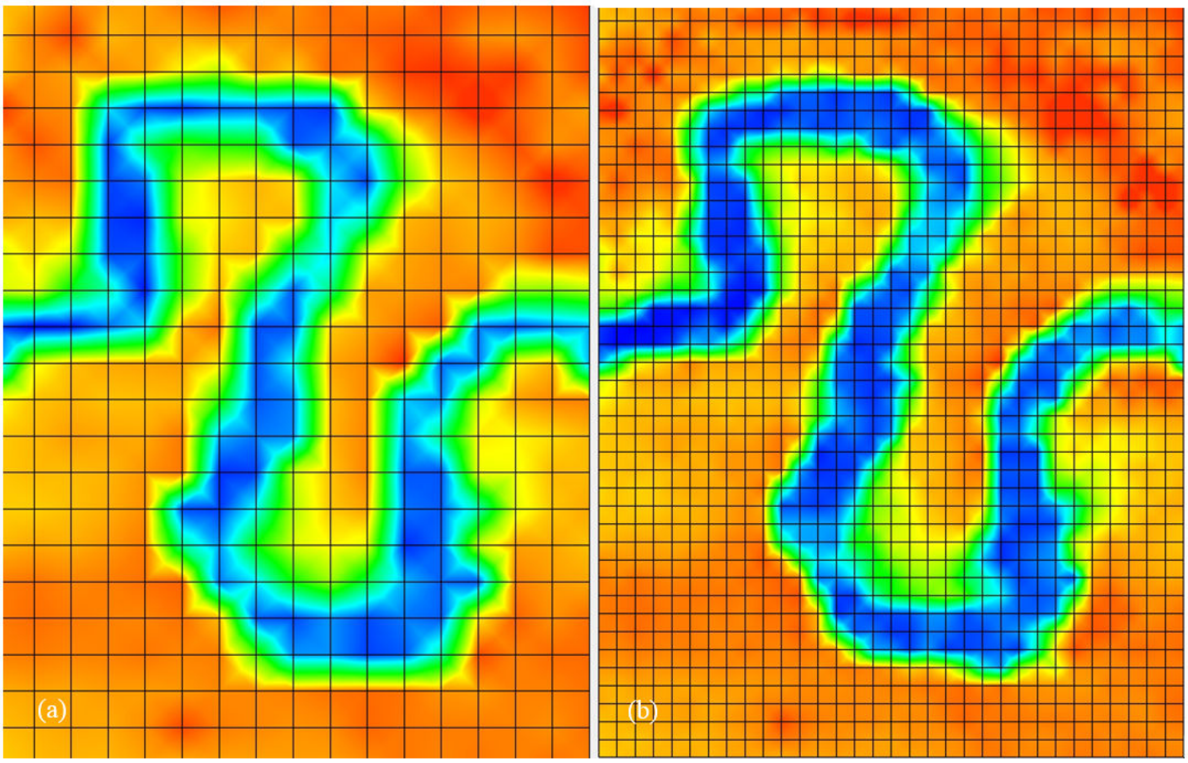


Figure 6.4.12. Showing (a) poor resolution of a sinuous flow path on a coarse square grid, and (b) improved, but still relatively poor resolution at a finer grid scale.

By comparison, flexible mesh models typically use a combination of unstructured triangular and quadrilateral mesh cells as shown in Figure 6.4.13(a). These can be aligned more closely with the main flow paths allowing good resolution with larger mesh cells than with a corresponding structured grid model, as shown in Figure 6.4.13(b). Larger mesh sizes can sometimes also be used in regions with relatively uniform flow and away from particular areas of interest. The main disadvantages of the flexible mesh models are that they take more time and skill to set up the initial model mesh and, due to their increased complexity, they generally take more computer time to run, even with the use of larger mesh sizes.

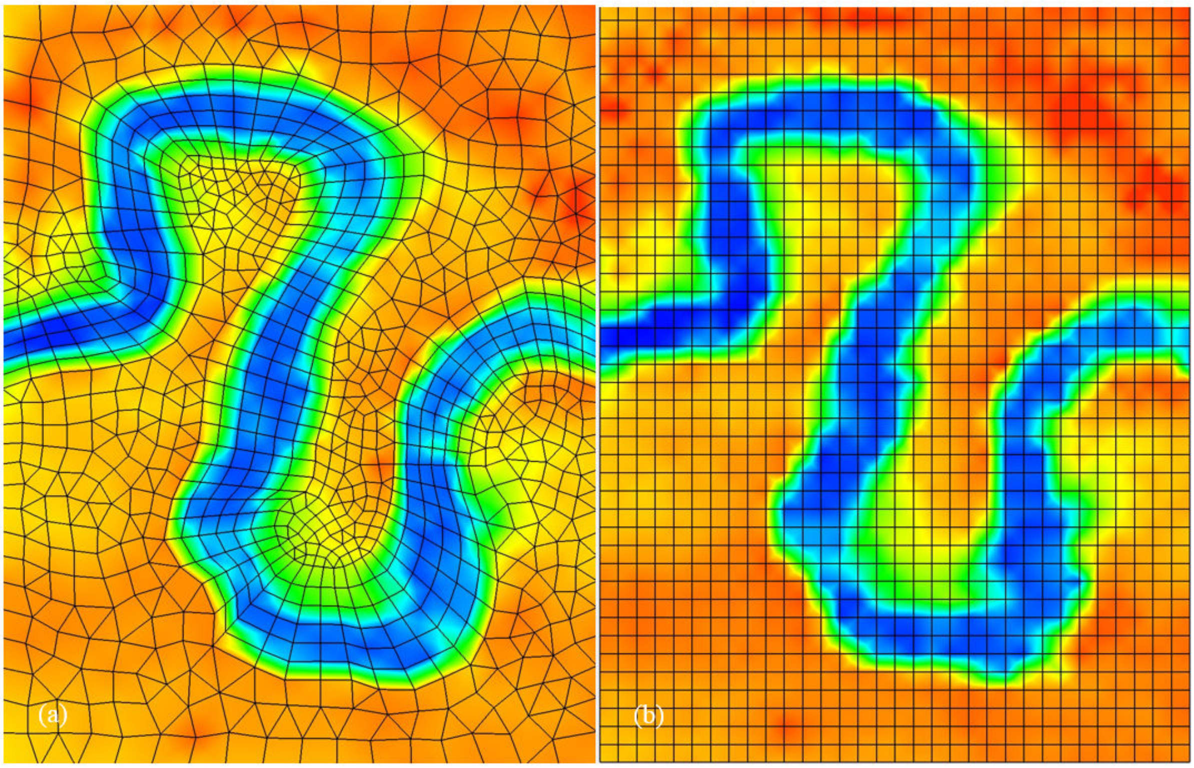


Figure 6.4.13. Showing (a) improved resolution of a sinuous flow path using a flexible mesh, relative to (b) a fine square grid.

Model Schematization

The first task in schematizing a model is to decide on the model domain. This is the area to be included within the model. It should include the main area of interest and should extend out far enough to include all the areas that are likely to be inundated in the most extreme flood to be considered. It should also extend sufficiently upstream and downstream such that any irregularities in the flow at the model boundaries will not affect the model results in the main area of interest. Wherever possible, the model boundaries should be located in areas of relatively uniform flow.

For structured grid models, a square or rectangular is overlaid on the model domain. This grid should, where possible, be aligned with the main flow direction. The grid size should be selected to provide adequate resolution of the main features to be modelled (e.g., channels, levees, bridges, etc). Care should be taken to ensure that the main flow paths to be considered can be adequately described, particularly when they are aligned at an angle to the grid.

For unstructured flexible mesh models, a model mesh covering the model domain must be developed. This mesh may typically be formed using triangular mesh cells, or a combination of triangular and quadrilateral mesh cells, although some software allows the use of other shapes as well. As seen in Book 6, Chapter 4, Section 7, one advantage of the flexible mesh approach is that the mesh cells can be aligned to the main flow paths irrespective of their orientation. Another is that finer cell sizes can be used to provide greater resolution in particular areas of interest. Conversely, larger cell sizes can be used to reduce computational requirements in less important areas.

Clearly, the smallest feature that can be resolved will be one grid or mesh cell wide. However, if realistic simulation of flow separation and eddy formation behind structures such as bridge abutments is required, then these structures will need to be resolved by a minimum of 6 to 8 grid or mesh cells.

The selection of the appropriate cell size is generally a trade-off between model resolution and computational requirements. In this respect it is noted that the computational time required by a model is roughly proportional to the cube of the cell size. That is, halving the cell size could be expected to increase the computational time by a factor of 8.

The time step is generally set to the largest value that can be used without affecting the accuracy of the model results. For explicit models, the time step is set close to the maximum allowable under the Courant stability criterion, discussed in Book 6, Chapter 4, Section 7. Although implicit models are not affected by this constraint, their timestep is usually limited by accuracy requirements, and the optimum timestep is typically determined by sensitivity testing during model calibration.

Key Model Inputs

Some of the key inputs to a 2D model include: the model topography, bed resistance values and, where appropriate, the eddy viscosity formulation. These are discussed briefly below.

Topography: The model topography forms the basis of any 2D hydraulic model. It is the numerical analogue of the actual terrain over which the water flows. It comprises survey data (e.g., from a digital terrain model) that has been interpolated onto the model grid or mesh. Following the initial interpolation process, some degree of manipulation may be required to ensure that the main flow paths and flow controls, such as road embankments or levee banks, have been adequately resolved. For flow paths that are less than a few cell widths wide, some degree of schematization may be required to ensure that the model flow path provides a realistic description of the flow path conveyance. This is particularly true for structured grid models. Care taken in setting up the model topography can significantly reduce the amount of time required to calibrate a 2D model.

Bed Resistance: 2D models also require bed-friction coefficients to be specified for application to each computational cell. These are generally specified in terms of different Mannings “n” coefficients that are assigned to a range of different land-use categories. These can be determined on the basis of a combination of information gained through site inspections, from aerial imagery and from cadastral data.

Eddy Viscosity: Eddy viscosity is used in 2D models to represent the effects of turbulence and sub-grid scale processes. The use of appropriate eddy viscosity values is necessary in simulations where the realistic representation of flow separations and eddy formation, or of momentum transfers between the main channel and overbank areas are important. Depending on the software being used eddy viscosity coefficients can be described as constant values, spatially varying values, or values computed internally by a turbulence closure model. In this respect, it is noted that a “Smagorinsky-type” eddy viscosity formulation has been found to be suitable for describing the effects of sub-grid scale processes in many applications.

Initial Conditions

For any model computation to move forward in time there must be a known starting point. For numerical hydrodynamic models this starting point is known as the model "initial conditions". The initial conditions necessary for two-dimensional hydrodynamic models consist of water surface elevation ς and u and v velocity values at every grid/mesh element that is "active" at the start of the computation.

These values can be specified by:

- A "cold start", where initial estimates of the water surface elevation ς values are made, and the u and v velocity values are set to zero (i.e., no flow).
- A "hot start", where the initial water surface elevation ς, and u and v velocity values are specified from the results of a previous model simulation.

Cold Starts: In many flood modelling applications, the precise values of the initial conditions are not that critical, provided the model computation starts in a reasonably realistic manner; that is, relatively smoothly and with no initial instabilities. However, in applications where there are lakes, wetlands, retarding basins, or other depressions that may provide initial flood storage, it is important that the initial water surface elevations provide the correct amount of initial storage in these areas. If the initial amount of water in these storage areas is underestimated, this may cause the model to artificially attenuate the flood peak. Conversely, if it is overestimated, the flood peak may be artificially enhanced.

Hot Starts: Hot starts are not used extensively in practice. Their main uses tend to be limited to:

- Providing initial conditions for model simulations where significant computing time may be required for the model "warm-up" period required. In these cases, the results of a single prolonged "warm-up" simulation can be used to provide initial conditions for a subsequent series of model simulations.
- Breaking-up very long model simulations into more manageable sub-sections.

Urban Applications: In many urban applications, the area to be modelled may be initially dry. In these cases, the initial water surface elevation ς values will typically be set to the corresponding ground surface elevation z values, and the initial u and v velocity values set to zero. This approach could be considered as a special case of the cold start. It is applicable in model simulations where there is no initial overland flow, including direct rainfall on grid applications.

Boundary Conditions

With the initial conditions specified, the model boundary conditions are the remaining pieces of necessary information required for the model computation to proceed. In this respect it is noted that boundary conditions are required at every grid/mesh element along the model boundaries. These boundaries include both external boundaries and internal boundaries, where:

- External boundaries are located along the external edges or boundaries of the model, where water can flow into or out of the model domain.
- Internal boundaries are located within the model domain, and include the interface between wet (water) cells, where the computation is to be carried out, and dry (land) cells where there is no computation.

The subject of boundary conditions for two-dimensional flow models is quite complex and the following discussion is, of necessity, relatively superficial.

External Boundaries: The model requires boundary conditions in terms of either water levels or discharges along both the upstream and downstream boundaries.

The upstream boundary conditions for a 2D flood model are generally provided by a discharge hydrograph. This has to be converted to discharges or flow velocities and water depths at each boundary cell. For this to be done, some assumptions need to be made with respect to the distribution of the flows along the boundary, and to the direction of the flow. Depending on the modelling package being used, the flow distribution along the boundary may be computed internally using a range of possible assumptions (e.g., uniform flow or the use of Manning's equation). There may also be options for providing user specified flow distributions and directions.

The downstream boundary conditions are generally specified in terms of water surface elevations. These may be specified as a constant, a times series, or computed internally using a rating curve. As a first approximation, it can be assumed that the water surface elevations along the boundary are horizontal. As such, the water surface elevation specified at each individual boundary cell will be the same. Depending on the modelling package being used, the flow directions may be specified as being normal to the boundary, or there may be options for them to be computed as a function of the upstream flow conditions, or specified externally by the user.

Whatever forms of boundary conditions that are being used, it is important to recognise that the model has no information regarding the flow conditions upstream or downstream of the model boundaries. As such, it is important that, wherever possible, the model boundaries should be located in areas where the flow is expected to be relatively uniform. The model boundaries should also be placed far enough upstream and downstream of the area of interest to ensure that any errors in flow distribution and/or direction do not have a significant effect on the model results.

Internal Boundaries: There are two types of internal boundaries used in 2D models:

- The first occurs at the land/water interfaces within the interior of the model domain. These internal boundaries can be considered as a special case of a velocity boundary where the flow velocity between adjacent pairs of wet and dry cells is set to zero. The locations of these internal boundaries are dynamic as different cells are brought into the computation as the flood rises, and taken out again as the flood recedes.

- The second type of internal boundary occurs where there are source or sink terms (related to local inflows and outflows), hydraulic structures, or where there are links to embedded 1D model branches.

Hydraulic Structures

Depending upon the particular software being used, hydraulic structures such as weirs, culverts, bridges and regulating structures can be introduced into the model. This may be done either by introducing a structure equation to replace the momentum equation between two adjacent computational cells (similar to the 1D model approach), or by introducing a 1D model branch containing the required structures (see Book 6, Chapter 4, Section 5) and coupled to the 2D model domain via internal boundaries.

4.7.8. Advantages and Disadvantages

The main advantage of a full 2D model is that it can provide a more realistic description of the flows and flow distributions throughout a river and flood plain system. When compared with the disadvantages of 1D models discussed above, it can be said that:

- The floodplain flow paths do not need to be pre-determined by the user, as they are computed directly as a function of the model terrain and the applied flows.
- The flow paths can change with increases in water level in much the same way as they do in real life.
- Losses due to two-dimensional effects such as bends, flow separations, etc, are automatically included within the computation, and do not need to be lumped into the bed-friction parameter (as such, the bed-friction coefficients can be specified directly as a function of bed-roughness only).
- Model results can provide details of the flow distribution within individual flow paths.
- Model results can be used directly for mapping flood extents and depths of inundation.

In the early days of full 2D modelling, the main disadvantages of the 2D models were that they required significantly more survey data than 1D models, and that they were very heavy computationally. The advent of LiDAR has, to some extent, overcome some of the survey requirements. There have also been significant increases in computing power, which combined with the introduction of parallel processors and GPUs has greatly increased the computational capacity of modern computers. However, along with the increase in computing capacity, there has been a tendency for modellers to use smaller cell sizes, to get better resolution, and also to use much larger model domains. As a result, long simulation times can still be an issue with 2D models. Further, the result files of these model runs can become very large, and can make significant demands on data storage and processing capability.

Another disadvantage of 2D models is that flow paths and channels can only be resolved at the same scale as the model grid or mesh. Even when a channel may be several computational cells wide, the in-bank flows may not be described as well as in a 1D model with detailed cross-sections.

Figure 6.4.14 shows the topography of the floodplain region of the Lindsay and Murray River system considered in Book 6, Chapter 4, Section 6, where the channel of the Murray River is shown in white. To provide adequate resolution of the complex channel system throughout the floodplain in a 2D model would require a relatively fine grid or mesh size resulting in very large computational arrays and correspondingly long run times.

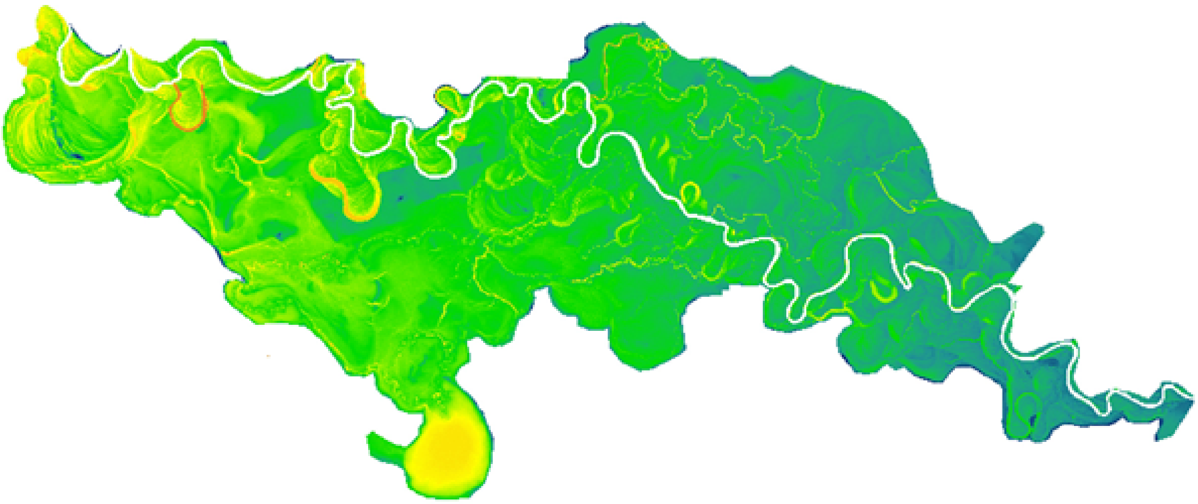


Figure 6.4.14. Topography of The Lindsay and Murray River System

4.7.9. Coupled 1D/2D Hydraulic Models

From the discussions above, it can be seen that 1D models are well suited to modelling in-bank flows and flows along long river and floodplain reaches (of the order of 10s to 100s or even 1,000s of kilometres) while full 2D models are well suited to modelling the details of the flow in smaller areas (of order 100s of metres to 10s of kilometres). This led to the proposal for integrated modelling (Carr and McCowan, 1988) whereby overall 1D models are used to provide water level and discharge boundary conditions for use in a detailed 2D model of a particular area of interest. This concept of model integration has been taken further (references needed) with 1D models being dynamically coupled with 2D models.

The coupling can work in different ways depending upon the particular software package being used. These can include:

- The use of a 1D model to simulate an overall river and floodplain system dynamically coupled to a 2D model providing detailed flow computations in particular areas of interest.
- The use of dynamically coupled 1D model branches to provide a better description of in-bank channel flows within a 2D model domain. The coupling of these branches can provide for the exchange of water between the 1D in-bank flows and the 2D model flood plain flows.
- The use of dynamically coupled 1D model branch to introduce hydraulic structures (such as weirs, culverts, bridges, etc.) into the 2D model domain.

A further extension of the integrated 1D/2D modelling concept has been the dynamic coupling of 2D models with 1D pipe network models. This has significantly enhanced the capabilities of 2D models for urban flood modelling applications.

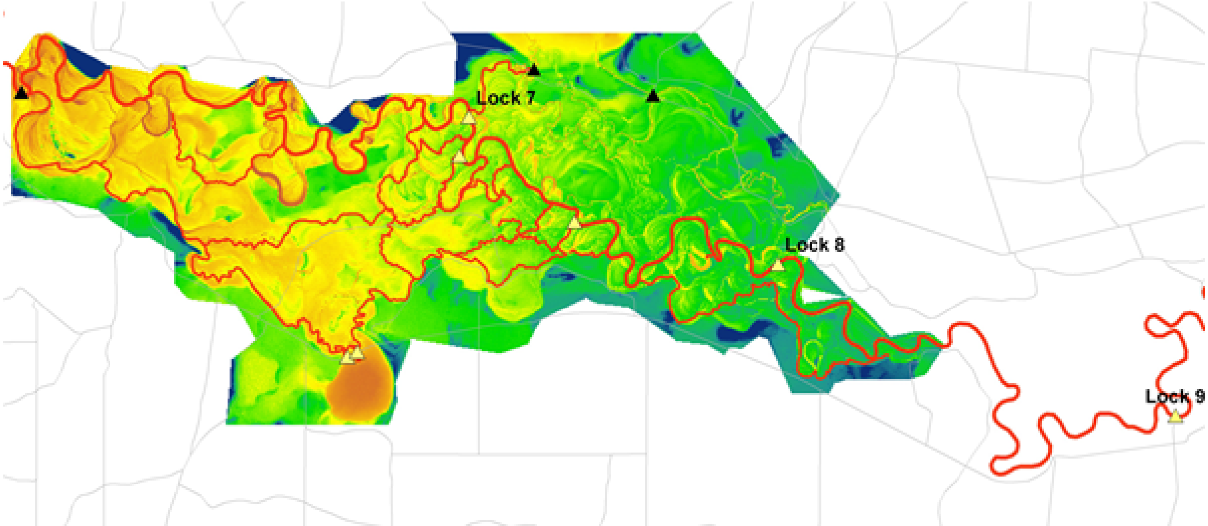


Figure 6.4.15. Example of a Coupled 1D/2D Model of the Lindsay River System

4.7.10. Direct Rainfall Models

A relatively recent development in 2D hydraulic modelling models has been the use of direct rainfall to estimate flows in catchments or sub-catchments where the local rainfall within the 2D model domain is contributing to the flow that we are interested in. Under these circumstances it is difficult to use traditional approaches such as the use a hydrologic model to provide flows at the model boundaries.

With the direct rainfall approach, the rainfall-runoff process is simulated by applying rainfall directly to each cell within the model domain. Losses are accounted for using different approaches depending upon the software package being used. With the simplest approach, the losses are applied directly to the rainfall with only a resulting rainfall “excess” being applied to the model cells. More sophisticated approaches may use infiltration models incorporated within the 2D modelling software and, ultimately, may involve coupling with a groundwater model.

The use of direct rainfall on a 2D model makes it possible to simulate the rainfall-runoff process, as well as the hydraulic routing of the resulting overland flows throughout model domain. This provides a more realistic representation of catchment storage and runoff effects. It is, however, essential to have good topographic data. The selection of appropriate roughness coefficients is critical to the success of this approach (Muncaster et al., 2006). Further, roughness values may need to be increased for describing shallow flows in rural areas, or decreased to allow for more rapid runoff from rooves and some paved areas in urban applications (Caddis et al., 2008).

The use of a 2D hydraulic model in this way integrates both the hydrologic and hydraulic aspects of the rainfall-runoff process into a single model. The use of direct rainfall is, however, an area of on-going research and care should be used when interpreting the model results. Wherever possible models should be calibrated to measurements. Where calibration is not possible (as in many cases), sensitivity testing should be carried out to assess the sensitivity of the model to variations in the main model parameters.

4.7.11. Limitations of 2D Hydraulic Models

One of the main limitations associated with 2D hydraulic models is that they can provide plausible results in situations where the underlying data may be inadequate, the model schematization does not adequately describe all the physical system being represented, or in flow cases where the assumptions of two-dimensional flow or of hydrostatic pressure becomes invalid.

Examples of situations where the two-dimensional assumption becomes invalid include:

- Modelling the details of “helicoidal” flows around river bends
- Modelling the details of flows at bridges, culverts and intake and outlet structures
- Modelling separation zones and wakes behind structures where the horizontal dimensions of the structure, and of the cell size, are much smaller than the water depth

In these situations care needs to be taken in interpreting the model results. In some models the effects of three-dimensional flows can be included through additional loss terms, or the inclusion of sub cell-scale pier loss formulae. If, however, it is necessary to model the details of these types of flow, then a full three-dimensional model should be used.

Situations where the hydrostatic pressure assumption becomes invalid include free overflows of water over levees and embankments. In these cases the hydrostatic pressure assumption can lead to a significant over-estimation of the overflows. Depending upon the software being used, these effects can be overcome by incorporating weir equations into the computation, or by widening the model description of the levee or embankment to include two cell widths.

4.8. Summary and Conclusions

The aim of a hydraulic flood model is to provide a realistic representation of flood flows in river and floodplain systems. In general, it can be said that the more realistic the modelling approach, the greater the probability of achieving a successful outcome. However, the use of the most sophisticated modelling approach available will not, in itself, guarantee success. This is because the skill of the modeller adapting a generic modelling system to a specific application, and the quality of the data used as model input can be equally (or even more) important in determining the success of a modelling exercise.

The reliability of a hydraulic model is determined by a three stage process:

- Validation: to confirm that the modelling software is doing what it is supposed to do.
- Calibration: the process of adjusting model parameters to obtain a best fit with measured flood data.
- Verification: ideally, a check of the model calibration against an independent set of flood data.

In cases where there is insufficient or no calibration data, sensitivity tests should be carried out to assess the sensitivity of the model to variations in the main model parameters

Steady flow hydraulic models are best suited to modelling flows along relatively short reaches of river with well-defined flow paths, and/or for modelling flows at structures. However, unsteady hydraulic models should be used to describe flows where there are:

- Rapidly changing hydrographs;
- Flat channel slopes where wave propagation effects can become important;
- Wide floodplains and/or other features where storage effects may affect the flow; and
- Channel networks where the flow splits are not well defined.

With respect to the different types of unsteady hydraulic models that are available, it can be said that:

- The use of CFD and other non-hydrostatic models, including physical models, is generally limited to simulating the details of complex flows in relatively short reaches of a river, or at or within hydraulic structures.
- 3D models are very heavy computationally, and are best suited to modelling the details of complex flows in relatively short reaches of rivers, around structures and in other flows cases where three-dimensional effects become important in determining localized flood effects.
- 1D models are computationally quick to run and are well suited to modelling flows along well-defined channel and floodplain systems. However, the more general use of 1D models in flood studies has been largely superseded by 2D models.
- 2D models can provide a much more detailed description of flood flows and flow distributions within individual flow paths and have become virtually the standard for rural and urban flood studies. 2D models are, however, more demanding on input data and on computing resources.
- The integration of 1D models with 2D models has made it possible to include 1D model branches within a 2D model domain to provide a better description of in-bank flows and/or to introduce hydraulic structures (such as weirs, culverts, bridges, etc.) into the 2D model domain.
- The integration of 1D pipe network models with 2D models has significantly enhanced the capabilities of 2D models for urban flood modelling applications.
- The use of rainfall on grid has made it possible to integrate both the hydrologic and hydraulic aspects of the rainfall-runoff process into a single model. The use of direct rainfall is, however, an area of on-going research and care should be used when interpreting the model results.

4.9. References

Abbott, M.B. and Ionescu, F. (1967), On the Numerical Computation of Nearly Horizontal Flows, Journal of Hydraulic Research.

Abbott, M.B. (1979), Computational Hydraulics - Elements of the Theory of Free Surface Flows, Pitman, London.

Abbott, M.B. and Basco, D.R. (1989), Computational Fluid Dynamics: an Introduction for Engineers, Longman.

Abbott, M.B. and Rasmussen, C.H. (1977), On the Numerical Modelling of Rapid Contractions and Expansions in Models that are Two-Dimensional in Plan, Proc. 17th Congress IAHR, Baden-Baden.

American Society of Engineers (1996), River Hydraulics - Technical Engineering and Design Guides as Adapted From the US Army Corps of Engineers, No. 18, ASCE Press, New York.

BMT WBM (2008), TUFLOW User Manual. GIS Based 2D/1D Hydrodynamic Modelling.

Benque, J.P., Hauguel, A. and Viollet, P.L. (1982), Numerical Models in Environmental Fluid Mechanics, Pitman.

Bishop, W.A., McCowan, A.D., Sutherland, R.J. and Watkinson, R.J. (1995), Application of Two-Dimensional Numerical Models to Urban Flood Studies, 2nd International Symposium on Stormwater Management, Melbourne.

Caddis, B.M., Jempson, M.A., Ball, J.E. and Syme, W.J. (2008), Incorporating Hydrology into 2D Hydraulic Models - the Direct Rainfall Approach, (9th Aust. Conf. on Hydraulics in Water Engineering, Darwin.

Carr, R.S. and McCowan A.D. (1988), An Integrated Approach to Two?Dimensional Flood Plain Modelling, ACADS Workshop on 2D Flood Plain Modelling, Monash University.

Connor, J.J. and Brebbia, C.A. (1976), Finite Element Techniques for Fluid Flow, Newnes-Butterworths

Cunge, J.A., Holly, F.M. and Verwey, A. (1980), Practical Aspects of Computational River Hydraulics, Pitman, London (Reprinted by University of Iowa).

DHI Water and Environment (2005), MIKE 21 - Coastal Hydraulics and Oceanography - Hydrodynamic Module, Scientific Documentation, DHI Software, Hørsholm.

Engineers Australia (2012), Australian Rainfall and Runoff Revision Project 15: Two-Dimensional Modelling in Urban and Rural Floodplains.

King, I.P. and Roig, L.C. (1988), Recent Applications of RMA's Finite Element Models for Two-Dimensional Hydrodynamics and Water Quality, Proc 2nd Int. Conf. on Finite Elements in Water Resources, Pentech Press.

LeVeque, R.J and Bale D.S. (2012), Wave Propagation Methods Conservation, Journal of Hyperbolic Problems, Zurich.

Leendertse, J. (1967), Aspects of a Computational Model for Long-Period Water Wave Propagation, Rand Memorandum RM-5294-PR, Santa Monica.

Leonard, B.P. (1979), A Stable and Accurate Convective Modelling Procedure Based on Quadratic Upstream Interpolation. Computer Methods in Applied Mechanics and Engineering, 19(1), 59-98.

McCowan, A.D., Rasmussen, E.B. and Berg, P. (2001), Improving the Performance of a Two-dimensional Hydraulic Model for Floodplain Applications, Proceedings of the 6th Conference on Hydraulics in Civil Engineering, Hobart.

Muncaster, S.H., Bishop, W.A. and McCowan, A.D. (2006), Design Flood Estimation in Small Catchments Using Two-Dimensional Hydraulic Modelling - A Case Study, 30th Hydrology and Water Resources Symposium, Launceston.

Preissmann, A. (1961), Propagation des Intumescences dans les Canaux et Rivieres, First Congress of the French Association for Computation, Grenoble.

Roache, P.J. (1998), Fundamentals of Computational Fluid Dynamics, Hermosa.

Sherwin S.J., Peiro J., (2005), Finite difference, finite element and finite volume methods for partial differential equations, Handbook of materials modeling, Editors: Yip, Berlin, Publisher: Springer, pp: 1-30, ISBN: 9781402032875

Stelling, G.S. (1984), On the Construction of Computational Methods for Shallow Water Flow Problems, Delft University of Technology.

Stelling, G.S., Kernkamp, H.W.J and Laguzzi, M.M. (1998), Delft Flooding System: A Powerful Tool for Inundation Assessment based upon a Positive Flow Simulation, in Hydroinformatics '98, V. Babovic and L.C. Larsen (ed's), Balkema.

Syme, W.J. (2001), TUFLOW - Two and One-Dimensional Unsteady Flow Software for Rivers, Estuaries and Coastal Waters, I.E.Aust Workshop on 2D Flood Modelling, Sydney.

United States Army Corps of Engineers (USACE) (2010), HEC-RAS River Analysis System, Hydraulic Reference Manual, Version 4.1, Hydrologic Engineering Center.

Violeau, D. (2012), Fluid Mechanics and the SPH Method, Theory and Applications, Oxford University Press.

Zienkiewicz, O.C., Taylor, R.L. and Nithiarasu, P. (2014), The Finite Element Method for Fluid Dynamics, 7th Edition, Butterworth-Heinemann.

Chapter 5. Interaction of Coastal and Catchment Flooding

Seth Westra, Michael Leonard, Feifei Zheng

Chapter Status	Final
Date last updated	14/5/2019
Minor edits	27/08/2024

5.1. Introduction

Floods in estuarine areas can be caused by runoff generated by an extreme rainfall event, an elevated ocean level generated by a storm surge and/or a high astronomical tide, or a combination of both processes occurring simultaneously or in close succession. Research in Australia (Zheng et al., 2013) and internationally (Svensson and Jones, 2002; Svensson and Jones, 2004; Hawkes and Svensson, 2006) has shown that extreme rainfall and storm surge processes are statistically dependent, and therefore their interaction needs to be taken into account for areas affected by both processes.

This chapter describes procedures that can be used to estimate design flood levels in the 'joint probability zone', defined as the region in which the dependence between riverine and ocean processes has the potential to influence the design flood level. This region is illustrated in Figure 6.5.1, and shows that the range of possible flood levels corresponding to a given Annual Exceedance Probability are enclosed in an envelope bounded by cases where flood events and ocean levels are perfectly dependent (upper curve) and independent (lower curve).

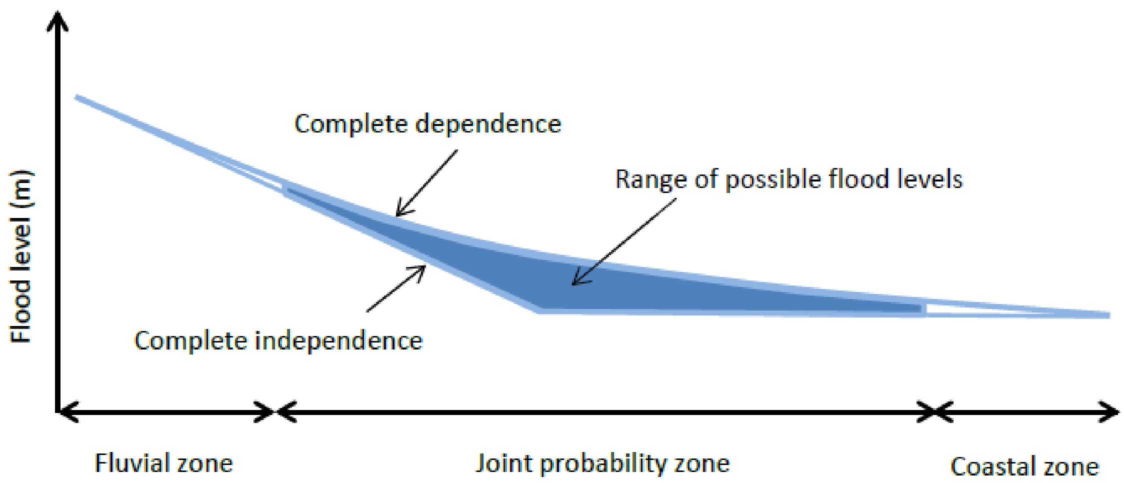


Figure 6.5.1. Schematic of a longitudinal section of an estuary, which shows two hypothetical water levels: the level obtained by assuming that fluvial floods will always coincide with storm tides of the same exceedance probability (upper curve); and the level assuming fluvial processes and ocean processes are completely independent and thus will almost never coincide (lower curve).

This chapter provides practical guidance on estimating the exceedance probability of floods in the joint probability zone. The focus of this chapter is on the 'design variable method', which has been developed as a flood estimation approach that can be applied across Australia's diverse climates. The method has been tested for design floods from 50% to 1% Annual Exceedance Probabilities , and can account for the influence of climate change by adjusting both the design rainfall and design ocean levels that are required as inputs.

The theory and practice of flood estimation in the joint probability zone is considerably more complex than many traditional flood estimation problems, and engineering judgement is required on whether the design variable method described in this chapter is suitable for a given situation. This judgement should be based on a sound knowledge of joint probability theory, combined with an understanding of riverine and oceanic flood processes. Alternative methods that may be appropriate under certain conditions are discussed briefly in Book 6, Chapter 5, Section 3. The approach presented in this chapter is also valid for overland flooding problems.

5.2. Background to Flood Processes in Estuarine Areas

The combination of processes that can cause flooding in the joint probability zone is illustrated using a hypothetical flood occurring in an estuarine region bounded above by flow from an upstream catchment and below by ocean water levels at the estuary mouth (Figure 6.5.2). Factors that can influence the magnitude of a flood in this region have been numbered in Figure 6.5.2, and each factor is described is more detail below.

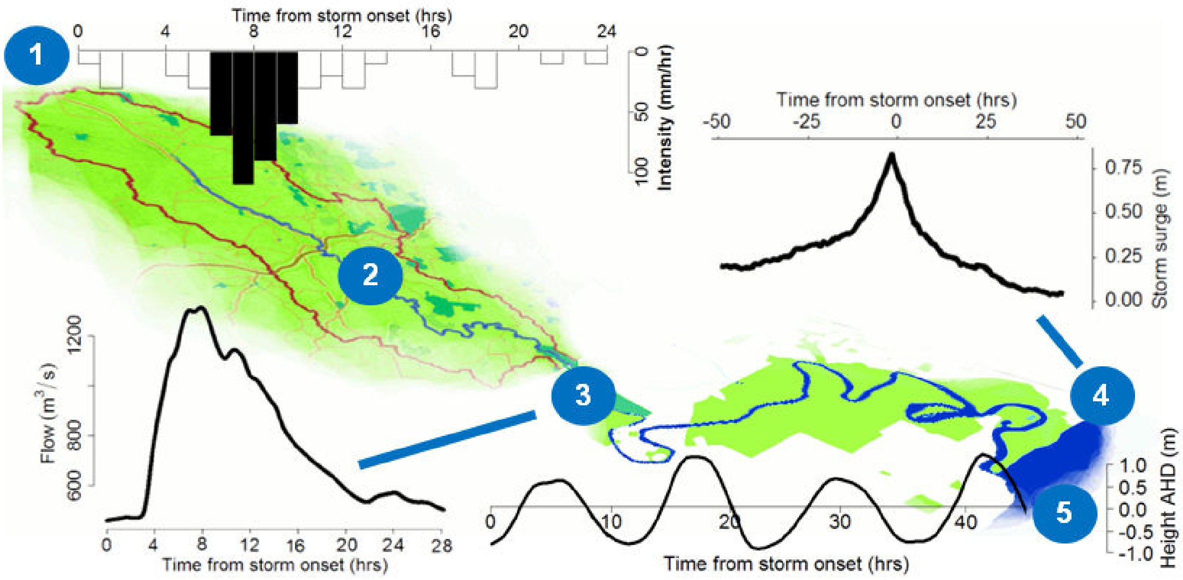


Figure 6.5.2. Timing factors affecting the magnitude of a flood in the joint probability zone

1. *Rainfall* - A flood can be initiated by a sustained burst of intense rainfall (often referred to as a storm burst, shown in Figure 6.5.2 as the shaded four hour period) over an estuarine catchment. This storm burst can be characterised by its duration, spatial extent, temporal pattern and rainfall intensity. The storm burst is often embedded in a longer period of rainfall, which can be caused by large-scale meteorological features such as a frontal rainfall system or a tropical cyclone.

2. *Runoff Generation*- The shape, size, slope, soil type, vegetation and level of urbanisation all contribute to the way a catchment translates rainfall into runoff. The time of concentration refers to the time it takes all of the catchment to contribute runoff at the

catchment outlet, and is often assumed to be equivalent to the time taken for water to travel from the most distant point in the catchment to the catchment outlet. The time of concentration of the hypothetical catchment in Figure 6.5.2 is four hours, which is equivalent to the duration of the storm burst.

3. *Hydrograph at Catchment Outlet:* - The time it takes for the hydrograph to enter the joint probability zone causes a lag between the flood producing rainfall event and the hydrograph peak. The hydrograph represents the fresh water contribution to floods in the joint probability zone, and may form the upstream boundary condition for hydrodynamic models of this region.

4. *Storm surge* - The ocean level forms the downstream boundary to the system, and typically comprises a deterministic component (the astronomical tide) and a random component (usually dominated by the storm surge). The storm surge is caused by anomalous wind and atmospheric pressure that are linked to large-scale weather patterns, and the magnitude of the surge at a particular location will be influenced by the coastal geography and bathymetry. Figure 6.5.2 shows a composite of ten storm surge events near Perth, with the composite exhibiting a sharp peak lasting several hours, yet with some effects still apparent for a day or longer both before and after the peak.

5. *Astronomica*l tide- Tidal patterns, whether diurnal (24 hour), semi-diurnal (12 hour) or mixed, can vary substantially with location. The astronomic tide level is usually assumed to be independent of the rainfall intensity.

As illustrated in Figure 6.5.2, the question of whether or not a large fluvial flood will coincide with an elevated ocean level will depend on several timing issues, which are influenced by a combination of meteorological, catchment scale and oceanographic processes. In particular, the timescale of both rainfall and storm surge events are determined by meteorological influences, whereas the timescale of the runoff depends on specific catchment features that are related to the catchment's time of concentration.

A further complicating factor is that the same meteorological events can drive both rainfall and storm surge events, and this has led to the finding in Australia (Zheng et al., 2013) and internationally (Svensson and Jones, 2002; Svensson and Jones, 2004; Hawkes and Svensson, 2006) that extreme rainfall and storm surge is statistically dependent. The dependence strength between extreme rainfall and storm surge in Australia was found to vary as a function of geographic location and the duration of the rainfall burst (Zheng et al., 2013; Zheng et al., 2014a). Each of these factors will need to be taken into account when selecting a method for estimating flood exceedance probabilities in Australia's estuarine catchments.

5.3. Flood Estimation Approaches for the Joint Probability Zone

Several approaches have been developed to estimate the exceedance probability of floods in the joint probability zone, each with different assumptions, data and modelling requirements. The three most commonly used approaches are described here, with key features summarised in Table 6.5.1:

Flood Frequency Analysis (FFA) - This approach involves fitting a probability distribution to a time series of historical streamflow. The approach is relatively easy to implement, but requires long, high-quality historical flood records at the location of interest. The advantage of this approach is that, by directly focusing on the statistical characteristics of historical floods, it may be possible to avoid modelling the complex processes that lead to estuarine floods as depicted in Figure 6.5.2**.**

However, the approach assumes that the upstream catchment conditions and the bathymetry of the estuary are unchanged over the historical record and are reflective of future conditions, and that the statistical characteristics of the upper and lower boundary conditions (e.g. extreme rainfall, sea level, storm surge) will remain constant into the future. For most of Australia's estuarine catchments, one or more of the assumptions underpinning FFA will be violated; therefore this approach is unlikely to be practically applicable in most situations.

Further information on implementation of flood frequency approaches is provided in Book 3, Chapter 2.

Continuous simulation - As discussed in Book 6, Chapter 5, Section 2, floods in the joint probability zone can be influenced by a large number of processes operating at a range of timescales, including sub-daily variability in tides, storm surges and the flood hydrograph from the upstream catchment, superimposed on lower-frequency variability at daily, seasonal, annual and inter-annual timescales. In many cases, dynamical features, such as the progression and attenuation of tides up the estuary, can significantly influence flood behaviour.

Continuous simulation approaches aim to simulate these complex dynamics, by running continuous hydrological and hydraulic models to generate a long time series of a response variable (e.g. flood level) that can then serve as the basis for Flood Frequency Analysis. To capture these dynamics, the models will usually need to be run at fine sub-daily timescales. Furthermore, the hydrological and hydraulic model will require long continuous observational time series of rainfall (representing the upstream boundary condition) and storm tides (representing the downstream boundary condition). To estimate flood characteristics such as level at specified exceedance probabilities, it is possible to apply a univariate Generalised Extreme Value analysis to extreme simulated flood values. Alternatively, it is possible to stochastically generate long continuous time series of the forcing variables and then use the empirical probabilities.

The computational load of continuously running hydrological and hydraulic models at the short time steps required for capturing tidal dynamics—while producing long runs required for estimating floods with low exceedance probabilities—is often extremely high. Furthermore, in many cases, long historical time series of both extreme rainfall and storm tides are unlikely to be available at the location of interest. If implemented correctly, continuous simulation is likely to be a technically rigorous approach for flood estimation in the joint probability zone, but given its numerous practical challenges, the design variable method has been developed as an alternative approach for flood estimation problems along the Australian coastline.

Further information on implementation of continuous simulation approaches is provided in Book 2, Chapter 7.

The design variable method -This approach has been developed as a simpler alternative to continuous simulation, without the limiting assumptions of Flood Frequency Analysis. For further information on the theory and practical limitation of the method, refer to Book 6, Chapter 5, Section 4 and Book 6, Chapter 5, Section 5, respectively. The primary assumptions of the approach are:

- The statistical dependence between extreme rainfall and storm surge can be represented through a bivariate logistic extreme value dependence model, discussed in further detail in (Zheng et al., 2014a);
- The dependence strength can be interpolated between gauged locations along the Australian coastline, and therefore can be represented by a map of dependence strength (given in Figure 6.5.13 and discussed further in Book 6, Chapter 5, Section 5); and
- The Annual Exceedance Probability of the rainfall event is equivalent to the Annual Exceedance Probability of the flood event (probability neutral);
- Ocean water levels are assumed to be 'static', as tidal dynamics are not considered explicitly in the method; and
- Anthropogenic climate change will have negligible effect on the strength of dependence between extreme rainfall and storm surge, although the effects of climate change can be accounted for by changing the marginal distributions (ie. the extreme rainfall intensity and the ocean level).

The validity of the assumptions of the design variable method need to be considered when applying the method to a specific flood estimation problem, and weighed against assumptions associated with alternative approaches. For many situations, the design variable method is a pragmatic approach that can be applied across a range of estuarine flood estimation approaches.

A comparison between each of the methods is provided in Table 6.5.1.

Table 6.5.1. Comparison design flood estimation methods in the joint probability zone

Aspect \ Model	Flood Frequency Analysis	Design Variable Method	Continuous Simulation
Domain of Applicability	Analysis restricted to locations with gauged data.	Can be applied throughout the joint probability zone.	Can be applied throughout the joint probability zone.
Models Required	Univariate extreme value model or other statistical model of extremes (see Book 6, Chapter 4).	Event-based hydrological and hydraulic models, and a bivariate extreme value model.	Continuous hydrological and hydraulic models, and a univariate extreme value model.
Technical Complexity	Low	Intermediate	Advanced
Computational Demand	Low	Medium	High
Capacity to Account for Dynamic Tidal Effects	N/A	Static ocean levels only	Dynamic Tides.
Parametric Uncertainty	Well understood likelihoods and methods for parameter uncertainty (refer to Book 3, Chapter 2 on FLIKE).	It is feasible to estimate the uncertainty of each parameter in a bivariate extreme value model, but this	Model-dependent.

Aspect \ Model	Flood Frequency Analysis	Design Variable Method	Continuous Simulation
		is beyond the scope of this Chapter.	
Capacity to Account for Climate Change	Cannot account for climate change.	The method enables the distribution of both the extreme rainfall and ocean level to be modified by adjusting AEPs. The dependence between extreme rainfall and storm surge is assumed to remain constant in a future climate.	Requires the full distribution of future changes to rainfall and ocean levels to be modified, rather than just the extremes. This is likely to require some form of dynamical and/or statistical downscaling.

5.4. Theory of Joint Probability

This section describes the theory of joint probability concepts in general, and also provides a more detailed overview of the design variable method. A practical description of the implementation of the design variable method is provided in Book 6, Chapter 5, Section 5, and worked examples in Book 6, Chapter 5, Section 6 are provided in.

5.4.1. Joint, Marginal and Conditional Distributions

Consider two random variables, X and Y[1]. The joint probability distribution (or, equivalently, the bivariate probability distribution) of these variables describes the probability that $(X,\ Y)$ equals a particular set of values $(x,\ y)$ or falls in any particular range of values for that variable. This enables the relationship between two variables to be considered. The joint probability distribution can be generalised to any number of random variables, in which case it is referred to as a multivariate probability distribution. The following text presents basic statistical properties of the joint, marginal and conditional distributions, using bivariate distributions by way of illustration. For more information on the theory of joint, marginal and conditional distributions, the reader is referred to statistics references such as Ang and Tang (2006).

The joint probability density function is written as $f_{x,y}(x,y)$, and has the property:

$$\int_x \int_y f_{X,Y}(x,y)dydx = 1 \tag{6.5.1}$$

For independent variables, the joint probability distribution can be expressed as:

$$f_{X,Y}(x,y) = f_X(x)f_Y(y) \tag{6.5.2}$$

The conditional probability density function given the occurrence $X = x$ is given as:

[1]In this chapter, the variable X can be thought of as denoting daily or sub-daily rainfall, and Y denotes either storm surge or storm tide. However, the theory can be applied more generally to any pair of variables.

$$f_{Y|X}(y|X=x) = \frac{f_{X|Y}(x,y)}{f_X(x)} \tag{6.5.3}$$

A corollary of the definition of independence in Equation (6.5.2) is that substitution into Equation (6.5.3) leads to:

$$f_Y(y|X=x) = \frac{f_X(x)f_Y(y)}{f_X(x)} = f_Y(y) \tag{6.5.4}$$

In other words, the conditional distribution becomes equivalent to the marginal distribution of Y when the two variables are independent.

Finally, a marginal distribution can be written as:

$$f_X(x) = \int_y f_{X,Y}(x,y)dy = \int_y f_{X|Y}(x|y)f_Y(y)dy \tag{6.5.5}$$

These concepts are illustrated in Figure 6.5.3. The main panel shows a joint Gaussian probability density function $f_{x,y}(x,y)$with simulated data that has been drawn from this distribution function given as light blue dots. The marginal distributions $f_y(y)$ and $f_x(x)$ are shown as solid lines in the left and bottom panels, respectively. A conditional distribution $f_y(y|X) = x)$ is represented as a slice through the joint density at X=2, and the conditional probability density function is shown as the dashed line in the left panel.

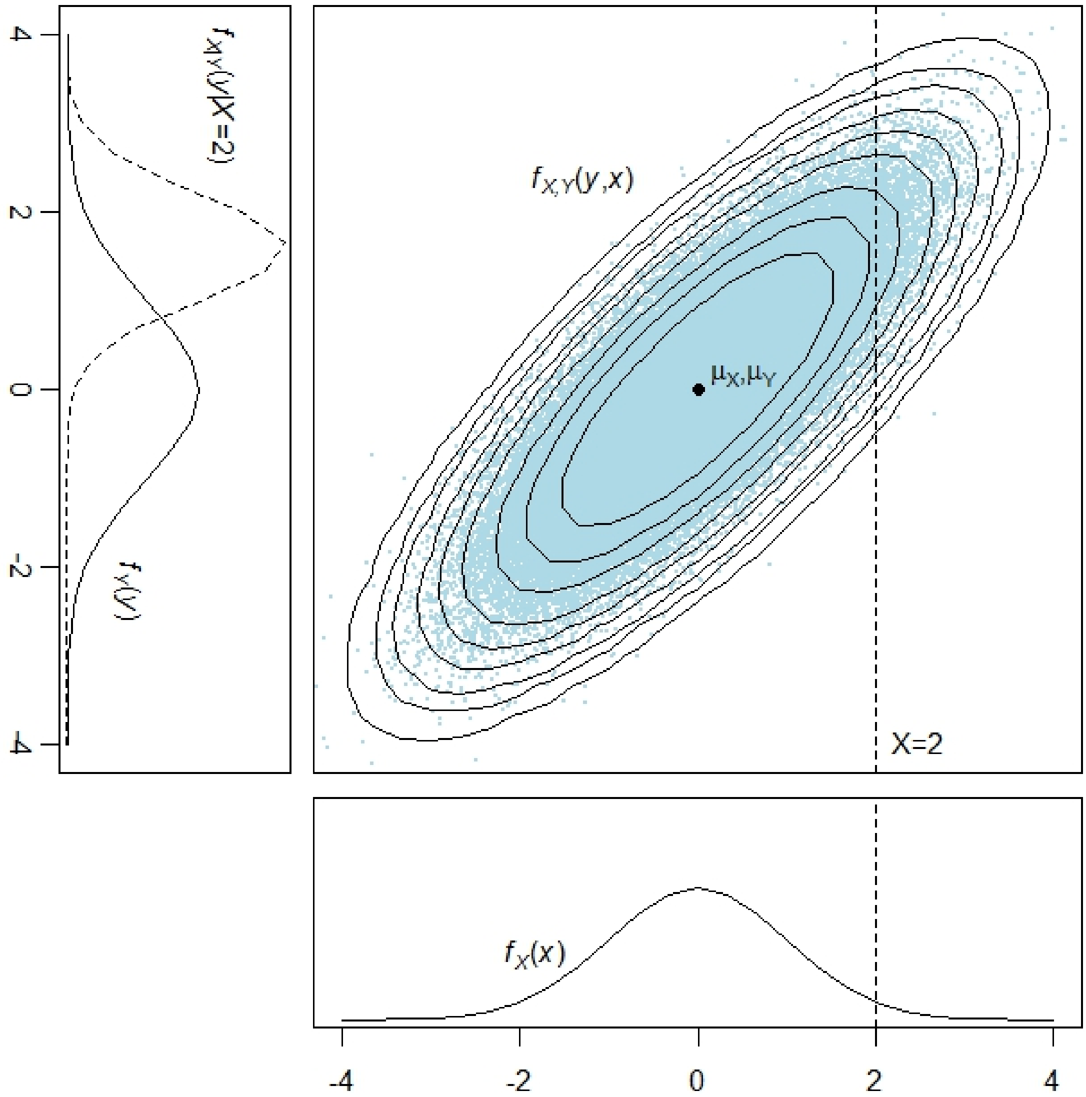


Figure 6.5.3. Joint, Marginal and Conditional Probability Density Functions

5.4.2. Representations of Univariate and Multivariate Extremes

Extreme value theory focuses on the statistical behaviour of the extremes of a random variable. Most of the theory is derived asymptotically as one or multiple variables become increasingly extreme, however a large body of literature now shows that the theory performs well in modelling finite extremes commonly encountered in hydrological applications (e.g. Refer to discussion in Coles (2001)).

Univariate extreme value theory is now a mature field, and the reader is referred to the text by Coles (2001) for a detailed overview of the theory and practical applications of extreme value models. Probably the most well-known representation of univariate extremes are 'block maxima', which are the maximum values of a process of independent and identically distributed random variables over a period of time such as a year. These maxima are commonly modelled using a Generalised Extreme Value (GEV) distribution, with the cumulative GEV distribution function given as:

$$F(x;\mu,\sigma,\xi)=\exp\left\{-\left[1+\xi\frac{x-\mu}{\sigma}\right]^{-1/\xi}\right\} \tag{6.5.6}$$

for $1+\xi(x-\mu)/\sigma>0$, where $\mu\in\mathbb{R}$ is the location parameter, $\sigma>0$ is the scale parameter, $\xi\in\mathbb{R}$ is the shape parameter, and $F_{(x)}$ is the cumulative distribution function.

An alternative representation that is widely used is the 'threshold-excess' representation, which is defined as exceedances $(x-u)$ over some suitably high threshold u. These maxima are commonly modelled using the Generalised Pareto distribution, with the cumulative distribution function given as:

$$F(y)=1-\left(1+\frac{\xi y}{\sigma}\right)^{-1/\xi},\ y=x-\mu \tag{6.5.7}$$

For both univariate representations, the definition of an ‘extreme’ event is clear. In contrast, the definition of an ‘extreme’ event becomes more ambiguous in the multivariate context. Four characterisations were identified in Zheng et al. (2014b), and are summarised briefly herein (refer also to the illustration in Figure 6.5.4). For more theoretical treatment of multivariate extremes, the reader is referred to Kotz and Nadarajah (2000) and Beirlant et al. (2004).

Component-wise block maxima -This is a direct analogue of univariate block maxima, but has the limitation that the component-wise maxima may occur at different times in the block. As such, the joint maxima will not necessarily correspond to 'real' (ie. simultaneously occurring) events. This representation is also very wasteful of data, as only the maximum values in each block contribute to the analysis. In practice these are severe limitations, and therefore component-wise are rarely used in multivariate extreme value analyses.

Threshold-excess extremes - (Figure 6.5.4, left panel): A high threshold (u_x *and* u_y) is set for both variables X and Y, and the multivariate threshold-excess model simulates the dependence between extremes that exceed both thresholds (illustrated by blue 'plus' symbols in Figure 6.5.4). Identifying appropriate thresholds (u_x *and* u_y) represent a compromise between maximising the amount of data exceeding both thresholds, and ensuring that the asymptotic assumptions that support the Generalised Pareto distribution are approximately valid; diagnostics for threshold identification are discussed in more detail in Coles (2001). A disadvantage to this characterisation is that, by only focusing on cases where both thresholds are exceeded, situations where only one variable is extreme are not modelled.

Point process representation - (Figure 6.5.4, middle panel): In this representation, the data are first transformed to radial ($r=x+y$) and angular ($w=x/(x+y)$) components, which is a transformation from Cartesian to pseudo-polar coordinates. Here, r represents the distance of each data point from the origin (and therefore describes the ‘extremeness’ of the observation), and w measures the angle on a [0,1] scale (and thus describes whether the variable is mostly influenced by x,y, or a combination of both variables) (Coles, 2001). Extreme events are those above the radial threshold r_0 (red 'plus' symbols in Figure 6.5.4), and the identification of an appropriately high threshold r_0 is based on asymptotic arguments, with diagnostic measures given in Coles (2001). As can be seen from the figure, this representation characterises the situation where both margins are extreme as well as the situation where only a single margin is extreme.

Conditional extremes distribution - (Figure 6.5.4, right panel): This representation is based on conditional distributions in both the X and Y dimensions, with the distribution of Y conditioned on the threshold exceedances in X (ie. $Y \big| X > u_{Y|X}$) and vice versa. The threshold $u_{Y|X}$ (vertical green line in Figure 6.5.4) needs to be specified, and then all points with $X > u_{Y|X}$ (green open circles) are defined as extremes when modelling the distribution of $Y|X$. The extremes when modelling the distribution of $X|Y$ are defined analogously, with the horizontal green line representing $u_{X|Y}$ and the green 'plus' symbols representing extremes above this threshold. The extreme events in the upper right quadrants (the combination of green circles and plus symbols) are based on combining $Y|X$ and $X|Y$, with further details in Heffernan and Tawn (2004). Similar to the point process representation, this characterisation models the situation where both margins are extreme as well as the situation where only a single margin is extreme.

The decision of how to represent multivariate extremes can have important implications in the context of estimating flood exceedance probabilities in the joint probability zone, with different models potentially leading to different probability estimates. In particular, given that the dependence between extreme rainfall and storm surge is generally statistically significant but not very strong (refer to Book 6, Chapter 5, Section 5), it is necessary to assess the probability of floods for situations when only a single variable is extreme, as well as when both variables are extreme, suggesting that the point process and conditional representations may be most suitable for coastal flood problems.

A detailed study in Zheng et al. (2014b) compared the three methods illustrated in Figure 6.5.4, with results summarised in Table 6.5.2. Zheng et al. (2014b) generated synthetic data from a bivariate logistic model with dependence α = 0.9 and Gumbel margins, and the threshold-excess, point process, and conditional methods were used to fit an extreme value model to this simulated data. Zheng et al. (2014b) concluded that the point process representation was most suitable for estimating the exceedance probability of floods in Australia's estuarine regions, as the conditional model tended to underestimate the dependence strength, and the parameter estimates are also highly variable. It was noted, however, that the dependence parameter estimates can be biased when simulating extremes using the point process representation, and that to overcome this issue it may be appropriate to estimate the dependence parameter the threshold-excess model. This was the approach taken to develop Figure 6.5.13, and is discussed in more detail in Zheng et al. (2014a).

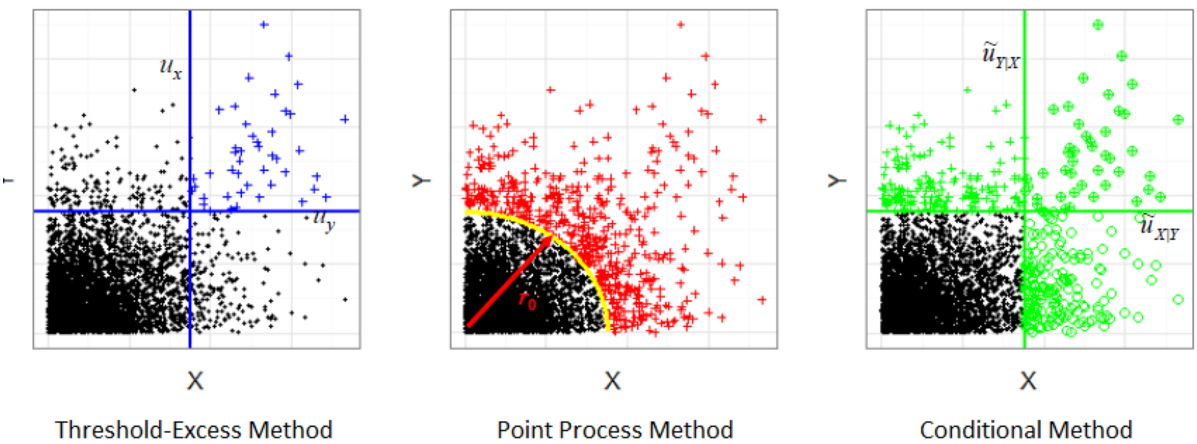


Figure 6.5.4. Three Representations of 'Extreme Values' Following Different Extreme Value Methods (After (Zheng et al., 2014b))

Table 6.5.2. Advantages and Disadvantages of Alternative Representations of Joint Extremes(based on Zheng et al (Zheng et al., 2014b))

Method	Advantages	Disadvantages
Component-wise Block Maxima	Some of the original theory on multivariate extremes has been developed using component-wise maxima, but there are few benefits of using this approach to estimate the exceedance probability of floods in estuarine regions.	Does not necessarily correspond to 'real' events, since the maxima of each variable can occur at different times of the year.
Threshold-excess Extremes	Corresponds to 'real' meteorological events, and enables unbiased estimates of the dependence parameter.	Does not represent the situation where only a single variable is extreme.
Point process	Corresponds to 'real' meteorological events, including the situation when only one variable is extreme. The models are typically parsimonious, and the variance is often low.	The dependence parameter is typically biased for weak dependence parameter values (α>0.8), and will lead to an overestimate of the dependence strength.
Conditional Extremes	Corresponds to 'real' meteorological events, including the situation when only one variable is extreme.	The dependence parameter is typically biased for weak dependence parameter values (α>0.8), and will lead to an underestimate of the dependence strength. The variance of the estimator is also high, and the model can be difficult to implement in practice.

5.4.3. External Dependence

In addition to the large number of alternative definitions of a 'multivariate extreme' discussed in Book 6, Chapter 5, Section 4, there are also a range of statistical models available for simulating extremal behaviour of multivariate processes. Five alternative models were compared in Zheng et al (Zheng et al., 2014b): the logistic, negative logistic, bilogistic, negative bilogistic and dirichlet models (refer also to Kotz and Nadarajah (2000)). The conclusion was that the differences in the performance of each model were minor. The bivariate logistic model was the simplest and most widely used model, and has therefore been recommended for use in implementing the design variable method (Book 6, Chapter 5, Section 5).

The cumulative distribution function of the bivariate logistic model is given as (Tawn, 1988):

$$F(x,y) = \exp\left\{-\left(\tilde{x}^{-1/a} + \tilde{y}^{-1/a}\right)^{a}\right\},\ \tilde{x} > 0, \tilde{y} > 0, 0 < a \leq 1 \tag{6.5.8}$$

where $\tilde{x}$ and $\tilde{y}$ are standard Fréchet-transformed values of original observations x and y, and α represents the dependence strength with α→0 and α=1 representing complete dependence and independence, respectively.

The Fréchet transformation is given as:

$$\tilde{z} = \begin{cases} -\left(\log\left\{1 - \hat{\varsigma}_{u_z}[1 + \frac{\hat{\xi}(z-u_z)}{\hat{\sigma}_z}]^{-1/\hat{\xi}_z}\right\}\right)^{-1}, & z > u_z, \xi_z \neq 0 \\ -\left(\log\left\{1 - \hat{\varsigma}_{u_z}\exp(-\frac{z-u_z}{\hat{\sigma}_z})\right\}\right)^{-1}, & z > u_z, \xi_z = 0 \\ -\left\{\log\hat{F}(z_i)\right\}^{-1}, & z \leq u_z \end{cases} \tag{6.5.9}$$

where z represents one of the original margins (either x or y), $\tilde{z}$ is the standard Fréchet value corresponding to the z in the original scale. $\hat{\varsigma}_{u_z} = \Pr\left\{Z > u_z\right\}$, and u_z is an appropriately high threshold for the margin z, and $\hat{\sigma}_z$ and $\hat{\xi}_z$ are the maximum-likelihood estimated parameters of the Generalised Pareto distribution. Finally, $\hat{F}$ is the empirical distribution function of z, estimated by $\hat{F}(z_i) = i/(n+1)$, where i is the rank of z_i and n is the total number of data points.

The application of the bivariate logistic model (Equation (6.5.8)) and the Fréchet transformation (Equation (6.5.9)) are illustrated in Figure 6.5.5 for an example dataset near Perth, Western Australia. First, a pairwise scatterplot of daily rainfall and daily maximum storm tide is presented (Figure 6.5.5a). Prior to applying the Fréchet transformation, it is necessary to identify marginal thresholds u_x and u_y. The choice of threshold values represents a trade-off between bias and variance: if the threshold is too low, then the parameters will likely be biased as the asymptotic justification of the extreme value model may not be valid; conversely if the threshold is too high then the limited sample size will result in parameter estimates with high variance. Based on visual inspection of two diagnostic plots—the mean residual life plot and the plot of parameter estimates against threshold - at multiple rainfall-storm surge pairs across Australia, it was found that the 1% daily exceedance probability (ie. the top 1% of rainfall and storm tide days) led to reasonable model performance for most locations along the Australian coastline (Zheng et al., 2013). These thresholds are shown as grey lines in Figure 6.5.5b.

The Frechet transformation in Equation (6.5.9) is applied to each margin, and the transformed data are shown on a logarithmic scale in Figure 6.5.5c. As discussed in Book 6, Chapter 5, Section 4, the point process representation focuses only on data above a radial threshold r_0; values below this threshold are represented in the figure as solid blue shading. The bivariate logistic model can then be fitted to this transformed data, with dependence represented using a single dependence parameter, α. For this case, a weak dependence parameter (α=0.95) was used. The bivariate probability density function, $f(x,y)$, and the bivariate cumulative distribution function, $F(x,y)$, are presented as dashed blue contours and solid black contours, respectively, in Figure 6.5.5d.

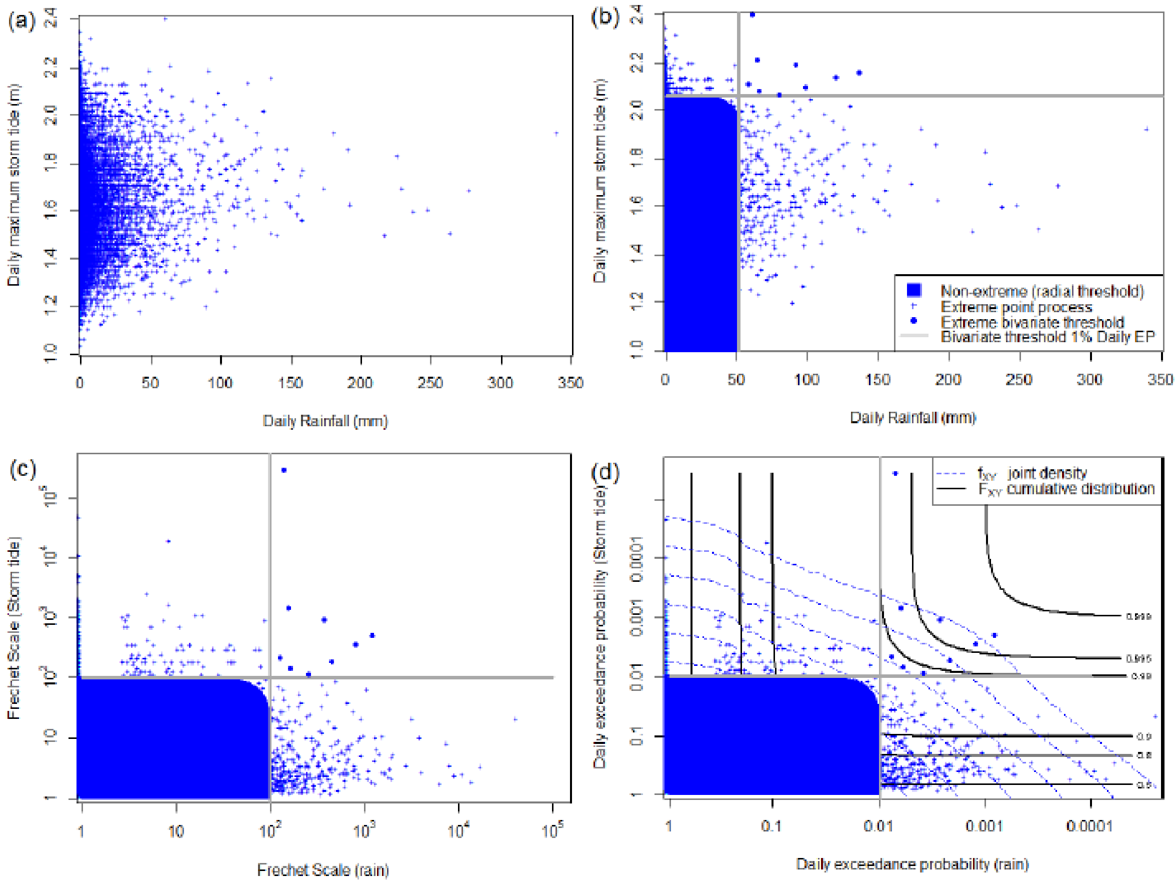


Figure 6.5.5. (a) Pairwise Plot of Daily Maximum Storm Tide and Daily Rainfall; (b) Application of Marginal Thresholds (Based on the 1% Daily Exceedance Probability for Each Margin), with Events Below the Radial Threshold r_0 Shaded in Blue; (c) Transformation of Events to Unit Fréchet Scale; and (d) Fitting the Joint Probability Distribution

To assist in the interpretation of the dependence parameter (α), the relationship between α and the number of events that exceed a bivariate threshold are shown in Figure 6.5.6 (refer also to Zheng et al. (2013)). The analysis was based on a study of 13 414 pairs of daily rainfall and daily maximum storm surge data located throughout the Australian coastline, and a marginal threshold of the 99th percentile of observed rainfall or storm surge data was used, which corresponds to an average of 3.65 exceedances per year. Assuming statistical dependence, it would be expected on average that one event every 100 x 100 = 10 000 days exceeds the joint threshold by random chance. The actual number of exceedances was then plotted against the fitted dependence parameter $\hat{a}$, to see the relationship between this parameter and the number of events exceeding the joint threshold.

There is a close relationship between $\hat{a}$ and the number of joint exceedances of both thresholds. As will be discussed in Book 6, Chapter 5, Section 5, the value of $\hat{a}$ typically varies from about 0.8 to 0.95 throughout most of the Australian coastline, therefore it is expected between eight and 27 more exceedances above the joint 99% threshold compared to what might be expected had the processes been independent. This is an order of magnitude increase in the probability of a ‘joint’ flood event (ie. a flood event caused by the combination of extreme rainfall and storm surge), and highlights the importance of accounting for joint probability issues in the Australian estuarine zone.

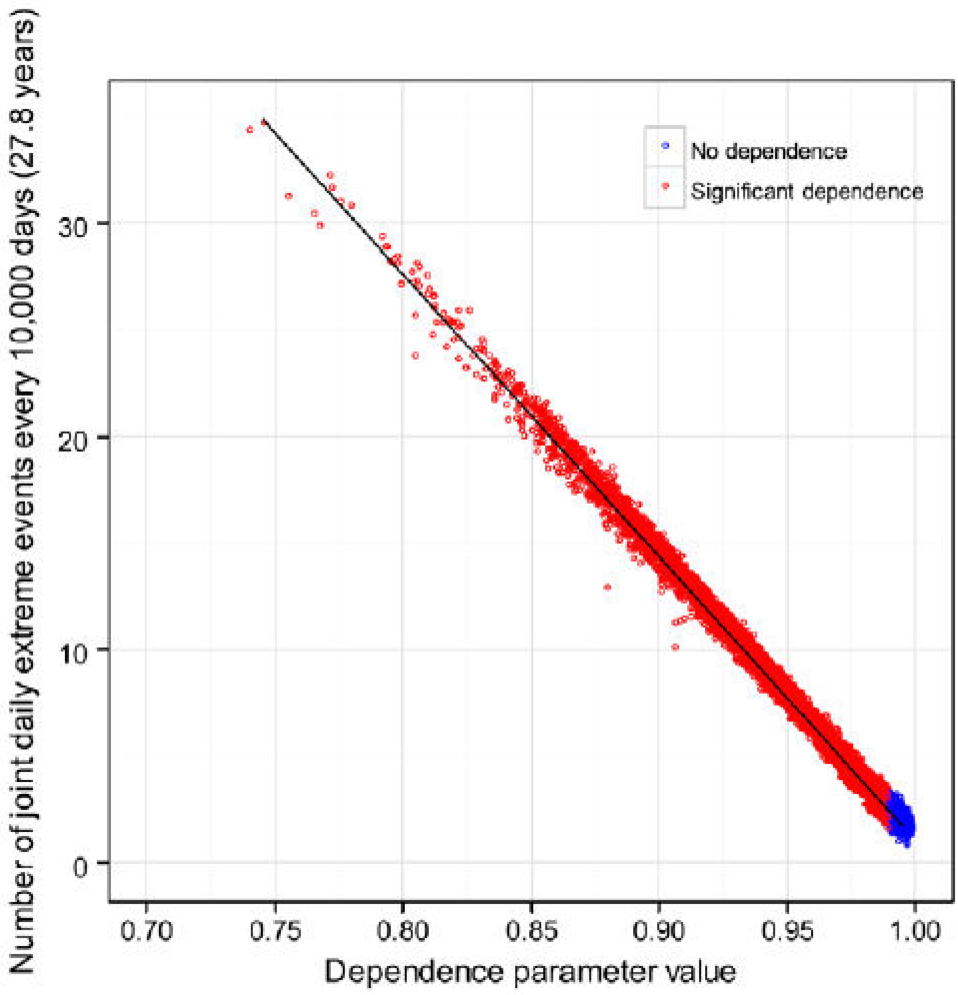


Figure 6.5.6. The Relationship Between the Dependence Parameter and the Number of Joint Extreme Events per 10 000 days

5.4.4. Design Variable Method

In this section, the theory is presented for translating information on extremal dependence into estimates of flood exceedance probability, commencing with a brief review of univariate flood estimation concepts. Further details on the design variable method can be found in Coles and Tawn (1994), with a more recent review by Zheng et al. (2015).

Univariate estimation methods are used for many flood estimation problems, whereby the frequency of a single forcing variable (e.g. extreme rainfall or storm tide) is assumed to be equivalent to the frequency of the corresponding flood level. The exceedance probability $\Pr(H \geq h)$ for a given flood level, *h* can be defined as:

$$\Pr(H \geq h) = \int_{x = x_0}^{\infty} f(x)dx, \quad h = B(x_0) \tag{6.5.10}$$

where $B(x)$ is a function relating the flood level to a single forcing variable X (e.g. rainfall or storm tide); x_0 is the value of the forcing variable that causes the flood level h, and $f(x)$ is a density function of X at extreme levels. To obtain $\Pr(H \geq h)$, one needs to first estimate the corresponding x_0 that causes the flood level h, then estimate the exceedance probability that a value of the forcing variable will be greater than x_0, and assign this to $\Pr(H \geq h)$, ie. $\Pr(H \geq h) = \Pr(X \geq x_0)$. Typically, an annual maximum, r-largest or a peak-over-threshold method is used to obtain the tail distribution of X (Coles, 2001).

The estimation procedure becomes complicated in a multivariate setting since the exceedance probability of any given forcing process is no longer equivalent to the exceedance probability of the flood level, ie. $\Pr(X \geq x_0) \neq \Pr(H \geq h)$. Considering the design variable H forced by two variables X and Y, Coles and Tawn (1994) defined a 'failure region' A_h as:

$$A_h = \left\{(x, y) \in \mathbb{R}^2 : B(x, y) > h\right\} \tag{6.5.11}$$

where B(x,y) is a 'boundary function' that maps the two dimensional space of the forcing variables to the one dimensional response variable. In flood estimation, a combination of hydrologic and hydraulic models are typically used to obtain a flood level $h = B(x, y)$ as a function of boundary conditions such as rainfall and storm tide (x, y). The failure region A_h can be interpreted as the set of values of the constituent processes (x, y) that cause flood levels greater than a specified design flood level h. The corresponding exceedance probability $\Pr(H \geq h)$ is given as:

$$\Pr(H \geq h) = \int\int_{A_h} f(x, y) \tag{6.5.12}$$

where $f(x, y) = \partial F(x, y)/\partial x \partial y$ is the joint density function of the two variables X and Y at extreme levels, and $F(x, y)$ is their corresponding joint cumulative distribution function.

Figure 6.5.7 illustrates the difference between the univariate method (top panel) and the design variable method as an example of a joint probability method (bottom panel) for a hypothetical scenario in which floods are caused by two forcing variables X and Y. In the top panel, the grey shaded region represents the exceedance probability $\Pr(H \geq h)$, where h (the red dashed line) is determined by a single forcing variable (e.g. rainfall or storm tides). The grey shaded region in the bottom panel illustrates the exceedance probability for the region A_h where h (the solid red line) depends on both forcing variables. In the bivariate case, the probability $\Pr(H \geq h)$ can then be evaluated as the integral of the joint density $f(x, y)$ (thin blue contours) across the whole failure region A_h (Equation (6.5.12)).

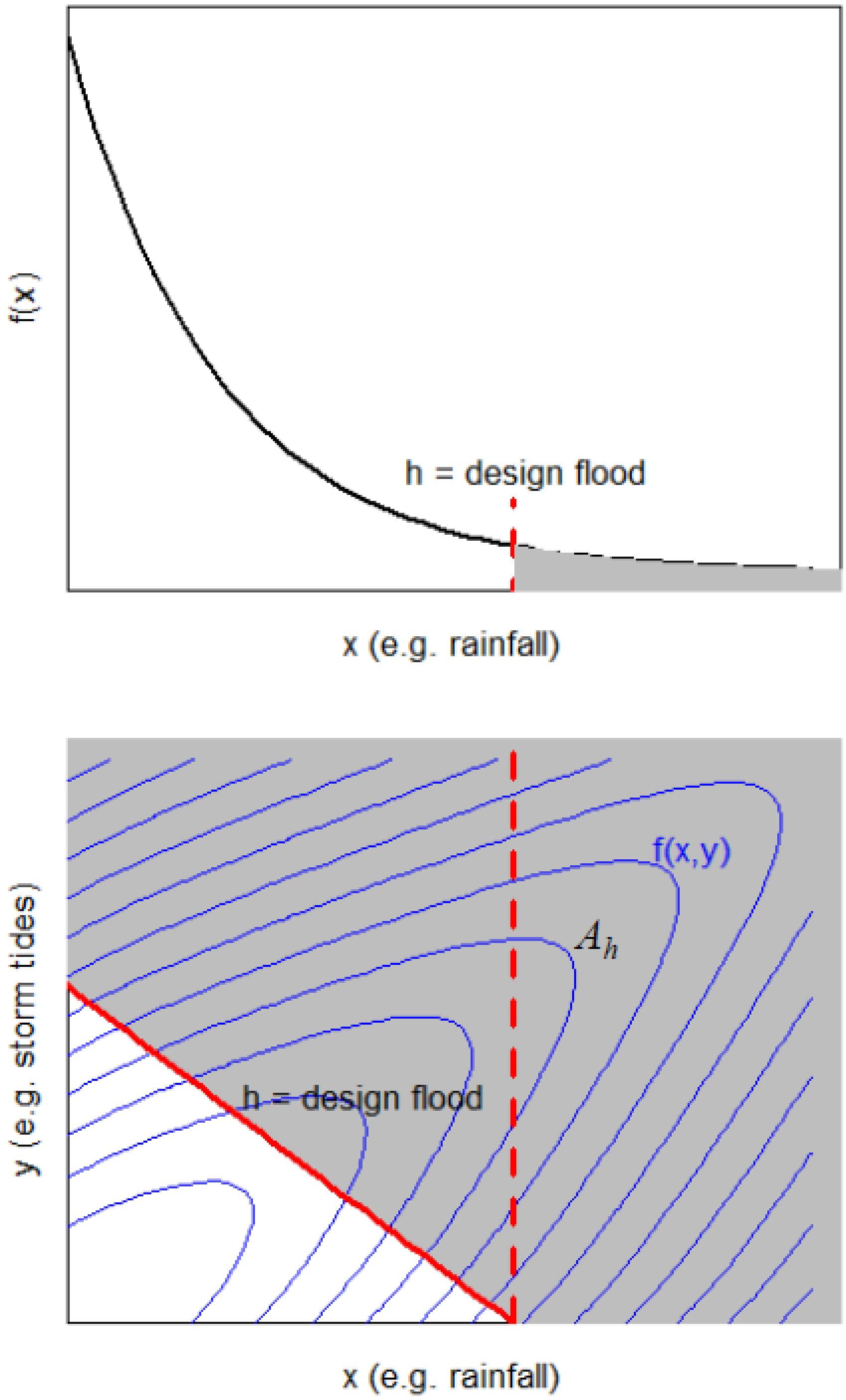


Figure 6.5.7. Exceedance Probabilities Obtained from a Univariate Analysis (top panel) and a Bivariate Analysis (bottom panel)

It is possible to compute the integral in Equation (6.5.12) using two dimensional numerical integration or Monte Carlo techniques, but these approaches can be slow for the required levels of precision. It is more computationally efficient to exploit the properties of the joint cumulative distribution function $F(x,y)$ to reduce the bivariate integral to a univariate line-integral along the boundary function $B(x,y) = h$. This is implemented numerically as:

$$\Pr(H \geq h) = 1 - \Pr(H < h) = 1 - \sum_{j=1}^{m-1} \left[F\left(x_j,y_j\right) - F\left(x_j,y_{j+1}\right) \right] \quad (6.5.13)$$

defined as $(x_j, y_j){:}B(x_j, y_j) = h, y_{j+1} - \Delta y$, where m is the number of points (x_j, y_j) that are used to discretise the boundary line h (the red solid line in Figure 6.5.7) and $\Delta y \geq 0$. By taking the finite difference of F(x,y) in only one dimension, the probability of being less than x_j is obtained for an increment of width Δy. Moving along the boundary function line, the probability increments are obtained for all corresponding pairs (x_j, y_j), and the non-exceedance probability $\Pr(H \geq h)$ is the sum over all increments.

Finally, the exceedance probability of flood event from the univariate method (the grey shaded region in the top panel) is also illustrated on the bivariate plot (the grey shaded region to the right of the red dashed line in the bottom panel). The univariate failure region is smaller than the A_h that would be obtained by the joint probability method, demonstrating that the univariate method will underestimate the exceedance probability of the flood in this case.

Asymptotic Dependence or Independence?

Flood estimation is often concerned with understanding the behaviour of the upper tail of a probability distribution. In the context of multivariate extremes, this requires assumptions about how the dependence between variables changes as the variables become increasingly extreme.

Multivariate probability distributions can be classified based on how they behave in the limit as each variable becomes increasingly extreme (refer to Coles (2001) for additional coverage of the theory of asymptotic dependence). Examples of an asymptotically independent and an asymptotically dependent distribution are given the Figure 6.5.8: the dependence between variables for the Gaussian distribution decreases with extremity of X or Y (evidenced by the increased scatter of points away from the leading diagonal), where dependence remains high for the asymptotically dependent bivariate logistic distribution.

A detailed study along the Australian coastline ((Zheng et al., 2013)) found that at most locations, the bivariate distribution between extreme rainfall and storm surge was asymptotically dependent, meaning that rarer events are more likely to occur jointly compared to more frequent events. This is the basis for the recommendation to use a bivariate logistic distribution for dependence analysis, and provides a cautionary note for using correlation-based measures (which assume Gaussianity) for representing joint dependence.

5.4.5. Illustration of Joint Probability Concepts

Several of the theoretical concepts of joint probability described above are now illustrated through a set of joint probability problems. The solution to each problem has been derived based on the simplifying assumptions of statistical independence or complete dependence, which means that the solutions in the tables below can be easily verified using hand calculations. For situations with intermediate levels of dependence, it is necessary to apply the full design variable method to calculate flood exceedance probabilities.

Results are presented both in terms of Annual Exceedance Probabilities (AEPs) and Average Recurrence Intervals (ARIs), using the conversion $AEP = 1 - e^{\frac{-1}{ARI}}$

The probability of two independent events Z = X>x or Y>y: Consider two independent random variables, X and Y. What is the probability of a 'failure event' Z = X > x or Y > y? This type of question might arise when (i) a system is considered to 'fail' when any component of the system fails, and (ii) the failure of any component of the system is independent of the failure of any other component. For example, a road might 'fail' when either of two bridges are overtopped, and the bridges are sufficiently far from each other so that it is possible to assume the flood producing mechanisms are approximately independent[2].

The set of events X>x is shown by the vertical blue lines in Figure 6.5.8, and the set of events Y>y is shown by the horizontal blue lines in Figure 6.5.8. Start by considering only the probability of two variables exceeding their respective thresholds in a given year (but potentially on different days). The question of calculating the probability of the two events coinciding (ie. occurring at the same time) is considered in a later example.

The exceedance probabilities corresponding to those thresholds is AEP_x = Pr{X>x} and AEP_y = Pr{Y>y}. The Annual Exceedance Probability of the failure event, Z, is then given as AEP_z = Pr{X>x or Y>y}. With reference to the illustration in Figure 6.5.8, it is straightforward to see that $AEP_z = AEP_x + AEP_y - (AEP_x * AEP_y)$; the reason for the subtraction term is because the cross-hatched region in Figure 6.5.8 would otherwise have been counted twice. Example calculations of AEP_z assuming a number of different combinations of X and Y are presented in Table 6.5.3.

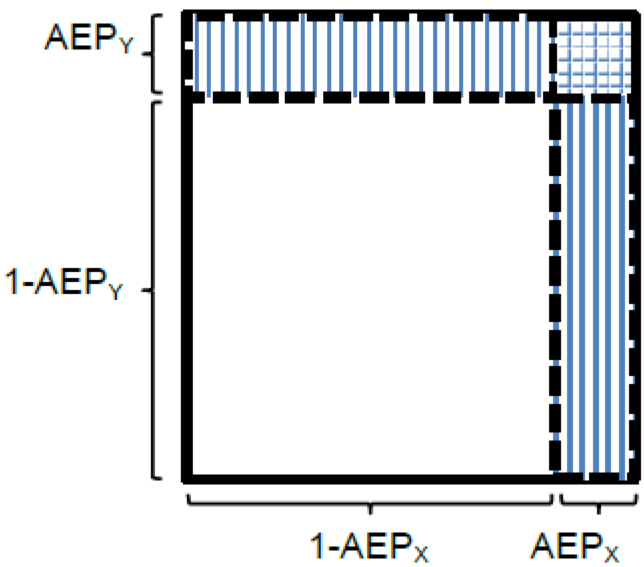


Figure 6.5.8. Illustrating the Probability of Two Independent Events Z = X>x or Y>y

Table 6.5.3. Worked Examples of the Probability of Two Independent Events Z = X>x or Y>y

AEP_X	AEP_Y	AEP_X(years)	AEP_Y(years)	AEP_Z	ARI_Z (years)
1.00%	65.0%	99.5	1.0	65.4%	0.94
2.00%	40.0%	49.5	2.0	41.2%	1.88
5.00%	20.0%	19.5	4.5	24.0%	3.64

[2]Given that rainfall is a spatial process, the assumption that extreme rainfall at two nearby locations is statistically independent is unlikely to be valid; it is made here for illustration purposes only.

10.00%	10.0%	9.5	9.5	19.0%	5.75

The probability of Two Independent Events Z = X>x and Y>y: An alternative question concerns the probability of *both* X and Y exceeding their specified thresholds.

This situation is illustrated as the hatched region in Figure 6.5.9. Defining AEP_z as Pr{X>x and Y>y}, AEP_z = AEP_x * AEP_x can be estimated, with a number of specific examples shown in Table 6.5.4.

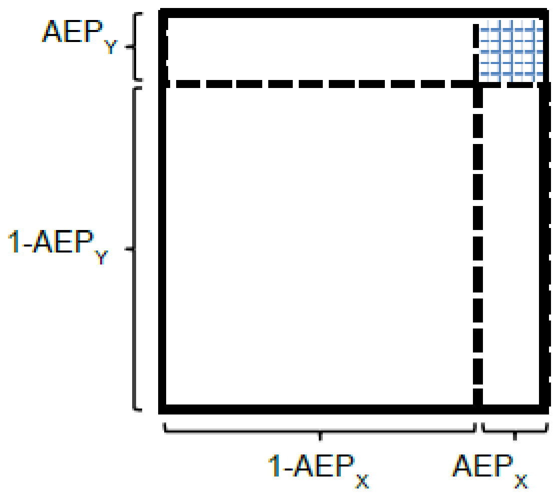


Figure 6.5.9. Illustrating the Probability of Two Independent Events Z = X>x and Y>y

Table 6.5.4. Worked Examples of the Probability of Two Independent Events Z = X>x and Y>y

AEP_X	**AEP_Y**	**AEP_X(years)**	**AEP_Y(years)**	**AEP_Z**	**ARI $_Z$(years)**
1.00%	65.0%	99.5	1.0	0.65%	153.3
2.00%	40.0%	49.5	2.0	0.80%	124.5
5.00%	20.0%	19.5	4.5	1.00%	99.5
10.00%	10.0%	9.5	9.5	1.00%	99.5

The Probability of Two Completely Dependent Events Z = X>x and Y>y: In situations of perfect dependence between X and Y, the probability of X>x would be equal to the probability of Y>y for all x and y. Because of this, Pr{X>x} = Pr{Y>y} = Pr{X>x and Y>y}. For example, if AEP_x and AEP_y are both 10%, then AEP_z = 10%.

The probability that Two Events X>x and X>y with Specified Annual Exceedance Probabilities Pr{X>x} and Pr{Y>y} Occur on the Same Day: In the previous examples, the interest was in the probabilities that two random variables X and Y exceeded thresholds x and y in a given year. However, when estimating the exceedance probability of floods, our interest is in the probability of these two variables coinciding. Therefore, one must consider the probability that both variables reach their maxima at the same time within a given year.

This issue can be illustrated by considering the case where the daily maximum storm tide is assumed to coincide with the daily maximum rainfall. This is still a conservative assumption (since the peak of the hydrograph will not always occur at exactly the same time as the peak of the storm surge within a given day), but less conservative than the assumption that the annual maximum of variable X will always occur at the same time as the annual maximum of variable Y.

The conversion between an AEP and a Daily Exceedance Probability (DEP) is DEP=AEP/365. For the example of 1 year ARI event, there is a 63% chance that any given year will exceed that level (AEP), and a corresponding 0.17% chance that any given day will exceed that same level (DEP).

The earlier example is now revisited (the probability of two independent events Z = X>x and X>y), but now first converting to daily values. Table 6.5.5 shows the results using the example of two coinciding 10% AEP events. It is clear from this example that the joint exceedance probability is much lower than the results presented in Table 6.5.4 (since, in the absence of dependence, the most likely case is that the two extreme events would occur on different days). In contrast, had complete dependence been assumed, then AEP_z would remain at 10%, as by the definition of complete dependence, high values of X and Y will always occur at the same time.

Table 6.5.5. Calculating the Probability of Two Independent Events Z = X>x and Y>y, When Adding the Constraint that Both Events Must Occur on the Same Day

AEP_X	AEP_Y	DEP_X	DEP_Y	ARI_X(years)	ARI_Y(years)	DEP_Z	AEP_Z	ARI_Z (years)
10.0%	10.0%	0.027%	0.027%	9.5	9.5	(7.51E-06)%	0.00274 %	36500

Why the Probability That X>x or Y>y is Non-commensurate with the Probability that H>h Where $h = B(x, y)$, Even When Random Variables X and Y are Independent: This illustrates a basic problem in joint probability analysis of floods in the coastal zone, where interest is in a quantity such as a flood level (h) that is some complex function of forcing variables such as rainfall (X) and storm tide (Y). A 'boundary function' $B()$ is used to represent the complex mapping from rainfall/storm tide to the flood level. In most practical applications this mapping would be achieved using hydrologic and hydraulic models that take rainfall and storm tide as their boundary conditions, and produce flood level as their output.

Consider the situation concerning the exceedance probability of a flood of height H>h, and after a hydrological/hydraulic analysis concluded that this height can be caused by a rainfall event with AEP_x= 5% but with no significant storm tide, or a storm tide event with AEP_y= 20% but with no significant rainfall. Furthermore, assume that the processes X and Y are independent. In this case, it is tempting to refer to Table 6.5.3 and suggest that the AEP of the flood becomes 24.0%.

A problem with this calculation is that it neglects floods that can occur through a combination of smaller values of rainfall and storm tide. This is illustrated by the red hatched region in Figure 6.5.10. Therefore, even if it is assumed that extreme rainfall and storm tide are statistically independent, it will still be necessary to apply the design variable method to compute flood exceedance probabilities.

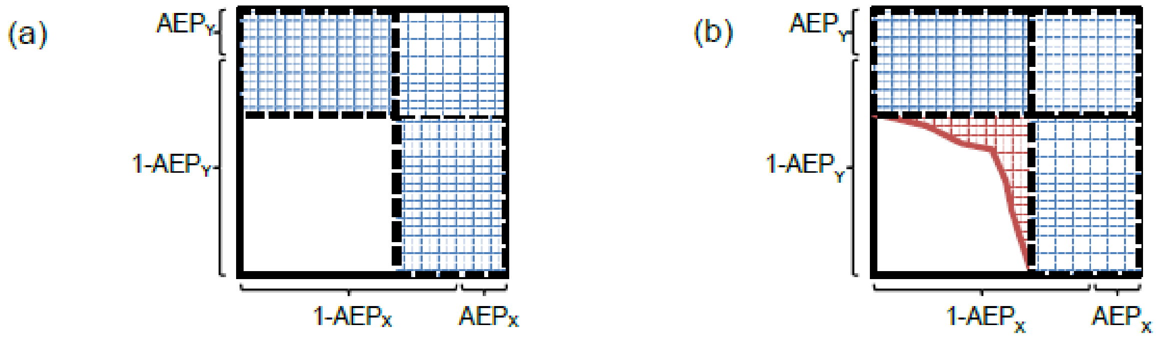


Figure 6.5.10. Conceptual Diagram (a) Probability of Floods Caused by Either a Significant Rainfall Event or a Significant Storm Tide Event, and (b) Additional Probability of Floods Produced by Combinations of Smaller Rainfall and Storm Tide Events

Accounting for Intermediate Levels of Dependence Between X and Y: The previous examples illustrated the extreme situations whereby the processes X and Y were either completely independent, or perfectly dependent. The design variable method has been designed to cater for cases with intermediate levels of dependence. The method superimposes the joint probability distribution of the forcing variables X and Y onto the boundary function $B()$ describing the relationship between forcing variables and flood level. The practical implementation of this method is discussed in Book 6, Chapter 5, Section 5.

5.5. The Design Variable Method

This section describes a practical approach to implement the design variable method, which comprises four distinct steps (Book 6, Chapter 5, Section 5). Further detail on each step is given below.

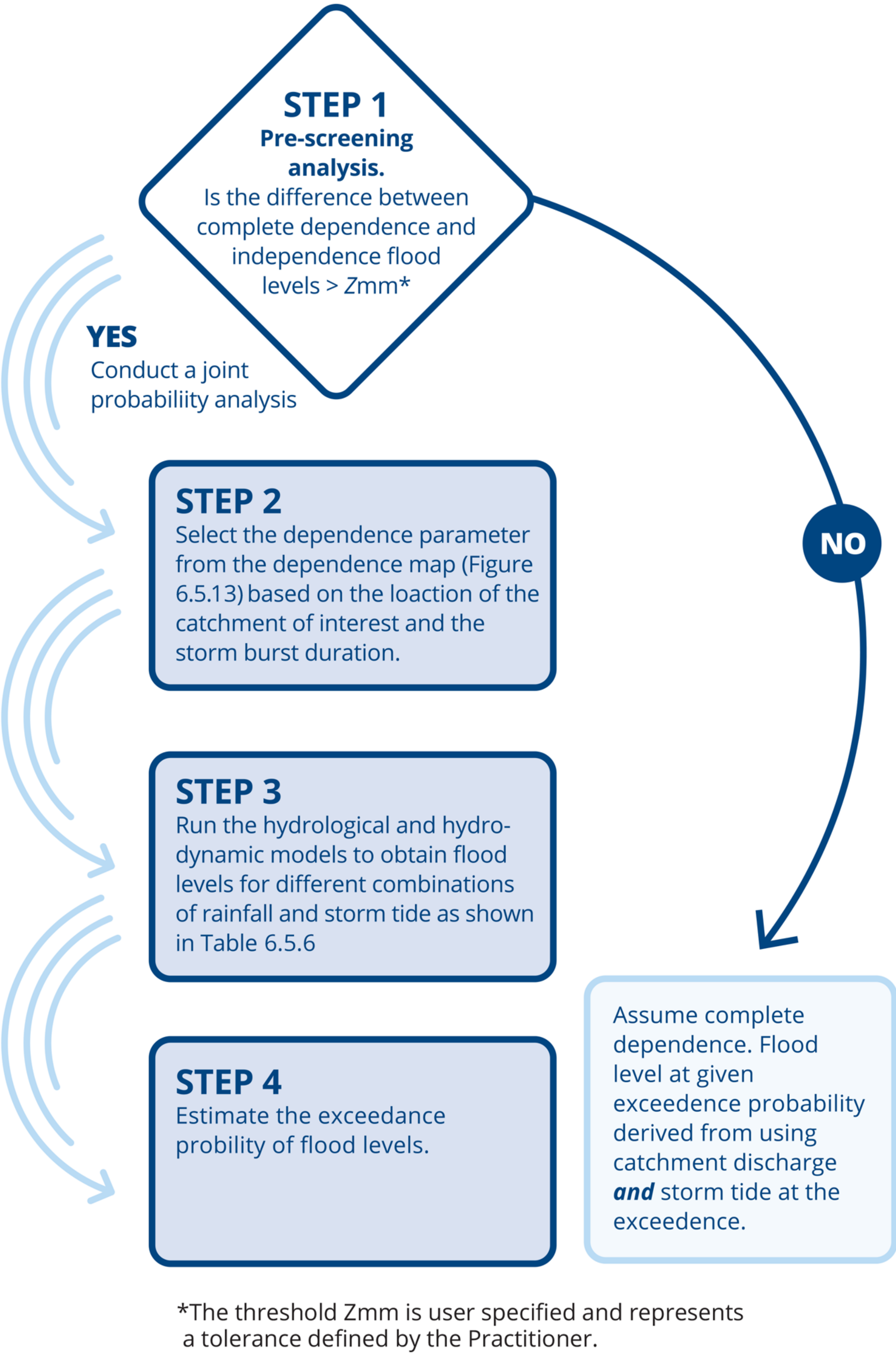


Figure 6.5.11. The Design Variable Method

Step 1: Pre-Screening Analysis

Accounting for dependence between extreme rainfall and storm surge as part of estuarine flood assessments represents significant additional computational effort when compared to traditional univariate methods. Therefore, a pre-screening analysis is recommended to determine whether the additional complexity of a joint probability analysis is warranted.

The aim of this step is to calculate the outer envelope of flood estimates obtained from the joint probability method. This involves calculating a minimum number of cases to determine the magnitude of flood differences between independence and full dependence:

1. the independence case where a fluvial flood occurs in the absence of an ocean event; (2) the independence case where a coastal flood occurs in the absence of a rainfall event; and

2. the full dependence case where both a fluvial flood and a coastal flood occur simultaneously.

The specific number of runs required in the pre-screening analysis will depend on the number of AEPs that need to be evaluated. Table 6.5.6 presents the pre-screening analysis for three AEPs, which requires nine instances ('runs') of a hydrological and hydrodynamic model. The boundary conditions are specified in terms of their AEP rather than in their corresponding dimensional value (e.g. m^3/s, m, etc), so that it will be necessary to consider the probability distribution of the extremes of each boundary to enable translation between an AEP and the dimensional value.

Table 6.5.6. Flood Levels of Different Combinations of Rainfall and Storm Tide in Terms of Annual Exceedance Probability, for a Particular Storm Burst Duration. Only the highlighted cells need to be evaluated.

Rainfall Events in AEPs	**Storm Tide Events in AEPs**			
	Lower Bound	20%	2%	1%
No Rainfall		blue	blue	blue
20%	green	red		
2%	green		red	
1%	green			red

The pre-screening analysis should be undertaken at each cross-section or grid cell in the floodplain where information is required, with a longitudinal profile of the independence and complete dependence cases illustrated for a single exceedance probability in Figure 6.5.12. The pre-screening analysis involves classifying each cross-section and AEP into one of the following cases.

Case 1 - If the flood levels in the green cells are similar to those in the red cells for each rainfall AEP (ie. the difference is less than some tolerance threshold value of Z mm), the flood levels for the catchment of interest are completely dominated by the rainfall (the 'fluvial zone' in Figure 6.5.12). Normally such catchments are in upstream reaches of the river. For this case, complete dependence should be assumed; the AEP of a flood level is obtained when the same AEP of the rain and storm surge are assumed to coincide (red cells). While this is a conservative assumption, it eliminates the need for modelling a much larger number of combinations.

Case 2 - If the flood levels in the blue cells are very close to those in the red cells for each storm tide AEP (ie. the difference is less than the tolerance threshold of Z mm), the flood levels are completely dominated by the storm tide (the 'coastal zone' in Figure 6.5.12). Normally this location is in lower reaches of the river. As with Case 1, complete dependence should be assumed, and the flood level should be obtained based on the combinations in the red cells.

Case 3 - If the flood levels in the red cells are significantly higher than those in the green and blue cells (ie. the difference is greater than the tolerance threshold of Z mm) with the same rainfall and storm tide AEPs, this indicates that the joint dependence has a significant influence on the flood level (the 'joint probability zone' in Figure 6.5.12). It will be necessary to continue to Step 2 and conduct a full joint probability analysis.

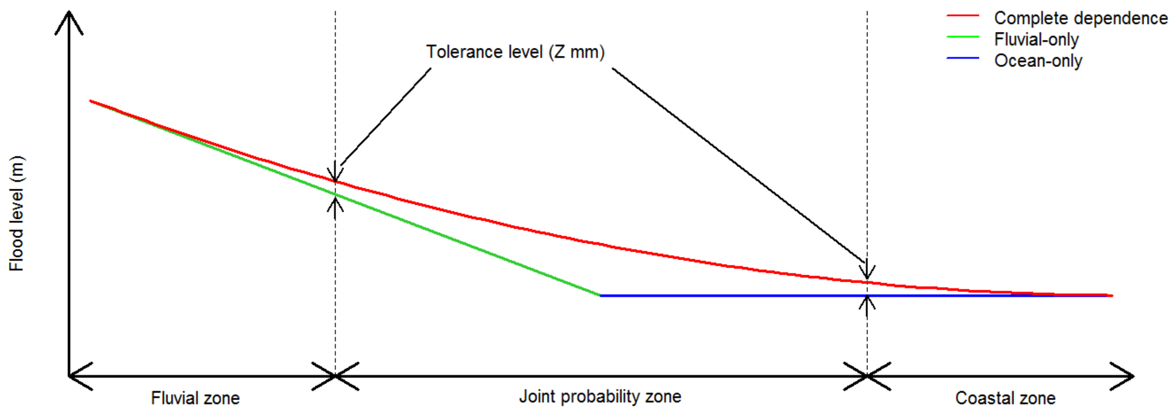


Figure 6.5.12. Pre-Screening Step, which Involves Calculating the Outer Envelope of the Possible Flood Levels.

The threshold value of Z represents a tolerance defined by the practitioner. This tolerance is a trade-off between the benefit of a more accurate assessment of flood exceedance probabilities (obtained through the joint probability calculation) and the additional effort required to implement a joint probability analysis. A joint probability analysis has additional computational cost and this cost should be proportional to the benefit of the additional precision. This trade-off will vary according to different locations and design problems. As illustrated in Figure 6.5.12, the tolerance is also used to formally define the 'joint probability zone' that was first introduced in Figure 6.5.1.

It should be noted that if one is only interested in a single AEP (rather than a range of AEPs) then only three runs are required instead of the nine runs in Table 6.5.11. For example, if only the 1% AEP is of interest, then the three model runs are: (i) an event with 1% AEP rainfall combined with the lower bound of the storm tide; (ii) an event with 1% AEP storm tide combined with no rainfall; and (iii) an event with the 1% AEP rainfall combined with the 1% AEP storm tide.

If a joint probability analysis is required then proceed to Step 2.

Step 2: Dependence Parameter Selection

A map of dependence parameters from the bivariate logistic extreme value model has been created for the Australian coastline (Figure 6.5.13). The map was derived based on an analysis of the joint dependence using data from 64 tide gauges, 7684 daily rainfall gauges and 70 sub-daily rainfall gauges, and is described in more detail in Zheng et al (Zheng et al., 2014a). The dependence parameters are available for storm burst durations shorter than 12 hours, between 12 and 48 hours, and between 48 and 168 hours. Note that values closer to one represent weaker dependence, and values closer to zero represent stronger dependence.

This step involves selecting the dependence parameter from the map (Figure 6.5.13). The duration should be estimated with reference to the catchment time of concentration. Values closer to 1 represent weaker dependence, and values closer to 0 represent stronger dependence.

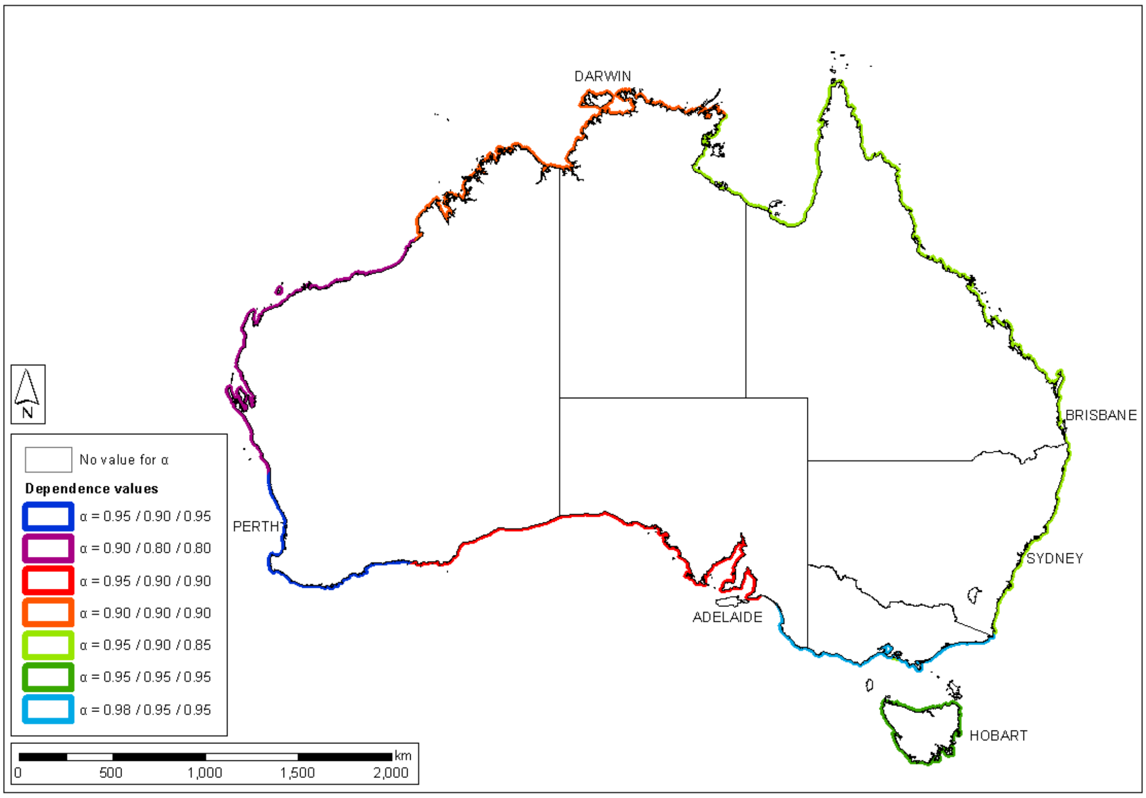


Figure 6.5.13. Dependence Parameter (α) Map for the Basins of the Australian Coastline - shorter than 12 hours, 12 to 48 hours, and 48 to 168 hours

Step 3: Flood Level Modelling

In this step, the flood level corresponding to a number of scenarios of rainfall and storm tide needs to be evaluated to accurately estimate flood levels incorporating dependence. The scenarios should include no rainfall and the lower bound of the storm tide cases to represent the lowest possible value of each variable. The scenarios should also consider cases with exceedance probabilities lower than the smallest AEP (ie. largest flood level) of interest. Up to the 1% AEP, a typical example is given in Table 6.5.7 which has seven cases for each variable leading to 49 runs of a hydrologic and hydrodynamic models.

Table 6.5.7. Flood Levels of Different Combinations of Rainfall and Storm Tide in Terms of Annual Exceedance Probability with a Particular Storm Burst Duration

	Storm Tide Events (AEP)

		Lower Bound	**50%**	**20%**	**10%**	**2%**	**1%**	**0.2%**	**0.05%**
Rainfall Events (AEP)	**No Rain**								
	50%								
	20%								
	10%								
	2%								
	1%								
	0.2%								
	0.05%								

Step 4: Estimate the Exceedance Probability of Flood Levels

The final step of the analysis involves superimposing the flood level table (Table 6.5.7) onto the joint probability density function of the bivariate logistic extreme value distribution, and in the context of estimating the probability of a specific design flood level, *h* will involve the following steps:

1. Using the bivariate logistic extreme value model of Equation (6.5.8) with the dependence parameter estimated in Step 2, estimate the bivariate probability distribution function corresponding to the extreme rainfall and storm tide. This is represented as the blue contours the hypothetical example that was presented in Figure 6.5.7.

2. Using the data obtained in Step 3 (Table 6.5.7), estimate the set of all possible combinations of extreme rainfall and storm tide that would produce flood level h. This involves interpolating over the values in Table 6.5.7. The contour of fixed, *h*, was illustrated as a solid red line in the hypothetical example presented in Figure 6.5.7.

3. Integrate the bivariate probability distribution function to the right of (ie. above) the design flood level to obtain the exceedance probability of that flood event. This is represented in Figure 6.5.7 as the integration of the blue contours over the grey shaded region.

If the objective of the analysis is to find the design flood level corresponding to a specific AEP, then the above steps need to be repeated for a number of flood levels until the flood level corresponding to the desired AEP is identified.

A software tool[3] has been developed to perform these calculations. This tool requires as inputs the dependence parameter and the flood level table, and will produce a plot of water levels against AEPs. Implementation of the software is illustrated using worked examples in Book 6, Chapter 5, Section 7. At present the software implementation of the method has been tested for flood levels for the 50% to 1% AEP. The software tool is needed primarily for Step 4.2 and 4.3. To determine contour lines in Step 4.2 there are a number of standard libraries, but to determine the integral in Step 4.3 a customised routine is required for implementing Equation (6.5.13).

Finally, a note of caution is required regarding the identification of the storm burst duration in Step 2. In that step, it was recommended that the storm burst duration was selected based on the time of concentration of the catchment, with the reasoning that this would lead to the

[3]http://p18.arr-software.org/

maximum flow rate. Assuming static tailwater levels (which is a fundamental assumption of the design variable method; see Book 6, Chapter 5, Section 3) and a constant dependence parameter, this would be equivalent to the duration that would lead to the largest flood event. However, because the dependence parameter depends on duration, it is possible that some storm burst duration could result in a lower peak flow rate but nonetheless lead to a higher flood level because the peak flow is more likely to coincide with the peak ocean level. Therefore, if the identified storm burst duration identified in Step 2 is close to a threshold between dependence parameters, it may be necessary to test the implications of an adjacent duration with a lower dependence parameter (ie. stronger dependence).

Accounting for Climate Change

Anthropogenic climate change is likely to increase the exceedance probabilities of flooding in estuarine regions, owing to a combination of elevated ocean levels arising from increases in both mean sea level and possible changes in storm surges, as well as increases in extreme rainfall. Furthermore, climate change may result in changes to the frequency and magnitude of different types of extreme weather events, which may affect the magnitude of dependence between extreme rainfall and storm surge/tide.

Information on how the dependence between extreme rainfall and storm surge/tide may change in a future climate is currently unavailable, and therefore guidance on possible changes to the dependence parameters in Figure 6.5.13 cannot be provided at this stage. Guidance is available, however, on possible changes to both extreme rainfall and mean sea level (Book 1, Chapter 6). As an interim measure, it is recommended that estimates of the impact of climate change on flooding in the joint probability zone be accounted for as follows[4]:

- Changes to extreme rainfall and ocean level should be estimated using the approach described in Book 1, Chapter 6); and
- The dependence parameters described in Figure 6.5.13 that correspond to the historical climate situation should be used unless more precise estimates of future dependence parameters are available.

The implication of changing both the extreme rainfall intensity and the ocean levels is illustrated in Figure 6.5.14. Using the adjusted rainfall and ocean levels, the four step methodology described earlier in this section then can be applied. It is noted that a possible effect of climate change is that the tidally affected part of a river is likely to change (for example, it may reach further upstream due to the effects of sea level rise), and this will influence the area classified requiring a full joint probability analysis. Therefore the pre-screening analysis in Step 1 will also need to be repeated when considering climate change.

[4] This section was written before the latest climate change guidance in Book 1, Chapter 6 (2024). A minor change to the text has been made to reflect the change in guidance.

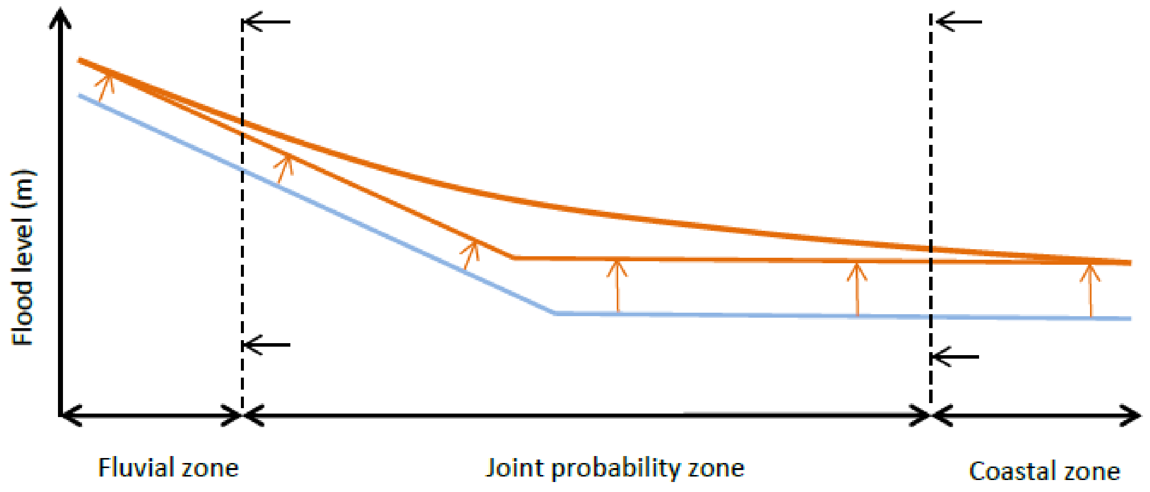


Figure 6.5.14. Interim Approach to Account for the Effects of Climate Change

To account for climate change, two alternative methods for adjusting the flood level table (Table 6.5.7) are proposed:

- The table can be populated using the climate change-affected rainfall and ocean levels as upper and lower boundary conditions to the hydrologic/hydraulic models, which would require repeating all the simulations to account for the changes in the rainfall and ocean level values; or
- The historical flood level table can be used but the exceedance probabilities of the extreme rainfall and storm tide can be modified to reflect future exceedance probabilities. This can eliminate the need for additional hydrologic and hydraulic runs, although it is possible that additional simulations may still be required for low exceedance probability events.

5.6. Worked Example 1 — Hawkesbury/Nepean River

This worked example illustrates the basic implementation of the design variable method and should be read in conjunction with Book 6, Chapter 5, Section 5. The example demonstrates each of the four steps of the method applied to multiple sections of a river reach, and provides flood level estimates with respect to location, dependence level and AEP.

The hydraulic and hydrologic models for the Hawkesbury-Nepean River system were originally developed as part of an Environmental Impact Statement in the 1990s for works to upgrade the spillway capacity of Warragamba Dam. The study included a detailed analysis of the existing flooding behaviour and was carried out by Webb McKeown and Associates (1996). The outcomes were subject to rigorous technical reviews by a range of parties including Sydney Water, the then Department of Land and Water Conservation, the Bureau of Meteorology and other experts.

The hydrologic model used was RORB, and the hydraulic model was RUBICON. The RORB model was calibrated and then evaluated using historical records available for five of the events between June 1964 and April 1988. The RUBICON hydrodynamic model software was used to quantify the hydraulic aspects of the flood behaviour (e.g. flood levels and velocities). RUBICON is a fully dynamic one dimensional (1D) model and uses different elements to simulate complex flow over floodplains and through channel systems. The hydraulic model covers the entire area from Lake Burragorang to the ocean at Broken Bay. The process of calibrating and evaluating the RUBICON model was undertaken using recorded information for 11 individual historical events. The models were then used to determine design flood behaviour of the system. The calibrated RUBICON model of the Hawkesbury-Nepean has been maintained by WMAwater (previously Webb McKeown and Associates) since this original study and was selected for this project as there is a very high flood gradient along the river even though the river is tidal for 140 km upstream of the estuary inlet under non-flood conditions.

Step 1: Pre-Screening Analysis

As described in Book 6, Chapter 5, Section 5, a pre-screening analysis is recommended as a first step to identify whether a full joint probability analysis is warranted for the problem being considered. The basis of the pre-screening analysis is to assess whether flood levels corresponding to the extreme cases of complete dependence and complete independence are sufficiently different from each other, which is determined with reference to a tolerance threshold.

Consider the location of Liverpool which is 80 km upstream from the ocean boundary. Assume that the practitioner specifies a tolerance of 0.1 m for the design in question. Assume also that the user is interested in a design at the 2% AEP level. Three model runs are required:

1. *Completely dependent* — flow boundary at 2% AEP and ocean boundary at 2% AEP.

2. *Flow boundary only* — flow boundary at 2% AEP and ocean boundary at 100% AEP.

3. *Ocean boundary only* — flow boundary at 100% AEP and ocean boundary at 2% AEP.

The water levels resulting from these three runs are summarised in Table 6.5.8. For the dependent case, the water level is 9.590 m. For the independent case, the water level is obtained by taking the highest water level from either the flow boundary only case or the ocean boundary only case. For this location, the flow boundary only case dominates, and leads to a flood level of 9.537 m. The difference between the dependence and independence cases is 0.053 m, which is within the specified tolerance. This implies that this cross-section is not highly sensitive to the ocean level, and is thus in the 'fluvial zone'. The fully dependent value of 9.590 m is therefore used as the best approximation to the 2% AEP event, without having to implement the design variable method. This analysis is only valid for the 2% AEP level, and should be repeated for other AEPs if there is interest in analysing other exceedance probabilities.

Table 6.5.8. Model-Derived Water Levels (mAHD) for Given Pairs of Tide and Rainfall Boundary Input Conditions for a Cross-section Located at Liverpool (Chainage–80 300)

		Storm Tide (% AEP)	
		Lower Bound	**2**
Flow (% AEP)	**No Rain**		*1.381*
	2	*9.537*	*9.590*

By repeating the pre-screening analysis at multiple locations along a river, the extent of the joint probability zone can be defined. Two examples of dependent locations for this case study are Olga Bay and Spencer, with 2% AEP model runs shown in Table 6.5.9 and Table 6.5.10 respectively. The difference between the dependent and independent cases at both these locations is greater than the tolerance of 0.1 m, indicating the influence of both boundary conditions. At these locations, it is therefore necessary to implement the design variable method to determine the 2% AEP water level. AtSpencer, in particular, the difference in flood level based on the dependent and independent cases is 0.614 m, suggesting potentially significant discrepancies depending on the joint probability assumption at this location.

Table 6.5.9. Pre-Screen Analysis Pairs at Olga Bay (Chainage–20 400)

		Storm Tide (% AEP)	
		Lower Bound	**2**
Flow (% AEP)	**No Rain**		*1.258*
	2	*0.286*	*1.397*

Table 6.5.10. Pre-Screen Analysis Pairs at Spencer (Chainage–34 700)

		Storm Tide (% AEP)	
		Lower Bound	**2**
Flow (% AEP)	**No Rain**		*1.306*
	2	*1.876*	*2.490*

Step 2: Dependence Parameter Selection

For the location of this case study, assume that the dependence parameter is 0.9 (refer Figure 6.5.9).

Step 3: Flood Level Modelling

The design variable method requires many combinations of boundary conditions. Table 6.5.11 and Table 6.5.12 are examples of the hydraulic response at Olga Bay and Spencer, respectively. The design variable method does not require the same number of runs for each boundary condition (here there are five storm tide cases and 10 flow cases), nor do the marginal probabilities have to be identical. Where there is sufficient prior opportunity, the marginal probabilities could be selected to be standard values (e.g. 1, 2, 5, 10, …) but in many instances (as in Table 6.5.11 and Table 6.5.12), they will be back-calculated from existing model runs.

The hydraulic response table should include runs where the 100% exceedance probability of each margin is considered, and there should be a wide range of AEPs. The range of AEPs for the margins should include events that are rarer than the AEPs being calculated for the water level (since, for example, a 1% AEP water level could hypothetically arise from the combination of a 10% AEP flow and a 0.1% AEP storm tide). The total number of model runs (here 5 x 10 = 50 runs) is likely to be the limiting factor for the feasibility of the method (especially where two dimensional (2D) hydrodynamic models are used) and this will govern the resolution at which the table is evaluated (Zheng et al., 2015).

Table 6.5.11. Model-Derived Water Levels (mAHD) for Given Pairs of Storm Tide and Rainfall Boundary Input Conditions for a Cross-section Located at Olga Bay (Chainage–20 400).

	Storm Tide (% AEP)

		Lower Bound	30	2	0.25	0.0025
Flow (% AEP)	**No Rain**	0.001	1.048	1.258	1.466	1.678
	18.1	0.114	1.084	1.279	1.482	1.687
	9.5	0.129	1.094	1.292	1.494	1.696
	4.9	0.175	1.129	1.321	1.515	1.708
	2	0.286	1.209	1.397	1.586	1.776
	1	0.429	1.316	1.498	1.682	1.866
	0.5	0.686	1.509	1.681	1.854	2.03
	0.2	0.982	1.735	1.895	2.057	2.222
	0.1	1.364	2.033	2.177	2.325	2.476
	0.01	1.449	2.101	2.241	2.387	2.534

Table 6.5.12. Model-Derived Water Levels (mAHD) for Given Pairs of Storm Tide and Rainfall Boundary Input Conditions for a Cross-section Located at Spencer (Chainage–34 700)

		Storm Tide (% AEP)				
		Lower Bound	**30**	**2**	**0.25**	**0.0025**
Flow (% AEP)	**No Rain**	0.002	1.09	1.306	1.519	1.737
	18.1	0.913	1.626	1.782	1.941	2.104
	9.5	1.007	1.694	1.845	2.000	2.159
	4.9	1.29	1.909	2.049	2.193	2.341
	2	1.876	2.374	2.49	2.612	2.737
	1	2.497	2.895	2.99	3.089	3.194
	0.5	3.343	3.643	3.714	3.791	3.873
	0.2	4.122	4.345	4.402	4.462	4.526
	0.1	4.919	5.091	5.134	5.181	5.231
	0.01	5.083	5.247	5.286	5.334	5.378

Figure 6.5.15 is a plot of the water level contours that have been interpolated from the hydraulic response tables for Olga Bay and Spencer. These plots provide a consistency check of the water levels in the tables. Vertical lines imply that the storm tide (in this case the X variable) is the dominant process affecting the water level, whereas horizontal lines imply that the flow (the Y variable) dominates the water level. Any other slope between these two indicates variation with respect to both inputs.

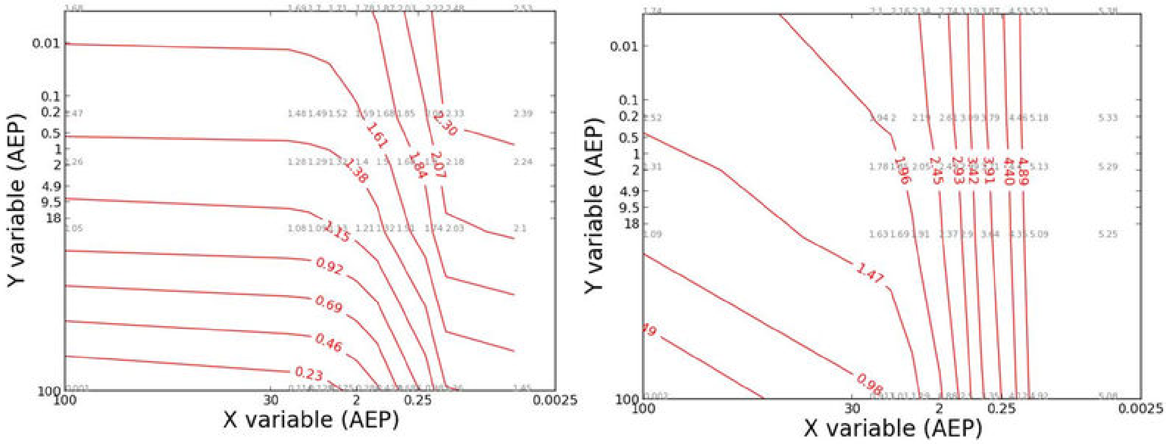


Figure 6.5.15. Interpolated Contours for Olga Bay (left) and Spencer (right) Corresponding to the Water Levels in Table 6.5.11 and Table 6.5.12.

Step 4: Estimate the Exceedance Probability of Flood Levels

Figure 6.5.16 shows the output water levels at the two locations. For Olga Bay the best estimate (solid black line) is very similar to the independence case. Figure 6.5.16 also shows that the difference between complete dependence and independence is a function of the AEP (AEPs >10% are very similar, but AEPs <10% diverge between these two cases). For Spencer, the best estimate lies approximately midway between the complete dependence and independence cases. This demonstrates that the relationship of α is non-linear with respect to the resulting water levels and that it varies with location. Specifically, although α varies from zero (complete dependence) to one (independence), α=0.90 does not necessarily mean the water level is ‘near independence’.

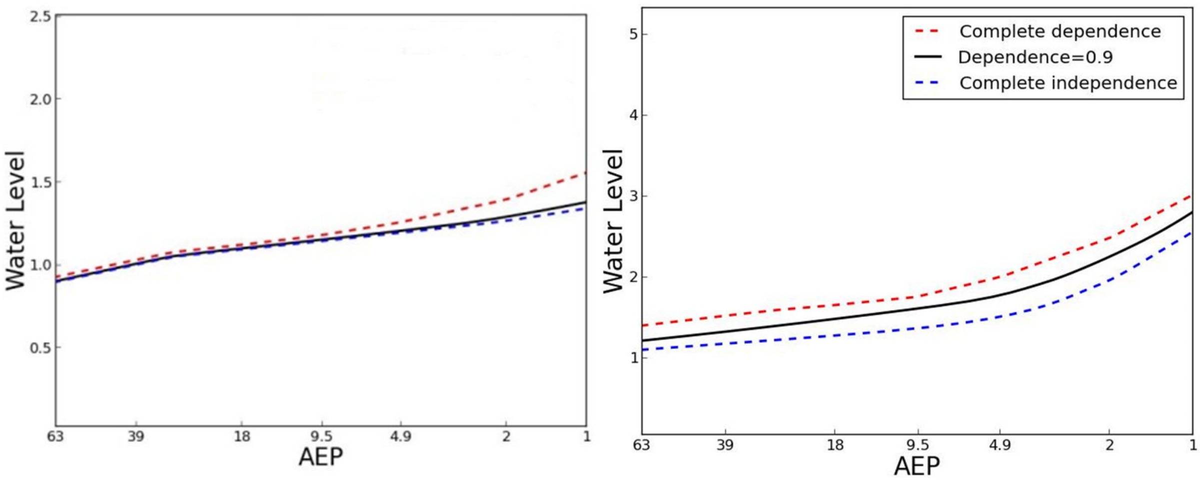


Figure 6.5.16. Water Levels at Olga Bay (left) and Spencer (right) Corresponding to Cases of Complete Dependence, Complete Independence and the Best Estimate when α=0.9

A longitudinal plot can be generated by repeating the analysis for multiple cross sections (Figure 6.5.17). The joint probability zone is indicated as the region where the difference between the complete dependence and independence cases is greater than the defined tolerance. From Figure 6.5.17 it is clear that Spencer is situated in the middle of this zone, and that Olga Bay – being closer to the ocean boundary – is less affected by the joint dependence. The extent of the zone also depends on AEP, as the joint dependence is more important for more frequent events, and this leads to a longer extent of the zone (e.g. compare the range of distance over which there is a noticeable difference between dependence and independence cases, for the 10% and 1% AEP respectively).

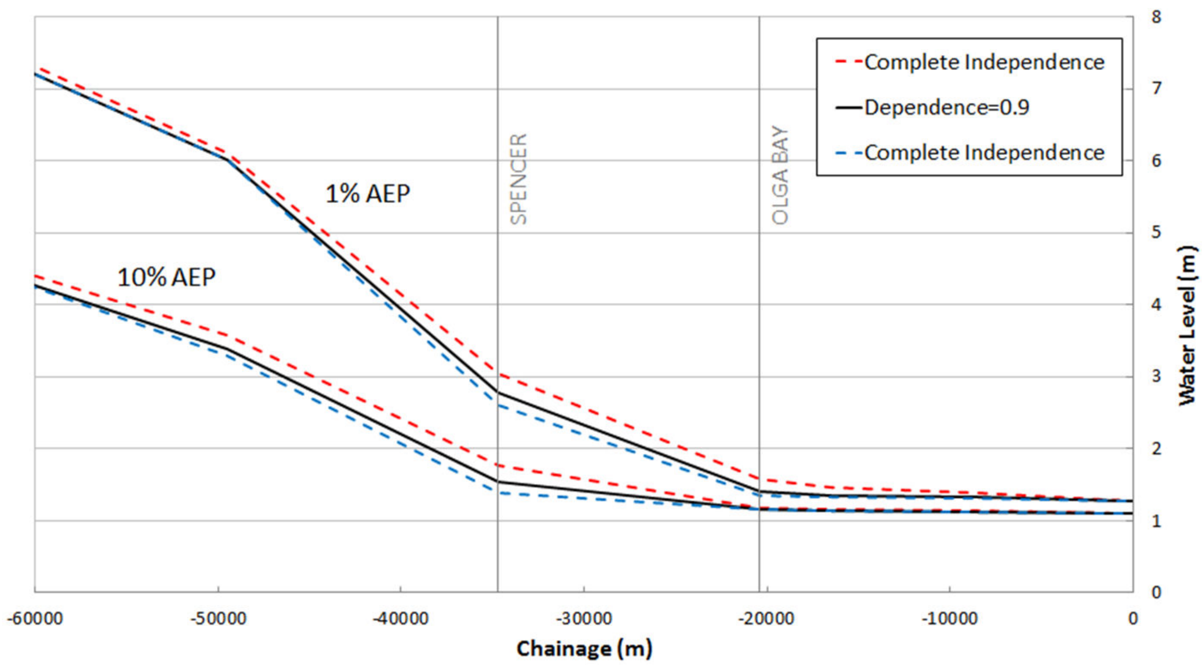


Figure 6.5.17. Longitudinal Comparison of 1% AEP and 10% AEP Water Levels

5.7. Worked Example 2 — Nambucca River

The Nambucca River catchment is located in northern New South Wales. Based on work prepared for the Nambucca Shire Council, modelled flood levels for combinations of boundary conditions were provided from a Tuflow 1D-2D hydrodynamic model ((WMAwater, 2013)). The model is of the Nambucca River, Warrell Creek and tributaries, and covers a catchment area of 1315 km^2. The model was calibrated to peak flood survey levels (1890-2011) and large historical events (1972, 1977, 2009) recorded at gauges located at Bowraville, Macksville, Stuarts Island and Utungun.

Step 1 Pre-Screening Analysis

Model runs for a pre-screening analysis at three different AEPs (9.5%, 2% and 1%) are shown in Table 6.5.13. For the 9.5% AEP the difference between independence and complete dependence is 0.21 m, for the 2% AEP it is 0.12 m and for the 1% AEP it is 0.12 m. The importance of accounting for joint probability effects therefore appears to be greater for more frequent events. If a tolerance of Z=0.1 m was specified for the design, a joint probability analysis would be required for each AEP to obtain more accurate estimates of the water level corresponding to a specified exceedance probability.

Table 6.5.13. Pre-Screening Analysis Pairs for Macksville

	Storm Tide (% AEP)			
	Lower Bound	**9.5%**	**2%**	**1%**

Flow (% AEP)	No Rain		1.45	1.52	1.55
	9.5%	2.26	2.47		
	2%	3.32		3.46	
	1%	3.68			3.80

Step 2 Dependence Parameter Selection

The critical duration of the Nambucca River catchment is between 36 and 48 hours. Given this storm burst duration and the location of the Nambucca River catchment, α =0.90, taken from Figure 6.5.13, was used to represent the dependence between extreme rainfall and storm tide.

Step 3 Flood Level Modelling

Table 6.5.14 shows flood levels at Macksville for various combinations of critical-duration rainfall and storm tides in terms of AEP.

Table 6.5.14. Flood Levels for Various Combinations of Rainfall and Tide Levels at Macksville (Pacific Highway Bridge) Nambucca River

		Storm Tide (% AEP)											
		Lower Bound	**63.1%**	**39.3%**	**18.1%**	**9.5%**	**4.9%**	**2%**	**1%**	**0.5%**	**0.2%**	**0.1%**	**0.05%**
Rainfall levels (AEPs)	**No Rain**	0.60	1.35	1.38	1.42	1.45	1.48	1.52	1.55	1.58	1.62	1.65	1.68
	63.1%	1.29	1.70	1.73	1.75	1.77	1.80	1.82	1.84	1.87	1.90	1.92	1.94
	39.3%	1.61	1.92	1.94	1.96	1.98	2.00	2.02	2.04	2.06	2.08	2.10	2.12
	18.1%	1.83	2.08	2.09	2.11	2.12	2.14	2.16	2.18	2.19	2.21	2.23	2.25
	9.5%	2.26	2.43	2.44	2.46	2.47	2.49	2.21	2.52	2.54	2.56	2.58	2.59
	4.9%	2.82	2.96	2.96	2.98	2.98	2.99	3.00	3.01	3.02	3.04	3.05	3.06
	2%	3.32	3.42	3.42	3.43	3.44	3.45	3.46	3.46	3.47	3.48	3.49	3.50
	1%	3.68	3.76	3.76	3.77	3.78	3.78	3.79	3.80	3.81	3.82	3.82	3.83
	0.5%	4.20	4.27	4.27	4.28	4.28	4.29	4.29	4.30	4.30	4.31	4.32	4.32
	0.2%	4.95	4.99	4.99	4.99	5.00	5.00	5.00	5.01	5.01	5.02	5.02	5.03
	0.1%	5.48	5.51	5.51	5.51	5.52	5.52	5.52	5.52	5.53	5.53	5.53	5.53
	0.05%	5.91	5.93	5.93	5.93	5.93	5.94	5.94	5.94	5.94	.595	5.95	5.95

Step 4 Estimate the Exceedance Probability of Flood Levels

Figure 6.5.18 shows the flood levels at the Macksville cross-section (Pacific Highway Bridge) for various AEPs. As with the pre-screening analysis, the difference between the flood levels is larger for more frequent AEPs. For rarer AEPs, the small difference between the independence and complete dependence-based estimates indicates that one flood-producing mechanism has negligible effect and that the other dominates. Based on the results in Table 6.5.14 rainfall is the dominant mechanism (there is a larger variation with changes in rainfall than with changes in tide).

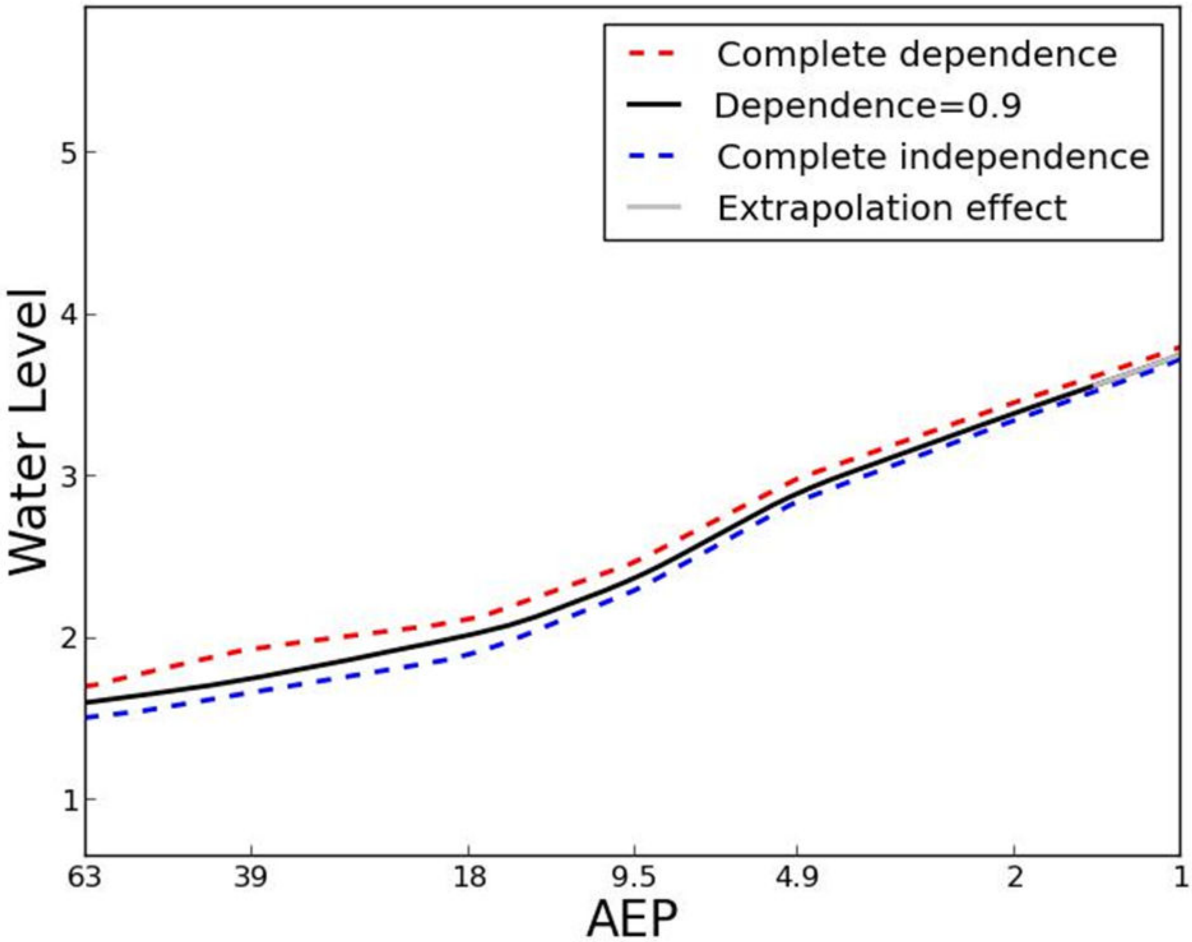


Figure 6.5.18. Water Levels at Macksville Corresponding to Cases of Complete Dependence, Complete Independence and the Best Estimate when α=0.95

Evalaution Against Observed Water Levels

Given that the location is jointly affected by storm tides and streamflow, it is preferable to compare the modelled water levels to observed levels (rather than flows). The observation gauge at Macksville has records from 1890 to 2011, giving 121 annual maximum events. Of these, 93 were censored below the 2 m threshold due to the tidally influenced nature of the location, leaving 28 uncensored gauged values. Of the 28 values, one value – the largest on record – could not be specified precisely but instead was suggested to have a range between 3.5 m-4 m ((WMAwater, 2013)).

Figure 6.5.19compares the observed water levels at Macksville (blue points) to the range of estimates from the design variable method, from complete dependence to independence, with the range depicted in the figure as grey shading. The fitted model gives reasonable agreement for less frequent events (AEP < 5%) that were the focus of hydraulic model calibration, but there is noticeable discrepancy for more frequent events (AEP > 5%). These observations lie outside the bounds produced by the dependence parameter, suggesting that variability in the dependence between extreme rainfall and storm tide is insufficient to explain this discrepancy.

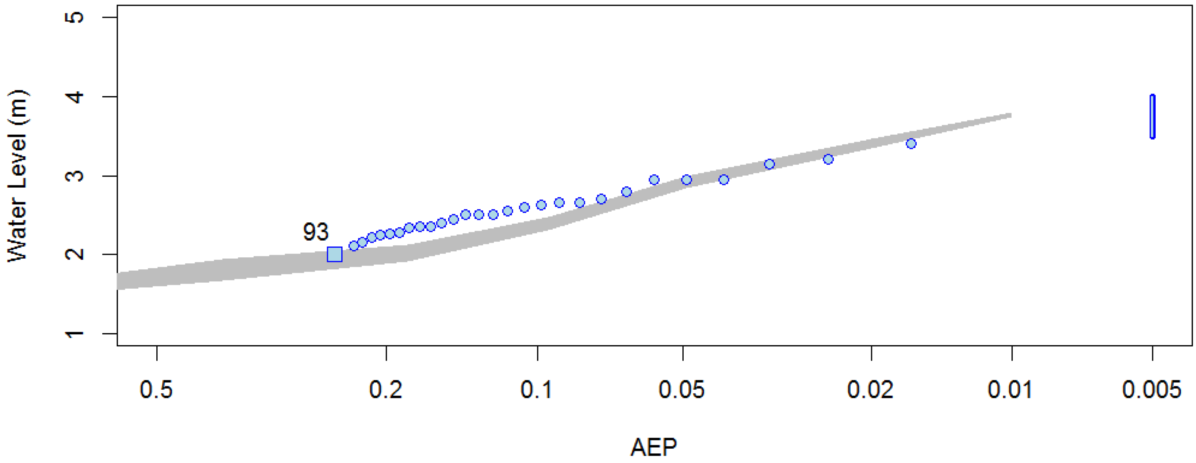


Figure 6.5.19. Comparison of Observed Water Levels at Macksville with Range of Estimates from Design Variable Method from Complete Dependence to Complete Independence

In addition to uncertainty in the representation of dependence (grey shading in Figure 6.5.19), alternative explanations for the discrepancy between simulated and observed water levels could include:

1. Parametric uncertainty in the rainfall, storm tide and observed water levels;

2. The hydraulic model and how it is represented via the hydraulic response table;

3. The assumed entrance conditions being too efficient for these more frequent events; and

4. The upper and lower boundary models (ie. the hydrological and storm tide models).

Uncertainty Assessment

When comparing models to observations, uncertainty assessment provides a useful mechanism for assessing the relative magnitude of a discrepancy. Uncertainty intervals were estimated for the frequency analysis of both the observed water levels and the rainfall/storm tide data, to enable an assessment of the magnitude of the discrepancy between observed and modelled water levels relative to their uncertainty intervals.

To estimate the 90% confidence intervals for the water levels, the procedures outlined in Book 3, Chapter 2 were used to implement a Flood Frequency Analysis. The 90% confidence limits from a fitted Generalised Extreme Value distribution are represented as grey dashed lines in Figure 6.5.20, and appear to encompass the simulated flows for most AEPs.

There is also uncertainty in the distributions used to model the design variable method, such that the rainfall AEPs and storm tide AEPs may differ from those in Table 6.5.14. To estimate the 90% confidence intervals to account for the effects of rainfall and storm tide, the censored threshold likelihood of (Zheng et al., 2015) was used. In the analysis, the joint distribution of storm tides at Stuart Island and rainfall from the Utungun gauge were extracted, jointly dependent Generalised Pareto distributions were fitted and the corresponding parameters were sampled using a Markov Chain Monte Carlo method to adjust the AEPs in Table 6.5.14. This method is beyond the scope of this chapter, but it is nonetheless useful for diagnosing the discrepancy with observations. Figure 6.5.20 shows the 90% confidence limits of the uncertainty analysis of the design variable method.

Comparing the observations to the confidence limits in Figure 6.5.20, the design variable method has considerable uncertainty in the upper tail, but less uncertainty in the lower tail. The observed water levels in the lower tail lie outside the confidence limits, which suggests that this discrepancy is not accounted for by considering parametric uncertainty. Nonetheless, the confidence intervals between the two methods overlap for the majority of AEP estimates suggesting general agreement.

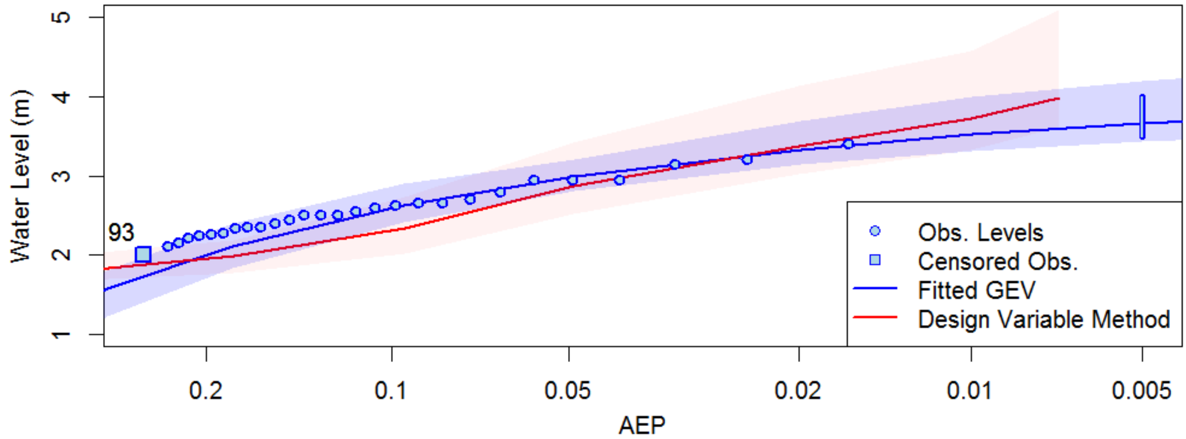


Figure 6.5.20. Comparison of Observed Water Levels to 90% Confidence Limits from Generalised Extreme Value Distribution and Design Variable Method

Hydraulic Model

The hydraulic model entails a number of assumptions that could lead to misspecification of water levels for a given set of boundary conditions. For example, typical issues such as simplified model representations and coarse grids could affect the flood level estimates. If the water levels specified for Table 6.5.14 are different, this leads to different water level contours and exceedance probabilities. For this study a two dimensional model was used with a rigorous calibration ((WMAwater, 2013)). The focus of the calibration was to less frequent events, whereas the discrepancy in Figure 6.5.19 is with respect to the more frequent events. One potential issue with respect to more frequent events is an assumption about the river entrance. The model assumed that events generated sufficient flow to blow-out the river entrance, but for more frequent events this may not hold, leading to higher observed water levels than those modelled. This issue will be further considered in the following section by adjusting AEPs corresponding to the water levels (since it is computationally expensive to rerun the hydraulic model).

A related issue may be due to the coarseness and range of the hydraulic response table (Zheng et al., 2015), but Table 6.5.14 extends to a 0.05% AEP event and has a relatively large number (twelve) increments for each dimension. Based on these considerations, the discrepancy does not seem to be due to the coarseness or extent of AEPs in Table 6.5.14.

Boundary Model

The boundary model refers to the methods by which the boundary conditions of the hydraulic model were derived and linked with exceedance probabilities (e.g. the AEPs in Table 6.5.14). For example, the probability distribution for the ocean boundary may have been specified using a surrogate location or may itself be derived from a coastal model. The probability distribution for the streamflow boundary may have assumptions in how the streamflow was derived from rainfall, for example, loss parameter values, temporal patterns, representativeness of the rainfall gauge, and the coincidence of rainfall across multiple tributaries. In short the probabilities associated with water levels in Table 6.5.14 may not be correct.

Visual inspection of Table 6.5.14 shows that the water level at Macksville is more responsive to the rainfall distribution rather than storm tide. This suggests that the design variable method at this location will be more sensitive to the assumptions made when associating the rainfall AEPs to water levels. Rather than reassess the hydrological model, a heuristic method is to manually adjust the AEPs and determine whether an improved fit to water levels is plausible. Taking this approach, the frequent rainfall AEPs in Table 6.5.14 were modified from {63.1%, 39.3%, 18.1%} to {63.1%, 50%, 39.1%} with all other AEPs remaining the same. The result of this approach is shown in Figure 6.5.21 giving the strongest indication that the discrepancy is due to the association of water levels to the frequent rainfall AEPs. As noted previously, the hydraulic model assumption that the river entrance is blown-out for frequent events is the most likely plausible explanation for this observation. However, it cannot be ruled out that the issue may instead be with the hydrological model and further inspection would be required to isolate the specific issue (beyond the scope of this chapter).

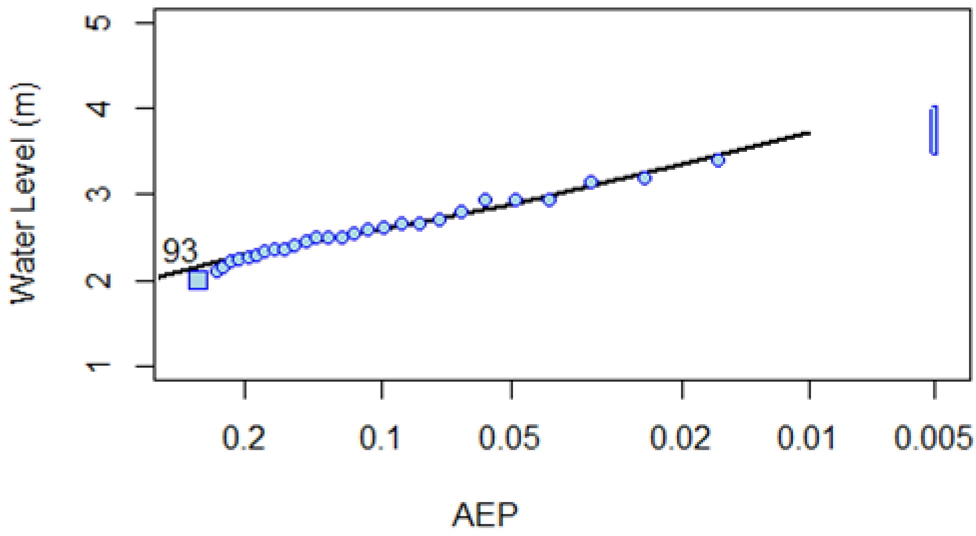


Figure 6.5.21. Comparison of Observed Water Levels at Macksville to Best Fit Estimates from Design Variable Method Assuming Correction to Frequent Rainfall AEPs

From this example, it is clear that care is required when interpreting the results from the design variable method. This example has illustrated the type of issues that should be taken into account, including the uncertainty of data sources, hydraulic model assumptions and boundary model assumptions.

Consistency of Flood Level Table

The flood level table for Macksville was constructed from output from a 2D hydraulic model. At other locations the flood level table may be only partially complete because some flow/ tide combinations do not cause the water level to exceed the base elevation of that grid cell. As an example, Table 6.5.15 presents a water level table from a different location, and has a number of NA (not available) values indicating that for these combinations the free water surface was not high enough to wet the grid cell. Provided that there are not too many NAs and they are in a consistent block, the design variable method can handle partially wetted flood level tables by ignoring the region of missing values.

Another issue is that the hydraulic model output should be 'well behaved' for all combinations of boundary conditions. This issue can be seen in Table 6.5.15 for the column of 0.05 % AEP storm tide, which shows two instances where a larger rainfall value results in a lower water level. When a rarer rainfall (or, equivalently, storm tide) event yields a lower water level this is referred to as being non-monotonic increasing. Strictly, this is not physically possible, but could be produced by a model for various reasons. One explanation is that the hydraulic model itself has spurious numerical artifacts.

Another explanation is cases where the boundary conditions have been derived inconsistently. For example, the practitioner may have switched between critical durations, used different temporal patterns or changed the way hydrographs are derived from tributary catchments. A practical workaround is to enforce monotonicity by artificially raising the water levels to be at least as high as water levels from more frequent events (in Table 6.5.15, the two events would be set to be 10.44 m). See the underlined cases of tide= 0.05% with rain = (5% or 0.05%) that are not monotonic increasing when compared to the values at lower AEPs.

Table 6.5.15. Hydraulic Response Table for cell (294, 200)

		Storm Tide (% AEP)											
		Lower Bound	**63%**	**39%**	**18%**	**10%**	**5%**	**2%**	**1%**	**0.5%**	**0.2%**	**0.1%**	**0.05%**
Rainfall levels (AEPs)	**No Rain**	NA	NA	NA	NA	5.68	7.04	7.63	8.31	8.79	9.43	9.98	10.44
	63%	NA	NA	NA	NA	5.73	7.04	7.63	8.31	8.79	9.44	9.99	10.44
	39%	NA	NA	NA	NA	5.73	7.04	7.63	8.31	8.79	9.44	9.99	10.44
	18%	NA	NA	NA	NA	5.73	7.04	7.63	8.31	8.79	9.44	9.99	10.44
	10%	NA	NA	NA	NA	5.73	7.04	7.63	8.31	8.79	9.44	9.99	10.44
	5%	NA	NA	NA	NA	5.74	7.04	7.63	8.31	8.79	9.44	9.99	10.43
	2%	NA	NA	NA	NA	5.74	7.04	7.63	8.31	8.79	9.44	9.99	10.44
	1%	NA	NA	NA	NA	5.74	7.04	7.63	8.31	8.79	9.44	9.99	10.44
	0.5%	NA	NA	NA	NA	5.75	7.05	7.63	8.31	8.79	9.44	9.99	10.44

	0.2%	NA	NA	NA	NA	5.75	7.05	7.63	8.31	8.79	9.44	9.99	10.44
	0.1%	NA	NA	NA	NA	5.75	7.05	7.64	8.31	8.79	9.44	9.99	10.44
	0.05%	NA	NA	NA	NA	5.76	7.05	7.64	8.31	8.79	9.44	9.99	10.43

5.8. Summary

Flood estimation in estuarine regions is generally more complicated than for other locations due to the range of processes and timescales that can lead to flood events. As described in Book 6, Chapter 5, Section 2, these processes can include extreme rainfall events on the upstream catchment, combined with storm surges and high astronomical tides in the lower reaches of the estuary. The strengths and limitations of three alternative methods - Flood Frequency Analysis, continuous simulation and the design variable method—were reviewed in Book 6, Chapter 5, Section 3, with the design variable method identified as the most appropriate method for general application in Australia.

A detailed overview of the theory joint probability modelling was presented in Section Book 6, Chapter 5, Section 4, and a practical approach to implementing the design variable method was provided in Book 6, Chapter 5, Section 5. The recommended approach includes a pre-screening analysis that can be used to ensure that a detailed joint probability analysis is only conducted for cases where the additional complexity is warranted. The implementation of the method has been designed to be generally applicable to a range of situations across the Australian coastline, and the method is able to accommodate changes to extreme rainfall and ocean levels as a result of anthropogenic climate change. Worked examples describing the implementation of the method were provided in Book 6, Chapter 5, Section 6 and Book 6, Chapter 5, Section 7.

The additional complexity of joint probability modelling means that the methods described here should only be implemented by users with sufficient understanding of the theoretical basis of each method. In all cases, the assumptions and limitations of each method (summarised in Book 6, Chapter 5, Section 3) should be taken into account to ensure that the selected method is appropriate for the problem. The theory, computational methods and supporting datasets required to implement joint probability approaches will continue to advance, and users should maintain familiarity with on-going developments in this field.

5.9. References

Ang, A.H.S. and Tang, W.H. (2006), Probability Concepts In Engineering Planning and Design. Emphasis on applications in Civil and Environmental Engineering, John Wiley and Sons.

Beirlant, J., Goegebeur, Y., Segers, J. and Teugels, j. (2004), Statistics of Extremes - Theory and Applications, 490 pp., John Wiley and Sons, West Sussex, England.

Coles, S.G. (2001), An Introduction to Statistical Modelling of Extreme Values, pp.208, Springer, London.

Coles, S. G. and Tawn, J.A. (1994), Statistical Methods for Multivariate Extremes: An Application to Structural Design, Journal of the Royal Society. Series C (Applied Statistics), 43(1), 1-48.

Hawkes, P.J. and Svensson, C. (2006), Joint Probability: Dependence Mapping and Best Practice. R&D Technical Report FD2308/TR1Rep., DEFRA.

Heffernan, J. and Tawn, J.A. (2004), A conditional approach for multivariate extreme values (with discussion), Journal of the Royal Society. Series B (Methodological), 66(3), 497-546.

Kotz, S. and Nadarajah, S. (2000), Extreme Value Distributions: Theory and Applications, Imperial College Press.

Svensson, C. and Jones, D.A. (2002), Dependence between extreme sea surge, river flow and precipitation in east Britain, International Journal of Climatology, 22: 1149-1168.

Svensson, C. and Jones, D.A. (2004), Dependence between sea surge, river flow and precipitation in south and west Britain, Hydrological Earth Systems Science, 8(5), 973-992.

Tawn, J. (1988), Bivariate extreme value theory: Models and Estimation, Biometrika, 75(3), 397-415.

WMAwater (2013), Hydraulic modelling report - Nambucca River and Warrell CreekRep.

Webb McKeown and Associates (1996), Warragamba Dam Auxiliary Spillway EIS flood study parts A-ERep.

Zheng, F., Westra, S. and Sisson, S.A. (2013), The dependence between extreme rainfall and storm surge in the coastal zone, Journal of Hydrology, 505: 172-187.

Zheng, F., Leonard, M. and Westra, S. (2015), Efficient joint probability analysis of flood risk, Journal of Hydroinformatics, in press (accepted 14/1/2015).

Zheng, F., Westra, S. and Leonard, M. (2014a), Australian Rainfall and Runoff Revision Project 18, Stage 3: Coincidence of fluvial flooding and coastal water levels in estuarine areasRep., pp: 145.

Zheng, F., Westra, S., Leonard, M. and Sisson, S.A. (2014b), Modelling dependence between extreme rainfall and storm surge to estimate coastal flood risk, Water Resources Research, 50(3), 2050-2071.

Chapter 6. Blockage of Hydraulic Structures

William Weeks, Ted Rigby

Chapter Status	Final
Date last updated	14/5/2019

6.1. Introduction

6.1.1. Background and Scope

The capacity of drainage systems can be severely impacted by blockage. However, there are situations where significant blockage may not impact flood behaviour to any great extent. Determination of likely blockage levels and mechanisms, when estimating design flows, is therefore an important consideration in quantifying the potential impact of blockage of a particular structure on design flood behaviour.

This chapter provides guidance on the assessment of blockage in drainage systems to assist in drainage analysis and design for both urban and rural catchments. While there are a range of locations and conditions where blockage of a drainage network may be a concern in hydraulic design, this chapter concentrates specifically on blockage of cross drainage structures, in particular culverts and small bridges.

Blockage of drainage structures is a subject where a range of advice has been provided in different guidelines. Many drainage guidelines do not mention blockage at all (Pilgrim, 1987), therefore blockage is ignored in many cases. In other situations, especially where there has been an observed blockage problem in historical flood events, blockage may be specified for extreme conditions. Other guidelines provide inconclusive advice.

In fact, the actual evidence for the impact of blockage on design flood events is very limited and the evidence for any clear quantitative design advice is lacking. This is the case internationally as well as in Australia.

This chapter is not a definitive approach, but is an attempt to provide an approach that allows a consistent analysis methodology, while not becoming too extreme in either direction since there are risks in either under- or over-estimating the influence of blockage on design flood levels. It draws heavily on the findings of an earlier report prepared by the ARR Revision Project 11 team (Weeks et al., 2009). Materials upon which this guideline has been based are referenced in the Bibliography of this chapter and in the earlier project reports and papers released on the ARR website (http://arr.ga.gov.au/).

It is expected that this chapter will be updated and revised as more information becomes available and designers gain experience in the assessment of blockage and how it affects the drainage system and calculated design flood behaviour.

6.1.2. Limitations of the Procedure

This procedure has been developed to quantify the most likely blockage level and mechanism for a small bridge or culvert when impacted by sediment or debris laden floodwater. It has not been developed for and is not appropriate when considering the impact of what are known as hyperconcentrated flows, mudflows or debris flows, on blockage of a structure. Hyperconcentrated flows are typically defined by a solids content of 20% or more by volume (or about 40% by weight) of the water column. Mud and debris flows include even higher levels of solids. At these much higher levels of suspended or fully integrated solids, blockage levels are likely to be much higher than those assessed in accordance with this chapter. Care should be taken in the review of catchment conditions where bed grades are relatively steep (say > 3%), to confirm bed and banks would remain relatively stable, such that flows would remain in the sediment or debris laden category and not become hyperconcentrated during the event under consideration.

While this procedure includes consideration of the impact of non-floating (sediment) on blockage of a structure, it is restricted to the likely impact of such material arriving at the structure during a design event. It cannot reflect the impact of any pre-existing build-up of sediment on the subsequent blockage of a structure.

6.2. Types of Structures and Drainage Systems

The types of structures or drainage elements affected by blockages can generally be grouped as follows:

Bridges and Culverts

These cross drainage structures carry roads, railways, pipelines or other infrastructure across watercourses. These structures can be affected by a number of different types of blockage mechanisms, resulting in consequences including increased flood levels, changes to stream flow patterns, changes to erosion and deposition patterns in channels, and physical damage to the structure. Blockage of these structures is the subject of this chapter.

Drainage system inlets and pipes

This includes components of urban drainage systems located within road reserves and urban overland flow paths. Frequently blockage of this type of system is generally less likely to cause the same extent of damage associated with blockage of bridges and culverts, but the consequences can still be serious from a traffic and safety perspective, and can cause serious inconvenience and nuisance. However in certain circumstances, in densely developed urban areas, pit blockage can cause significant monetary damage due to flooding of buildings upstream. While this type of blockage can be a significant nuisance, it is not covered in this chapter.

Open channels and waterways

Blockage of natural and constructed waterways can occur at any location, typically as a result of large debris snagged against bank vegetation, or debris passing slowly down the channel. The consequences of such blockage are increased flood levels, diversion of surface flows and the possible relocation of the waterway channel as a result of severe bank erosion. Blockage of these structures is not covered in this chapter.

Overland flow paths

This category covers various surface flow paths that are not normally recognised as drainage channels but do act to convey surface flows in larger events. Blockage of these flow paths can result from the deposition of sediment or the material blockage of structures built across the flow, such as property fences blocked by litter and grass clippings. Blockage of these structures is not covered in this chapter.

Weirs and dams

Debris can cause blockage within the spillways of weirs and dams, especially where there is a significant constriction to the flow area. This could increase the water level in the storage, possibly threatening the security of the structure. The sudden release of large debris rafts from dam spillways can cause significant damage to downstream road crossings. Blockage of these structures is not covered in this chapter.

6.3. Factors Influencing Blockage

6.3.1. Overview

The factors that most influence the likely blockage of a bridge or culvert structure are:

Debris Type and Dimensions

Whether floating, non-floating or urban debris present in the source area and its size;

Debris Availability

The volume of debris available in the source area;

Debris Mobility

The ease with which available debris can be moved into the stream;

Debris Transportability

The ease with which the mobilised debris is transported once it enters the stream;

Structure Interaction

The resulting interaction between the transported debris and the bridge or culvert structure; and

Random Chance

An unquantifiable but significant factor.

These various factors which impact debris movement and interaction with the structure are discussed further in the following sections.

6.3.2. Debris Type and Dimensions

6.3.2.1. Overview

All blockages that do occur arise from the arrival and build-up of debris at a structure. There are three different types of debris typically present in debris accumulated upstream of or within a blocked structure. This debris may be classified as

- Floating (e.g. trees);
- Non-floating or depositional (e.g. sediment); and
- Urban (e.g. cars and other urban debris).

Debris comprising natural materials is discussed in Book 6, Chapter 6, Section 3 and Book 6, Chapter 6, Section 3 and urban debris in Book 6, Chapter 6, Section 3. A means of determining the relevant dimensions of the debris is discussed in Book 6, Chapter 6, Section 4.

6.3.2.2. Floating Debris

Floating debris in rural or forested streams is generally vegetation of various types:

Small floating debris
: less than 150 mm long, can include small tree branches, sticks, leaves and refuse from yards such as litter and lawn clippings and all types of rural vegetation. This type of debris can also be introduced into a stream by earlier windstorms, bank erosion and land mass failures or from seasonal leaf falls. It is important to note that this material is available in both urban and rural catchments, and is usually available for transportation at any time.

Medium floating debris
: typically between 150 mm and 3 m long, mainly consists of tree branches of various sizes. This material is usually introduced into the flow path by channel erosion undermining riparian vegetation or through wind gusts during storms. It can also be present as a result of the breakdown of larger floating debris.

Large floating debris
: more than 3 m long, consists of logs or trees, typically from the same sources as for medium floating debris. Transport and storage of this material depends on discharge, channel characteristics, the size of the drift pieces relative to the channel dimensions, and the hydraulic characteristics (depth and slope) of the system. In small and intermediate size channels, this material is not easily transported and can easily become snagged mid-stream acting as a collection point for smaller material (i.e. a debris raft or log-jam). Whole trees can be retained within streams by being temporarily anchored either to the bed or banks of the stream. Large floating debris is usually transported during larger floods or prolonged periods of high river-stage where the floodplain is engaged and the ability of the debris to become snagged is reduced. This type of debris can cause significant problems for both culverts and bridge structures.

Small items of vegetation will usually pass through drainage structures during floods, while larger items may be caught in the structure. Once larger items are caught, this then allows smaller debris to collect on the structure.

6.3.2.3. Non-Floating Debris

Non-floating debris in rural or forested streams is usually sediment of all types. These can be classified as:

Fine sediments (silt and sand)
: typically consist of particles ranging from 0.004 to 2 mm. The deposition of finer clay-sized particles is normally a concern in tidal areas, with lower flood surface gradients and velocities. This type of debris is either transported along the streambed as bed load or within the water column as suspended load. Such material is normally sourced from sheet and rill erosion, landslip and landmass failures and channel erosion. Yield rates for this material can be significantly influenced by the conditions of, and changes to, a catchment due to urbanisation and/or rural land use practices.

Gravels and cobbles
: consist of rock typically ranging in size from 2 to 63 mm and 63 to 200 mm respectively. The source of this material may be from gully formation, channel erosion, landslips or land mass failure although landslips and/or land mass failures of any size will likely create hyperconcentrated or even debris flows which are not covered by this guideline.

Once mobilised, gravels and cobbles are primarily transported as bed load within high gradient streams. The deposition of cobbles can readily block the entrance of culverts or reduce the flow area under bridges.

Boulders

comprise rocks greater than 200 mm. The source of boulders is mostly from gully and channel erosion, landslips and the displacement of rocks from channel stabilisation works. Like gravel and cobbles, this material is typically transported as bed load in high gradient streams. This material can readily block the entrance to a structure and/or cause damage to the structure from the force of impact/collision.

6.3.2.4. Urban Debris

Urbanisation of catchments introduces many different man-made materials that are less common in rural or forested catchments and which can cause structure blockage. These include fence palings, building materials, mattresses, garbage bins, shopping trolleys, fridges, large industrial containers and vehicles. Garbage bins can for example be easily washed down a street and into a stream or drainage structure, a situation made worse if a large rainfall event occurs on the same day as rubbish collection within the catchment, when bins are placed in streets for collection. Urban Debris can be floating or non-floating.

6.3.3. Debris Availability

Defining the source area is an important consideration, when discussing debris availability and mobility. The source area is that area from which debris could be sourced during an event. In a small event it may be restricted to the immediate confines of the creek and its banks but in larger events will likely extend to the full extent of the floodplain and possibly the full extent of the upstream catchment area. As this procedure is used to initially establish debris potential in a 1% Annual Exceedence Probability (AEP) event , the relevant source area will typically be limited to the 1% AEP flood extents. Steep sided tributaries and larger rills may however extend the source area beyond the limits of the 1% AEP flood.

The following factors affect the availability of debris material within a source area:

Potential for soil erosion

Soil erosion exposes soil and rock particles, thus increasing their availability. The potential for soil erosion is dependent on a number of factors including soil erodibility, rainfall erosivity, surface slope length and gradient, vegetation cover and changes in catchment hydrology, this latter factor being often closely linked to the effects of urbanisation.

Local geology

The geology of the debris source area, particularly the exposed geology of the watercourse, influences the availability of materials such as clay, silt, sand, gravel, rocks and boulders.

Source area

Increasing the area supplying debris typically increases the quantity of available blockage material. It is noted however, that once blockage occurs at a given structure, the debris source area for the next downstream structure may be much less than that of the upstream structures source area.

Amount and type of vegetative cover

Cover can vary from grasses and shrubs to thick forests and plantations as well as a variety of crops and agricultural uses. Increasing the cover density in the source area will typically increase the availability of debris. Some types of cover are also more prone to produce debris than others (eg Cora trees). The type of cover in the source area can also impact availability

Land clearing

This is associated with both rural and urban land use practices. Deforestation and urbanisation can alter the long-term flow regime of streams and may lead to gully erosion and channel expansion.

Preceding wind and rainfall

The occurrence of frequent flood events typically reduces the availability of debris in the source area, however, the occurrence of frequent windstorms will typically increase the quantity of debris available in the source area.

Urbanisation

Such areas make available a wide range of debris typically influenced by the extent of flood inundation and proximity of such debris to the stream. In most circumstances this a manageable factor linked to town planning and drainage design.

6.3.4. Debris Mobility

The following factors affect the mobilisation of debris material within the catchment:

Rainfall erosivity

Different regions experience a range of frequencies of rainfall intensity, and in general, those areas that experience more intense rainfall have a greater potential to mobilise debris than areas of lower rainfall intensity.

Soil erodibility

This can vary from weathered rocks to cohesive clays, all soils have different abilities to become eroded, entrained and available for mobilisation.

Slope

For sediment and boulder movement, there is a relationship between the mobilisation of such debris and the slope of the catchment, with respect to overbank areas where debris may be sourced and the stream channel which conveys the debris.

Storm duration

The mobilisation of materials generally increases with increasing storm duration.

Vegetation cover

Sparse vegetation cover can increase sediment mobility.

6.3.5. Debris Transportability

Once debris has been mobilised, it then needs to be transported down the stream if it is to present a hazard to downstream structures. Stream power, velocity, depth, presence of snags and bends and the overall dimensions of the water course play a large part in determining whether the mobilised debris lodges where it first enters the stream or is transported downstream to a receiving structure. There is a reasonably strong correlation between the waterway width and the maximum size of floating debris that a stream can transport. The event magnitude is also a major factor in controlling the quantity of debris transported. Rarer events produce deeper and faster flowing floodwaters which are able to transport large quantities and larger sizes of debris, smaller events may not be able to transport larger bridging material at all.

6.3.6. Structure Interaction

The likelihood of blockage at a particular structure depends on whether or not debris is able to bridge across the structure's inlet or become trapped within the structure. As bridging occurs, the clear expanse of each opening reduces, thus increasing the likelihood of further bridging and further blockage by smaller or similar material. Smaller blockage matter is unlikely to cause full blockage of a structure without the presence of suitable larger bridging matter, the material that initially bridges across the opening or inlet of a structure. Bridging matter can be as small as leaves caught on a kerb inlet grate, or as large as logs, cars and shipping containers caught at a culvert inlet or on bridge piers.

Exposed services attached to the face of culverts or bridges or obstructing the culvert waterway opening can significantly increase the risk of blockage. Similarly, some through-culvert features introduced to improve fish passage can also collect and hold debris increasing the risk of internal blockage problems. Many other factors such as skew alignments, opening aspect ratios, opening height to overtopping height ratios, culvert hoods, sloping inlet walls and the smoothness of transitions can also modify the likely interaction between the arriving debris and the bridge or culvert structure.

In urban drainage systems, any individual culvert in the system is not an individual structure, it is part of a system, generally with culverts and other structures in a series down the water course. As a consequence, upstream culverts are likely to collect a portion of the transported debris in the stream, reducing the quantity of debris that would otherwise reach the downstream culverts so the risk of blockage in these downstream structures is reduced.

Consideration of multiple structures is discussed further in Book 6, Chapter 6, Section 4.

6.3.7. Random Chance

While an unquantifiable factor, random chance plays a significant role in the blockage of structures. Antecedent conditions can in particular substantially alter the likely level of blockage at a structure. Recent floods can for example reduce the availability of debris but increase the transportability of debris of a particular size by cleaning out the waterway. Even the alignment of a limb approaching a structure can substantially alter its likelihood of being caught on the inlet and triggering a more substantial blockage. Blockage of a structure in any event of a particular magnitude will therefore vary in response to these random changes in behaviour, creating a distribution of blockage levels associated with such an event. This chapter attempts to quantify the average or most likely blockage level associated with a design event of a particular magnitude, as this presents an probability neutral approach to simulation of the resulting flood surface.

6.4. Assessment of Design Blockage Levels

6.4.1. Overview

Blockage of cross drainage structures such as culverts and bridges could have an impact on the capacity of these structures and also on flood levels. Hydraulic analysis of these structures should include some consideration of these impacts. This section describes a procedure for the inclusion of the impacts of blockage in analysis.

The design blockage is the blockage condition that is most likely to occur during a given design storm and needs to be an “average” of all potential blockage conditions to ensure that the calculated design flood levels reflect the defined probability. For example, an assumption of a higher than average level of blockage would lead to the calculated design flood level upstream of the structure being higher than would be appropriate for the defined probability. Downstream flood levels would be lower because of the additional flood storage created upstream of the structure. On the other hand, an assumed lower than average level of blockage would result in lower flood levels upstream and higher flood levels downstream. This is a similar concept to that of probability neutrality used in various aspects of design flood event analysis. It is also noted that actual blockage levels vary greatly from event to event with a potential spread from “all clear” to “fully blocked” even in floods of comparable magnitude. Antecedent catchment conditions and random chance are major factors in determining blockage levels in an actual event. The selected design blockage must aim for probability neutrality (the concept of ensuring that the AEP of the design flood discharge is the same as the AEP of the design rainfall input) so design floods are appropriate for the particular circumstances. As with other similar aspects of design flood estimation, such as losses, each individual historical flood may have quite different amounts of blockage compared to the design event.

Flood mapping is an exercise in probabilities that involves the estimation of ‘average’ catchment conditions for various storm and flood frequencies to ensure that the rainfall of the defined probability produces a flood event of the same probability. In such work, design blockage conditions must be considered when predicting flood levels of a given frequency. In situations where the consequences of flooding (including the impact of blockage) are high, planning rules typically require design for a lower probability (rarer) event. An increase in the design event probability is typically adopted for planning purposes, when the consequences of flooding are low.

This chapter is based on a design event type analysis, where a flood of a defined flood probability is required. For Monte Carlo analysis of flood risk, a probability distribution of blockage is required, as an input. Considering the uncertainty in the assessment of blockage, analysis of probability distributions is even more difficult. This topic is discussed more detail in Book 6, Chapter 6, Section 5. The procedure presented in this chapter is based on a qualitative assessment of debris likely to reach a structure, and the likely interaction between that debris and the structure regarding its potential for blockage. It is based on the various papers prepared by Barthelmess, Rigby, Silveri and others.

The procedure initially involves a series of decisions leading to estimation of the likely magnitude of debris reaching a structure in a 1% AEP event and the most likely blockage level that would develop at the structure under consideration. Subsequent adjustments are then made to reflect the most likely design blockage levels in lesser or greater AEP events and to establish the associated most likely blockage mechanism. This procedure provides an probability neutral approach to the assessment of an appropriate level of blockage for the simulation of design flood behaviour , but may not reflect specific conditions in an equivalent historical event. Such is the random nature of the many variables controlling blockage behaviour.

6.4.2. Appropriate Investigation

It is important to recognise the impact that different levels of investigation can have on the confidence associated with any blockage estimate. Estimates based on aerial imagery alone cannot for example provide the level of confidence that would be obtained from a field visit to the site, specifically aimed at assessing the various factors influencing blockage levels at the site or likely blockage mechanisms.

Where the structure/site under consideration is located in a particularly flood sensitive area and blockage of the structure could significantly impact flood behaviour in that area, then a high level of investigation is warranted. This should include a field inspection of the upstream catchment/source area to confirm the types of debris likely to reach the site, their availability, mobility and transportability together with the average size of the largest 10% of each debris type likely to reach the site. Any structures upstream of the target structure/site should be inspected and consideration given to their ability to trap debris reaching the target structure/site. Any photographs/records of past blockage material and extents should be used to validate the choice of L_{10} and debris type. Although seldom available, any photos/records of the blockage mechanism (Location – Type – Timing) that have been observed in past events will help to validate the chosen blockage mechanism to be used in the hydraulic model. However it must be stressed that it is the most likely (probability neutral) blockage mechanism that is required, not the worst case scenario. Flood mapping, aerial photography, annual rainfall and rainfall IFD data, rainfall and soil erosivity maps, topographic maps, vegetation and soil maps should be consulted when available to further consolidate conclusions as to the types of debris likely to reach the site and the quantum of such debris.

Conversely, when the structure under consideration is in an area where changes in flood behaviour would have no significant consequences on safety, property damage or amenity, then an extensive investigation to support the blockage assessment process, as outlined above, may not be warranted. This decision should be documented.

The final decision as to what is an appropriate level of investigation must ultimately be the responsibility of the person making the assessment. It will vary greatly between sites and will to some extent be constrained by what information is available. Whatever the approach adopted, it is important that the level of investigation undertaken should be relevant to the importance of the assessment of blockage at the site and is documented, so that others relying on the assessment can be aware of the confidence limits attaching to that particular assessment.

6.4.3. History of Blockage

The history of blockage in the drainage system is an important input to any risk based approach to blockage, and should always be explored in so far as available data permits. While the procedure outlined in this chapter provides a generic assessment of likely design blockage levels and mechanisms, local observations and history can be important in ensuring that this procedure results in reasonable answers. All available history should be sought from relevant local stakeholders, including residents, in assessing the reasonableness of blockage levels and mechanisms produced by this chapter.

In particular, if there has been no long term history of blockage at a particular structure and similar drainage structures in the catchment have not demonstrated blockage problems, blockage may not need to be considered, or a nominal allowance only may be appropriate in design.

6.4.4. Assessment Procedure

6.4.4.1. Debris Types and Dimensions

In using this procedure it is necessary to first assess the type of debris likely to arrive at the structure under consideration and the likely dimensions of that debris. Where more than one type of debris is present in quantity in the source area, the procedure will need to be repeated for each debris type to establish the debris type with the most impact on the performance of the blocked structure.

The types of debris available in their respective source areas will normally be readily apparent during a field visit or from aerial photographs, but relevant dimensions may be more difficult to assess.

The ratio of the opening width of the structure (e.g. diameter or width of the culvert or bridge pier spacing) to the average length of the longest 10% of the debris that could arrive at the site (termed here as L_{10}) is a well correlated guide to the likelihood that this material could bridge the openings of the structure and cause blockage. This L_{10} value is defined as the average length of the longest 10% of the debris reaching the site and should preferably be estimated from sampling of typical debris loads. However, if such data is not available, it should be determined from an inspection of debris on the floor of the source area, with due allowance for snagging and reduction in size during transportation to the structure.

For debris of any particular type and size to reach the structure, the debris must:

- be available in the source area;
- be able to be mobilised into the stream and not snagged by bank vegetation as it enters the stream; and
- be delivered into a stream able to transport the debris from the source area down to the structure, without floating debris being snagged by bank vegetation or stream bends or constrictions, or without non-floating debris being deposited prior to reaching the structure as the stream grade and velocities reduce. For smaller more turbulent streams (less than say 6 m bank to bank) the width between banks of the stream through the source area will normally limit the size/length of larger floating debris to less than the stream width. The bed grade immediately upstream of the structure will normally limit the size of the larger non-floating debris reaching the structure to that capable of being moved by the flow.

Any loose material and pockets of debris lying within or in close proximity to the channel are likely to be representative of the debris that could cause downstream blockage. A detailed inspection of the waterway upstream of the target structure, particularly after a flood, will assist with assessing the above factors and deriving a realistic value for L_{10}.

In an urban area the variety of available debris can be considerable with an equal variability in L_{10}. In the absence of a record of past debris accumulated at the structure, an L_{10} of at least 1.5 m should be considered as many urban debris sources produce material of at least this length such as palings, stored timber, sulo bins and shopping trolleys.

6.4.4.2. Debris Availability

The availability of a particular type of debris (floating, non-floating or urban) in a source area limits the level of that particular debris that can be ultimately mobilised and transported to a structure. As noted in Book 6, Chapter 6, Section 4, there may be significant quantities of more than one type of debris present in the source area, requiring more than one type of debris to be assessed. The characteristics of high, medium and low availability are hard to quantify, so there is some judgment required in their evaluation. Table 6.6.1 describes typical source area characteristics and a corresponding ranking for the likely availability of a particular type of debris in that source area. It should be noted that the characteristics included are not exhaustive or presented in any particular order. Some will only be applicable in respect to certain debris types. They are provided to provoke thought about the factors that could be relevant to the level of availability. As this procedure is based on a 1% AEP flood (with later adjustment for other AEPs) the effective source area is that associated with a 1% AEP event.

Table 6.6.1. Debris Availability - in Source Area of a Particular Type/Size of Debris

Classification	Typical Source Area Characteristics (1% AEP Event)
High	• Natural forested areas with thick vegetation and extensive canopy cover, difficult to walk through with considerable fallen limbs, leaves and high levels of floor litter. • Streams with boulder/cobble beds and steep bed slopes and steep banks showing signs of substantial past bed/bank movements. • Arid areas, where loose vegetation and exposed loose soils occur and vegetation is sparse. • Urban areas that are not well maintained and/or where old paling fences, sheds, cars and/or stored loose material etc., are present on the floodplain close to the water course.
Medium	• State forest areas with clear understory, grazing land with stands of trees. • Source areas generally falling between the High and Low categories.
Low	• Well maintained rural lands and paddocks with minimal outbuildings or stored materials in the source area. • Streams with moderate to flat slopes and stable bed and banks. • Arid areas where vegetation is deep rooted and soils are resistant to scour. • Urban areas that are well maintained with limited debris present in the source area.

6.4.4.3. Debris Mobility

The ability for debris to become mobilised from the source area into a stream has an effect on the amount of debris that can then be ultimately transported to a structure. Table 6.6.2 describes typical source area characteristics and a corresponding rank for the likely mobility of debris from the sorce area into receiving streams.

Table 6.6.2. Debris Mobility - Ability of a Particular Type/Size of Debris to be Moved into Streams

Classification	Typical Source Area Characteristics (1% AEP Event)
High	• Steep source areas with fast response times and high annual rainfall and/or storm intensities and/or source areas subject to high rainfall intensities with sparse vegetation cover. • Receiving streams that frequently overtop their banks. • Main debris source areas close to streams.
Medium	• Source areas generally falling between the High and Low mobility categories.
Low	• Low rainfall intensities and large, flat source areas. • Receiving streams infrequently overtops their banks.

Classification	Typical Source Area Characteristics (1% AEP Event)
	• Main debris source areas well away from streams.

6.4.4.4. Debris Transportability

The ability for debris to be transported by a stream down to a structure has an effect on the amount of debris arriving at the structure. Table 6.6.3 describes typical stream characteristics and a corresponding rank for the likely transportability of debris.

Table 6.6.3. Debris Transportability - Ability of a Stream to Transport Debris Down to the Structure[a]

Transportability	Typical Transporting Stream Characteristics (1% AEP Event)
High	• Steep bed slopes (> 3%) and/or high stream velocity (V > 2.5 m/s) • Deep stream relative to vertical debris dimension ($D > 0.5L_{10}$) • Wide stream relative to horizontal debris dimension.($W > L_{10}$) • Stream relatively straight and free of major constrictions or snag points. • High temporal variability in maximum stream flows.
Medium	• Stream generally falling between High and Low categories.
Low	• Flat bed slopes (< 1%) and/or low stream velocity (V < 1m/s). • Shallow depth relative to vertical debris dimension ($D < 0.5L_{10}$). • Narrow stream relative to horizontal debris dimension ($W < L_{10}$). • Stream meanders with frequent constrictions/snag points. • Low temporal variability in maximum stream flows.

[a]Where V = velocity, D is depth, W is width and L_{10} is average length of the longest 10% of the debris that could arrive at the site

6.4.4.5. Debris Potential

Where reliable long term data is available on the quantity and type of debris typically present at a structure, this should be used to directly quantify the debris potential at the structure. Where such data is not available, the potential quantity of debris reaching a structure at a site from a contributing source area in a 1% AEP event can be estimated from Table 6.6.4. If there is a significant quantity of more than one type of debris in the source area that could induce blockage, this will require more than one type of debris to be assessed.

Table 6.6.4. 1% AEP Debris Potential

Classification	Combinations of the Above (any order)
High	HHH or HHM
Medium	MMM or HML or HMM or HLL
Low	LLL or MML or MLL

6.4.4.6. Adjustment for Annual Exceedence Probability

Observation of debris conveyed in streams strongly suggests a correlation between an event's magnitude and debris potential at a site. This is accommodated in Table 6.6.5 as follows.

Table 6.6.5. AEP Adjusted Debris Potential

Event AEP	(1% AEP) Debris Potential at Structure		
	High	Medium	Low
AEP > 5%	Medium	Low	Low
AEP 5% - AEP 0.5%	High	Medium	Low
AEP < 0.5%	High	High	Medium

6.4.4.7. Design Blockage Level

Inlet Blockage (Floating or Non-Floating)

In conjunction with the quantity of debris likely to arrive at the site, Table 6.6.6 provides an estimate of the 'most likely' inlet blockage level should a blockage form from floating or non-floating debris bridging the inlet.

Table 6.6.6. Most Likely Inlet Blockage Levels - $B_{DES}\%$

Control Dimension Inlet Clear Width (W) (m)	AEP Adjusted Debris Potential At Structure		
	High	Medium	Low
$W < L_{10}$	100%	50%	25%
$L_{10} \leq W \leq 3{*}L_{10}$	20%	10%	0%
$W > 3{*}L_{10}$	10%	0%	0%

Barrel Blockage (Non Floating)

An alternative blockage mechanism is however possible for non-floating material (typically sediment) when this material progressively arrives and is deposited at the inlet and in the barrel or waterway of the structure. This typically leads to a bottom up blockage of both the barrel and inlet to the structure. Blockage in this form can arise because velocities through the structure fall below the level required to maintain the material in motion or, in extreme cases, because the depth of sediment in the bed load is sufficient to overwhelm the inlet, leading to sediment with little water completely blocking the inlet and filling a substantial proportion of the barrel of the structure.

Table 6.6.7 classifies the likelihood of deposition in the barrel or waterway based on sediment size and velocity through the structure. Using this likelihood of deposition Table 6.6.8 then combines the likelihood of deposition with the debris potential to provide a most likely depositional barrel or waterway blockage level for the structure.

Table 6.6.7. Likelihood of Sediment Being Deposited in Barrel/Waterway (HML)

Peak Velocity Through Structure (m/s)	Mean Sediment Size Present Clay/Silt 0.001 to 0.04 mm	Sand 0.04 to 2 mm	Gravel 2 to 63 mm	Cobbles 63 to 200 mm	Boulders >200 mm
>= 3	L	L	L	L	M
1.0 to < 3.0	L	L	L	M	M
0.5 to < 1.0	L	L	L	M	H
0.1 to < 0.5	L	L	M	H	H
< 0.1	L	M	H	H	H

Based on Hjulstrom's diagram as modified by Sundborg (Sundborg, 1956)

Table 6.6.8. Most Likely Depositional Blockage Levels – B_{DES}%

Likelihood that Deposition will Occur (Table 6.6.7)	AEP Adjusted Non Floating Debris Potential (Sediment) at Structure High	Medium	Low
High	100%	60%	25%
Medium	60%	40%	15%
Low	25%	15%	0%

It is noted that Table 6.6.8 (blockage caused by non-floating debris) is to be read in conjunction with Table 6.6.6 (blockage caused by floating debris) and the blockage mechanism creating the worst impact on flood behaviour should be used in design.

While the above tables provide a means of estimating a realistic value for the magnitude of a likely (probability neutral) blockage, they do not address the other characteristics required to properly describe the blockage mechanism (viz the blockage type, location and timing) and its impact on the hydraulics of flow through the structure. These issues are discussed further in Book 6, Chapter 6, Section 4.

6.4.4.8. Minimum Opening Height Considerations

Consideration of likely inlet blockage levels as presented in Table 6.6.6 assumes that the greatest dimension (length) of debris relative to the structures opening width is the dominant factor influencing inlet blockages. All debris however has three dimensions and a lesser dimension, such as the debris height, could also trigger vertical bridging across the opening height if the structure's opening height was substantially less than the structures opening width. In the absence of detailed data on likely debris geometry, it is recommended that structures be designed with a clear opening height of at least one third their width to reflect the assumptions inherent in this procedure. In an existing structure where the opening height is less than one third of the opening width, it is recommended that analysis be based on the likely vertical dimension of the debris and the vertical opening height of the structure in lieu of the likely debris length and horizontal opening width. Unless data is available to support the choice of L_{10} (vertically), it should be taken as not less than one half of the assessed debris L_{10} (length).

6.4.4.9. Blockage of Multi Cell/Span Structures

Limited observation of blockages at multi cell culverts or multi span bridges suggests that all cells/spans often do not block to the same extent. The main factors influencing this variability appear to be the main stream approach alignment and location relative to the multiple culverts or spans and the relative width of flow carrying debris to the total opening width. These two factors are somewhat related as they both influence the uniformity of presentation of debris, carried by the flow, to the individual cells or spans.

Where the main stream width is considerably less than the total structure width, it is likely that more debris will be delivered to and accumulate at or in the cells/spans falling within the main stream width, than at the cells/spans located on the adjacent floodplains. This may not be the case when the mainstream flow is only a small proportion of the total flow reaching the structure. In such cases the presentation of debris to the multiple cells/spans may become more uniform resulting in more consistent levels of blockage.

As an initial guide it is suggested that, where the width of that part of the approach flow that is capable of transporting the debris under consideration, is comparable with or greater than the total width of the structure, then the assessed B_{DES} be applied uniformly to all cells/ spans.

Where the width of that part of the approach flow that is capable of transporting the debris under consideration is significantly less than the total width of the structure, then the culverts/spans within the effective transport width be assessed as blocked to B_{DES} and those outside of that zone be reduced to half B_{DES}. Measurements of observed distributions are however essentially non-existent at this time. More information, to permit refinement of guidelines for blockage of multiple spans/cells, is needed.

6.4.4.10. Assessment of Multiple Structures

It is fundamental to the consideration of the interaction between multiple culverts that any individual culvert/bridge could be ‘all clear’ or ‘guideline blocked’ in a design event.

The question then arises as to what are the ‘likely’ probability neutral combinations of blockage that could occur across a catchment. Clearly an ‘all clear’ (B_{DES}=0) global solution is possible in any event and even probable in lesser events. In these lower probability events the single site B_{DES} is probably also low so the change in catchment floods behaviour between different mixes of sites with B_{DES}>0 and B_{DES}=0 may not be great. In larger events however substantial differences in flood behaviour can be created from different mixes of ‘all clear’ and B_{DES} structures across the catchment. Simple math shows that n independent sites with two choices for blockage presents 2n combinations. A catchment with 6 interacting culverts therefore could involve 64 possible blockage scenarios. In analysing these combinations it is therefore critical both with respect to probability neutrality and computation time that only likely combinations are considered. Seldom will all structures be responding in a truly independent manner. There is unfortunately no pre-prepared solution for this problem – all catchments will be different. While not a truly probability neutral approach, modelling all structures 'all clear' and 'guideline blocked' ensures individual structure impacts are properly simulated in the envelope solution together with the 'all clear' impacts. If these scenarios are then augmented with 'likely' mixtures of clear and 'guideline blocked' structures, the resulting flood surface envelope should reasonably represent the likely envelope flood surface levels that could be reached at any site in the catchment. It should be noted however that in any single historic event of a given AEP, the recorded flood surface will likely only reach the envelope levels at some locations (due to the variability in actual historic blockages).

As previously noted, where there are multiple structures on a contiguous water course, the debris availability will normally reduce downstream since debris will be captured by the upstream structures. Therefore for downstream structures, the debris availability, as defined in Table 6.6.1 will normally be reduced.

6.4.4.11. Risk Based Assessment of Blockages

In general, the consequences of a flood event of given probability will be used to establish risk in an area or at a site and this level of risk will in turn be used to establish the appropriate event AEP to be used as the planning event for that particular area or site. What this approach does not reflect is the relative uncertainty in all of the various parameters influencing design flood estimation. With even the most careful approach to the selection of parameters like design rainfall intensity, rainfall temporal patterns, stream roughness or most likely blockage levels, there is a significant likelihood that error in the assessment of these parameters may in turn lead to errors in the predicted design flood behaviour.

In an event based approach to modelling it is therefore prudent to undertake various sensitivity runs to quantify how reasonable variation in the chosen parameters could affect the model’s results. Where such an analysis generates significant changes in the flood surface, it indicates that the parameter creating that change needs very careful review to confirm that the value selected was as appropriate as available data permits. A sensitivity analysis of alternate reasonable blockage levels and mechanisms is therefore strongly recommended for design or analysis involving blockages. It is recommended that the sensitivity to such a variation in design blockage levels be incorporated into analysis by considering both an 'all clear' and blocked at twice the calculated 'guideline blocked' level (max 100%) scenarios, to identify sites where flood behaviour upstream or downstream of the structure is particularly sensitive to the adopted design blockage level. Where such a site is identified, all inputs into the assessment process should be carefully reviewed to confirm the adopted design blockage level before proceeding with design or analysis based on that level.

As blockage of a structure with significant upstream available flood storage can lead to a reduction in flood flow and levels downstream of the structure, effectively protecting downstream properties, it is important to review the all clear analysis to see if the all clear scenario results in significantly increased flows downstream of the structure. If this is found to be the case then the all clear and 'guideline blocked' results should be enveloped for design flood estimation purposes.

In reviewing risk, inclusion of blockage in a Monte Carlo analysis is a valuable means of quantifying the impact of blockage on uncertainly in the flood assessment process. A distribution of blockage values is however needed for Monte Carlo analysis. Considering the uncertainty inherent in the factors influencing blockage levels and the lack of data in respect to the variation of blockage levels over time, it is however difficult to determine a suitable distribution. What little research has been done on this distribution suggests that the probability distribution is likely to be dual peaked with the ‘all clear’ and ‘most likely’ values ranking higher than adjacent values. Much more data is however needed before these characteristics can be confirmed.

6.4.5. All Clear

This is the condition where there is no allowance for blockage, and the hydraulic analysis assumes that the structure flows freely.

This condition should be considered as referenced above as an important sensitivity case, since the 'all clear' condition will reduce the upstream flood level and may increase flood levels downstream depending on the storage and flood immunity of the structure being considered.

Secondly, and perhaps more importantly, as referenced in Book 6, Chapter 6, Section 4, blockage may not need to be considered at all or may need consideration as a nominal allowance, if there is no history of blockage at this site or at similar neighbouring sites, especially if there is low risk of damage or disruption caused when blockage is neglected.

6.4.6. Implementation

A form has been prepared to assist in implementing this procedure and is available on the ARR website[1].

6.5. Hydraulic Analysis of Blocked Structures

6.5.1. Overview

Where blockage has historically been included in analysis or design, it has often been applied as a reduction factor to the 'all clear' flow through the structure. This is a simple and rapid means of making some allowance for blockage and in the absence of information on likely blockage mechanisms and extents can provide an answer commensurate with the associated uncertainty in such an approach.

This chapter enhances our understanding of likely design blockage mechanisms at a structure by quantifying likely blockage levels at a structure based on assessable catchment and structure parameters and understanding the blockage mechanism that will likely develop at the structure. Given this information, a more deliberate approach to hydraulic analysis of design blockages is now available, although most current hydraulic modelling software currently lacks the functionality to simulate the blockage mechanisms described in this chapter. It is hoped that this functionality will however be made available in the more capable software packages, in use in Australia, in the not too distant future.

6.5.2. Blockage Types

As previously noted, a blockage mechanism can be described by its type, its location and its timing and extents. With respect to type, there are three types of blockages that could occur:

A top down blockage

occurs, when a floating debris raft builds up at the entrance to a structure, obstructing the inlet. This is a very dynamic type of blockage with the raft volume and elevation varying over time. These changes occur in response to both the flow rate and the difference between debris being added and lost from the raft as the blockage develops. On the flood recession this material may settle to fully block the inlet even though the inlet may have been only partly blocked by the raft at the flow peak. While rarely available, the temporal history of such a blockage, in an historic event, can be an important factor in realistically reproducing the actual flood behaviour at the blocked structure. While top down blockages are common in heavily vegetated areas, realistic simulation of this form of blockage is very complex.

[1]BLOCKAGE ASSESSMENT FORM via http://arr.ga.gov.au/downloads-and-software/revision-project-reports

A bottom up blockage

occurs, when non-floating material is deposited at the inlet and/or in the barrel or waterway of the structure. This also is a dynamic type of blockage with sediment being both added and removed from the blockage as time passes. Because of the dynamic nature of this process, the debris apparent at the conclusion of the event may have little relationship to the debris level at any point in time during the event. As with the top down blockage, the temporal history of blockage in an historic event can be important in realistically reproducing actual flood behaviour during the event. Bottom up blockages are relatively common in steep lightly vegetated catchments with unstable stream banks or easily eroded stream beds. As the geometry of a bottom up blockage does not directly vary with flood stage (as in a top down blockage), hydraulic analysis of a bottom up blockage is more straightforward.

A porous plug blockage

typically occurs when larger vegetative debris (often rapidly) bridges across the inlet of the structure covering the entire inlet but with sufficient porosity to allow some flow through the plug. It typically arises from a rapid bank or slope collapse, releasing a substantial pulse of vegetation and sediment into the stream. Unlike a top down or bottom up blockage, the porosity of this plug will likely only diminish as the event continues, with ever finer material being trapped on the bridging material that triggered the initial blockage. As blockage geometry does not vary with flood stage (as in a top down blockage), hydraulic analysis of a porous plug blockage is also more straightforward.

6.5.3. Blockage Mechanisms

While the number of possible blockage mechanisms is considerable, there appears to be a strong correlation between the dominant debris type arriving at a structure and the blockage mechanism it triggers. This correlation forms the basis of Table 6.6.9 where the blockages ‘most likely’ location, timing and extents are described. It should be noted that this table is based heavily on limited observations and should be updated as further data becomes available.

Progressive floating raft inlet blockages are assumed in this chapter to significantly impact flow through the structure only after the flow peaks (being mostly clear at higher flows as the raft lifts clear of the inlet and possibly overtops the structure. Pulse like blockages of floating material at an inlet mostly arise from vegetation injected into the stream from collapsing banks, as floodwater rise or from litter swept off the floodplain as streams overtop their banks. Neither of the above blockage types is likely to create a significant barrel/waterway or outlet blockage although non-floating debris, if present in any quantity can build up under the raft at the inlet and in the barrel, particularly as the flood recedes. It should be noted that factoring of 'all clear' flow will not necessarily provide a good estimate of the impact of either of these mechanism as both are inlet control mechanisms and the 'all clear' structure could be operating under strong outlet control.

Non-floating material reaching a culvert or bridge will mostly build up progressively but can occur as a pulse of debris in streams with unstable banks. Typically, non-floating material (sediment) will build up throughout the structure (inlet, barrel and outlet) as increasing flows mobilise ever increasing amounts of bed and bank material. Material will be continuously lost from the accumulated debris mass, but the rate of supply is likely to exceed the rate at which material passes on downstream, at least while flows are increasing and new material is being mobilised.

These observations and assumptions on the likely type, location and timing of a blockage are summarised in Table 6.6.9 In this table, the following designations are used to describe the timing of key trigger points in the blockage process.

$T_{TOTB/SA}$
Is the time when flow that first overtops the stream's banks in the source area reaches the structure.

$T_{OT/F\ \&\ OT/L}$
Are the times when flow first and last overtops the structure.

T_P
Is the time at which the upstream water level peaks at the structure.

$T_{OBV/FL}$
Is the time on the falling limb when the upstream water level drops back to the obvert level of the structure.

Table 6.6.9. Likely Blockage Timing and Extents

Dominant source material	Delivery and Type	Likely Blockage Locations and Timings			
		Inlet	Barrel	Outlet[a]	Handrails[b]
FLOATING	Progressive Top Down	0 @ T_P to B_{DES} @ $T_{OBV/FL}$	Unlikely	Unlikely	B_{DES} @ $T_{OT/F}$ to B_{DES} @ & $T_{OT/L}$
	Pulse Porous Plug	B_{DES} @ $T_{OTB/SA}$	N.A	N.A	B_{DES} @ $T_{OT/F}$ to B_{DES} @ & $T_{OT/L}$
NON FLOATING	Progressive Bottom Up	0 @ $T_{OTB/SA}$ to B_{DES} at T_P	$T_{OTB/SA}$ to B_{DES} at T_P	$T_{OTB/SA}$ to B_{DES} at T_P	Unlikely
	Pulse[c] Porous Plug	Unlikely[d]	N.A	N.A	Unlikely

[a]Unlikely - but could become likely if inlet is open and outlet grated.

[b]B_{DES} is for the handrail geometry and will normally be much higher than for the culvert/bridge waterway as L_{10} is likely to be much greater than the horizontal opening width/spacing of the balusters. In modelling B_{DES} can be assumed at t=0 as the model will not apply handrail blockages until flow reaches the level of the handrails.

[c]Pulse blockage is more likely in systems subject to irregular flooding and/or streams with unstable banks.

[d]Unlikely – but could become likely if upstream bed/banks unstable and/or prone to scour.

As previously noted in Book 6, Chapter 6, Section 5, modelling the hydraulics of a progressively accumulating floating raft is quite complex as the blockage is not fixed in regard to its own geometry or in relation to the structure's opening geometry. While applying a blockage progressively from T_P to $T_{OBV/FL}$ provides a reasonable approximation of when a floating blockage most impacts flow through a culvert or bridge that overtops, it does not sensibly reflect behaviour when floodwater carrying floating debris does not reach the obvert of the structure. In the absence of any better information it is recommended that a progressive top down blockage by floating debris that does not reach the structures obvert be initiated at $T_{OTB/SA}$ and ramped up to B_{DES} at T_P. It should also be noted that a floating raft creates a top down blockage only as a consequence of the projection of floating debris below its water surface. Relative to the structures opening height this projection will lift on the rising limb and fall on the falling limb creating a quite variable level of blockage of the structure itself during the event. Under such circumstances, blockage levels of the structure will be controlled by both the water depth and projection of the raft below the water level. Detailed simulation of such a process is however considered beyond the scope of this chapter. This chapter assumes that a top down blockage will be simplistically modelled by lowering the obvert of the structure over the tabulated time to then reflect the tabulated blockage level. Where the consequences of this form of blockage are high, and more realistic simulation is deemed necessary, it may be necessary to develop a site specific procedure. More information on this process can be found in Parola (2000), US DOTFHA (2005) and USGS (2013).

While the temporal pattern of a structure's blockage when it blocks prior to the flood peak in a system with little flood storage will have minimal impact on downstream peak flows or upstream peak flood levels, it can substantially alter the duration that upstream flood levels are above a certain level (floor or structure overtopping) level. In a system with significant flood storage, the timing of a structure's blockage can significantly alter upstream peak flood level, downstream peak discharge and overtopping duration. Consideration of the temporal pattern of a blockage can therefore be extremely important in realistically simulating the hydraulic impact of a blockage.

In establishing the key timings referred to in Table 6.6.9, it will normally be necessary to first run a simulation with estimated blockage levels and timings in place.

When modelling a historic event, hydraulic analysis will need to reflect (as far as available data permits) the actual blockage mechanism that developed at the structure during the event. It should be noted that this may vary significantly from what this chapter provides as the ‘most likely’ blockage scenario for the structure, such is the impact of near random chance on the many parameters influencing actual blockage. However, where data for multiple historic events is available and blockage appear to consistently differ from these chapter recommendations, further investigation is warranted, with historic data, if of reasonable quality, being given precedence.

6.6. Management of Blockage

6.6.1. Design Considerations

Even though floodway crossings can be subject to blockage issues, by far the greatest attention is given to the management of blockage at culvert and bridge crossings.

To minimise the adverse impacts of debris blockage on bridges the following design considerations should be given appropriate consideration:

- Minimise the number of in-stream piers.
- Minimise the exposure of services (i.e. water supply pipelines) on the upstream side of the bridge, and/or minimise the likelihood of debris being captured on exposed services.

To minimise the effects of debris blockage on culverts the following design consideration should be noted:

- Take all reasonable and practicable measures to maximise the clear height of the culvert, even if this results in the culvert hydraulic capacity exceeding the design standard. This minimises the likelihood of debris being caught between the water surface and obvert, and also minimises the risk of a person drowning if swept through the culvert (i.e. the culvert is more likely to be operating in a partially full condition).
- The risk of debris blockage can also be reduced by using single-cell culverts, or in the case of floodplain culverts, spacing individual culvert cells such that they effectively operate as single-cell culverts without a common wall/leg (Figure 6.6.1 and Figure 6.6.2).

Figure 6.6.1. Series of Floodplain Culverts

Figure 6.6.2. Floodplain Culvert

- One means of maintaining the hydraulic capacity of culverts in high debris streams is to construct debris deflector walls (1V:2H) as shown in Figure 6.6.3 and Figure 6.6.5. The purpose of these walls is to allow the debris that normally collects around the central leg to rise with the flood, thus maintaining a relatively clear flow path under the debris. Following the flood peak, the bulk of the debris rests at the top of the deflector wall allowing easier removal (Figure 6.6.4).

Figure 6.6.3. Debris Deflector Walls

Figure 6.6.4. Post Flood Collection of Debris on Top of Deflector Walls

- Sedimentation problems within culverts may be managed using one or more of the following activities:
  - Formation of an in-stream sedimentation pond or trap upstream of the culvert.
  - Formation of a multi-cell culvert with variable invert levels such that the profile of the base slab simulates the natural cross section of the channel (Figure 6.6.6).
  - Installation of sediment training walls on the culvert inlet (Figure 6.6.3 and Figure 6.6.5). Sediment training walls reduce the risk of sedimentation of the outer cells by restricting minor flows to just one or two cells.

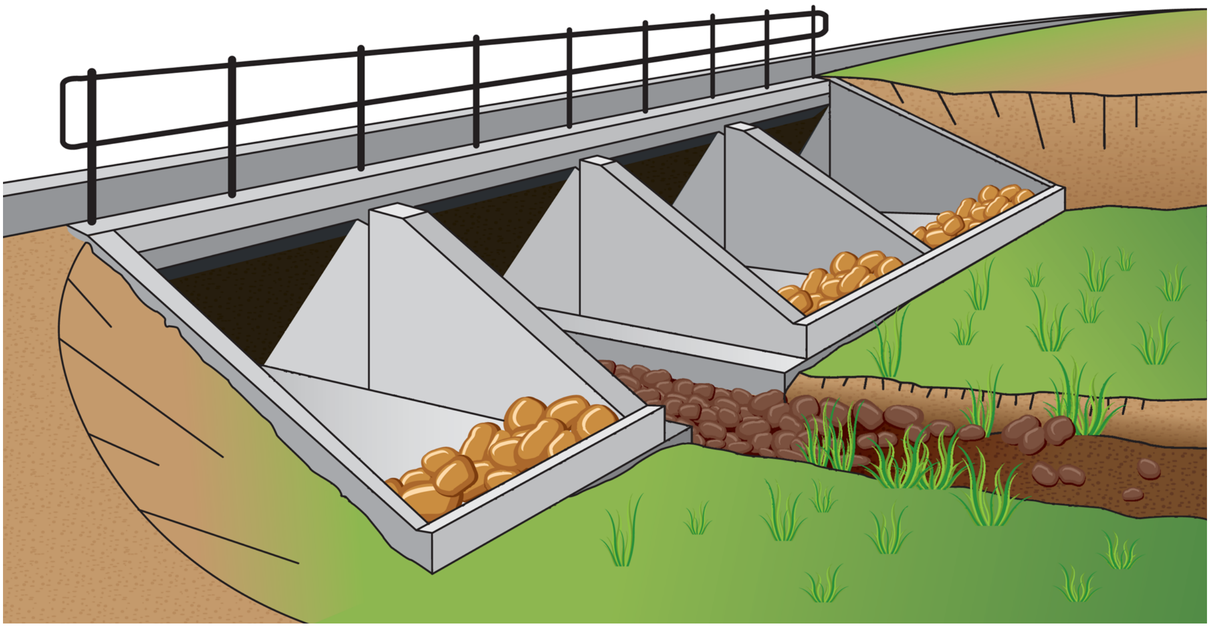


Figure 6.6.5. Sediment Training Walls Incorporated with Debris Deflector Walls (Catchments & Creeks Pty Ltd)

Figure 6.6.6. Multi-Cell Culvert with Different Invert Levels

Figure 6.6.7. Debris Deflector Walls and Sediment Training Wall Added to Existing Culvert

- Where space allows, a viable alternative to increased culvert capacity (in response to the effects of debris blockage) may be to lengthen the roadway subject to overflow (i.e. the effective causeway weir length).
- Where high levels of floating debris are present and frequently become trapped on hand rails, collapsible hand rails may be considered. Such systems typically include pins or bolts designed to fail when water becomes backed up by the handrails and therefore require ongoing maintenance. If used as traffic barriers the downstream rail fixing can be problematic. They can however limit rises in floodwater levels upstream of the structure.

6.6.2. Retro-fitting Existing Structures

Structures can be modified to allow debris to be directed through the structure with a reduced risk of blockage. These modifications can include improved inlet performance through the use of debris deflection walls and/or sediment training walls (Figure 6.6.7) or an increase in the size of the structure.

6.6.3. Debris control structures

Debris control structures or traps are structural measures provided in a watercourse upstream of critical structures to collect debris before it reaches the structure and causes problems. These can be (a) fences, posts or rails providing a much larger ‘interception area’ for debris than a pipe or culvert entrance, (b) storages or dry basins in which boulders or other debris can collect, or (c) diversion structures designed to provide safe bypass of debris or water. Such structures can occasionally be incorporated into a water quality management plan for a catchment.

Where debris control structures or at-source control measures have been implemented, these should be incorporated into the assessment of the drainage system, which could mean a reduction in the allowance that needs to be made for blockage. Ongoing maintenance is however fundamental to the successful operation of these measures. Unless a deliberate maintenance program is in place and has been demonstrated to work, it would not be prudent to lower design blockage levels.as a consequence of such works.

Care should also be taken to ensure that the hydraulic impact of the debris control structure does not itself aggravate flooding in the system.

6.7. Conclusion

The inclusion of blockage in the analysis of hydraulic structures in drainage systems is an important consideration in the realistic simulation of flood behaviour. The impact of blockage is however a complex and difficult problem to analyse. It is important to ensure that the estimate of blockage used in analysis is probability neutral and not over or under-estimated as this can influence the performance of the total system. This chapter has presented an approach to the assessment of design blockage that has been developed in consultation with Australian experts and provides a consistent and logical approach to assist in the effective planning and design of drainage systems. Future investigation will refine this approach.

For further information on the background to this chapter, readers are referred to the following bibliography.

6.8. References

Parola, A.C. (2000), Debris Forces on Highway Bridge Issue 445, Prepared by the U.S. Transport Research Board of the National Research Council Queensland Department of Natural Resources and Water (2008).

Pilgrim, DH (ed) (1987) Australian Rainfall and Runoff - A Guide to Flood Estimation, Institution of Engineers, Australia, Barton, ACT, 1987.

Sundborg, A. (1956), The River Klarålven: Chapter 2. The morphological activity of flowing water-erosion of the stream bed: Geografiska Annaler, 38: 165-221.

U.S. Department of Transportation Federal Highway Administration (US DOTFHA), (2005), Debris Control Structures - Evaluation and Countermeasures. (DOTFHA) Hydraulic Engineering Circular No.9 Third Edition, 2005. Publication No. FHWA-IF-04-016.

U.S. Geological Survey (USGS) (2013), Potential Drift Accumulation at Bridges. URL: http:// tn.water.usgs.gov /pubs/FHWA-RD-97-028/accumwid.htm.

Weeks, W., Barthelmess, A., Rigby, T., Witheridge, G. and Adamson, R. (2009),'Blockage in Drainage Structures', Hydrology and Water Resources Symposium, Engineers Australia, Newcastle.

Chapter 7. Safety Design Criteria

Grantley Smith, Ron Cox

Chapter Status	Final
Date last updated	14/5/2019

7.1. Introduction

The safety of people in floods is of major concern in floodplain management for both rural and urban areas. Consideration of the circumstances for individual flood fatalities, both in Australia and internationally, indicates that flood fatalities occur most commonly when people enter floodwaters either on foot or in a vehicle (French et al., 1983; Coates and Haynes, 2008; Haynes et al., 2016). However, where floodwaters rise rapidly and unexpectedly in flash flood areas, people may also perish trapped inside buildings as occurred in the Lockyer Valley QLD in 2010 (Rogencamp and Barton, 2012) and in Dungog, NSW in 2015. Further, recent analysis of the Queensland floods in 2011 by the Queensland Commission for Children, Young People, and Child Guardian (CCYPCG, 2012) and the Queensland Fire and Rescue Service, 2012 has reinforced the conclusion that floodwaters are extremely dangerous to both people trapped inside building or wading or driving vehicles of all types in floods. While floodplain management activities aim to reduce the risk arising from flooding events, ongoing human interaction with floodwaters during flood events is largely unavoidable, as significant areas of existing development and transport infrastructure in Australia remain within flood prone regions.

Records for past floods show that exposure of the community to flooding can result in significant death tolls and 1859 flood fatalities have occurred nationally between 1900 and 2015 (Haynes et al., 2016). Flood fatalities are significantly higher in flash flood events with rapidly rising violent flood flows than in comparably slower rising and moving riverine flooding. Two hundred and six (206) flash flood fatalities occurred in Australia between 1950 and March 2008 (Coates and Haynes, 2008). The cause of death for the majority of these cases was drowning. Other fatalities were a result of heart attacks or overexertion, or indirect causes such as electrocution or fallen trees (Coates and Haynes, 2008). Similarities have been observed in the United States, where 93 % of flash flood deaths can be attributed to drowning (French et al., 1983). Details about the activity of flash flood victims immediately prior to death are available for just under 50 % of the victims. Of these, almost 53 % perished attempting to cross a watercourse, either by wading/swimming, or by using a bridge or ford (Coates and Haynes, 2008). These values include those in vehicles. The motivation behind the activity leading to the death was known for 47 % of the study group. Of these, almost 22 % were undertaking business as usual, either attempting to reach a destination, ignoring the flood warnings or unaware of the flood intensity (Coates and Haynes, 2008).

The majority (31 %) of the Australian flash flood fatalities, for which the mode of transport is known, were inside a vehicle at the time of death. Similar results have been observed around the world, 42 % of the 93 % US flash flood drowning fatalities were vehicle-related (French et al., 1983) and 63 % of US riverine and flash flood fatalities were found to be vehicle-related (Ashley and Ashley, 2008). Jonkman and Kelman (2005) noted that vehicle-related fatalities occurred most frequently (33 %) in European and US floods.

The Lockyer Valley floods of January 2011 dramatically demonstrated that sheltering in a residential building was also not a safe option where flood flows have high force and damage potential. Of the nineteen people who perished in the Lockyer Valley floods, thirteen were sheltering in buildings that were either completely inundated or collapsed under the force of the flood flows (Rogencamp and Barton, 2012).

Regardless, the high numbers of people that die in vehicles or on foot highlights the considerable risk in fleeing flash flood events. In many cases, people become exposed to greater risk when attempting to flee a flood affected area (Ashley and Ashley, 2008; Coates, 1999; Drobot and Parker, 2007; Jonkman and Kelman, 2005). The risks to those fleeing are not just the floodwaters themselves, but also include poor driving conditions, the danger of being hit by falling debris, electrocution from fallen power lines, lightning and mudslides (Haynes et al., 2016).

Whilst evacuation is generally considered the safest of emergency management options during flood events, it is not always possible. Subsequently, it is an important aspect of emergency planning to ensure that in flood prone locations where timely evacuation may not be possible people will not be in greater danger remaining in their homes.

Jonkman and Kelman (2005) highlighted that in most floods, people are more likely to be killed or injured if they are outside of their home or in their cars during the flood. Subsequently, undertaking evacuation at inappropriate times, such as when the floodwaters have risen in depth and velocity, is likely to increase chance of death (Cave et al., 2009).

Sound floodplain management and emergency planning requires identification of the location, timing and duration of potentially hazardous floodplain areas for design flood conditions and the careful assessment of the most suitable mitigation options taking into consideration the specifics of each floodplain location. The intention of this chapter of Australian Rainfall and Runoff is to provide background information and guidance on the application of approaches for prediction of flood hazard in those locations; additional background information can be obtained from Cox et al. (2010), Shand et al. (2011) and Smith et al. (2014). Note that while guidance is provided on predicting the flood hazard and flood risk, it is the role of the relevant floodplain management authority to define the acceptability or otherwise of the predicted risk.

A detailed discussion of risk with respect to floodplain management is presented in Book 1, Chapter 5 of Australian Rainfall and Runoff.

7.2. Flood Hazard

7.2.1. General Introduction

In terms of floodplain management, hazard can be defined as a source of potential harm or a situation with potential to result in loss. Hence, the primary hazard is the result of a flood event that has the potential to cause damage or harm to the community. Associated with the hazard is the probability of its occurrence.

There are a number of factors to be considered where assessing the hazard associated with floods. The usual starting point is to predict the flood characteristics and particularly the flow characteristics of the inundated areas of the floodplain. The main characteristics of interest typically are the flow depth and the flow velocity. In addition, the assessment of the flood hazard needs to consider a range of other social, economic and environmental factors, though these are often more difficult to quantify.

The magnitude of flood hazard can be variously influenced by the following factors:

- Velocity of Floodwaters;
- Depth of Floodwaters;
- Combination of Velocity and Depth of Floodwaters;
- Isolation During a Flood;
- Effective Warning Time; and
- Rate of Rise of Floodwater;

When quantifying and classifying flood hazard, it is important to understand the underlying causes of the hazard level. For example, if the hazard level is classified as ‘high’ then it is important to understand the key reason that it is high e.g. high depth, high velocity, high velocity and depth in combination, isolation issues, short warning time? If the core reasons that the hazard is high are not well understood, then attempts to modify and lower the hazard level may not be successful.

7.2.2. Flood Hazard Assessment

The base data that underpins assessment of floodplain risk typically comprises the flow characteristics (the flow depth and velocity) in the flood-affected areas of the catchment. A common approach to obtaining this information is the analysis of predictions obtained from catchment numerical modelling systems although physical models of the flood affected area may be used. More information on the application of catchment modelling systems is presented in Book 4 to Book 7 of Australian Rainfall and Runoff.

The data used for assessment of the floodplain hazard are presented commonly as maps of flood depth (see Figure 6.7.1) and flood velocity (speed and direction). Typically, these maps are shown as an envelope of maxima; a time series of flow behaviour, however, is an alternative presentation format.

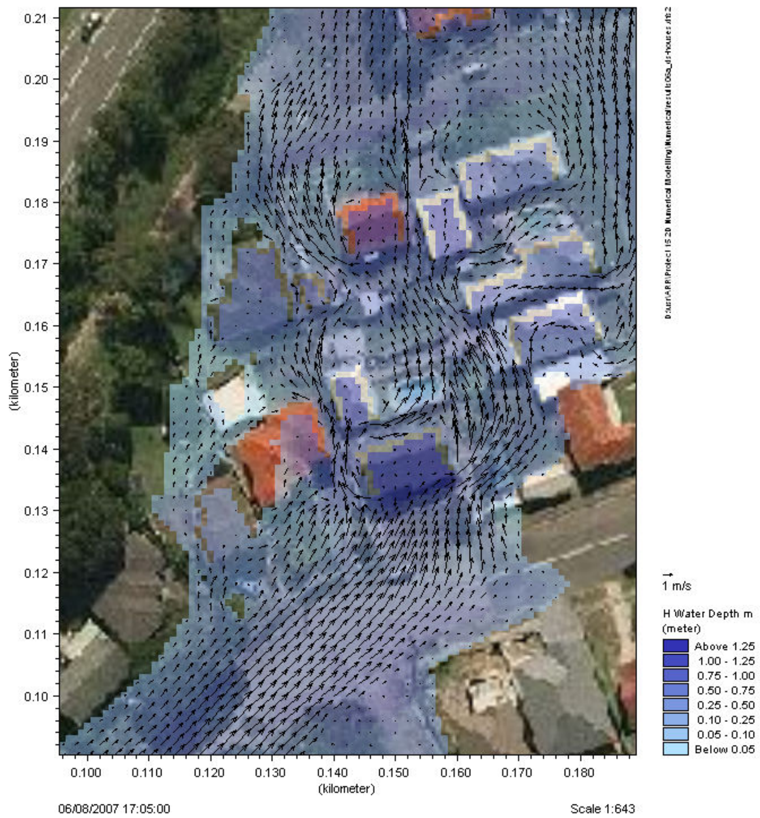


Figure 6.7.1. Example of a Flood Study Depth Map (Smith andWasko (2012))

Smith andWasko (2012) investigated the effects of alternative grid resolutions on prediction of the flood hazard. The predicted flood hazard (computed as D.V – flood depth times flood velocity) for two model grid resolutions, namely 1m and 10m, for the 2007 flood event at Merewether, NSW are shown in Figure 6.7.2. Comparing the predicted flood hazard estimates, it can be seen that those derived using the 1m grid are higher than those obtained with the 10m grid. Furthermore, Smith andWasko (2012) report that the predicted velocity and depth characteristics for the 1m grid more closely replicated those obtained from a physical model of the same area.

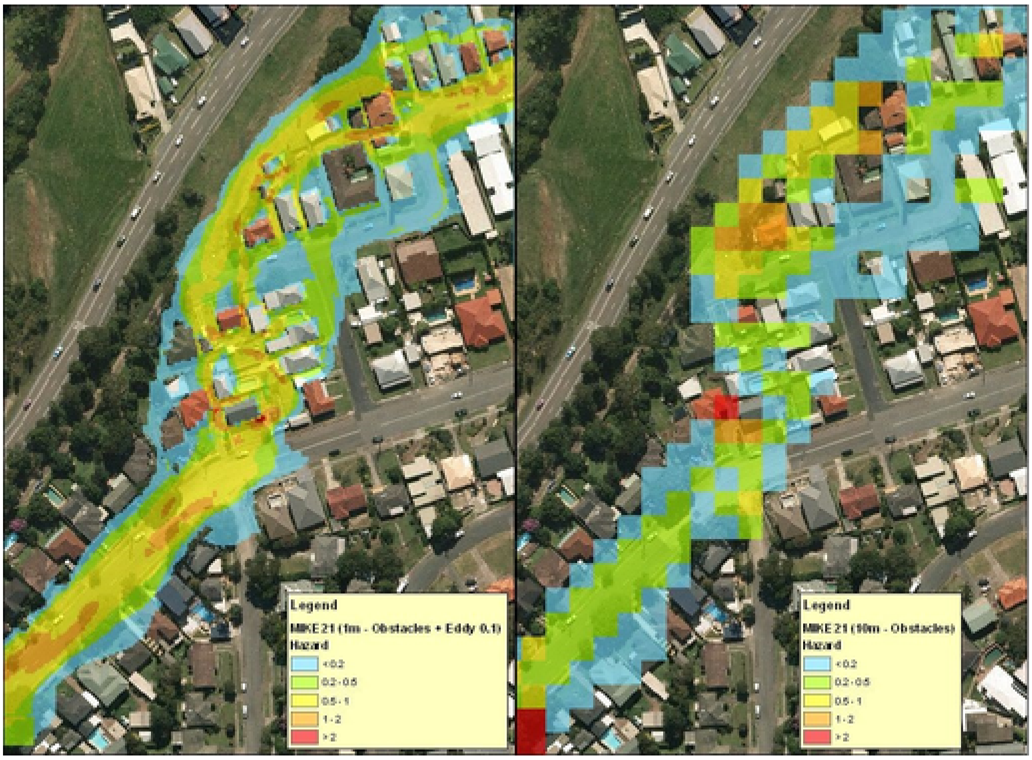


Figure 6.7.2. Comparison of Provisional Flood Hazard Estimates from Numerical Models at Differing Grid Resolutions (after Smith and Wasko, 2012)

a) 1 m grid resolution b) 10 m grid resolution

An important conclusion of Smith andWasko (2012) was that predictions from 2D numerical hydrodynamic models require further interpretation in order to ensure that suitable, representative flood behaviour information was obtained for application, especially in emergency planning and management decisions.

As described in the introduction to this chapter, people tend to be at risk in one of three main categories; on foot, in vehicles or in buildings. Subsequently, in order to further assess the vulnerability of a flood under the predicted conditions, flood hazard assessment can be divided into three categories; people stability, vehicle stability and structural stability.

7.2.3. People Stability

The two recognised hydrodynamic mechanisms by which people may lose stability in flood flows are *moment instability* and *friction instability* (Figure 6.7.3). A summary description of these mechanisms is provided here based on the comprehensive discussion of the topic presented in Cox et al. (2010).

In brief, moment (toppling) instability occurs when a moment induced by the oncoming flow exceeds the resisting moment generated by the weight of the body (Abt et al., 1989). This stability parameter is sensitive to the buoyancy of a person within the flow and to body positioning and weight distribution.

Frictional (sliding) instability occurs when the drag force induced by the horizontal flow impacting on a person's legs and torso is larger than the frictional resistance between a person's feet and the ground surface. This stability parameter is sensitive to weight and buoyancy, clothing type, footwear type and ground surface conditions.

Additionally, loss of stability may also be triggered by adverse conditions, which should be taken into account when assessing safety such as:

- **Bottom conditions:** uneven, slippery, obstacles;
- **Flow conditions:** floating debris, low temperature, poor visibility, unsteady flow and flow aeration;
- **Human subject:** standing or moving, experience and training, clothing and footwear, physical attributes additional to height and mass including muscular development and/or other disability, psychological factors;
- **Other issues:** strong wind, poor lighting, etc.

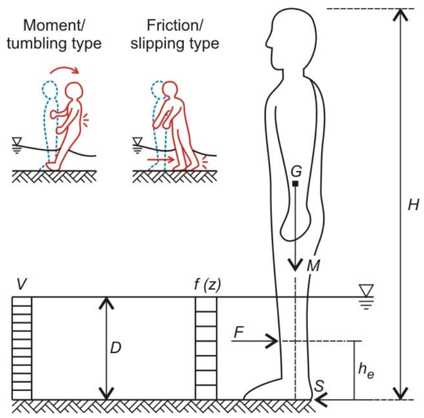


Figure 6.7.3. Typical Modes of Human Instability in Floods (Cox et al., 2010)

Determining safety criteria for people requires an understanding of the physical characteristics of the subjects along with the nature of the flow. The best measure of physical attributes for human stability is the parameter H.M (mkg), the product of subject height (H; m) and mass (M; kg) (Cox et al., 2010). The measure of flow attributes is the parameter D.V (m^2s^{-1}), the product of flow depth (D, m) and flow velocity (V, ms^{-1})

While distinct relationships exist between a subject's height and mass and the tolerable flow value, definition of general flood flow safety guidelines according to this relationship is not considered practical given the wide range in such characteristics within the population.

In order to define safety limits, which are applicable for all persons, hazard regimes are defined based on H.M for representative population demographics. Each classification is based on laboratory testing of subject stability within floodwaters. The following suggested classifications, after Cox et al. (2010) are:

- Adults, where H.M > 50 mkg;
- Children, where H.M is between 25 and 50 mkg; and
- Infants and very young children, where H.M < 25 mkg.

Several hazard regimes are recommended based on D.V flow values for each H.M classification. The hazard regimes, as suggested from laboratory testing of subject stability and response within variable flow conditions, are:

- Low hazard zones where D.V < 0.4 m^2s^{-1} for children and D.V < 0.6 m^2s^{-1} for adults;

- A Significant hazard zone for children exists where flow conditions are dangerous to most between D.V = 0.4 to 0.6 m^2s^{-1};

- Moderate hazard zone where conditions are dangerous for some adults and all children is defined between D.V = 0.6 to 0.8 m^2s^{-1} for adults. This is inferred to define the limiting working flow for experienced personnel such as trained rescue workers;

- Significant hazard zone where flow conditions are dangerous to most adults and extremely dangerous for all children is suggested between flow values of D.V = 0.8 to 1.2 m^2s^{-1}; and

- Extreme hazard where flow conditions are dangerous to all people is suggested for D.V > 1.2 m^2s^{-1}.

Cox et al. (2010) concluded that self-evacuation of the most vulnerable people in the community (typically small children, and the elderly) is limited to relatively placid flow conditions. Furthermore, a D.V as low as 0.4 m^2s^{-1} would prove problematic for people in this category, i.e. the more vulnerable in the community.

These hazard regimes for tolerable flow conditions (D.V) as related to the individual's physical characteristics (H.M) are presented in Figure 6.7.4 and Table 6.7.1.

Table 6.7.1. Flow Hazard Regimes for People (Cox et al., 2010)

DV (m^2s^{-1})	Children (H.M = 25 to 50)[1]	Adults (H.M > 50)
0	Safe	Safe
0 - 0.4	Low Hazard if depth < 0.5m and velocity < 3m/s otherwise Extreme Hazard	Low Hazard if depth < 1.2m and velocity < 3m/s otherwise Extreme Hazard
0.4 - 0.6	Significant Hazard; Dangerous to most if depth < 0.5m and velocity < 3m/s otherwise Extreme Hazard	
0.6 - 0.8	Extreme Hazard; Dangerous to all	Moderate Hazard; Dangerous to some[2] if depth < 1.2m and velocity < 3m/s otherwise Extreme Hazard
0.8 - 1.2		Significant Hazard; Dangerous to most[3] if depth < 1.2m and velocity < 3m/s otherwise Extreme Hazard
> 1.2		Extreme Hazard; Dangerous to all

Maximum depth stability limit of 0.5 m for children and 1.2 m for adults under good condition. **Maximum velocity stability limit of 3.0 ms^{-1}** for both adults and children.

[1]More vulnerable community members such as infants and the elderly should avoid exposure to floodwater. Flood flows are considered extremely hazardous to these community members under all conditions

[2]Working limit for trained safety workers or experienced and well equipped persons (D.V < 0.8 m^2s^{-1})

[3]Upper limit of stability observed during most investigations (D.V > 1.2 m^2s^{-1})

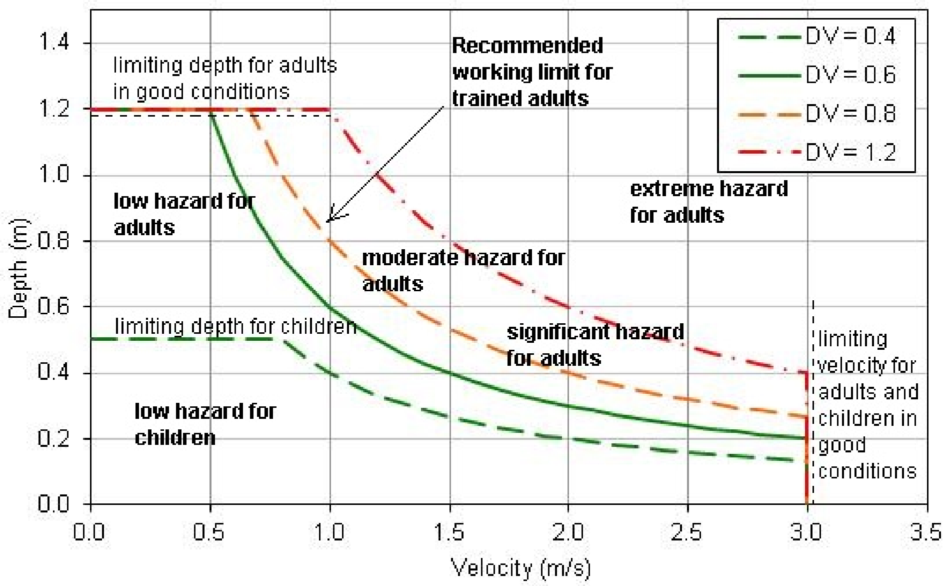


Figure 6.7.4. Safety Criteria for People in Variable Flow Conditions Cox et al. (2010)

7.2.3.1. Physical considerations

There is a lack of test data for infants and very young children as well as frail/older persons (Cox et al., 2004). These populations are unlikely to be safe in any flow regimes and as such, care is required in locating aged care and retirement villages as well as childcare centres and kindergartens.

For physically and/or mentally disabled people, similar intolerance criteria to the very young children and frail/older persons should be applied as subjects are considered vulnerable to all flow values. This is because while the H.M values may be similar to regular adults, they are clearly at a physical (e.g. muscular development, control of limbs) and/or psychological disadvantage (e.g. cognisant of the real/perceived danger, inability to cope with external stimulus).

Emergency personnel tasked with carrying evacuees should be aware that the additional H.M gained by carrying a person is not necessarily a benefit to their stability. This was demonstrated in a particular laboratory test of human stability criteria, Jonkman and Penning-Roswell (2008), who note that their test subject (a trained stuntman) considered balancing in the flowing water more difficult when carrying extra weight such as a child or elderly person.

It should also be noted that while these criteria are based on experimental data for loss of stability for persons wading in floodwaters, it is also inherently dangerous to swim through floodwaters. Swimming through floodwaters should not be attempted.

7.2.3.2. Psychological/behavioural considerations

A person's ability to withstand flood flows is affected by their mental disposition, perception, specific training and experience.

Where specific training has been undertaken or a subject has recent and relevant experience, personnel are able to tolerate situations of high D.V (Jonkman and Penning-

Roswell, 2008). A limiting working flow of D.V = 0.8 m^2s^{-1} is suggested for experienced personnel such as trained rescue workers. These personnel should, where possible, be equipped for dangerous flow conditions with safety restraints, floatation aids and other safety apparatus, and be trained to cope with high D.V situations. It is trained emergency personnel who are likely to be instructing, driving and guiding evacuation paths, and consequently to whom the upper limit of safety design criteria is directed.

7.2.4. Vehicle stability

The two recognised hydrodynamic mechanisms by which stability of vehicles is lost include *buoyancy* or *floating* and *friction instability or sliding instability* (refer Figure 6.7.5). More comprehensive discussion is presented within Shand et al. (2011) but briefly, vehicle floating instability occurs when the upward buoyancy force exceeds the downward force exerted by the vehicle mass. This instability is dominant in low velocity, high depth flows. Frictional or sliding instability occurs when the horizontal force exerted on one or more car panels is greater than the vertical restoring force, which is dependent on the vehicle mass, buoyancy and the friction between the car tyres and road surface.

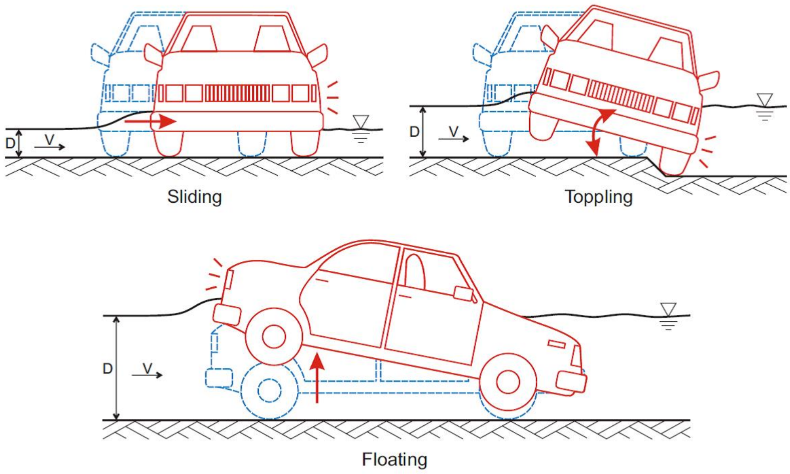


Figure 6.7.5. Typical Modes of Vehicle Instability (Shand et al., 2011)

Note that in the context of this discussion *friction instability* is associated with slow moving or stationary cars as distinct to *hydroplaning*, which occurs when a vehicle at high speed encounters very shallow, evenly distributed water covering a road, typically a highway or freeway. Hydroplaning is not considered further within this report.

Determining safety criteria for vehicles requires an understanding of the physical characteristics of the vehicle along with the nature of the flow.

The measure of physical attributes for vehicle stability analysis is the vehicle classification as based on length (L, m), kerb weight (W, kg) and ground clearance (GC, m). Three vehicle classifications are suggested:

- Small passenger: L < 4.3 m, W < 1250 kg, GC < 0.12 m
- Large passenger: L > 4.3 m, W > 1250 kg, GC > 0.12 m
- Large 4WD: L > 4.5 m, W > 2000 kg, GC > 0.22 m

The measure of flow attributes for vehicle stability analysis is D.V m^2s^{-1}, determined as the product of flow depth (D, m) and flow velocity (V, ms^{-1})

Limiting conditions exist for each classification based on limited laboratory testing of characteristic vehicles. The upper tolerable velocity for moving water is defined based on the frictional limits, and is a constant 3.0 ms^{-1} for all vehicle classifications.

The upper tolerable depths within still water are defined by the floating limits:

- Small passenger vehicles: 0.3 m
- Large passenger vehicles: 0.4 m
- Large 4WD vehicles: 0.5 m

The upper tolerable depths within high velocity water (at 3.0 ms^{-1}) are defined by the frictional limits:

- Small passenger vehicles: 0.1 m
- Large passenger vehicles: 0.15 m
- Large 4WD vehicles: 0.2 m

While specifically equipped vehicles may remain stable in water of greater depths, the intention of the presented criteria is to focus on the more vulnerable of typical vehicle types in common use.

Note that for all flow conditions in all vehicle classes, *the proposed vehicle safety criteria remain below the moderate hazard criteria for adults* (Cox et al., 2010). This ensures that adults occupying vehicles are, in principle, safe if exiting a vehicle in floodwaters with attributes within the specified hazard ranges.

During flood events, the majority of flood deaths are vehicle related, more than half of all deaths during floods in the United States are vehicular-related (Gruntfest and Ripps, 2000). Regardless of how often people see the power of water in flash floods or are notified through community advertising that driving through high water is dangerous, there remain sections of the community who will continue with ‘business as usual’ irrespective of the flooding conditions (Gruntfest and Ripps, 2000).

7.2.4.1. Vehicle Modernisation and Scale

A limiting aspect of the advice provided in this report is that vehicle stability data sets are limited to dated laboratory data (Shand et al., 2011). The properties of contemporary vehicles have significantly changed vehicle stability criteria through modified buoyancy properties (e.g. improve dust sealing), weight and ground clearance. These changes apply to all scales of vehicles from small passenger to large commercial vehicles.

As a result, the hazard criteria provided in this report are identified as interim recommended limits based on interpretation of existing information. The criteria presented here are subject to change once acceptable data for modern vehicles becomes available.

7.2.4.2. Stability Criteria for Vehicles

Stability criteria based on the best available information for stationary small passenger cars, large passenger cars and large 4WD vehicles in various flow situations are presented in Figure 6.7.6 and Table 6.7.2.

Table 6.7.2. Interim Flow Hazard Regimes for Vehicles (Shand et al., 2011)

Class of vehicle	Length (m)	Kerb Weight (kg)	Ground clearance (m)	Limiting still water depth[1]	Limiting high velocity flow depth[2]	Limiting velocity[3]	Equation of stability
Small passenger	< 4.3	< 1250	< 0.12	0.3	0.1	3.0	$DV \leq 0.3$
Large passenger	> 4.3	> 1250	> 0.12	0.4	0.15	3.0	$DV \leq 0.45$
Large 4WD	> 4.5	> 2000	> 0.22	0.5	0.2	3.0	$DV \leq 0.6$

[1]At velocity = 0 ms^{-1}; [2]At velocity = 3.0 ms^{-1}; [3]At low depth

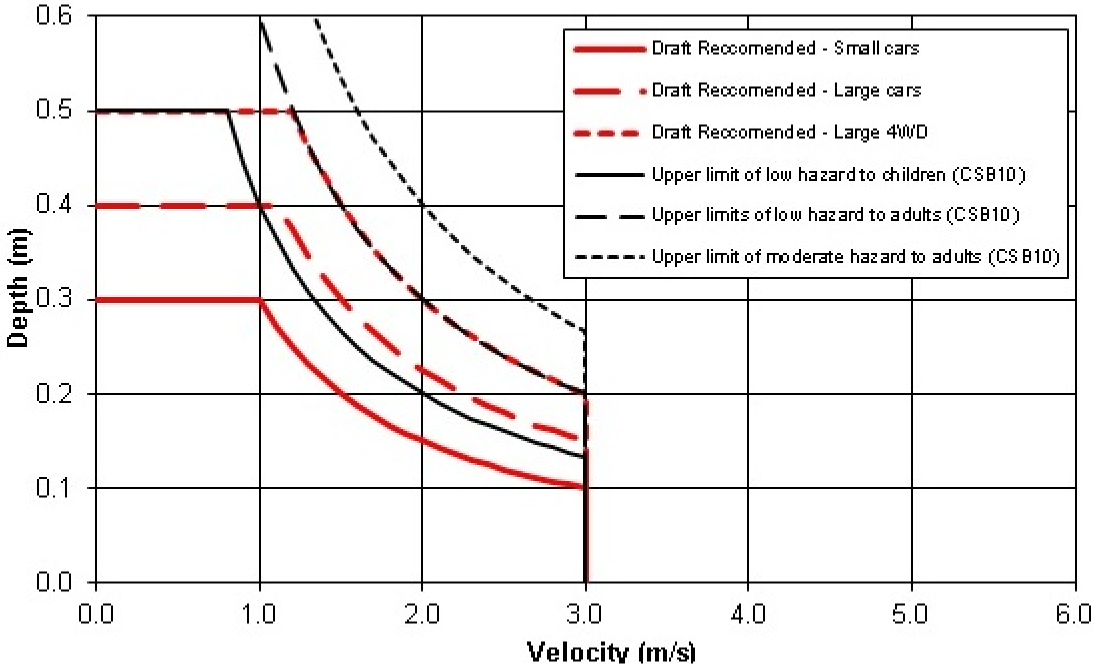


Figure 6.7.6. Interim Safety Criteria for Vehicles in Variable Flow Conditions (Shand et al., 2011)

Shand et al. (2011) concludes that the available datasets do not adequately account for the following factors and that more research is needed in these areas:

- Friction coefficients for contemporary vehicle tyres in flood flows;
- Buoyancy changes in modern cars;
- The effect of vehicle orientation to flow direction (including vehicle movement);
- Information for additional categories including small and large commercial vehicles and emergency service vehicles

7.2.5. Building Stability

A comprehensive summary of available literature describing the stability of buildings in floodwaters is provided in Smith et al. (2014). Numerous hazard threshold curves for building were collated from international literature and are compared in Figure 6.7.6. The collated curves have a variety of origins. As discussed by (Leigh, 2008), it can be difficult to synthesise the different building stability curves and associated data, as they are derived by various means of analysis. Subsequently, comparison between theory based curves, (e.g. Black (1975); Dale (2004)) field derived curves (e.g. Clausen and Clark (1990)) and curves derived from modelling and analysis (e.g. Becker *et al.*, 2011) is difficult. Further, different damage thresholds may apply for each of these curves e.g. some threshold curves presented represent the initiation of structural damage, while others represent the flood conditions for complete destruction of the building. The spread of curves in Figure 6.7.6 highlights the overall uncertainty surrounding building stability during flood events.

Investigation and review of the available information concerning the failure of building structures under flood loads has also been conducted by Kelman and Spence (2004) and (Leigh, 2008). Amongst a range of relevant conclusions, these reviews noted that while a series of studies had theoretically analysed incident flood forces compared to the resisting strength of various building structures, most of these studies had considered components of flood forces in isolation or in limited combinations e.g. hydrostatic and simplified hydrodynamic (velocity head) or buoyancy and drag forces.

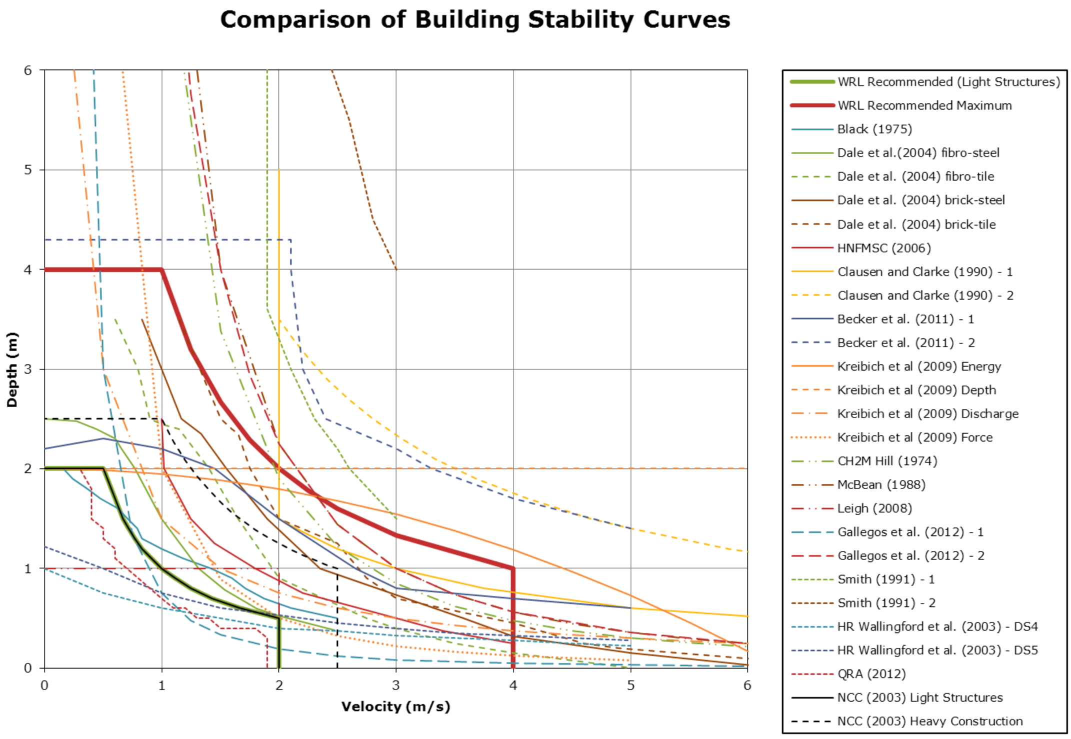


Figure 6.7.7. Comparison of Building Stability Curves

While the considerable variability in building construction is acknowledged, the analysis of building damage leading to collapse reported by Mason *et al.*, (2012) for the Lockyer Valley floods in January 2010 is compelling. This analysis shows that buildings constructed for Australian conditions are vulnerable to damage and collapse under flood hazard conditions at the lower end of the scale presented in Figure 6.7.6.

On this basis, the green curve in Figure 6.7.6 is proposed as a lower threshold for residential homes, built without consideration of flood forces. This curve can be used as a minimum criterion for building stability in existing flood affected areas.

The hazard zone between the green curve and the upper limit red curve in Figure 6.7.6 identifies flood hazard conditions where it is considered, if required, possible to construct a purpose built structure that is an appropriately engineered structure specifically designed to withstand the full range of anticipated flood forces including:

- **Hydrostatic forces**

  resulting from standing water or slow moving flow around the structure;

- **Buoyant forces**

  due to displaced volume of water;

- **Hydrodynamic forces**

  arising from moderate-to-high-velocity water flow around the structure;

- **Impulsive Forces**

  caused by the leading edge of the water impacting the structure;

- **Uplift forces**

  on elevated floors of a structure that are submerged during a flood event;

- **Debris Impact Forces**

  generated by floating debris colliding with the structure;

- **Damming of Waterborne Debris**

  due to the accumulation of debris on the upstream side of the structure, which results in an increase in the hydrodynamic force.

- **Wave actions**

  from wind and wakes; and

- **Erosion and Scour**

  due to flood actions.

In locations where timely evacuation is not possible, such purpose built structures may be required for vertical evacuation, not dissimilar to the process used in Japan for tsunamis. However, it would be important to ensure the structure was purpose built for the conditions it would be likely to encounter, up to and including the PMF or a similar extreme flood event. The bottom floor of such structures may need to be somewhat sacrificial during a flood event, for example, the windows and doors may ‘blow out’ under high flow conditions, however the building’s structural members will be required to remain intact.

The red curve in Figure 6.7.6 is a suggested upper limit for all buildings. Buildings in areas classified with flood hazard above this threshold are considered vulnerable to collapse under these extreme flood conditions.

7.2.6.

Previous hazard classification curves (e.g. SCARM (2000);HNFMSC (2006)) provided a single set of hazard curves that divide flood hazard levels into generic classifications of low, medium, high etc. While the thresholds between these classifications had some basis in data collected for stability/vulnerability of people and risk to life, in practice, such threshold curves have been widely interpreted (sometimes mis-interpreted) and applied in myriad ways.

It is interesting to compare the curves summarised for people, vehicle and building stability compiled for this report. Figure 6.7.7 provides a direct comparison of these three sets of curves. The first observation to be made is that for slow moving floodwaters at depths greater than 0.5 m, adults wading through floodwater are generally considered more stable than vehicles i.e. in most cases, vehicles are equally unstable or more unstable than adults wading through the same flow conditions. Secondly, the stability limit for an untrained adult walking through floodwater (D x V = 0.8) is almost the same level as the lower threshold limit for building stability (D x V = 1.0). Also, that for shallow fast moving flows, building stability (through foundation erosion/scour) may be less than the stability of a person walking through the same flow conditions. In some situations, this means that you would be safer to walk out through the prevailing floodwaters rather than sheltering in a poorly constructed building.

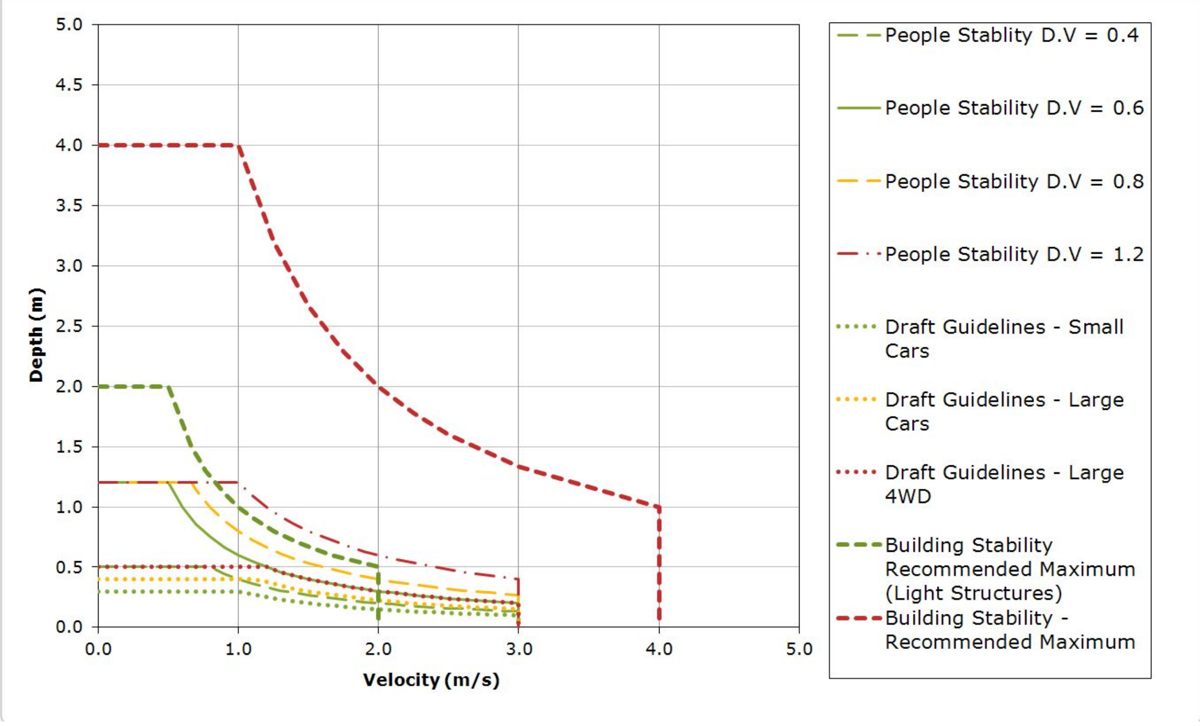


Figure 6.7.8. Comparison of Updated Hazard Curves (after Smith et al. 2014)

The third observation is that the flood water level that is used as the basis of a hazard depth varies between people and vehicle stability, where the flood depth is referenced to the ground level and building stability where the flood depth is nominally referenced to the floor level.

On a practical level, this would mean that once physical flood behaviour has been quantified in terms of flood depth and velocity, flood hazard could be classified individually for people, vehicles or building thresholds separately. In many instances, this will suit the requirements of specific analyses. For example, if the required assessment is to determine whether a road evacuation route is trafficable for a given flood event, then the vehicle stability threshold curves should be applied. Likewise, if the assessment is to determine which buildings would be suitable for shelter in place during a PMF event, then the building stability thresholds for flood hazard should be used in the analysis.

7.2.7. General Flood Hazard Curves

When dealing with specific floodplain management or emergency management analysis there may be a clear need to use specific thresholds as described above. However, particularly in a preliminary assessment of risks or as part of a constraints analysis such as might be applied as part of a strategic floodplain management assessment, there is also an acknowledged need for a combined set of hazard vulnerability curves, which can be used as a general classification of flood hazard on a floodplain. A suggested set of curves based on the referenced thresholds presented above is provided in Figure 6.7.9.

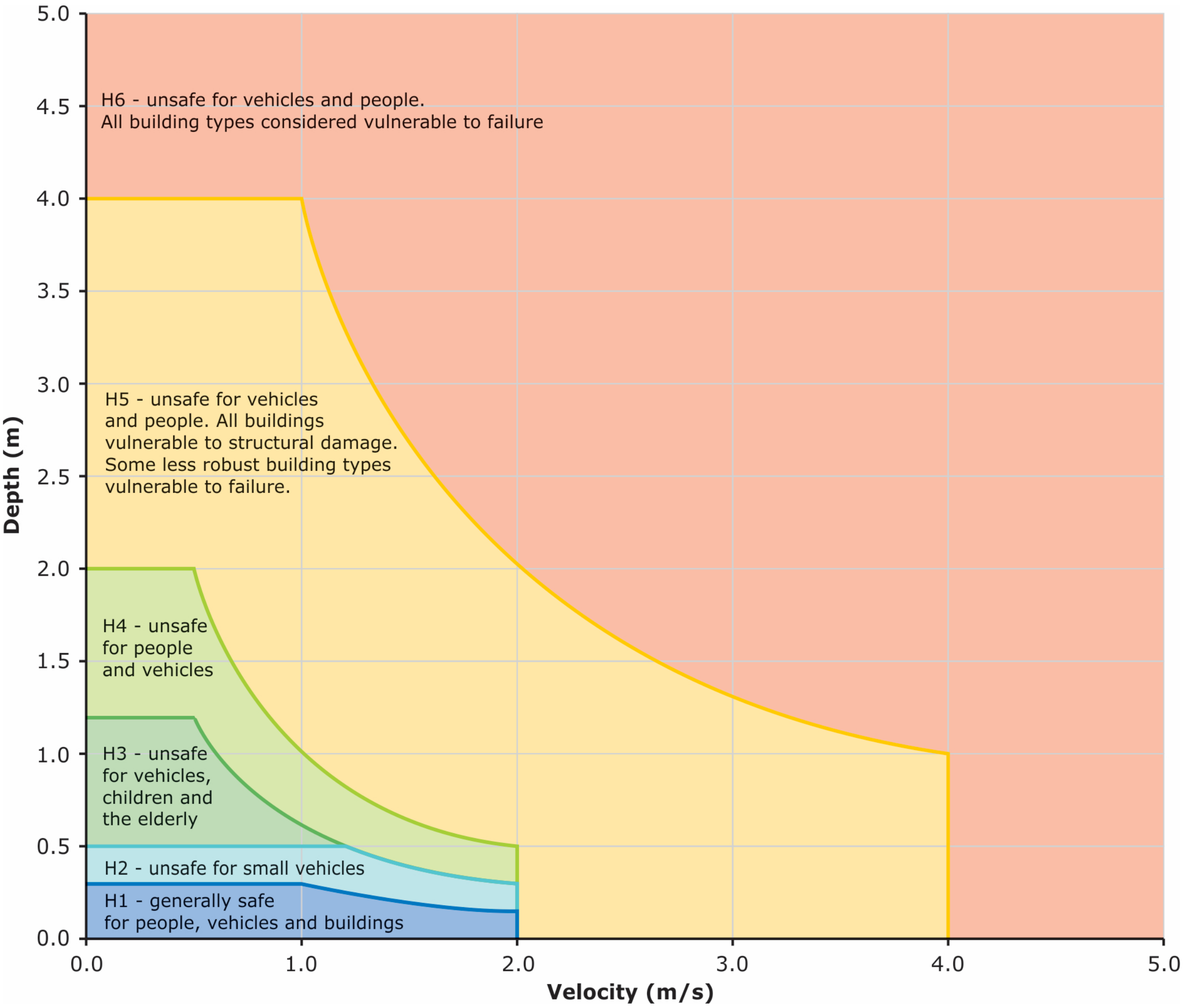


Figure 6.7.9. Combined Flood Hazard Curves (Smith et al., 2014)

The combined flood hazard curves presented in Figure 6.7.9 set hazard thresholds that relate to the vulnerability of the community when interacting with floodwaters. The combined curves are divided into hazard classifications that relate to specific vulnerability thresholds as described in Table 6.7.3. Table 6.7.4 provides the limits for the classifications in Table 6.7.3

Table 6.7.3. Combined Hazard Curves - Vulnerability Thresholds (Smith et al., 2014)

Hazard Vulnerability Classification	Description
H1	Generally safe for vehicles, people and buildings.
H2	Unsafe for small vehicles.
H3	Unsafe for vehicles. children and the elderly.
H4	Unsafe for vehicles and people.
H5	Unsafe for vehicles and people. All buildings vulnerable to structural damage. Some less robust buildings subject to failure.
H6	Unsafe for vehicles and people. All building types considered vulnerable to failure.

Table 6.7.4. Combined Hazard Curves - Vulnerability Thresholds Classification Limits (Smith et al., 2014)

Hazard Vulnerability Classification	Classification Limit (D and V in combination)	Limiting Still Water Depth (D)	Limiting Velocity (V)
H1	D*V ≤ 0.3	0.3	2.0
H2	D*V ≤ 0.6	0.5	2.0
H3	D*V ≤ 0.6	1.2	2.0
H4	D*V ≤ 1.0	2.0	2.0
H5	D*V ≤ 4.0	4.0	4.0
H6	D*V > 4.0	-	-

Importantly, the vulnerability thresholds identified in the flood hazard curves described above can be applied to the best description of flood behaviour available for a subject site. In this regard, the hazard curves can be applied equally to flood behaviour estimates from measured data, simpler 1D numerical modelling approaches, through to complex 2D model estimates with the level of accuracy and uncertainty of the flood hazard estimate linked to the method used to derive the flood behaviour estimate.

7.2.8. Isolation, Effective Warning Time, Rate of Rise and Time of Day

The effective warning time available to respond to a flood event, the rate of rise of floodwaters, the time of day a flood occurs, and isolation from safety by floodwaters and impassable terrain are all factors that may increase the potential for people to be exposed to hazardous flood situations. These factors are important considerations that influence the vulnerability of communities to flooding and are important considerations in managing flood risk.

7.2.8.1. Isolation

As outlined in AEM Handbook 7 (AEMI, 2014), flooding can isolate parts of the landscape and cut-off evacuation routes to flood-free land. This can result in dangerous situations, because people may see the need to cross floodwaters to access services, employment or family members. Many flood fatalities result from the interactions of people, often in vehicles, with floodwaters. Any situation that increases people's need to cross floodwaters increases the likelihood of an injury or fatality.

AEM Handbook 7 recommends that the floodplain be classified by precinct or community based on flood emergency response categories. This classification is separate to the quantification of hazard outlined in this guideline and is addressed in the complementary *Technical Flood Risk Management Guideline on Flood Emergency Response Classification of the Floodplain*.

7.2.8.2. Effective Warning Time

As outlined in of AEM Handbook 7, effective warning time is the time available for people to undertake appropriate actions, such as lifting or transporting belongings and evacuating.

Lack of effective warning time can increase the potential for the exposure of people to hazardous flood situations. In contrast, having plenty of effective warning time provides the opportunity to reduce the exposure of people and their property to hazardous flood situations.

7.2.8.3. Rate of Rise

Rate of rise of floodwaters is discussed in AEM Handbook 7. A rapid rate of rise can lead to people evacuating being overtaken or cut off by rising floodwaters. It is often associated with high velocities but it can be an issue if access routes are affected by flooding.

7.2.8.4. Time of Day

The time of day influences where people are and what they are doing. This can influence their ability to receive any flood warnings and respond to a flood threat. Inability to receive and respond to a warning can increase the potential for people to be exposed to hazardous flood situations.

7.3. Examples of Hazard Assessment

This section presents practical examples of the interpretation of flood hazard criteria in a floodplain management context. The examples are not intended to be a comprehensive analysis of all possible circumstances, but rather provide representative case studies, which illustrate practical interpretation of flood hazard criteria for floodplain planning and management.

7.3.1. Example - Warehouse Car Park

Flood behaviour quantification, including flood hazard analysis, is used to guide land use planning in floodplains (AEMI, 2014). Often, a general, first pass assessment of flood hazard is required to provide floodplain planners the opportunity to have a general overview of the magnitude of flood hazard and potential flood risks over a floodplain. This type of preliminary assessment of risks might also be used as part of a constraints analysis for a strategic floodplain management assessment.

Effective strategic land use planning is about responding to flood risks in a way that minimises future flood consequences. Consideration of flood hazard is therefore important so that development of land is encouraged in areas of low or no flood risk wherever possible. A clear understanding of flood risks early in the strategic land use planning process can help steer development away from areas that are not sustainable due to the likely impacts of the development on flood behaviour and guide land use zonings and development controls that support sustainable development on the floodplain in consideration of the flood risk

The following figures provide both a broad-scale example of the base data (variation in velocity and depth across a floodplain) and hazard mapping that can be developed as a first pass using standard two dimensional numerical model outputs from a flood study analysis. The flood hazard mapping presented in Figure 6.7.1 was developed by classifying the numerical flood model results shown in Figure 6.7.10 using the flood hazard thresholds listed in Table 6.7.4.

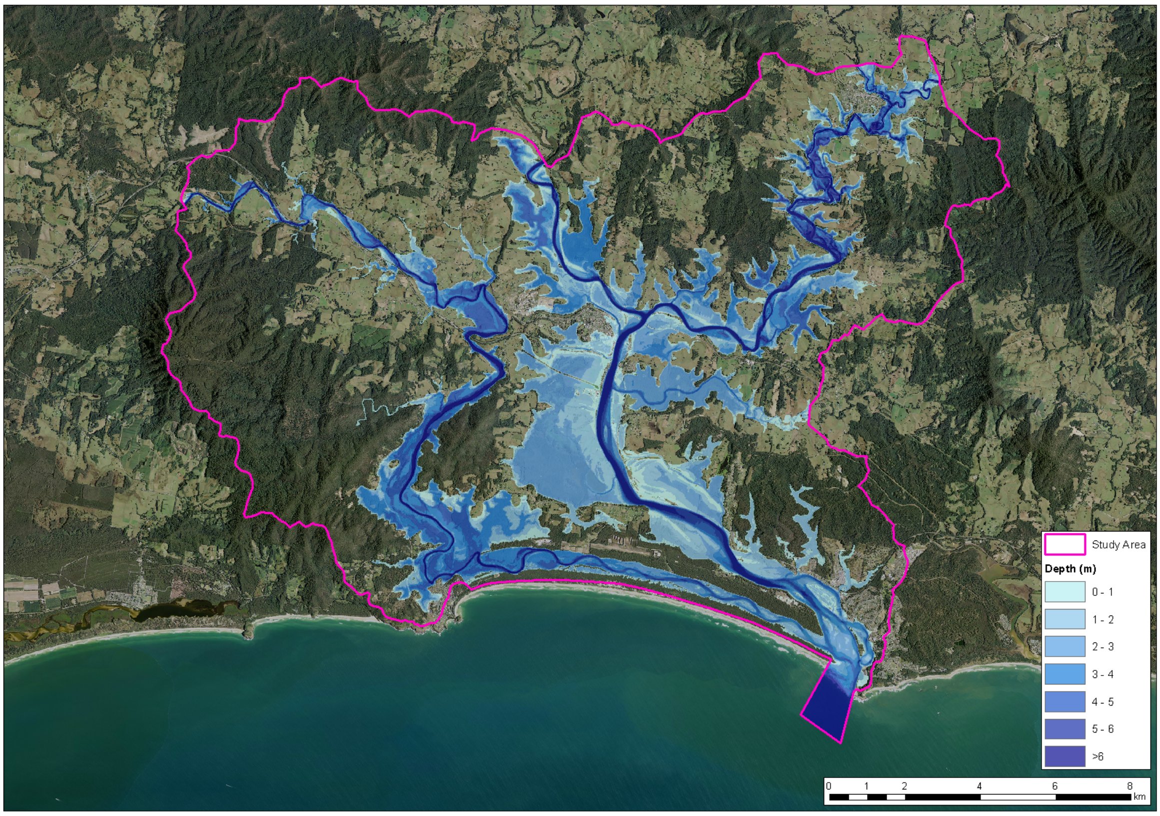


Figure 6.7.10. Flood Depth Map From Numerical Model Output (Courtesy WMAwater Pty Ltd)

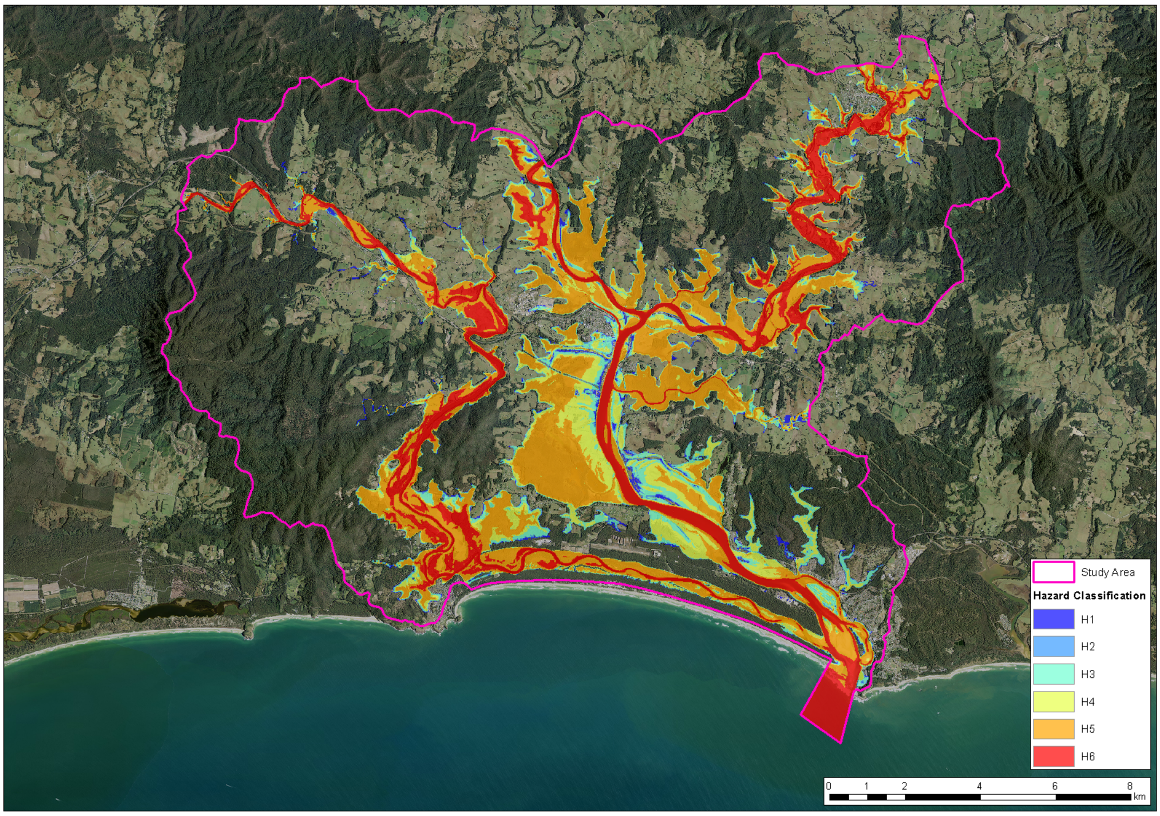


Figure 6.7.11. Flood Hazard Classification From Numerical Model Output (Courtesy WMAwater Pty Ltd)

7.3.2. Flood Mitigation - Warehouse Car Park

A large multinational retailer has identified an existing industrial area in an inner city suburb as a suitable site for re-development as a warehouse-style retail outlet. The site is gently sloping from the northwest to the southeast and, having been previously prepared for development, is clear of vegetation. An aging, concrete-lined channel runs along the eastern side of the site.

The retailer has submitted a development application, illustrated in Figure 6.7.12, "Schematic of Proposed Warehouse Development", which conceptually has a warehouse building situated in the northwest corner of the site with the area between the building and the concrete-lined channel earmarked as a car park.

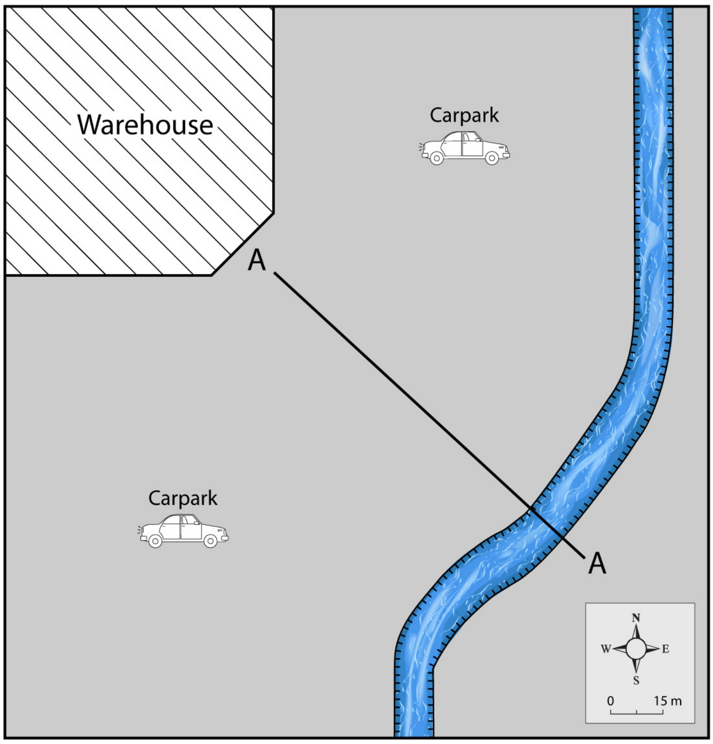


Figure 6.7.12. Schematic of Proposed Warehouse Development

The local council as the consent authority has identified a series of development constraints including flooding criteria. Amongst other criteria, the development must have:

- All retail floor space above the designated flood planning level defined as the 1% Annual Exceedance Probability (AEP) flood surface plus 500 mm freeboard;
- A flood free evacuation route for floods above the flood planning level; and
- All car park areas compliant with ARR flood hazard criteria for vehicle stability;

As there was no existing flood study, following consultation with Council, the developer engaged an experienced flood consultant to undertake flood modelling of the site to estimate 1% AEP flood behaviour. Flood modelling of the site for the 1% AEP event completed using industry best practice guidance as provided by ARR reference “Two Dimensional Modelling in Urban and Rural Floodplains” (Engineers Australia, 2012) predicted that while the warehouse building met the flood planning criteria, the car park was inundated by floodwaters. Figure 6.7.13 illustrates the extent of flood inundation for the existing site.

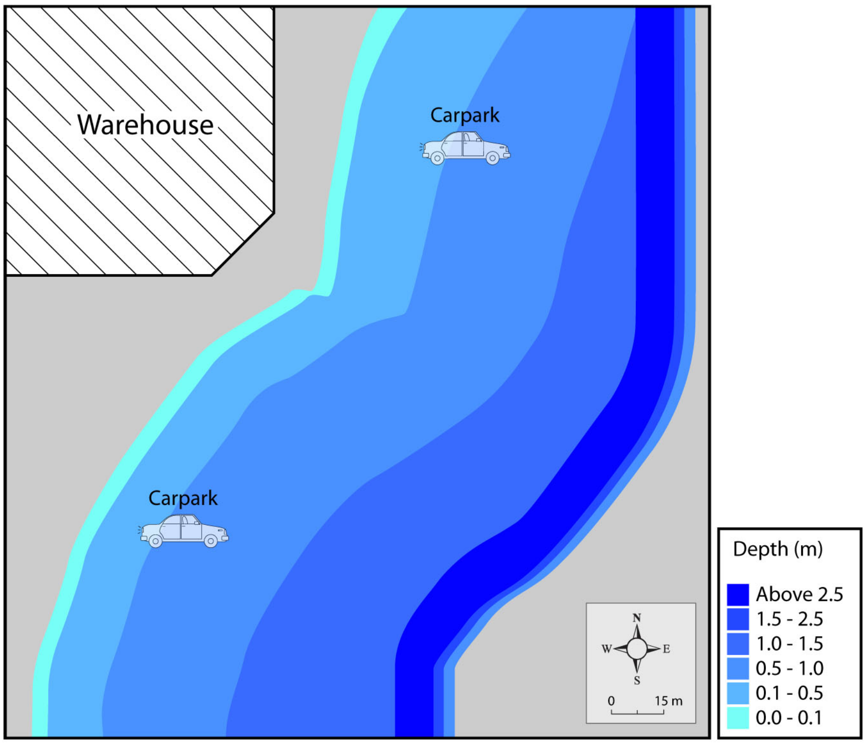


Figure 6.7.13. 1% AEP Flood Depth Map – Existing Site

Interrogation of the flood model results to determine provisional flood hazard as the product of flood depth (D) multiplied by flood velocity (V) showed that the peak flood hazard (D.V) corresponded with the maximum inundation of the site at the peak of the flood hydrograph. Provisional flood hazard for the 1% AEP flood on the existing site as illustrated by Figure 6.7.14 showed that flooding in most of the area identified for car parking exceeds the ARR stability criteria for small cars defined in Table 6.7.2 and illustrated in Figure 6.7.6. In Figure 6.7.14, areas coloured blue (D.V <0.3 m^2s^{-1}) indicate locations where small cars are likely to resist being moved by flood flows, whereas areas coloured green through red indicate areas where small cars are very likely to be pushed across the floodplain by floodwaters, with the flow having the potential to move larger cars closer to the creek channel. Based on this information, Council's preliminary advice to the developer was that the existing flood hazard conditions for the designated car parking area were incompatible with the nominated use.

Figure 6.7.14. 1% AEP Provisional Flood Hazard Map – Existing Car Park

As the concrete lined channel had low environmental value, Council was not opposed to the developer's proposed adjustment of the channel flow conveyance capacity to reduce the proportion of overbank flow at the site. The developer, in consultation with the flood consultant ran a range of cut and fill scenarios through the flood model aimed at expanding the channel capacity while raising the relative ground level of the car park to the flood peak. Figure 6.7.15 shows the adjustment of the channel and overbank area through the car park on the longitudinal section identified as 'A-A' in Figure 6.7.12.

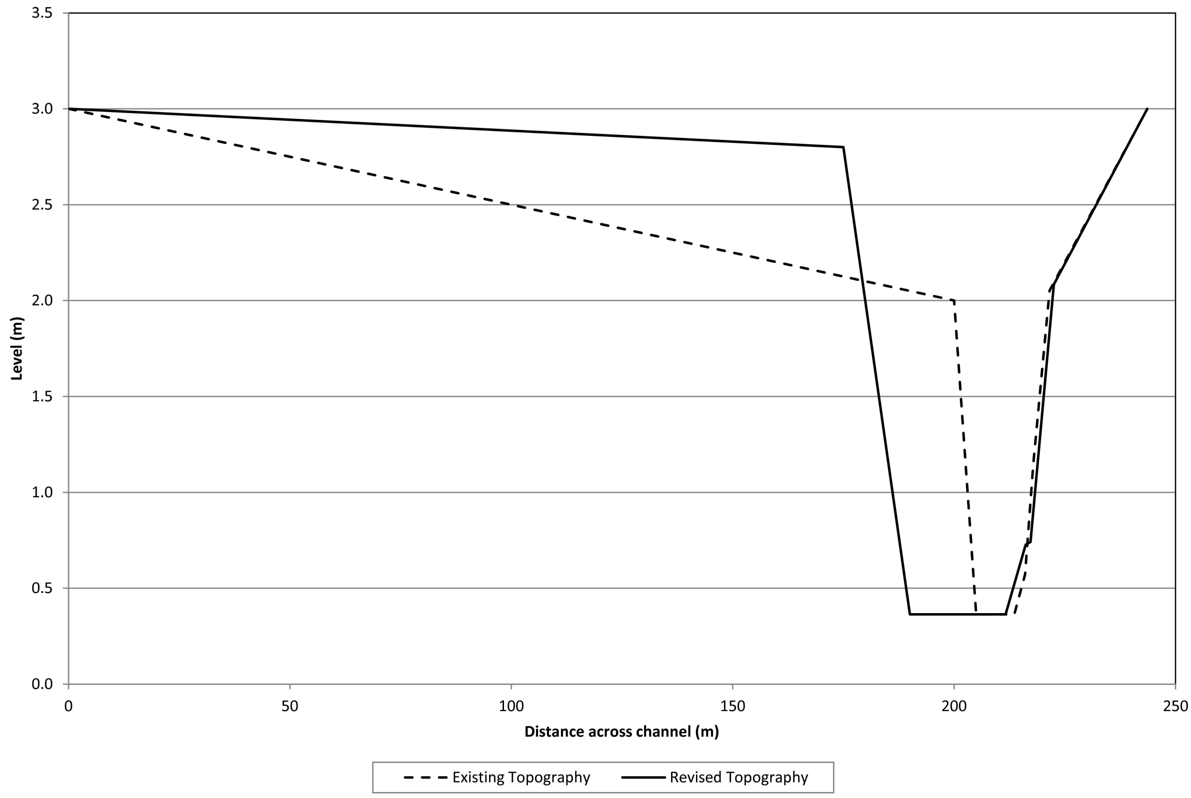


Figure 6.7.15. Comparison of Car Park Cross Sections (A-A)

This section is representative of a balance between the minimum volume of earthworks to meet the car parking capacity criteria for the development. The car park area adjacent to section A-A was graded to meet the natural surface areas outside of the development site.

Flood modelling for the revised site including the proposed earthworks is presented in Figure 6.7.16. The flood model results show that the site flood inundation area is significantly reduced with the works in place.

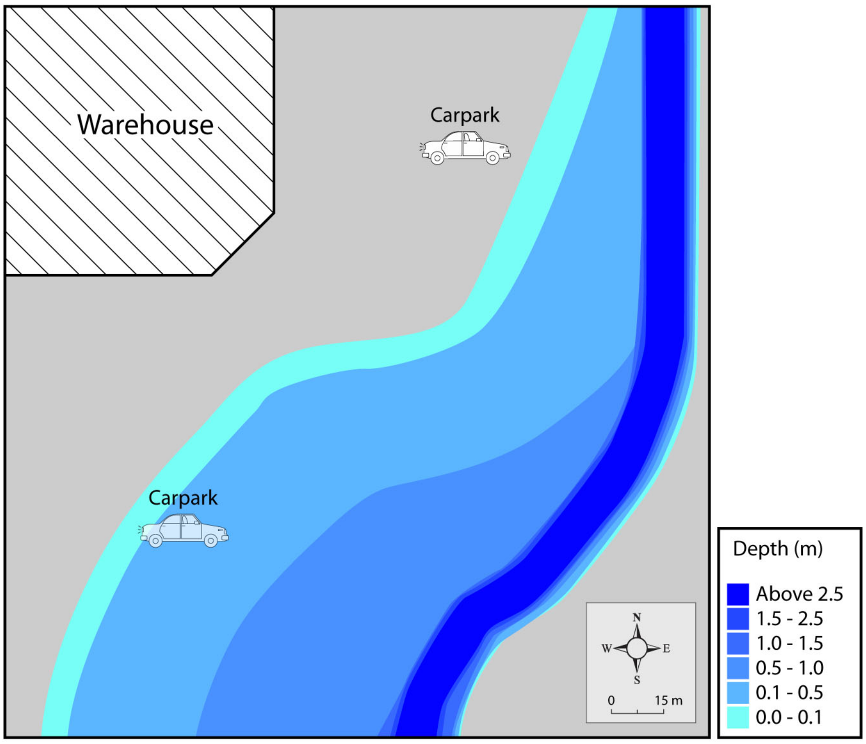


Figure 6.7.16. 1% AEP Flood Depth Map – Revised Car Park

Importantly, analysis of the provisional flood hazard as presented in Figure 6.7.17 shows that flood hazard for the area designated as car park in the conceptual site design now meets the ARR flood stability criteria for small vehicles. The nominated car park area has a D.V product of less than 0.3 m^2s^{-1} as indicated by the blue shaded area of Figure 6.7.17. This indicates that it is now unlikely that cars inundated by floodwaters in the 1% AEP flood will be pushed across the floodplain and potentially into the channel creating a possible downstream blockage hazard.

Council's planners also suggested to the developer that the yellow zone of Figure 6.7.17 could be landscaped as gardens providing clear separation of the car park from the channel. From a floodplain management perspective, this suggested change would provide a significant further reduction in exposure risk with little impact on available car parking space.

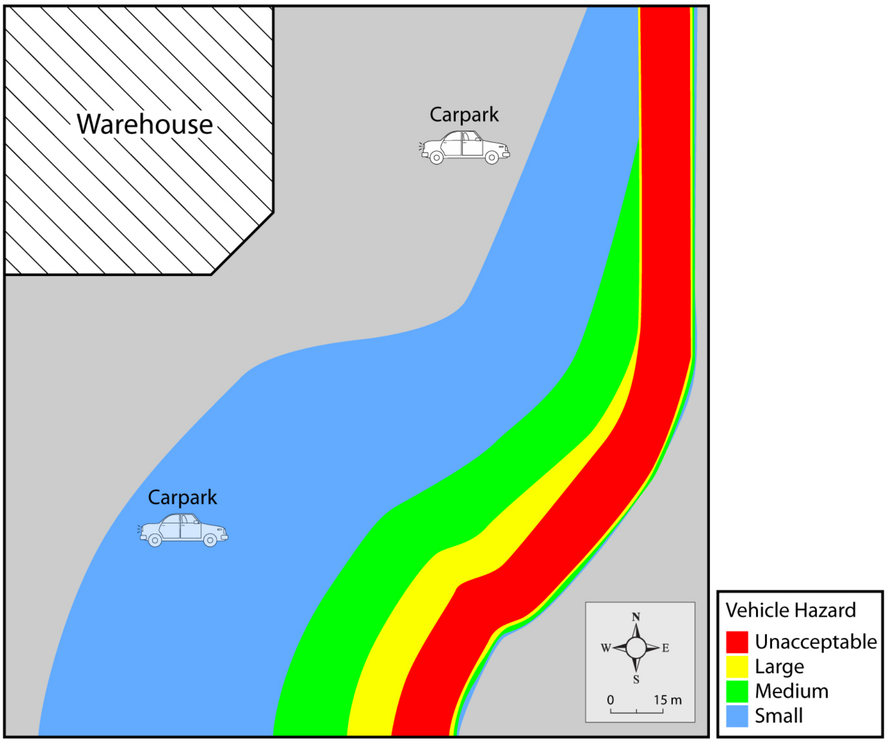


Figure 6.7.17. 1% AEP Provisional Flood Hazard Map – Revised Car Park

7.3.3. Example - Detention Basin

Council's conditions of consent for a proposed retirement village development require that on-site detention be provided so that peak flows from the site in floods up to the 1% AEP event remain similar to existing local runoff conditions.

The developer considers that a centralised detention basin on the site can be designed to have a dual use and integrated into the site grounds as a bowling green when not operating as a detention basin. Figure 6.7.18 illustrates the proposed design.

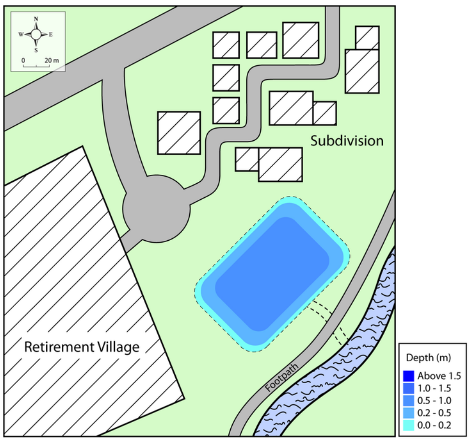


Figure 6.7.18. 1% AEP Flood Depth - Proposed Flood Detention Basin

As the catchment contributing runoff to the site is steep, Council's flood expert considers that there is some risk in a dual purpose design for the basin due to flash flooding in prevailing thunderstorms. Council's advice is that the developer engage a qualified flood expert to determine whether the basin meets the ARR hazard criteria for people safety.

The basin design philosophy is that the local catchment stormwater will collect both overland flows and also surcharge from the pit and pipe stormwater system. If the basin capacity is exceeded, flows spill at a designated location and flow overland to the adjacent creek channel. Flows that remain in the basin discharge through a grated pit in the lowest location in the basin floor.

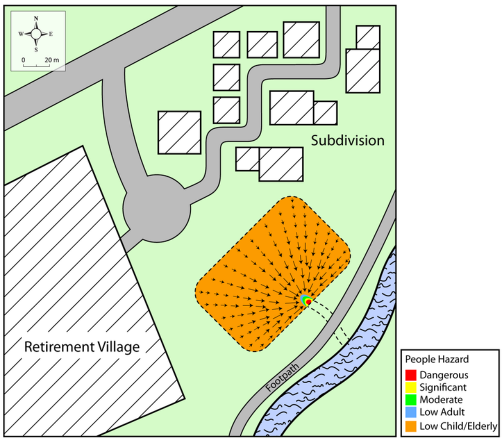


Figure 6.7.19. 1% AEP Provisional Flood Hazard Map - Proposed Flood Detention Basin

An assessment of the provisional flood hazard (flood depth multiplied by the flood velocity) is presented in Figure 6.7.19. When compared to flood hazard criteria presented in Table 6.7.1 and Figure 6.7.4, the provisional hazard meets the safety criteria for the elderly in most areas of the basin. This is because at full capacity, the basin is no greater than 0.5m deep. Analysis shows that a dangerous flood hazard is likely to occur near to the basin's outlet pit when the basin begins to drain at full capacity.

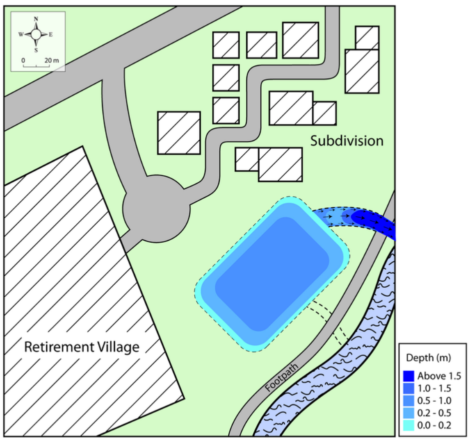


Figure 6.7.20. Basin Overflow Spillway – Flood Depth

Further analysis of the full design presented in Figure 6.7.20 shows that when the basin capacity is exceeded overflows will cross a public footpath adjacent the creek reserve. Provisional flood hazard analysis of the overflow path presented in Figure 6.7.21 shows that the flood hazard in the flow path will exceed the people stability criteria for adults and be a dangerous hazard to passing pedestrians.

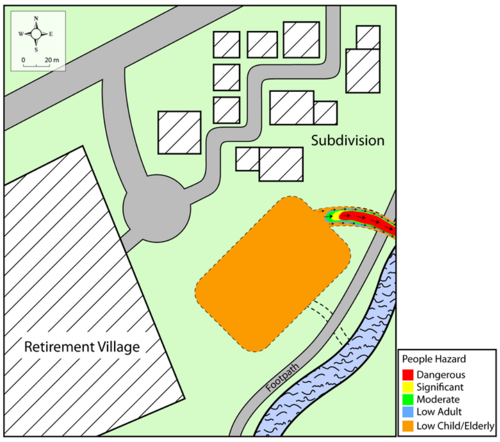


Figure 6.7.21. Provisional Flood Hazard Map – Basin Overflow Spillway

As a result of the analysis, Council consent criteria require signage to be placed in appropriate locations informing residents of the retirement village and the public passing the site on the public footpath of the dual purpose use of the basin and the danger of entering floodwaters when the basin is inundated. Further, Council's consent criteria require that the developer upgrade the footpath to include a bridge of suitable span over the basin spillway flow path so that safe thoroughfare of the footpath can be maintained during flood conditions.

7.4. Conclusions and Recommendations

Recent flood events on the east coast of Australia in 2010 and 2011 have highlighted that people continue to be exposed to dangerous and life threatening flow conditions in urban and rural floodplains. While floodplain management activities are continuing to mitigate problems associated the community's exposure to floodwaters, ongoing human activity in floods is largely unavoidable while significant areas of existing development and transport infrastructure in Australia remain flood prone.

This chapter of Australian Rainfall and Runoff presents a summary with an explanation of the limitations of recent analysis of flood stability thresholds for pedestrians and vehicles in floodwaters. Recommended flood hazard criteria for use within Australia based on the stability of people and vehicles in flood flows has also been presented.

Finally, the presented examples illustrate practical applications of these flood hazard criteria. While not exhaustive of all cases, the examples show how the criteria can be pragmatically applied to reduce the community's exposure to flood danger.

7.5. References

AEMI, Australian Emergency Management Institute. (2014). Technical flood risk management guideline: Flood Hazard. Australian Emergency Management Institute Barton, ACT.

Abt, S.R, Wittler, R.J, Taylor, A and Love, DJ. (1989). Human Stability in a High Flood Hazard Zone, Water Resources Bulletin, American Water Resources Association, 25 (4), pp 881-890.

Ashley, S.T. and Ashley, W.S. (2008), Flood Fatalities in the united States, Meteorology Program, Department of Geography, Northen Illinois University, DeKalb, Illinois, DOI: http://dx.doi.org/10.1175/2007JAMC1611.1.

Black, R.D. (1975), Flood proofing rural residences. Department of Agricultural Engineering, Cornell University.

Queensland Commission for Children, Young People, and Child Guardian (CCYPCG) (2012), https://www.fire.qld.gov.au/communitysafety/swiftwater/default.asp - accessed 11 August 2012

Cave, B., Cragg, L., Gray, J., Parker, D., Pygott, K. and Tapsell, S. (2009), Understanding of and response to severe flash flooding. Environment Agency, Bristol. Science Report No. SC070021.

Clausen, L. and Clark, P.B. (1990), The development of criteria for predicting dam break flood damages using modelling of historical dam failures. International conference on river flood hydraulics. John Wiley and Sons Ltd. Hydraulics Research Limited, Wallingford, England.

Coates, L. (1999), Flood Fatalities in Australia, 1788-1996, Australian Geographer, 30(3).

Coates, L. and Haynes, K. (2008), Flash flood shelter-in-place vs. evacuation research: Flash flood fatalities within Australia, 1950 - 2008. Report prepared for the New South Wales State Emergency Service.

Cox, R.J., Shand, T.D. and Blacka, M.J. (2010), Appropriate Safety Criteria for People in Floods, WRL Research Report 240. Report for Institution of Engineers Australia, Australian Rainfall and Runoff Guidelines: Project 10. 22 p.

Cox, R.J., Yee, M. and Ball, J.E. (2004), Safety of People in Flooded Streets and Floodways. 8th National Conference on Hydraulics in Water Engineering, Gold Coast. The Institution of Engineers, Australia.

Dale K.W., Edwards M.R., Middelmann M.H. and Zoppou C. (2004), Structural vulnerability and the Australianisation of Black's curves, in RISK 2004: Proceedings of a National Conference, Melbourne, 8-10 November, 2004.

Drobot, R. and Parker, D.J. (2007), Advances and Challenges in Flash Flood Warnings, Envrionmental Hazards, 7(3), 173-178, DOI:10.1016/j.envhaz.2007.09.001.

Engineers Australia (2012), Two Dimensional Modelling in Urban and Rural Floodplains. Project 15, Mark Babister, editor.

French J., Ing, R., Von Allmen, S. and Wood, R. (1983), Mortality from Flash Floods: a Review of National Weather Service Reports, 1969-81. Public Health Reports: Nov-Dec 1983, 98(6), 584-588.

Gruntfest, E. and Ripps, A. (2000), Flash floods: warning and mitigation efforts and prospects. In: Parker, D.J. (Ed.), Floods, vol. 1. Routledge, London, pp: 377-390.

HNFMSC (2006), Reducing Vulnerability of Buildings to Flood Damage, Hawkesbury-Nepean Floodplain Management Steering Committee, Parramatta.

Haynes, K. Coates, L., de Olivera, F.D., Gissing, A., Bird, D., van den Honert, R., Radford, D., D'Arcy, R. and Smith, C. (2016), An analysis of human fatalities from flood hazards in

Australia, 1900-2015. Proceedings 2016 Floodplain Management Association National Conference, Nowra, 2016.

Jonkman, S.N. and Kelman, I. (2005), An Analysis of the Causes and Circumstances of Flood Disaster Deaths. Disasters, 29: 75-97, doi: 10.1111/j.0361-3666.2005.00275.x.

Jonkman, S.N. and Penning-Rowsell, E. (2008), Human Instability in Flood Flows. Journal of the American Water Resources Association, 44(4), 1-11.

Kelman, I. and Spence, R. (2004), An overview of flood actions on buildings. Engineering Geology, 73(3), 97-309.

Leigh, R. (2008), Flash flood shelter-in-place vs. evacuation research: review of literature on building stability. Report prepared for NSW State Emergency Service.

Rogencamp, G. and Barton, J. (2012), The Lockyer Creek flood of January 2011: what happened and how should we manage hazard for rare floods, 52nd Annual Floodplain Management Association Conference. available at http://www.floodplainconference.com/papers2012.php., Vol.10.

SCARM (Standing Committee on Agriculture and Resource Management) (2000), Floodplain management in Australia: Best practice principles and guidelines, SCARM Report 73, CSIRO Publishing, Melbourne.

Shand, T.D., Cox, R.J., Blacka, M.J. and Smith, G.P. (2011), Appropriate Safety Criteria for Vehicles - Literature Review. ARR Report Number: P10/S2/020. Report for Institution of Engineers Australia, Australian Rainfall and Runoff Guidelines: Project 10. ISBN 978-0-85825-948-5.

Smith, G.P. and Wasko, C.D, (2012), Two Dimensional Simulations in Urban Areas: Representation of Buildings In 2d Numerical Flood Models. Australian Rainfall and Runoff Revision Project 15. Final Report, February 2012.

Smith, G.P., Davey, E.K. and Cox, R.J. (2014), Flood Hazard UNSW Australia Water Research Laboratory Technical Report 2014/07 30 September 2014.

BOOK 7

Application of Catchment Modelling Systems

Application of Catchment Modelling Systems

Table of Contents

1. Introduction 1
    1.1. Scope and Intent 1
    1.2. Application of guidelines 1
    1.3. Specific Terminology 1
    1.4. Relationship with other sections of Australian Rainfall and Runoff 2
2. Use of a Catchment Model 3
    2.1. Introduction 3
    2.2. Overview of Modelling Applications 4
        2.2.1. Conceptualisation of Modelling Approach 7
        2.2.2. Developing a Catchment Modelling System 7
        2.2.3. Testing Parameterisation, Calibration, and Validation of a Catchment Modelling System 7
        2.2.4. Application of Catchment Modelling System to a Specific Design Problem 7
        2.2.5. Interpretation of the Results and Understanding the Reliability and Uncertainty 7
3. Conceptualisation and Selection of a Catchment Modelling System 9
    3.1. Introduction 9
    3.2. Factors for Consideration 9
        3.2.1. Initial Scenario Identification 11
    3.3. Identify Key Processes, Inputs and Mechanisms 12
    3.4. Data Availability and Model Complexity 12
    3.5. Selecting Modelling Platform(s) 13
    3.6. References 14
4. Catchment Representation in Model 16
    4.1. Introduction 16
    4.2. Adapting an Existing Model 16
    4.3. Data 16
    4.4. Key Features of a Catchment 17
    4.5. Time step 17
    4.6. Boundaries between Models Elements and Platforms 18
    4.7. Hydrologic Modelling 18
        4.7.1. Spatial Resolution 18
        4.7.2. Process Representation 20
    4.8. Hydraulic Model Establishment 20
        4.8.1. Model Extents 21
    4.9. Boundary conditions 21
        4.9.1. Location of Boundary Conditions 22
        4.9.2. Type of Boundaries 22
    4.10. Run Times and Computational Resources 24
    4.11. Model Resolution 25
    4.12. Time Step 25
        4.12.1. Save Step 26
    4.13. References 26
5. Determination of Model Parameters 27
    5.1. Introduction 27
    5.2. Overview 28
        5.2.1. Physical basis of model 28
    5.3. Guiding Principles 28
    5.4. Parameter Determination for Catchment Modelling Systems 29

5.4.1. General Approach ... 29
5.4.2. Types of Calibration Data ... 30
5.4.3. Anecdotal Information ... 31
5.4.4. Range of Flood Data ... 31
5.4.5. Hydrologic and Hydraulic Models ... 31
5.4.6. Selection of Calibration Events ... 32
5.4.7. Calibration Processes ... 33
5.4.8. Objective Function for Calibration ... 34
5.5. Data Issues ... 35
5.5.1. Overview ... 35
5.5.2. Changes to Catchment Conditions ... 35
5.6. Acceptance of Calibration ... 36
5.6.1. Matching Timing and Magnitude ... 39
5.7. Ungauged Catchments ... 40
5.8. Adopted Parameter Set ... 41
6. Regional Relationship for Runoff-routing Models ... 43
6.1. Introduction ... 43
6.2. Regional Relationships ... 43
6.2.1. Regional relationships for RORB ... 44
6.2.2. Regional relationships for RORB using Area-Standardised Lag Parameter ... 50
6.2.3. Regional Relationships for WBNM ... 52
6.2.4. Relationships between RORB, WBNM, RAFTS and URBS Lag Parameters ... 53
6.3. Modelling Urban Catchments ... 53
6.4. References ... 54
7. Validation and Sensitivity Testing ... 58
7.1. Validation ... 58
7.2. Sensitivity Testing ... 59
8. Application to Design ... 61
8.1. Overview ... 61
8.2. Issues with Historical Calibration Floods ... 62
8.2.1. Introduction ... 62
8.2.2. Magnitude of the Calibration Events ... 62
8.2.3. Calibration Event Conditions ... 62
8.2.4. Applied Parameters ... 63
8.3. The ARR Data Hub ... 63
9. Uncertainty Determination ... 64
9.1. Introduction ... 64
9.2. Sensitivity Analysis ... 65
9.3. First Order Approximation Method ... 65
9.3.1. General Approach ... 66
9.3.2. Example: Flood Volume of an n-Day Flood Event with AEP 1 in T ... 66
9.4. Monte Carlo Simulation ... 69
9.4.1. General Approach ... 69
9.4.2. Example: Flood Volume of an n-Day Flood Event with AEP 1 in T ... 70
9.4.3. Example: Flood level downstream of the confluence between two rivers . 71
9.5. References ... 76
10. Documentation, Interpretation and Presentation of Results ... 78
10.1. Introduction ... 78
10.2. Audience Considerations ... 78
10.3. Documentation ... 79

10.3.1. General 79
10.3.2. Data Collation and Quality Checking 79
10.3.3. Model Development and Calibration 80
10.3.4. Modelling Results 80
10.4. Interpretation of Modelling Results 80
10.4.1. Model Representation vs Reality 80
10.4.2. Checking of Results 80
10.4.3. Accuracy of Results 82
10.5. Presentation of Results 82
10.5.1. General 82
10.5.2. Qualifications and Caveats 83
10.5.3. Use of Modelling Results in Decision Making 83
10.6. References 83

List of Figures

7.2.1. Conveyance Comparison – Different Manning's n Combinations 4

7.2.2. Steps in the Application of a Catchment Modelling System 6

7.3.1. Conceptual Relationship between Data Availability, Model Complexity and Predictive Performance (Grayson and Blöschl, 2000) 13

7.9.1. Representation of Relative Uncertainty of Outcome to Uncertainties Using a (a) Tornado Diagram and (b) Spider Plot 65

7.9.2. General Framework for the Analysis of Uncertainty using Monte Carlo Simulation. 70

7.9.3. Monte Carlo simulation of transposed flood volumes 71

7.9.4. Uncertainty in parameters of the log-Normal distribution (high and low bars represent 5% and 95% limits, the high and low boxes represent 25% and 75% limits, and the central bar shows the median). 73

7.9.5. Uncertainty in levels estimated from the regression equation. 75

7.9.6. Confidence limits on the flood level frequency curve determined using the general framework for the analysis of uncertainty using Monte Carlo simulation. 76

List of Tables

7.1.1. Terminology of Book 7 2

7.9.1. Errors Flood Volumes Estimated Using a Transposition Model for a Range of Assumptions. 67

7.9.2. Errors Flood Volumes Estimated Using a Runoff Coefficient Model for a Range of Assumptions 68

7.9.3. Errors Flood Volumes Estimated Using a Water Balance Model for a Range of Assumptions 68

7.9.4. 69

Chapter 1. Introduction

Mark Babister, Monique Retallick, Isabelle Testoni

Chapter Status	Final
Date last updated	14/5/2019

1.1. Scope and Intent

Catchment modelling has become the dominant flood estimation technique. This is because it:

- Allows different options to be simulated.
- Can be used with no data, limited data and in data rich situations
- Can be used to transfer estimations from one location to another
- Default parameters are available
- Well designed catchment modelling systems will reproduce flood behaviour over a range of floods
- A full hydrograph produced and can be used to assess storage
- Results can be easily visualised
- Is relatively easy to set up and reliable to do
- Can be reused by others for similar problems
- Helps to document the process of how the flood estimation was carried out

Scope is to provide practical guidance on the application of these models.

Because of the wide variety of flood estimation problems no modelling framework is suitable for all problems.

1.2. Application of guidelines

List types of problems the book is applicable to. Urban drainage design, detention basins, overland flooding, trunk drainage design, floodplain management, bridges and other infrastructure.

1.3. Specific Terminology

Generally the same terminology is used in this book as used elsewhere other than highlighting that while traditionally there has been rigid divide between hydrologic and hydraulic models the separation is largely artificial. Many components of a catchment model can be represented by either type of model. This book aims to guide the user in the application of a catchment model without predisposing which model is used to represent each of the processes.

An example of this divide is that routing does not only occur in hydrologic models and in this book when routing is referred to it includes routing in a hydraulic model.

A catchment modelling system refers a set of modelling processes or components that are used together to produce estimates of flood characteristics. These modelling processes can be available within a single modelling platform (such as a runoff-routing model) or can be the combination of a number of modelling platforms (where a runoff-routing model is used to generate inflows to a hydraulic model). Table 7.1.1 defines some key terminology for Book 7.

Table 7.1.1. Terminology of Book 7

Terminology	Description	Example
Modelling Process	Representation of conceptualised physical process in simulation models	Rainfall excess model, runoff-routing model
Modelling Platform	Software implementation of the modelling process (simulation model)	Software packages such as: RORB, RAFTS WBNM, MIKE SHE, TuFLOW, SOBEK, HEC-RAS, Spreadsheet software
Catchment Modelling System	Combines different modelling processes and may combine platforms	RORB to TuFLOW, or just RORB
Modelling Framework	Any statistical framework that is used to derive exceedance probability of flood characteristics from simulation model results	Ensemble or Monte Carlo framework

1.4. Relationship with other sections of Australian Rainfall and Runoff

This book draws together much of the advice and guidance in other books. Book 1 provides philosophy, Book 2 provides rainfall information, Book 3 provides alternative estimation techniques for comparison to results, Book 4, Book 5 and Book 6 provide theory and details on models discussed within this book. Book 8 deals with extreme flood and Book 9 deals with urban applications.

Chapter 2. Use of a Catchment Model

Isabelle Testoni, Mark Babister, Monique Retallick

Chapter Status	Final
Date last updated	14/5/2019

2.1. Introduction

A catchment model system is a very useful way of estimating how a system will perform under a number of different conditions. Catchment modelling systems are usually built from the series of modelling elements that are described in Book 4, Book 5 and Book 6. These are combined to replicate the key processes for a particular flood estimation problem.

The catchment modelling system can be used probabilistically (for estimating design flood behaviour) or can be used to estimate observed or historic flood behaviour. The catchment modelling system can be used to represent existing, historical or altered catchment conditions.

It is important when developing a catchment modelling system that the possible future uses of the model are properly identified so that the key processes are properly considered. The challenges in modelling are the need to represent various processes which introduces complexity, against the data available for calibrating these process and parameter and component interaction (Book 7, Chapter 3).

There are often subtle differences in how some of the key processes perform in frequent events than in rarer events. These differences mean that only rarer events can be used for the calibration which limits the data available for the calibration of complex models. Two simple examples are:

- during very frequent rainfall events the storage capacity of the soils is very important but during rarer intense events the rate of infiltration becomes more important ; and

- the hydraulics of a stream change significantly when flow moves from in-bank to the floodplain.

In both cases calibrating to just frequent events can give a very poor estimate of larger events.

In many modelling situations calibration exposes significant parameter interaction where very similar calibrations can be achieved with a range of parameters, while this often does not significantly change the behaviour of similar magnitude events it can make a significant difference to how larger events behave. When modelling components are combined into a catchment modelling system it is possible for this interaction to occur across modelling components. This problem is very common when only level data is available to calibrate a catchment modelling system that includes hydraulic and hydrologic components. A satisfactory fit can often be obtained for a range of flows and corresponding roughness values.

Catchment modelling system results can be sensitive to the chosen parameter values. Different combinations of parameters can give the same answer at a single point. However as is often the case, when extrapolating to larger events they give different answers and very different representations of the flow behaviour.

In many situations it is never completely clear what the correct combination of overbank and channel Manning's n is. The following example shows the results from a simple hydraulic model where the overbank and channel Manning's n were selected to match the 1% AEP flow and level. While different combinations give identical results at the adopted 1% AEP level and flow they give very different velocity distributions. The cases also give very different level vs flow relationships for different sized events. This is one of the key reasons why its important not to adopt models for problems outside the range they were designed for. Figure 7.2.1 depicts the difference in conveyance, K (Book 6, Chapter 2, Section 7) for a range of levels. At extreme flows the conveyance ranges from 11 500 to 16 000 m^3/s. At lower levels the flow can double.

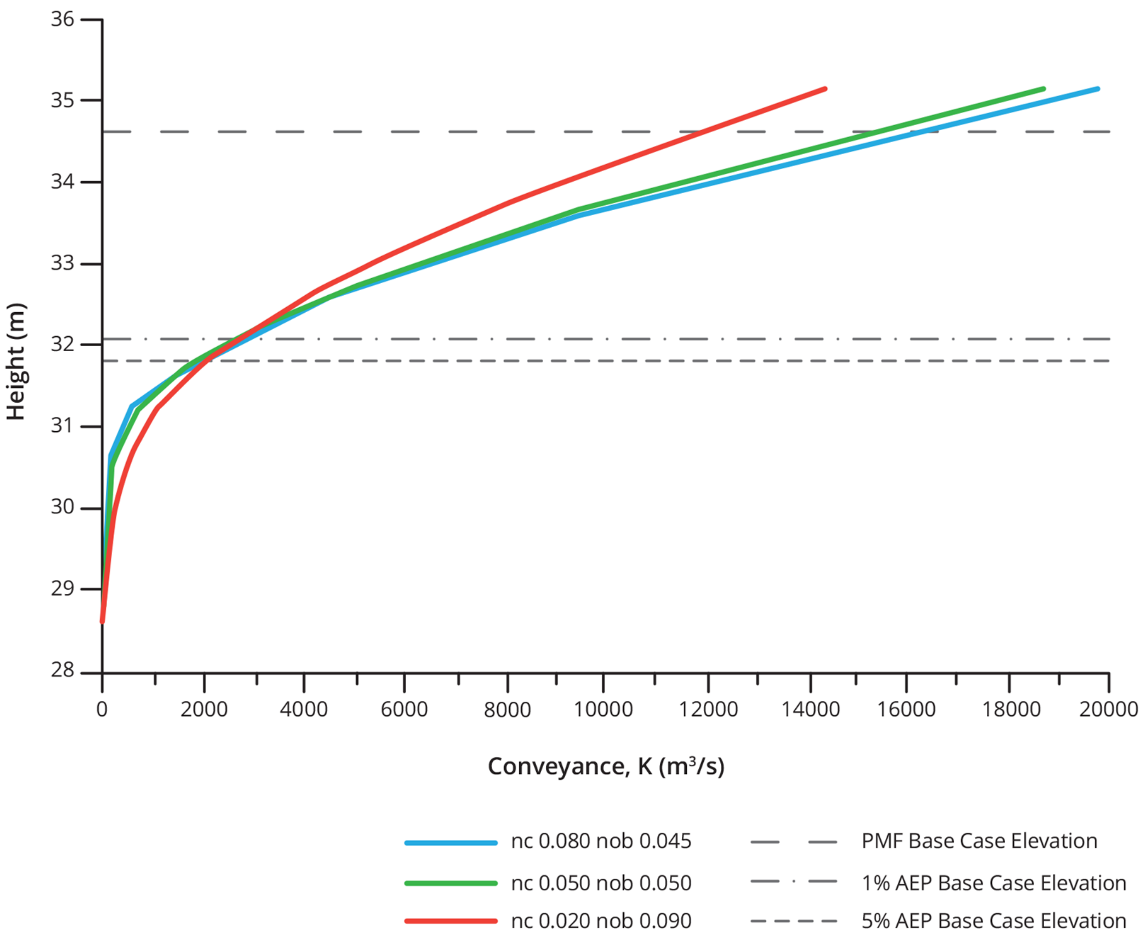


Figure 7.2.1. Conveyance Comparison – Different Manning's n Combinations

2.2. Overview of Modelling Applications

The application of a catchment modelling system should follow the process outlined within this chapter and book. While it is possible to create a catchment modelling system without following a rigorous process, it will lend itself to errors and wasted time in redoing work. A practitioner should first analyse the problem presented before deciding how to solve it. The data available must also be investigated, as it is likely that insufficient data exists for the ideal solution that the practitioner has already come up with.

A simplified overview of the steps involved in the application of a catchment modelling system (Figure 7.2.2):

- Conceptualisation of Modelling Approach (Book 7, Chapter 3);
- Developing a Catchment Modelling System (Book 7, Chapter 4);
- Testing Parameterisation, Calibration, and Validation of a Catchment Modelling System (Book 7, Chapter 5 and Book 7, Chapter 6);
- Application of Catchment Modelling System to a Specific Design Problem (Book 7, Chapter 7); and
- Interpretation of the Results and Understanding the Reliability and Uncertainty (Book 7, Chapter 9 and Book 7, Chapter 8).

These steps can be applied to an individual process, but it is important to apply them to the overall catchment modelling system. You need to confirm performance the overall CMS rather than just the individual components. Optimising individual components might not provide an overall robust CMS. The development of the CMS is constrained by the data that is available, the time/cost and experience of the practitioner.

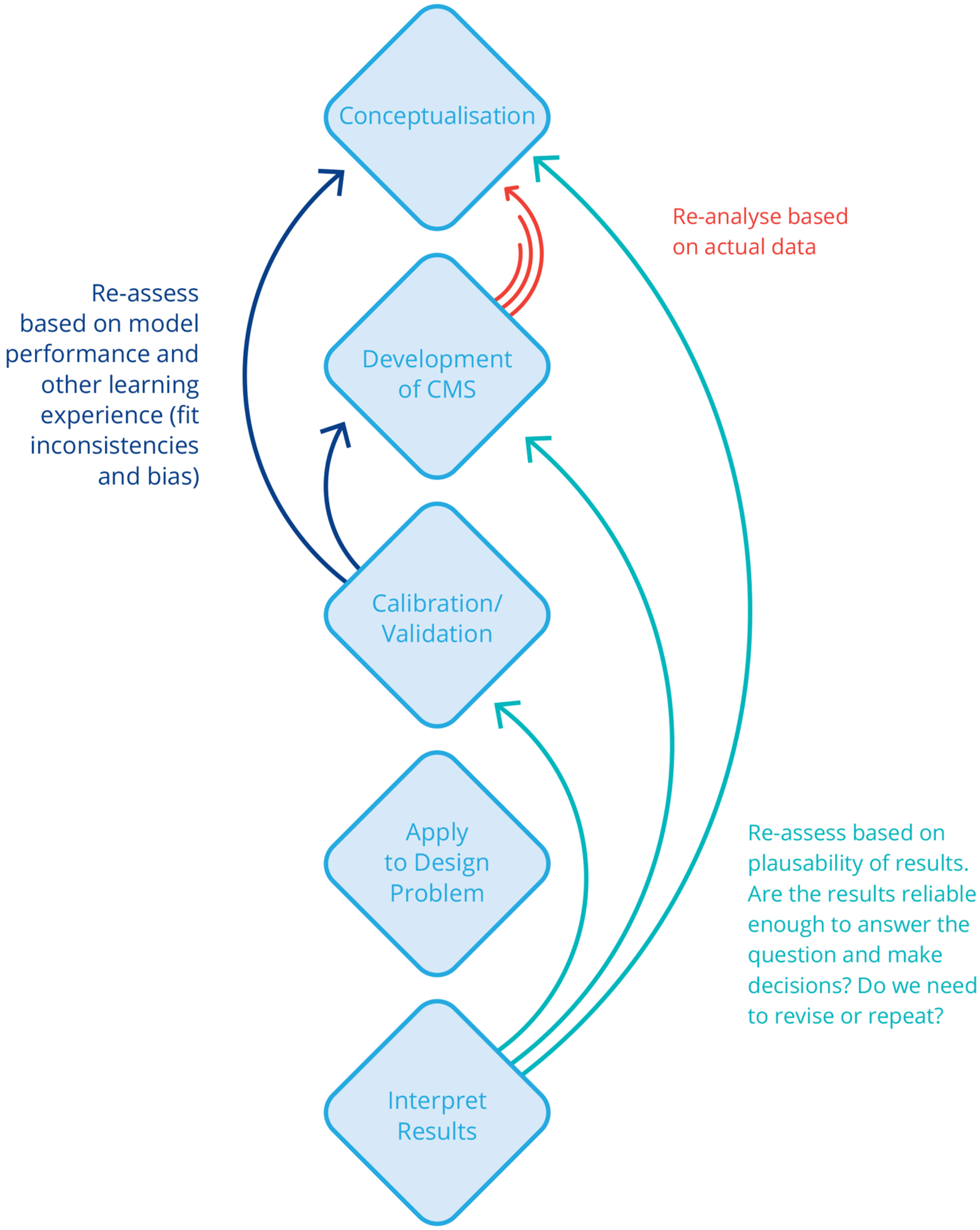


Figure 7.2.2. Steps in the Application of a Catchment Modelling System

Review of the conceptualisation of the catchment modelling system should be undertaken at each step in the process of creating and applying the catchment modelling system. This review does not have to be exhaustive. The reality is that most practitioners are undertaking this review as a sanity check already. It is highlighted here that this is a key step in the overall creation of a catchment modelling system or a component of it.

2.2.1. Conceptualisation of Modelling Approach

It is important at the start of a project to accurately define the problem and identify the key process/es that must be modelled in order to understand and accurately model the problem (Book 7, Chapter 3). In this stage limitations of the modelling approach must be explored. Available data, time, cost and model availability need to be defined for the problem in question. Preliminary selection of a catchment modelling system is carried out in this stage, though the selection may change as the practitioner develops the catchment modelling system.

2.2.2. Developing a Catchment Modelling System

The schematisation of a catchment in a modelling platform (Book 7, Chapter 4) depends heavily on the chosen modelling platform. Different modelling platforms have varied ways of representing the same catchment characteristics and features. In practice, the ease of representing key catchment features and key processes (discussed in Book 7, Chapter 3) plays a major role in developing a catchment modelling system. Decisions on conceptualisation and the representation of key features may be need to be revisited at this stage. Revising modelling platform choice is recommended if the initial selection is no longer appropriate in schematising the catchment.

2.2.3. Testing Parameterisation, Calibration, and Validation of a Catchment Modelling System

Ideally, all catchment models should be well calibrated and validated. However, data constraints mean this is not always possible or only limited calibration is possible. Book 7, Chapter 5 provides discussion on the best way to make use of the data that is available and discussion on data in general is in Book 1, Chapter 4. The calibration process of a model is not limited to matching historic records, but can include the overall estimation of parameters. The estimation of parameters aims to preserve the representation of the catchment characteristics as described by the conceptualisation of the catchment. Guidance on parameter values for ungauged catchments is provided in Book 7, Chapter 5. Validation techniques are used to independently test that the chosen parameters represent observed behaviour (Book 7, Chapter 6).

2.2.4. Application of Catchment Modelling System to a Specific Design Problem

Typically, at the start of setting up a catchment modelling system a specific design problem required a solution is already defined. Design applications of catchment modelling systems will vary depending on the specific problem under consideration. Book 7, Chapter 7 discusses different design applications after a catchment modelling system is established.

2.2.5. Interpretation of the Results and Understanding the Reliability and Uncertainty

The final step in the application of a catchment modelling systems is to provide information to decision makers, the community and designers regarding design flood behaviour. This information needs to be scrutinised and final checks should always be undertaken to ensure the modelled flow behaviour makes sense (Book 7, Chapter 8). Catchment modelling systems are only representations of the real world based on data and mathematical models.

Results can be inaccurate if any key processes or features are misrepresented in the catchment modelling system, which is not always easy to determine. This misrepresentation can be due to practitioner error, model error or incomplete and inaccurate data. The uncertainty surrounding design flood estimates should not be overlooked (discussed in Book 7, Chapter 9).

Chapter 3. Conceptualisation and Selection of a Catchment Modelling System

Monique Retallick, Isabelle Testoni, Mark Babister

Chapter Status	Final
Date last updated	14/5/2019

3.1. Introduction

A well thought out model conceptualisation and selection stage can result in significant project savings and the avoidance a lot of costly rework. While it is not possible to identify all potential issues, as learning experiences during a modelling project can identify issues and these can be addressed during this initial stage. This allows the limitations to be better understood and factored into decision making.

The conceptualisation stage in the catchment modelling process can be broken down into a series of steps that lead to informed decision making:

- Defining the problem under consideration and output needs;
- Identifying the key process/es that must be modelled to understand and model the problem;
- Identifying the available data;
- Selecting a level of modelling complexity that can be justified by the data available to calibrate or parameterise the modelling processes; and
- Selecting a modelling approach that matches these considerations with project constraints including, time, cost, model choices and modeller experience.

3.2. Factors for Consideration

The most important step in developing a catchment modelling system is to properly identify the problem under consideration, the purpose of the modelling and required outputs. Modelling is used to predict the behaviour of complex systems under different scenarios and conditions. Modelling will generally have a specific purpose. The purpose of modelling may include:

- Floodplain studies – inclusive of flood studies all the way through to mitigation impact assessment. This may include defining flood behaviour for land use planning.
- Flood Emergency Response – Model results can be used to enable emergency services to better prepare and respond to flood events by identifying potential flood hazard and planning evacuation routes. Model outputs can also enhance mapping outputs and improve flood intelligence for both responsible agencies and the community, leading to a reduction in flood impacts. Whilst not commonly used at present, it is possible that 2D models may be utilised more commonly for real-time flood warning in the future.

- Urban drainage studies – in such applications the hydraulic model may also perform the routing functionality typically carried out by a hydrologic model. The 2D model provides the “major” drainage layer and interfaces also with the “minor” drainage system (i.e. pits and pipes) dynamically;

- Dam Break assessments - Often a hyraulic model is used to route dam break hydrographs. 2D models are well-suited to this application as the flowpaths resulting from a dam break are often unexpected or different to typical flowpaths;

- Sizing of a spillway;

- Land filling for development;

- In any environment in order to assess the flood impact due to development;

- In-bank river flow modelling in 1D or 2D. This may be carried out in 2D in order to provide flow velocity that varies over the cross-section or in 1D in which velocity will be averaged over the cross-section. This approach is often used in ecosystem/habitat assessment;

- Wetland modelling - where routing paths are ill-defined and filling and draining processes are complex.

- Lake or estuary studies – often at the lower end of river systems the floodplain interacts with a lake or estuary and subsequently ocean or lake dynamics become important (tide, storm surge, or seiching).

- Water quality and sediment transport studies – these applications build on the two-dimensional hydrodynamics to provide information on water-dependent processes such as pollutant transport and river morphology.

Along with a specific purpose problems it is necessary to define the spatial extent and either the probability range of interest or parameter range. For example the spatial extents could be limited to just a dam, or a distance up and downstream. The following items should be defined at the start of the project:

- Spatial extent (note this might not be the same as the model extent);

- Probability extent (e.g. 5% AEP to 1% AEP);

- Parameter range;

- Types of outputs (flow, volume, level, rate of rise, warning time). These may be presented as either:

  - Peak;

  - Hydrograph;

  - Spatial Map; and/or

  - Animations.

The required outputs may be specified by the client, in the study brief.

While as part of the study a model of the entire catchment may be established typically a smaller specific location is the main focus of the study. If there are self-cancelling errors or bias in areas of the model not influencing the specific location of interest then the practitioner might not be concerned.

An important step in the conceptualisation of the problem is determining the likely scenarios that will need to be run (Book 7, Chapter 3, Section 2). For example if a future development scenario is to be run with urbanisation then a hydrologic model will be required which allows a change in pervious to impervious area.

3.2.1. Initial Scenario Identification

Along with defining the problem under investigation identifying, the types of scenarios that are likely to be assessed will significantly improve modelling decisions. For some problems the practitioner will only need to identify existing conditions. However for many problems the practitioner will be required to build a catchment modelling system that is capable of assessing different scenarios. Scenarios are broadly divided into these categories: modelling of historic and future conditions, mitigation options and management options. Most scenarios will fall into these broad categories. Some typical scenarios include:

- Existing conditions;
- Historic conditions;
- Change in landuse (impacts or restoration to pre-development conditions to manage the impact of urbanisation);
- Infrastructure (assessing and mitigation of the impact of a road and railway line);
- Structural flood mitigation measures (such as dam and levees);
- Future development scenarios;
- Change in dam operations;
- Changed catchment conditions assessment;
- Climate change;
- Parameter sensitivity tests; and
- Ocean interaction.

If the project requires only the definition of flood behaviour under existing conditions then this step can be ignored and focus is on the identification of key processes (Book 7, Chapter 3, Section 3). While in many situations, it will not be possible to identify all the scenarios at the conceptualisation stage that will need to be assessed, it is possible to identify the types of solutions, measures or works that are typically used to identify, mitigate or manage the problem. The ability to model scenarios is one of the powerful features of a catchment modelling system.

What Has Been Defined So Far

- The Problem;
- Likely Scenario (first pass);
- Spatial Extent (area of interest);
- Probability range or parameter range of interest

3.3. Identify Key Processes, Inputs and Mechanisms

The key processes and mechanisms in design flood estimation can include:

- Rainfall Models;
- Runoff generation;
- Overland flow;
- Hydrologic routing; and
- Hydraulic routing.

The key processes in flood estimation have been defined in Book 4, Book 5 and Book 6. The key design inputs have been defined in Book 2.

It is important to decide which key processes have the most influence on the scenarios of interest. For example, if the scenario of interest is land use changes then the key processes are runoff generation from different landuse types, catchment response from different land use types, resistance to flow for different landuse types. Therefore the chosen modelling platforms and catchment modelling system must be able to model these processes and allow for changes to parameters representing these processes.

3.4. Data Availability and Model Complexity

During the conceptualisation stage all data does not need to be collected. However an awareness of what data is or might be available will assist in the determination of which catchment modelling system should be used. Selecting the level of complexity of the model is a trade-off between data availability and predictive performance (Figure 7.3.1). Typically there is not enough observed data. Time and budget constraints are usually best addressed by reducing model complexity and the extent to which data is used.

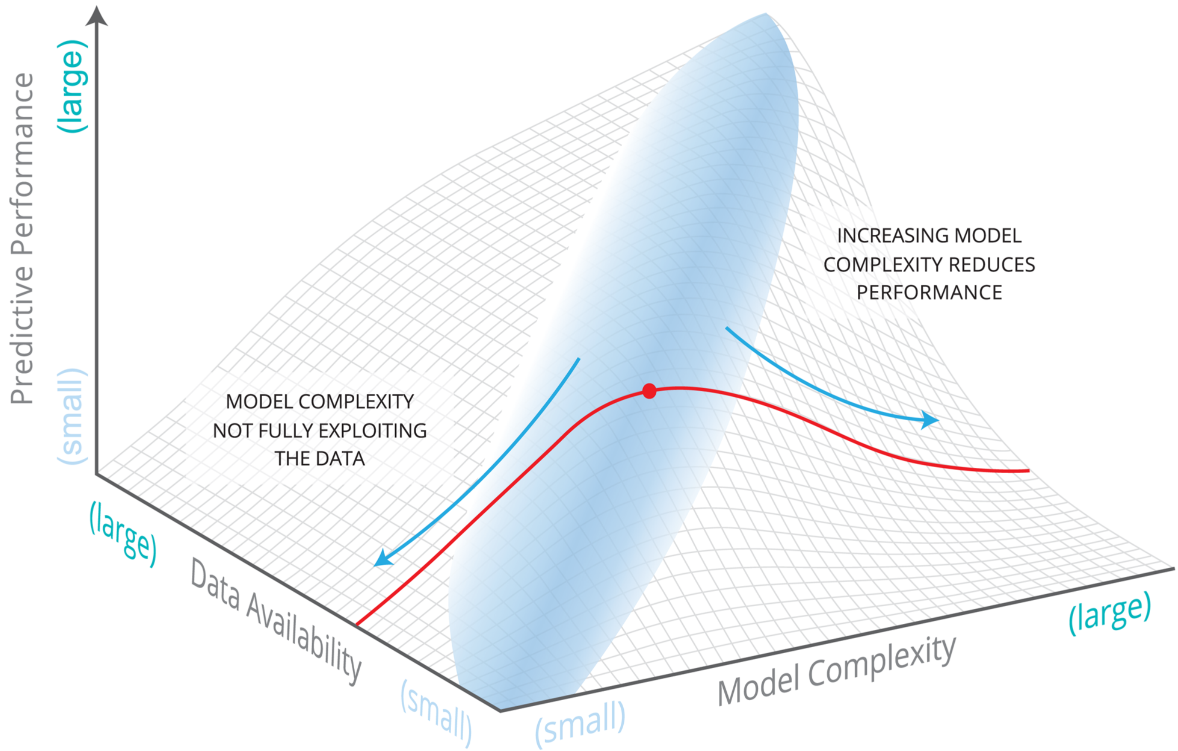


Figure 7.3.1. Conceptual Relationship between Data Availability, Model Complexity and Predictive Performance (Grayson and Blöschl, 2000)

Consideration must also be given to the resolution of modelling required. For example for a large catchment coarse representation may be sufficient. Therefore large subareas in the hydrologic model and a relatively large grid in the two dimensional hydraulic model may be used. For complex studies fine scale detail may be important and small grid and subareas may be needed in order to represent the hydraulic controls and key features.

What Has Been Defined So Far

An assessment of the available data and what can be achieve with it must be made. Following this some compromising on how key processes are represented must be made. It is a non-linear process.

3.5. Selecting Modelling Platform(s)

With a firm understanding of the problem, the key processes and data availability it is now possible to select a preferred modelling platform/s. A single modelling platform may contain all the key processes, inputs and mechanisms required to solve the design problem. However, in many cases is it more desirable to combine a number of modelling platforms. Reasons that influence the choice of modelling platform include:

- Reliable regional/default parameters for ungauged catchments (refer to Book 7, Chapter 5);
- Different modelling platforms are able to model specific features;

- Client preference;
- Standardisation;
- Likely run time;
- Anticipated resolution of the model and model outputs; and
- Ability to leverage existing modelling.

In many cases more than one modelling platform is often used. This is often the case where limited data is available as some modelling platforms are more suitable for ungauged catchments.

The other key inputs that must be considered at this stage are the project timeline, budget, experience with, and availability of modelling platforms. There is a certain art to modelling and there is no substitute for experience with a particular modelling platform. On many projects it is not practical to develop a job specific model and it is necessary to select one or a set of existing modelling platforms. This has major impacts on cost and timing. Likewise, selecting a modelling platform that the practitioner is familiar with can have significant impacts on cost timing and the reliability of results. Typically leading to a better outcome.

The advantages of selecting a platform that the practitioner is experienced with includes knowledge of appropriate parameter ranges, faster set up time, and knowledge of key features.

Selection of the Hydraulic Model

The selection of the appropriate type of hydraulic model is a critical decision in the application of catchment modelling systems process. In this step the physical system flow behaviour, which can commonly involve complex highly turbulent flows, must be reduced to an equation, or set of equations, describing the main characteristics of the flow. Here assumptions have to be made as to whether the flow can be considered as being one-dimensional (1D), two-dimensional (2D), or a combination of both, and whether the flow can be described as being steady (ie. constant with time), or unsteady (time-varying). In virtually all rural or urban floodplain modelling, vertical accelerations in the flow field are considered to be negligible and a hydrostatic pressure distribution is assumed, with computations and results based around a depth-averaged velocity. Further details are provided in Book 6, which outlines the governing equations utilised in hydraulic models. More detail on the application and selection of a hydraulic model is provided in Australian Rainfall and Runoff Supporting document – Two dimensional Modelling of Rural and Urban Floodplains (Babister and Barton, 2016).

What Has Been Defined So Far

A catchment modelling system has been chosen for the defined problem which makes the best use of available data. Consideration is given to model complexity and model representation of key processes.

3.6. References

Babister, M. and Barton, C. (eds) (2016). Australian Rainfall and Runoff Support Document: Two dimensional modelling in urban and rural floodplains. Project 15

Grayson, R.B. and Blöschl, G. (2000), Spatial Patterns in Catchment Hydrology: Observations and Modelling. Cambridge University Press, p: 404

Chapter 4. Catchment Representation in Model

Mark Babister, Monique Retallick, Erwin Weinmann, Isabelle Testoni

Chapter Status	Final
Date last updated	14/5/2019

4.1. Introduction

Once a catchment modelling system has been conceptualised and modelling platforms have been selected it is necessary to represent the catchment and floodplain in the modelling platforms. This requires a series of important decisions where all the key features and previously identified processes need to be represented. Model selection may need to be revised once data collection and analysis is undertaken. This chapter outlines all the key steps in establishing a catchment modelling system.

Schematisation of a catchment modelling system includes representing any physical properties of the catchment that affect the catchment's flood response. The selection of a catchment modelling system (Book 7, Chapter 3) will influence how the practitioner will schematise the catchment and its floodplain. Some catchment modelling systems are inherently easier to schematise certain key processes and features and therefore the ease of schematisation between different modelling platforms should be taken into account when selecting a modelling system.

The guidance within this chapter is divided into generic catchment modelling systems and that specific to hydrologic model and hydraulic models for ease of use.

4.2. Adapting an Existing Model

This is the most common mistake that practitioners and clients make. While it may be an easy choice from a time and budget perspective to choose to modify an existing model this often leads to a poorer representation of flood behaviour. Before making the choice to adopt an existing model consideration must be given to the original purpose of the model and how key processes relevant to the current design problem have been represented in the model.

Typical problems include:

- The original model was calibrated for a range of frequent floods and is not suitable for very frequent floods (or vice versa);
- the model might represent the processes for the existing case but cannot be adapted easily for new scenarios (changed catchment and floodplain conditions) that need to be run; and

The model is calibrated for rarer events and a different mechanism is dominant for smaller flows.

4.3. Data

The amount of historic data and terrain information available for the development of a catchment modelling system has a large impact on the model establishment. Book 1, Chapter 4 provides a detailed discussion of the types of data available and issues that the practitioner should look out for when using the data. Book 4, Chapter 2 discusses the balance between data availability and model complexity.

4.4. Key Features of a Catchment

In the conceptualisation and model selection stage the key features of a catchment should have been identified along with key processes. When developing a catchment modelling system the practitioner needs to ensure that they key features are properly represented in the model. A list of possible key features are:

- Landforms, vegetation and land use catchment areas influencing runoff response;
- Streams, stream network, floodplains and overflow paths;
- Natural and man-made flow constraints;
- Natural and man-made storages;
- Roads and railway lines;
- Weirs;
- Flow structures including levees, bridges and culverts;
- Levees;
- Flow diversions;
- Pits and Pipe network;

There are different ways of representing these key features within a modelling platform. Sometimes there are multiple options and it is important to select the method of representation that best suits the problem. Key features are identified so that most of the model effort is focused on them instead of other features that don't have a material effect on flow behaviour.

> There is a temptation to spend modelling effort on those features that can be readily measured. Yet it is often that the features that are hard to measure and quantify have a significant effect on flood behaviour. For example in a large river model small culverts will have little effect on flood behaviour.

4.5. Time step

Time step is typically more of an issue with hydraulic modelling, however, it is also an issue for some hydrologic processes. For example the time step at which a loss model is applied can change the amount of rainfall excess. Too coarse a time step will mean that the runoff hydrograph will be too coarsely represented. Continuous simulation models often need to be run at a finer time step to capture important details for simulating floods. Most hydrological processes (unless spatially distributed) the time step it is not a problem and computationally is no longer a challenge. For two and three dimensional hydraulic models this can still be a computation time issue.

4.6. Boundaries between Models Elements and Platforms

A key decision that has to be made is where the boundary is between modelling elements and platforms. The common example is between a hydrologic and hydraulic model. A model boundary usually means that there is no feedback between the modelling platforms. It is also important to understand at this point you could be using a very different modelling approach to represent the same process.

> A common problem is at the boundary between a hydrologic and hydraulic model. At the boundary care should be taken to ensure that there is no double routing of a flow hydrograph.

4.7. Hydrologic Modelling

As discussed in earlier chapters, the actual processes involved in converting rainfall inputs to a catchment into runoff and eventually a flood hydrograph are very complex, and they are represented in modelling platforms in a highly conceptualised form. The first decision to be made in representing a catchment is the level of spatial resolution to be adopted. At each level of resolution, from lumped models to fully distributed models further decisions then need to be made on how the key processes are to be represented in the model. In practice these decisions generally come down to selecting first an appropriate model platform and then possibly a particular model version.

4.7.1. Spatial Resolution

The appropriate degree of spatial resolution to be adopted in a hydrologic catchment model depends on the following factors:

- Catchment size
- Degree of spatial variation in catchment rainfall
- Variation in land use characteristics
- Presence of natural features or man-made structures that have a major influence on flood formation and need to be represented in the model
- Range of flood magnitudes to be simulated
- Requirement to estimate flood characteristics at internal points in the catchment

4.7.1.1. Lumped Models

In relatively small catchments there is often only limited spatial variation in rainfall and loss characteristics, and it is thus acceptable to treat the catchment as a homogeneous unit. In such situations, and when there is only interest on the flood hydrograph at the catchment outlet, a lumped model can give acceptable results.

While lumped flood hydrograph estimation models have the advantage of simplicity, they are limited in their application to the following situations:

- catchments with relatively uniform spatial rainfall, loss and baseflow characteristics or where the variation of these characteristics between events is relatively minor, so that the derived unit hydrograph or other model parameters are applicable to a range of design events

- catchments with no significant artificial storages (reservoirs or flood detention basins)
- applications that do not require extrapolation to the range of Very Frequent or Very Rare to Extreme floods.
- applications where a flood hydrograph is only required at the catchment outlet, as for the design of drainage structures on roads and railways

As lumped models do not represent the internal structure of the catchment explicitly and do not have direct links to physical catchment characteristics, they depend on the availability of observed flood hydrographs for their calibration. The scope for application to ungauged catchments is thus more limited.

4.7.1.2. Semi-distributed Models

Semi-distributed models allow the spatial variation of inputs and key processes to be modelled explicitly. This is particularly important in large catchments and in catchments where the natural flooding characteristics have been significantly modified by various forms of development, including the construction of reservoirs, flood mitigation works and transport and drainage infrastructure.

Most of the modelling platforms in common use in Australia belong to the group of semi-distributed models, owing to their flexibility and efficiency in representing the key factors that determine the flood formation under a broad range of catchment conditions. As explained in more detail in Book 5, Chapter 2 and Book 5, Chapter 6, the catchment is represented in the model through a network of nodes and links.

It is important that the development of the network structure used in the model is guided by a good understanding of the key catchment features described in Book 7, Chapter 4, Section 4. The catchment subdivision into model subareas should follow topographic features and match the degree of variation of the key influencing factors (spatial rainfall variability, soil and land use characteristics). The conceptualisation and level of detail in the representation of the flood producing and flood modifying processes (Book 7, Chapter 4, Section 7) should reflect their relative importance and their influence on the flood hydrograph outputs.

In large catchments the distributed runoff inputs experience a large degree of smoothing as they are combined and routed progressively through the stream or channel network to the hydrograph output location. This means that recorded flow hydrographs at the catchment outlet will provide only limited information on the flow contributions from different parts of the catchment and the influence of individual catchment features. However, the role played by different catchment features in the formation of flood hydrographs can be expected change for different flood magnitudes, and this needs to be reflected in the catchment representation.

Difficulties in calibrating a model to observed flood events of different magnitude should be taken as an indication of the changing role of processes, and the model representation thus needs to be adapted accordingly. In many cases a significant change occurs between floods that are mostly contained within the stream channel and floods in which floodplain storage plays an important role in the routing process. Large floodplain storage areas may need to be represented by special storage elements whose characteristics and parameters are determined from hydraulic calculations.

For more detailed guidance on the representation of catchments in node-link type models users should consult the user manuals of specific modelling platforms.

4.7.1.3. Distributed Models

In the fully distributed or grid-based flood hydrograph estimation models the catchment is represented by a large number of grid cells, based on topographic data from a digital elevation model (DEM), supplemented by more detailed survey information on the drainage network and the flow controlling features of the catchment. The two-dimensional hydraulic modelling approach adopted in these models allows their application in quite complex flow situations, e.g. floodplain areas with ill-defined drainage networks or urban areas with many flow obstructions.

In principle, distributed modelling allows the influence of spatial variability in rainfall inputs, runoff production and routing characteristics to be captured in more detail than in node-link type models. However, for this potential to be fully realised, the conceptualisation of the runoff producing processes has to be well matched to the scale of the basic model elements (grid cells) and has to reflect the change in processes as the cells are wetted up and the flow efficiency increases with flood magnitude.

The capabilities and limitations of distributed (rainfall-on-grid) models are further discussed in Book 5, Chapter 6, Section 5.

4.7.2. Process Representation

As explained in Book 4, Chapter 3, all models applied in flood hydrograph simulation employ a highly conceptualised representation of the actual hydrologic processes involved in runoff production and routing of the runoff inputs from the different parts of the catchment to form hydrographs at points of interest. It has to be kept in mind that the adopted conceptualisations are intended for the the simulation of probability-based design flood events rather than actual flood events. The model should reflect the typical flood response of the catchment to be expected in future events but may not reproduce the full range of variability between actual flood events.

In the event-based flood estimation methods, the influence of all the pre-event rainfall is only reflected in the initial catchment conditions (that determine initial loss), and in the delayed runoff contribution from baseflow which is modelled separately. The event rainfall is then divided into rainfall loss and rainfall excess which produces the surface runoff component that is modelled in detail.

The detailed guidelines for modelling losses and baseflow are provided in Book 5, Chapter 3 and Book 5, Chapter 4, respectively. The different approaches for modelling the production of runoff hydrographs from model subareas and for routing these through the drainage network to points of interest are discussed in Book 5, Chapter 6.

4.8. Hydraulic Model Establishment

The text below is largely reproduced from Babister and Barton (2016) which is focused on two dimensional modelling. Two dimensional models are used for the majority of problems because they provide spatial output showing the extent of flooding and flood characteristics. There is however a place for one dimensional models where a fast reliable computational model is required for flood forecasting or Monte Carlo modelling. More detail can be found in Babister and Barton (2016).

4.8.1. Model Extents

The primary goal in selecting a model extent is to represent key processes within the area of interest without significant influences driving runoff-routing or hydraulic behaviour from areas outside the model extent. Key considerations include, but not limited to:

- Ensuring that the model extent is sufficient to cover the likely inundation extent of the largest event to be modelled. The key here is that for the largest event to be modelled (typically the Probable Maximum Flood), the model extent does not artificially restrain water movement at its boundaries, and that the topographic data within the model extent also extends beyond the inundation areas. For a hydrologic model the model must cover the contributing area.

- Ensure that boundary conditions are located sufficiently far away so as to not unduly influence results within the area of interest.

- Minimise the inclusion of unnecessary (flood-free) areas, as this produces excessive results, impacts on computer memory requirements, increases model output file sizes and reduces efficiency.

If the likely maximum extent of the inundated area is difficult to define (e.g. very flat terrain or dam break studies) defining the extent can be an iterative procedure. A recommendation is to always start with a large model area and then narrow the model domain based on feedback of model results, as this is far less problematic than the reverse process. Using a coarser grid/ mesh resolution to reduce run times during these earlier stages of the modelling process can be an effective and efficient approach, especially for large model areas. Run time is typically not an issue for hydrologic models other than distributed runoff-routing models.

Model Study Area vs Model Applicability Area

In most cases there is a difference between the model extent and the area that the model can be used to produce reliable results. Just because a model extends over a certain area does not mean reliable results can be extracted in all areas of the model. Often these fringe areas are modelled just so that the model boundary conditions are sufficientlyfar away from the study area.

4.9. Boundary conditions

Boundary conditions should be located a sufficient distance away from the location of interest so as not to influence the results. The practitioner must decide which type of boundary condition to apply.

Beware of direct rainfall boundary conditions and pre-wetting of catchment.

When using direct rainfall water is applied to every grid cell so all calculation points are located on a boundary. A common problem is that the depression storage within the model which can be a combination of numerical and actual can be overlooked when applying losses.

4.9.1. Location of Boundary Conditions

The location of the boundary conditions is of critical importance. In general, boundaries should be located as far away from the study area as is practicably achievable. Any boundary condition on a hydraulic model requires a description of the water level, flow rate and velocity, flow direction and water surface slope across the boundary. In most situations, these flow conditions are rarely available as input data time series. Consequently, a range of assumptions are made in the definition of these conditions. While some models provide the ability to specify these explicitly, most models have generic assumptions incorporated into the model system to facilitate the automatic calculations of the range of parameters required.

As an example, the water level at a boundary condition is typically defined as a time series of recorded level. The other flow conditions are assumed or calculated based on the general assumptions or pre-defined conditions, such as an assumed flow direction across the boundary and assumed water surface slope. In this example, these assumptions, when combined with the water level time series, allow a discharge to be estimated across the boundary.

The specification of these conditions on the boundary introduces errors into the model predictions. Over time, these errors propagate through the model domain and may eventually pass through the model domain and out through another boundary. In a well developed and tested model, these errors become dampened as they propagate through the model domain. If the boundary conditions are located remotely then the errors become insignificant at the area of interest.

As an example, if a high flow rate is introduced through a topographic boundary condition that has small conveyance (restricted flow capacity) then high velocities and a significant velocity head results. This may cause large errors in the momentum flux into the system leading to errors in the flow patterns, water level and velocities downstream from the model boundary into the model domain. In this case, provided the boundary is located well away from the area of interest so that these effects have fully dissipated, the presence of these unrealistic flow patterns can be considered acceptable for the purposes of the investigation.

4.9.2. Type of Boundaries

The types of boundary conditions that are applied are important in determining the results produced by the model. The boundary conditions can be defined into two broad categories of;

- External boundary conditions; and
- Internal boundary conditions.

The most common boundary conditions applied in hydraulic models are external boundary conditions with a flow or discharge boundary defined along the upstream boundary of the model and a water level defined at the downstream external boundary.

The boundary condition type can be described using one of the following for specifications:

- Flow time series specified which is distributed across the model boundary grid/mesh points;
- Water level time series which is assumed to be constant across the model boundary;

- Flow and water level specified in combination as an input time series and distributed along the boundary;
- Flow or water level specified as a one dimensional line of values along the boundary for each time step;
- Transfer boundary where the water level, flow, velocity and water surface slope are provided from another model; and
- Rating curve along a model boundary (combination of water level and flow).

The combination of boundary types is important and must be considered in combination with the specification of initial conditions. In general, the boundary conditions for hydraulic models should be designed with upstream inflow or discharge boundaries and downstream water level boundaries. This ensures that any errors or uncertainties associated with initial conditions are “washed out” of the model. If other combinations of boundary conditions are used then the initial conditions will not necessarily be “washed out” of the model. The initial conditions will then significantly affect model simulation results and the results may not be reliable.

4.9.2.1. External Boundary Conditions

The schematisation of the external boundary conditions can vary across the range of model types and even within specific modelling platforms. The schematisation of external boundary conditions is therefore highly dependent on the specific case and modelling platform being used and it would be inefficient to describe all types of boundary conditions in detail. However we can define some general principles for schematising boundary conditions that are important to consider.

If general the practitioner should approach the schematisation of external model boundary conditions in a similar manner to how a boundary condition would be conceived for a physical model. The practitioner should consider the physical flow characteristics at the boundary in the real world and should attempt to schematise so as to minimise any artificial flow behaviour that is induced by the boundary condition. Issues that should be considered include:

- Align the model grid to be normal to the boundary flow streamlines if possible;
- Avoid rapid transitions in flow regime at the boundary;
- Avoid placing the boundary where turbulent flows are likely to be crossing the boundary;
- Minimise the wetting and drying on the boundary if the flooded boundary changes in width substantially during the simulation;
- Ensure that the boundary condition does not restrict or expand the flow substantially at the boundary; and
- Preference for specifying an upstream inflow discharge boundary and a downstream water level (or rating curve) boundary in combination.

As discussed, the boundary conditions should be located as far from the area of interest as possible. This will minimise the possibility of boundary effects and errors influencing the model results within the study area. The specification of the boundary conditions will therefore have a significant influence on the grid/mesh resolution. In general, the boundary condition should be identified as the first task that is carried out when conceptualising and schematising a model.

4.9.2.2. Internal Boundary Conditions

Internal boundary conditions are specified to control either the flow or the water level at grid/ mesh element(s) within the model and not along the edge of the model grid. There are generally two types of internal boundary conditions:

- Internal inflow points (sometimes called sources or sinks); and
- Internal flow or level controls.

The primary issue in defining internal inflow boundary points is to ensure that the flow rate is compatible with the grid or mesh resolution. There should be sufficient conveyance into or out of the element(s) where the boundary condition is specified to allow the model to accept the flow without introducing significant disturbance to the natural flow streamlines. If a large flow is forced as a boundary through a relatively small cell element with limited flow area; the model will produce an excessively large velocity and water level gradient to achieve continuity with the flow volume. If this occurs then significant momentum can be artificially introduced to the model at this location which will then influence water levels and flow patterns for a relatively large distance away from the boundary cell.

Internal control boundary conditions are a special form of boundary condition and are generally not recommended unless there is a strong compelling case for their use. An internal boundary condition will force the model to reproduce a predefined hydraulic behaviour within the model domain. The most common internal boundary condition is a forced rating curve at an internal cross-section of a one dimensional model. These boundary conditions are highly “reflective” and will introduce distortion and disturbance of the flow behaviour far from the actual boundary point. It is not recommended the use of this type of boundary for most catchment modelling applications.

4.10. Run Times and Computational Resources

The availability and type of computational resources will impact directly upon the efficiency and timeliness of any project. The efficiency of a gridded model study will be greatest where model run times are less than 24 hours. The shorter the run time the greater the efficiency. However, depending upon the extent and resolution of the model and the length of the modelled events there may be situations in which run times may be in the order of several days.

Excessively long run-times can introduce a significant bottle-neck in the study timeline and the decision to accept an excessively long model run time should be made carefully. Timeliness may be particularly affected during the calibration phase, where a large number of iterative simulations are necessary, mostly in series rather than parallel. With excessive run times the calibration essentially relies on the skill of the practitioner and their knowledge of likely calibration parameters.

In addition, the total number of runs required can be an important consideration if there are many scenarios to be considered, such as different event durations and Annual Exceedance Probailities, development scenarios, blockage scenarios or scenarios to parameter sensitivity tests (refer to Book 7, Chapter 7, Section 2). During the planning stage, the practitioner will need to consider the following factors to estimate the efficiency and timeliness of the study.

- The estimated length of time required to complete each run;
- The number of calibration and design events to be simulated;
- The number of computers/processors available;
- The ability of the computers to undertake multiple runs in parallel or not; and
- The number of licenses available if proprietary software is to be used.

The type and availability of computational resources can provide a real practical constraint. It may limit the number of design runs that can be achieved, the length of event that can be simulated, or the achievable resolution of the model. Such limitations and the resulting implications need to be identified as soon as possible in the process.

Consideration of run times can be particularly important for rural flood studies, or for studies involving continuous simulation of long flow periods. Such studies may require simulation of floods or flow sequences lasting several months. In these situations it may be appropriate to consider the use of a modelling package that can implement an adaptive timestep, using a longer timestep during periods of relatively steady flow conditions, which may significantly reduce computational run times. Adaptive timesteps are discussed further in Book 7, Chapter 4, Section 12.

The fast run times of hydrologic models lend themselves to Monte Carlo modelling. However, run times of two dimensional hydraulic models are somewhat prohibitive at this point in time. Fast run times are possible one dimensional hydraulic models. One alternative is to Monte Carlo or Ensemble hydrologic models then apply a selection of events to the two dimensional hydraulic model. Book 2, Chapter 5 recommends running an ensemble of ten temporal patterns in a hydrologic model and the selecting the pattern closest to the average (flow or volume depending on the problem of interest) through a hydraulic model.

4.11. Model Resolution

All models represent different processes at different resolutions:

- *One Dimensional Hydraulic Models* - the resolution is based on the space between cross-sections; and
- *Two Dimensional Hydraulic Models* - it is a simple representation of the topography.

Given other considerations such as run time it may not be possible to have the model resolution fine enough to represent the key features in the perfect detail. It is sometimes necessary to compromise on model resolution. For example, A levee is 30 m wide but chosen cell size is either 20 m or 40 m. Engineering judgement should be applied to decide which cell size should be used. An adjustment to the resolution of the model may be required in order to properly represent the flow behaviour.

4.12. Time Step

The model simulation time step is dependent on the model grid/mesh resolution and the schematisation of features in the model. As a consequence, the impact of poor model schematisation can lead to inefficiently small time steps which in turn will produce excessive run times. The impact of excessive run times should not be underestimated and in practice it becomes impractical to calibrate and apply the model effectively.

There are generally two choices for selecting a model time step which are:

- A fixed regular time step; or
- An adaptive time step.

The fixed regular time step allows the practitioner to pre-determine the model run time and to set the saving step (in which model results are saved) as a regular multiple of the simulation time step. However, the time step will need to be set at the shortest time interval necessary for stability of the model during the most energetic or deepest flows during the simulation. This typically occurs for only a very short period of time during the peak of the flood hydrograph. Consequently the model simulation time is longer than is necessary as it is fixed for the entire simulation. However, the practitioner can be sure that the simulation will complete within predetermined run time.

The adaptive time step allows the model to determine the appropriate time step necessary to maintain stability as defined by the Courant condition. The practitioner will typically set a maximum and minimum time step allowable. This allows the model to time step at relatively longer time steps when the flow is shallow or less energetic and shortens the time step during the peak of the flow event. In theory, this should allow the shortest run time for the simulation to be achieved whilst maintaining model stability. However, in practice the adaptive time step method can often lead to excessively long run times. This is due to the impact of a few minor locations in the model where short lived energetic fluctuations in the flow can lead to the minimum time step being selected for excessively long periods of time.

Run times can also become excessive if the period that it takes for the flood wave to propagate through the model is very long. For example, simulations of large river systems or of flat terrain where the critical rainfall duration is long, will have propagation times in the order of days, if not weeks. However, small catchments with short critical durations may only have propagation times in the order of hours. Therefore, some idea of the likely propagation period is needed before finalising the model resolution and extent.

4.12.1. Save Step

The model save step is an important issue to consider during the model schematisation process. As models (particularly hydraulic models and distributed models) will typically produce very large results files if all the results are saved, there is a requirement to select an appropriate saving step for the results.

The model saving step needs to be sufficient short to be able to define the shape of the hydrograph in time. The model save step also needs to be sufficiently short to enable the observation of stability issues that may occur during the simulation. If a model is being saved at a longer time interval than a higher frequency oscillation in the model then it would not be easily identified and could be missed. It is important that the model is checked thoroughly by saving all time steps at specific points or at small regions in the model domain. This allows for the observation and checking of stability issues without the need to save the entire model at all time steps. It is generally impractical to save all results at all time steps in a 2D model and it will typically exceed the limitations of most computer storage and hardware to do so.

4.13. References

Babister, M. and Barton, C. (eds) (2016). Australian Rainfall and Runoff Support Document: Two dimensional modelling in urban and rural floodplains. Project 15

Chapter 5. Determination of Model Parameters

William Weeks, Erwin Weinmann, Mark Babister

Chapter Status	Final
Date last updated	14/5/2019

5.1. Introduction

Following the selection of the catchment modelling system and the catchment and floodplain representation required, the next step is the estimation of appropriate model parameters to apply to the model platforms in the required application.

A flood model is a representation of the physical catchment processes affecting floods and the implementation is defined by a parameter set to apply the model to the specific problem being considered. The estimation of these parameters is often referred to as the calibration process.

While the term calibration strictly applies only where there is observed data to calibrate against, in this chapter calibration is defined here in general terms as the process for determining appropriate model parameters for the hydrologic and hydraulic models to ensure that they can be applied to the design flood estimation problem being considered. It involves varying model parameters to ensure that model results match observed data, to confirm that the model is performing adequately and is consistent with the records. Calibration can be carried out in a variety of ways and this chapter discusses appropriate methods of calibrating hydrologic and hydraulic models.

While the models used for these applications will generally have some representation of the physical characteristics of the catchment, meaning that the model parameters should be based on these physical features, there will always be uncertainty and the parameters will need to be estimated using available data to ensure that the model is at least consistent with the observed catchment performance. If the model represents physical processes closely, parameter values could be measured from catchment characteristics, but this is an uncommon situation.

The parameter estimation process may be based on recorded data (if there are suitable records in the project area) or may be based on regional estimates if the local catchment is ungauged or data is limited. There is a gradation between these two extremes however, it is rare that there is absolutely no available information to assist in setting parameters. It is also rare to find that there is sufficient data to allow a precise parameter determination, so the objective in determining parameters is to ensure that as much data as possible is used in this exercise.

Flood investigations usually require both hydrologic (calculation of design flood discharges) and hydraulic (calculation of flood levels, velocities and flow distributions as well as design of drainage systems) modelling applications, so this chapter covers both of these.

This chapter describes the different approaches to determining model parameters for the range of flood investigations and for the different amounts of available data.

5.2. Overview

The approach to parameter determination will depend on a number of factors which will determine the approach to the calibration and the level of detail sought in the process. This includes the model platform and the design problem.

5.2.1. Physical basis of model

Some models platforms may be purely ‘black-box’ or heavily conceptualised mathematical representations of the physical processes while others are more directly based on actual physical processes. Parameter estimation will be based on measurable catchment characteristics for model platforms where there is a direct physical basis for the parameters, and in these cases it is easier to establish model parameters. Most model platforms are likely to have at least some physical basis; it is thus possible to establish an acceptable range for model parameters, and model parameters calibrated to observed data should only be allowed to vary within this range.

During calibration, the parameters that are physically based should be defined using the catchment characteristics, while the other parameters can be varied so that the model results match the observed data, ensuring that the values remain within reasonable and acceptable limits.

The sensitivity of model parameters is also variable and some parameters have a greater influence on the model output than others. In some cases, there may be inadequate data to allow an accurate determination of actual parameter values. Therefore, these parameter values must be set using knowledge of model and catchment processes. There is a concern though that some parameters (e.g. Non-linearity parameters in runoff-routing models) may be important in design situations where rare floods are to be modelled but the observed data does not include any floods of the required magnitude. Therefore these parameters may appear insensitive during calibration but they have a major influence in the design situation.

5.3. Guiding Principles

Establishment of and applying models will vary depending on particular circumstances. However, for flood estimation applications, the following guiding principles will apply:

- All Available Data both formal and anecdotal should be considered in the calibration and the best use should be made of this data, in-line with its assessed accuracy and reliability. This data is the only way to ensure that the model application can be consistent with available local information. It is also important to carefully review the data to ensure that it is consistent and there are no obvious errors that will affect model performance.

- When calibrating the model parameters, it is important that the practitioner has an understanding of the role and relative importance of the different parameters and how they influence model operation. During calibration it is then important to concentrate on the most influential parameters, especially those that affect the model performance in the areas of particular concern for the specific model application.

- Model parameters when fitted to the data should be reasonable and within the range expected for the model platform and should be consistent with the physical features of the catchment being considered. If parameters are not within this typical range , the model conceptualisation could be incorrect and while the model may appear reasonable during calibration, there will be serious concerns for design events modelled where the event magnitude is run outside the range of that used for calibration. It is also possible that parameters outside the typical range may indicate errors in the observed data, and the calibration may be attempting to fit the model to these errors. The data quality and consistency then needs to be reconsidered and the calibration reanalysed accordingly.

Even if the data is of poor quality and incomplete, it is important that the model calibration be at least consistent with the available information, especially local or anecdotal information where formal data collection is lacking. Even very poor quality observations may be sufficient to apply a ‘common sense test’ and to ensure that even an essentially uncalibrated model can be a reasonable representation of local conditions.

5.4. Parameter Determination for Catchment Modelling Systems

5.4.1. General Approach

This section provides guidance on the parameter estimation for model platforms (both hydrologic and hydraulic models). Many principles are the same for these two calibration processes, but there are some differences in the data and approaches.

There are four basic approaches that will normally be dictated by the available calibration data and sometimes by the project budget and timeframe, and this classification is not as simple as the division into gauged or ungauged catchments.

The four primary categories are:

a. *No Data* - This is the lower limit to data availability, and having no data at all is probably not a common situation. In this situation, regional methods of some type are required. In addition to formal regional methods, parameters can be determined from experience with applications on similar systems or where the physical characteristics are similar.

b. *Very Limited Data* - The limited data may be some anecdotal records (Book 1, Chapter 4). An example for the design of drainage structures on a road or railway would be reports on the frequency of closure by flooding. In this case, it is possible to develop parameters that mean that the model is at least consistent with local observations. It may also be the case that the limited data is apparently inaccurate or inconsistent, though the exact source of this inaccuracy may be difficult to detect. In any case, efforts should be made to incorporate any information available in accordance with its assessed accuracy and reliability, no matter how limited this may be.

c. *Some Data* - In this case there may be a streamflow gauge with a very short period of record, records of flood levels for a single flood event or there may be records for a very frequent flood event. Some rainfall gauge information may be available. In this case, there will be a greater degree of confidence in the calibration, but the limited data means that there will still be uncertainty in the model performance, especially when the model is used for extrapolation to larger design events outside the range of the limited data or applied to alternative development scenarios.

d. *Extensive Data* - In this case, there is extensive data throughout the floodplain and catchment of interest. Data is available for a range of flood magnitudes and conditions and the flood data is accurate, reliable and consistent. In this case, the model calibration will be reliable and the model can be confidently used for design flood investigations.

These four categories blend together and there will be a gradation from one to the other. Projects where data is totally lacking are not common and projects with extensive data are also unusual. The objective is to consider all available data and to make the best use of all available information.

In the following sections the term *'calibration'* is applied to parameter estimation approaches c) and d), where the availability of flood data is sufficient to allow a calibration process that compares model results to observed flood data.

5.4.2. Types of Calibration Data

Types of calibration data are detailed in Book 1, Chapter 4 and include:

- Historical changes to topography, land-use, structures and drainage infrastructure;
- Records (photographs) of bed, bank and floodplain vegetation levels to assist with interpretation of roughness and provide record of prevailing conditions;
- Rainfall records (daily and pluviograph records), including in adjacent catchments;
- Gauged water level hydrographs, rating curves and derived flow hydrographs at streamflow gauge sites;
- Streamflow gauging at gauge sites and over the side of bridge structures (rare, but useful);
- Tidal level records if in a tidal area;
- Flood mark levels, location and measure of reliability. For example, debris marks, watermarks on/in buildings;
- Descriptive anecdotal information and past reports of flood behaviour in general;
- Observations of the rate of rise of flood waters and the time of peak;
- Photographs or videos of historical floods;
- Records or observations on water speeds and/or flow patterns;
- Records of blockage at hydraulic structures such as culverts and gully traps;
- Records and photography of the extent of inundation, noting the time of the photos; and

Information on road/railway closures.

> A flood occurred whilst calibrating a model. One of the local landowners phoned and asked if there was anything he could do? Make as many flood marks as you can, and if possible try to record when the marks were made. The local diligently went round hammering nails into trees until the flood peaked. After several weeks trying to calibrate to this fantastic data set, the practitioners were desperate, and visited the landowner. The model is always showing much higher levels than you've recorded. After a while the landowner took them over to the creek bank and showed them a levee hidden amongst the trees. Don't tell anyone he says, as I'm not sure if it's legal. In the end he agreed to have it surveyed, and lo and behold the model calibrated beautifully!

5.4.3. Anecdotal Information

While sourcing the observed flood data, it is important to also source descriptive or anecdotal information. This information can be just as valuable in the calibration process, especially where the observed data are scarce. Anecdotal information is best sourced through:

- *Discussions with Local Residents on their Recollections and Observations* - For example, they may have experienced a flood event and have noted features such as flow directions, water speeds and the timing of the flood's rise and fall. This information can be valuable to help check that the model's representation of flow behaviour is realistic; and
- *Information From Stakeholders* - For example, a road or railway authority may be able to advise how frequently a crossing is inundated and/or for how long. While this may not provide event specific observed data, it could be useful as to whether the model is in the right general area of performance.

> An old timer recalled how his grandfather remembered a large flood in the 1860s that broke across a ridge in two locations. Today, this would isolate the hospital and be a significant flood risk to homes. The 1% AEP flood did not show this flood behaviour, however, when the 0.2% AEP event was run, these floodways developed. This helped convince the old timer that the modelling was good, and the local council incorporated these floodways into their flood risk management planning.

> During a resident survey a local shop owner took the practitioner to look at a tree. "See that fork up there; well that was where a pig got stuck." Fortunately, the modelling for that event showed flooding to that height, and was proof to the local that the model was "doing the right thing".

5.4.4. Range of Flood Data

Calibration of flood models requires observed flood data (Book 1, Chapter 4) and generally data for several different flood events is needed. Application with data only from a single flood event means that there is less confidence in the calibration when extrapolating to design flood applications, especially considering that there may be errors in the data from the single event and this cannot be checked for consistency with others.

It is therefore desirable to have data for more than one flood event available for calibration, and hopefully several floods where a range of conditions is covered. Having floods of different magnitudes so that the flooding covers in-bank and floodplain flows and where flooding occurs in different seasons and with different rainfall distributions and catchment conditions will build confidence in the model performance. Successful calibration on a wide range of calibration events means that the model can be extrapolated to a wider range of design flood situations confidently.

5.4.5. Hydrologic and Hydraulic Models

The use of catchment modelling systems for design flood estimation generally involves two applications, namely the hydrologic and hydraulic components.

The hydrologic component, which is the model used to calculate flood peak discharges or flood hydrographs, is the more critical of the two, as any errors from hydrologic modelling will also transfer to the hydraulic modelling component. Calibration of the hydrologic model requires recorded flood flows, and these generally require a streamflow gauge. Availability of a streamflow gauge measuring discharges is less common than having flood level observations which may be provided by local residents and other non-experts. In some cases observed flood levels can be converted to flood flows by application of a stage-discharge relationship derived by a theoretical method (Book 1, Chapter 4), but this introduces another level of uncertainty into the calculation of discharge. The hydrologic model should be calibrated to ensure that the model can calculate flood flows to match the recordings. Calibration of hydrologic models must consider the accuracy of the recorded data and the consistency between different observations. These issues are discussed further below.

The hydraulic modelling process involves setting relevant parameters so that the modelled flood levels or flood hydrographs match the observed data. Observed flood levels are more commonly measured than flood discharges so there is often more extensive data. However, flood levels may be matched with a hydraulic model where the calculated discharges and hydraulic model parameters (primarily hydraulic roughness) are both incorrect and the errors compensate. While this is not necessarily a problem for the actual historical flood used for calibration, this can lead to significant errors when using the model for design applications over a larger range of flood events.

In many cases though, the hydrologic and hydraulic models may be calibrated together, ie. Joint calibration. In this situation, there may be observed flood levels but no recorded discharges, and the parameters for both the hydrologic and hydraulic models are adjusted together and the discharge determined such that the final flood levels are matched. As with the calibration of hydraulic models, this situation may lead to compensating errors in the two models, and the calibration may appear reasonable but the compensating errors mean that flood estimation for floods of different magnitude may be significantly in error. The compensating errors mean that the flood discharge is too low and the roughness is too high and the flood levels match, or the opposite.

5.4.6. Selection of Calibration Events

Prior to collecting and analysing all data for a calibration exercise, suitable historic events need to be identified and selected. The practitioner should primarily consider the:

- Amount, type and quality of suitable data available for each event; and
- Magnitudes of the events as to whether they are of a similar size to that of the primary design events.

Each calibration event must have sufficient historic flood observation and reliable topographic information and boundary data at the time of the flood. Often this means that events used for calibration are relatively recent, as the data sets are likely to be more complete. Larger floods that may have occurred longer ago may not be suitable for calibrating to due to the lack or scarcity of key data sets.

Calibration events should ideally also span the magnitude range of the intended design events with a preference for the more important design floods (e.g. 1% Annual Exceedance Probability event). This instils confidence in the ability of the model to replicate flow behaviour over the full range of event magnitudes. For example, a frequent flow event that is confined to the channel and drainage infrastructure will have a substantially different behaviour to a rare flood event that has broken the banks and is flowing overland. If the model has only been calibrated to the in-bank flow magnitude, confidence in its ability to replicate overland flow will be lower.

For tidal sections of a flood model, a tidal calibration is a useful additional calibration step, and is particularly recommended where storm tide inundation and interaction with catchment flooding is important. Tidal calibration data often exists, or can be readily measured, and is usually an accurate data set. It also provides a check that the model can reproduce any tidal amplification.

The 1998 flood in Katherine was larger than a 1% AEP event. There were extensive water level measurements taken throughout the town, many photographs and videos and the flood discharge was gauged at the gauging station. Therefore, the data available for calibration at Katherine for this event could be regarded as ideal: a large recent event with a reliable and extensive dataset.

5.4.7. Calibration Processes

The calibration process for flood models involves the adjustment of model parameters so that the model results match the recorded data. This process can proceed in one of several ways, though often a combination of different approaches is most effective.

When there is good quality data, there are automatic calibration algorithms that follow a defined search procedure to result in an “optimum” parameter set. While in theory, this procedure can result in a good quality parameter set with a minimum of effort, this approach is not as straightforward as first impressions indicate. The first step is to define an objective function that must be minimised for the optimisation. This may be minimising the root mean square error for the differences between observed and modelled flows. While this function may lead to a generally overall reasonable result, it may be more important to concentrate on high flows for example (a common requirement for flood studies), the rising limb of flood hydrographs (required for flood forecasting) or hydrograph volumes and shape (commonly needed for floodplains with extensive floodplain storage). These secondary details are often equally important and it is generally found that a purely automatic optimisation procedure does not converge to the optimum parameter set for a particular application, unless the objective function of the optimisation procedure has been carefully chosen.

Automatic parameter optimisation routines do not necessarily include an understanding of model processes and, if the objective function is not well selected, the optimisation may not represent the particular model application and produce realistic parameters. Manual parameter optimisation is the situation where the practitioner can vary model parameters based on the results of earlier model runs to progressively adjust model performance, and to incorporate an understanding of the model and catchment processes and the required model application.

Automatic optimisation procedures provide an approach for parameter estimation that in some situations can result in a good fit to the calibration data. However, in large and complex models there are usually many parameters, some of which only influence the model performance in particular circumstances. These automatic procedures may result in unrealistic parameter values and the performance outside the calibration range depends to a large extent on the objective function chosen for optimisation. Many objective functions will focus on the rarer floods while baseflow and frequent floods are poorly represented. These other details of the streamflow pattern are often important and it is difficult to find an objective function that can operate for all of the different conditions that may be needed.

Because of this, the most appropriate means of model parameter estimation should involve both automatic and manual parameter estimation where the modeller uses experience and understanding to estimate parameters appropriate for the particular application and the automatic procedures can refine and polish the optimisation.

Calibration, especially for large and complex models, may require a long process and tests on a large number of parameter combinations and variations. In this situation (which is common, except for the most simple situations) it is important that the practitioner maintains a log of calibration tests so that the impact of parameter changes can be understood and the calibration can proceed without retracing previous calibration tests.

5.4.8. Objective Function for Calibration

When a hydrologic or hydraulic model is being calibrated, the objective is to match the model results to observed data, but there are different ways of measuring the quality of the model fit.

A common application is to fit the model results to observed flood levels or flood hydrographs. Obviously, the objective is to fit the observations as closely as possible. However, the model will often show that it is over-estimating for some points and under-estimating for others or one flood may be consistently over-estimated while another is consistently under-estimated.

The aim therefore should be to provide the best “overall” match, though this is hard to define. Points to consider are that there should not be any consistent error, there should be some recorded points above the model results and some below and points of lower accuracy should not be weighted as heavily as those regarded of high accuracy. Estimates of rare design floods are most often required for flood studies, so the optimisation should normally be weighted towards the larger calibration events.

In most situations, flood peak levels are the most important objective, but in some cases, the hydrograph shape or flood volume may be of as much significance as the flood peak levels, so the model application must be considered when deciding on the objective function.

The objective function may be a mathematical parameter, such as minimising the sum of squares of the errors, or the function may be based more simply on fitting “by eye”, where judgement can be used to determine the quality of fit for different features of the observed flood record. There is a place for both of these approaches, even in a single application.

The calibration is assisted when the practitioner has a good understanding of the model processes and the influence of all parameters in the model. Knowledge of which parameters are most influential, and the influence of each parameter on different aspects of the flood process, is important in ensuring that the model parameters are maintained with realistic values and that efforts are not wasted working on insensitive parameters. Models with multiple parameters will usually exhibit interaction between the parameters so that it is possible that a similar calibration performance is achieved with different parameter sets. With incorrect parameter combinations, while the calibration performance may be similar, there are likely to be major differences in the design application results when the model is applied to conditions outside the range used for calibration. It is important therefore to have an understanding of the model operation and the relationship between parameters and physical characteristics to help keep parameters within reasonable bounds, especially when considering interactions between parameters.

Therefore a single objective function cannot be recommended for all model calibrations, a variety of methods will be applicable for particular applications.

5.5. Data Issues

5.5.1. Overview

When calibrating models for flood analysis, the first point is the assessment of the available data. Clearly maximising the quantity of data used for calibration will be a priority. However the data should be accurate and consistent or the calibration process may be impossible or it may lead to an incorrect model application and the catchment modelling system will be impossible to apply in practice. It is important to be aware that the flood estimation models need to be applied to practical problems, and the focus of model calibration is not just the preparation of a model that is well calibrated to the available flood data. Application of the model to the design requirements must be the primary focus, and the calibration must be prepared to the extent needed to have confidence in the design application.

While it is important to critically review the quality of the data available for calibration, it is also important to carefully review all available data and maximise the information available in this data to ensure the best possible calibration process. Formal data collection programmes are an immediately obvious source, but all available information should be examined. For example, old historic records from newspapers may be available to give an indication of major historic floods from before official records are available. These old records though do need careful study, since the survey datum may be hard to identify but some “detective” work can yield valuable information.

Careful review of the quality and properties of the data being applied for calibration is essential to ensure that it is appropriate and that the practitioner has a good understanding of the availability and applicability of the data. This is especially important for older historic data. Issues can include:

- The datum used for level survey where older data may use a different datum or two sets of survey data may be to two different datums;
- Streamflow gauge records of water levels are often measured to a local datum, which may be difficult to relate to topographic data;
- Stream channels may scour or silt up over time so current conditions may be different from those when the flood records were collected; and
- Floodplain roughness may vary with time, for example, sugar cane fields may be bare ground or very dense sugar cane depending on the time of year when the flood occurs.

Many types of informal data collection can assist in ensuring that model calibration is as accurate as possible, and these are discussed in Book 1, Chapter 4, where the value of data in all types of flood estimation is identified.

5.5.2. Changes to Catchment Conditions

The catchment condition data used in a model platform is typically that of the current day. This is due to the fact that an airborne and infrastructure survey is usually undertaken close to study commencement. In using this current day dataset, there are a number of potential calibration issues that the practitioner needs to consider.

As described in Book 1, Chapter 4, catchment conditions at each of the relevant historic calibration/verification periods must be established and used in the model. Changes to conditions that may affect flood behaviour include:

- dam construction;
- changes to initial dam storage levels and/or operations;
- dredging or siltation of river entrances;
- levee construction or raising;
- road/railway raising or duplication;
- new road/railway embankments;
- new culverts or bridges;
- upgraded drainage networks;
- development on the floodplain;
- different crop types or growth stage;
- changes in stream bed and bank profiles; and

changes to vegetation including seasonal variations.

> The last major river flood in one coastal area occurred in 1974 and resulted in extensive inundation of the floodplains. At this time, the floodplain was mostly utilised as grazing land. That land is now developed with extensive canal and flood mitigation works. While model calibrations for these rivers must rely on data from the 1974 flood, the drastically changed conditions mean that calibration results must be treated with appropriate caution.

> A 2D model was constantly producing flood levels that were too low in the upper tidal reaches of one branch of a coastal river. However, modelled flood levels matched recorded well in all other locations. Not even extremely high Manning's n values would lift modelled levels to those recorded. It was initially suspected that the recorded levels were erroneous, but this was proved incorrect when the recorded flood levels were independently resurveyed and found to be accurate. It was later revealed by a long term resident that a weir that had been installed to prevent saline water penetrating upstream, had never been completely removed and was still controlling flows. Once this partial weir was included in the model, a good fit was obtained with the same parameters used elsewhere in the model.

5.6. Acceptance of Calibration

When calibrating model parameters an important decision is to determine when the calibration is acceptable and when further refinement cannot be justified. There is often a temptation to continue to refine model parameters beyond what can be justified by the available data, which may be a lengthy process that does not lead to any improved performance in model application.

It is far more important to understand why a model may not be calibrating well at a particular location than to use unrealistic parameter values to ‘force’ the model to calibrate.

Considerations in the decision on when calibration can be accepted are:

- *Accuracy of Calibration Data* - The quality of calibration will depend on the assessed accuracy of the calibration data (refer to section on Data Issues above). For example, if the calibration of a hydraulic model is based on flood levels from observed debris marks, these levels may not be more accurate than ± 300 mm, so working towards matching a number of levels to a higher level of accuracy cannot be justified. Even where there is a streamflow gauge located on the catchment, the quality of the measured discharge will depend on the quality of the rating curve, which could cause quite significant inaccuracy in this measured data.

- *Representativeness of Calibration Data* - Calibration data may not be representative of the floods required for application of the model. For example, it is often the case that calibration floods are relatively frequent while design applications require much rarer floods. In this case, the value of refining the model calibration extensively to the frequent floods cannot be justified, since the significant extrapolation of the model means that the parameters may not be justified.

- *Number of Calibration Events* - The quality of calibration depends on the representativeness of the data and an important factor in this area is the number and range of events with suitable calibration data. In some cases, there may be only a single frequent flood event available for calibration and in this case, the quality of calibration will be poor especially where the model must be extrapolated to rare design events. When a model can be calibrated to several different flood events of a range of sizes and covering a range of different conditions (such as rainfall distribution or season), the resulting model can be applied with much more confidence than is possible where the data is limited.

- *Model Response and Catchment Consistency* - The calibration of models relies on the available data and the estimated parameters are based on the data used to estimate the parameters. However, the catchment conditions that applied during model calibration, especially if rare historic floods have occurred, may not be completely representative of conditions required for design applications. Because of this the model parameters required for design should be “generic” parameters based on the calibration but applicable for the design application. The exact catchment conditions for design applications may not be consistent with the particular conditions that applied for the calibration process. For example, vegetation coverage on a floodplain or the channel conditions in water courses will vary from time to time, so the conditions that applied for a single calibration flood event may not be representative of long term average conditions. Parameter values therefore must be modified to account for the expected future design conditions, rather than an unrepresentative calibration event.

- *Consistency of Data* - Review of data may indicate that the recorded data is inconsistent. For example, recorded flood levels for two different floods may be impossible to model with the same parameter set. There are several possible reasons for this possibility. For example, the recordings may be inaccurate, the catchment or floodplain may have changed between flood events or the model may be inappropriate for the analysis required. The effort should then be concentrated on resolving the source of the inconsistency rather than pursuing further calibration.

- *Requirements for Model* - The calibration acceptance may vary depending on the application required. For example, if the model is required for a bridge design, the calibration is only really critical for the bridge site, but model performance over a wider extent of the catchment is needed for floodplain planning. Also if the model is required for assessment of frequent floods, the performance for major overbank flooding is not as relevant so poor performance for these events is not a serious concern.

- *"Overfitting"* - This is the process where the model calibration process is taken to an extreme, and the model parameters are extended to possibly unrealistic values and can vary unrealistically throughout a catchment or floodplain to ensure that the model fit is close for all data points and all events. This situation may result when there are unrealistic calibration acceptance criteria adopted for the project and the only way of meeting the criteria is by an extreme and unrealistic parameter set. While the resulting model calibration may appear to be high quality and does meet calibration performance criteria, the resulting model parameters will not improve the performance of the model for extrapolation to the design situation.

> It is extremely rare that a flood model will fit all data well. This usually means one of the following:
>
> 1. The model has been overfitted to the data with unrealistic parameter values; and
>
> 2. Some of the data that does not fit well, has been ignored and not presented.
>
> It is extremely unlikely that your simple model is perfectly representing the complex real world well, all your data has been collected without error, or is unaffected by local factors.

For these reasons, it is difficult to define an acceptance criterion for model calibration and the quality of calibration may vary depending on particular conditions. It is important though to consider all the issues covered here when deciding on calibration performance. Unrealistic calibration criteria do not lead to an improvement in model design applications so the criteria need to be tailored for the particular application and local situation.

The quality of calibration depends on the quality of the data applied so the model application and results should consider this in interpretations of model results.

It is recommended that specifications for flood studies should not be prescriptive in defining calibration criteria, but should aim for realistic and applicable criteria.

It is important to note that a calibration process may not always result in a parameter set that is suitable for application to design conditions, and it is always necessary to approach calibration data critically. In these cases, the calibration process must be supplemented with other information such as regional parameter estimates as discussed in Book 7, Chapter 6.

Sensitivity testing of inputs and parameter values is a good way of understanding and resolving the importance of the input/parameter on the model’s calibration results. This is discussed further in Book 7, Chapter 7.

Following a large flood event that occurred in 1984, Council organised the survey of over 400 peak flood marks across the floodplains of the affected catchment. These were primarily flood debris marks. Prior to model calibration, Council specified that the calibration criteria was for modelled peak water levels to be within 300mm of recorded. However, calibration was accepted with 50% of points meeting this criterion in recognition of significant proven uncertainties in debris mark levels and some of the model inputs.

When calibrating a model to peak flood levels for one historic event, a good match between modelled and observed was obtained for all levels with the exception of the one recorded by the most upstream automatic gauge. The datum of the offending gauge was checked and no problem was found. In order to match this gauge, Manning's n values needed to be set at values that were outside the normal range and very different to elsewhere in the model. In addition, the peak level at this gauge looked out of place on a longitudinal plot of the river profile. Despite a strong desire to have the model calibrate well to this one gauge level, the client accepted the practitioner's advice that confidence in the accuracy of the observed level was low and it would be compromising the model to fit the data. Not long after the study was complete, a larger flood occurred and the model fitted all gauge data very well, including the troublesome gauge. It was concluded that something had gone wrong with the automatic gauge in the earlier event.

5.6.1. Matching Timing and Magnitude

Ideally, a model is calibrated to observed water level marks and hydrographs. Observed marks are usually at the flood peak and often spread throughout the model domain. Calibrating to these marks shows that the model is capable of reproducing the peak water level distribution. However, especially if the model only covers a small extent of the overall river/creek system, this does not necessarily mean that the model is well calibrated.

Also, fundamental to a good calibration is the demonstration that the model reproduces the timing of flood events. This may be achieved through calibrating to recorded water level hydrographs (if available), and to observations by locals (e.g. “the flood peaked around midday”). Water level hydrographs give the added benefit of showing whether a model is reproducing the shape (rise and fall) of the flood.

Calibrating to information on the timing of the flood shows that the flood dynamics are being reproduced, and this only occurs if the model’s input data and schematisation are satisfactory, parameter values are within typical ranges, the software is suited to the application, and most importantly, the hydrologic method is also reproducing the correct timing. The latter is particularly important when it comes to calibrating a hydraulic model. If the hydrologic method is inaccurate with respect to timing and/or magnitude, satisfactory calibration of the hydraulic model will be difficult, if not impossible. For this reason, jointly calibrating the hydrologic and hydraulic modelling is always recommended.

If parameters such as hydraulic roughness are outside standard values, the calibration may be “acceptable” for that particular event, but will very likely be compensating for inaccuracies in the hydrologic modelling, input data and model schematisation. In this case, the “calibrated” model is not suited to representing floods of smaller or larger size than the calibration event, and will be of limited use.

It is important to note that should flow/discharge hydrographs exist for a study area, the flows are not “recorded” but “derived”. A rating curve is used to convert the water levels recorded by the stream gauge into flows. Details on this process and its limitations are provided in Book 1, Chapter 4. However, it is worth reiterating that the reliability of discharge data is limited by the number and quality of manual gaugings undertaken at the site, the extent of extrapolation beyond the highest gauging of the rating curve and the means by which the rating curve is developed by the hydrographer. In undertaking a calibration using flow discharge hydrographs, it is essential to consider the quality and reliability of the rating curve used to derive the flows. Inaccurate rating curves produce inaccurate flows that will potentially mislead the practitioner into using inappropriate parameter values.

5.7. Ungauged Catchments

The model calibration processes discussed in this chapter apply when there is data (of varying levels of completeness and accuracy) to assist in the calibration. However in many cases, if not most, calibration data is either totally lacking or limited to sparse anecdotal information on flooding. The term ‘ungauged catchment’ here is meant to include also flooding areas with no or only very limited flood level observations. In these situations, the model parameters must be estimated to the best degree possible using what information is available. In these cases, while a complete calibration procedure is not possible, the model parameters can be estimated to some extent by other means, though there will obviously be additional uncertainty compared to the situation when there is adequate calibration data.

While many applications are required on totally ungauged catchments, it is common to have at least some minimal records of flooding available. The minimal descriptive data availability is discussed further in Book 1, Chapter 4, but where there is some anecdotal data, the parameter determination process must use this data to at least ensure that the model performance is consistent with this minimal data even if the data is insufficient to provide a calibration.

An important issue with the estimation of parameters for ungauged catchments is that the methods rely on transfer of parameter values from neighbouring catchments. The methods therefore rely on the assumption that the catchments used to estimate parameters are sufficiently similar to the catchment being analysed. It is important to carry out as many checks as possible to confirm that this is the case, but there will always be some uncertainty.

There are several different methods of estimating model parameters for ungauged catchments.

- *General Guidance* - Published documentation, including user guides for particular modelling platforms as well as textbooks and research publications, provide guidance for estimating parameters for ungauged catchments. These include advice on Manning’s n values for example which is widely available in textbooks and manuals. However many modelling platforms provide general guidance and in some cases, user forums can be of assistance.

- *Regional Relationships* - These are developed for particular model parameters and for particular regions. For example, there are published relationships for runoff-routing parameters which are related to catchment area, for several regions of Australia. In some cases, specific regional relationships are developed for particular project areas from limited data and then adopted for the whole project area.

- *Transfer from Neighbouring Catchments* - This is a special case of the regional relationship type approach. If the catchment being analysed is not gauged but there is a neighbouring gauged catchment that has similar characteristics, it is possible to calibrate a model on the neighbouring catchment and then transfer the parameters across perhaps with adjustments for the known differences, such as catchment area. There is a risk in this case that the neighbouring catchment may appear superficially similar, but may have a quite different catchment response.

The principal issue with parameter estimation for ungauged catchments is to use whatever data may be available, no matter how poor quality this may be, understand the physical processes represented by the models to ensure that the parameters are realistic, and to use regional relationships and information from neighbouring catchments to the maximum extent possible. The uncertainty in the resulting model operation must be considered in any model application for ungauged catchments, since this will be greater than would be the case for a well gauged catchment.

5.8. Adopted Parameter Set

The ultimate requirement for model parameter determination is to apply the calibrated model to certain design situations, as discussed further in Book 8. However some comments are provided here to give advice on the final step of the calibration process where the parameters resulting from the calibration process and from other sources of parameter estimates are accepted and reviewed further in a validation process (Book 7, Chapter 7) and then applied to design.

Often the calibration process will result in different parameter sets applying for different calibration events.

In general, this is not allowable, since a single parameter set will be required for application so after completing calibration on a number of different flood events, the calibration process must continue to calculate a single parameter set to best fit all of the available data. Therefore a procedure is needed to select a representative parameter set for application to the design situation.

The simplest approach would be to “average” the parameters, which will result in parameters that are representative, but may not result in a model that “averages” performance. An alternative approach to simple averaging would be to average them with a weighting towards the rarer floods. It is also possible to adopt the parameter set that has been estimated from the historic flood that is most similar to the design flood requirements, which may be the largest flood event.

Whichever technique is adopted to interpret the calibration results and adopt parameters for a design application, these adopted parameters should then be used with the model on all of the design flood events to confirm the performance for all the data. The results from this should show at least a reasonable performance for all of the calibration flood events and no bias in the results, that is the calibration on all historic floods should not be all under- or over-estimations.

It is desirable to compare the adopted parameter set from the calibration events with parameter estimates from catchments and flooding areas with similar characteristics and with parameter values obtained from regional parameter estimation procedures. If there are any significant discrepancies between the parameter estimates from different sources, the possible reasons should be investigated and the final parameter selection decision made in the light of the findings from these considerations.

Once the calibration has been accepted, the model should then be transferred to the validation phase, where the parameters are confirmed and determined to be available for application.

Chapter 6. Regional Relationship for Runoff-routing Models

Michael Boyd

Chapter Status	Final
Date last updated	14/5/2019

6.1. Introduction

Regional relationships can be used to estimate parameter values on ungauged catchments but they can also be used to test the plausibility of parameters derived from limited data. Where no data is available some insight can also be gained from comparing how adjoining catchments with data compare to the regional relationship. Relationships between model parameters and catchment characteristics have been derived for several regions. The most recent relationships available for Australia are given in the following section.

In all cases the reliability of regional relationships is likely to be less than parameter estimates derived from calibration from several recorded flood events on the catchment of interest. Regional relationships should be used with due caution, as most derived relations incorporate considerable scatter of the data from individual catchments. Also, different forms of relationships have been found to give equally good fits to the one set of data, but would give widely different estimates in some other cases (Sobinoff et al., 1983).

6.2. Regional Relationships

The following relationships for RORB and WBNM apply to catchments in natural condition. Regional data for RAFTS and URBS are not as extensive as for the other two models, and suggested parameter values for these models are included in Book 7, Chapter 5.

Regional relationships will contain some scatter about the fitted equation, partly due errors in rainfall and streamflow data, including insufficient spatial rainfall gauge coverage, but also due to limitations in the models themselves. Loy and Pilgrim (1989) quote typical errors of 10-20% for rainfall and 25% for streamflow data, with larger errors being quite possible, and note that as a consequence high correlation is unlikely to be obtained in the resulting regional relationships.

Scatter in the relationships can also be caused by different methods of treating the data when parameters were originally calibrated. These include different assumptions when separating baseflow, and different rainfall loss models, for example proportional loss as opposed to continuing loss rate. These different assumptions can lead to different calibrated parameter values, and hence contribute to scatter in the regional relationship. This problem will be reduced if the regional relationship is developed using consistent methods of treating the data. However, when parameters are combined from several different studies to develop a regional relationship, care should be taken to ensure that consistent parameter values are used.

Another cause of scatter can result from different parameters being derived from calibrations using floods of different magnitudes. Wong (1989) found that calibrated values of the RORB parameter k_c were larger for larger floods, when overbank flow became established, compared to smaller in-bank flows. Similar effects are likely in all runoff-routing models. The use of a single catchment parameter value in regional relationships, without regard to the magnitude of the flood, may therefore call into question the validity of the relationship (Wong, 1989).

It is important to note that that the value of the lag parameter *k* (Book 5, Chapter 5, Section 4) (or the corresponding k_c, C, B and β parameters in RORB, WBNM, RAFTS and URBS respectively) depends on the values of other parameters adopted during calibration of the model. The values of these lag parameters used in regional relationships will be dependent on the values of, for example, the nonlinearity parameter *m*, as well as the stream channel routing method used and the stream channel parameter values adopted. Another cause of variation in the lag parameter can occur if the basic model is modified, for example by allocating proportionally greater lag time to subareas and less to stream reaches (Kneen, 1982) in which case the calibrated *k* values will not be consistent with those calibrated for the same events using the basic model.

It is possible to obtain an approximate adjustment for *k* (or K_c, C, B or β) values which have been derived using other values of *m* so that they correspond to a base value, for example, *m* = 0.8 (Morris, 1982). This is done by adjusting *k* so that the same overall lag time K is maintained for the different *m* values, using Equation (7.6.1). This requires an average or representative discharge for the particular flood, which will be half or less than half of the peak discharge Q_p. Pilgrim (1987) used an average discharge equal to $Q_p/2$, giving the following adjustment:

$$k_{0.8} = k_m \left(\frac{Q_p}{2}\right)^{m-0.8} \tag{7.6.1}$$

where k_m is the lag parameter (K_c, C, B or β) corresponding to a specified value of *m*.

Most regional relationships relate the lag parameter to one or more physical characteristics of the catchment. These are most commonly the area A, stream length L and stream slope S_c, although other measures, such as elevation, average rainfall and drainage density are sometimes used. Different studies sometimes use different definitions of stream slope S_c so that caution is needed to ensure that the correct definition is used when applying the relationships. Measurements of stream length L are dependent on the map scale used (Cordery et al., 1981) and this should also be considered when applying the relationships. Stream length is strongly correlated with catchment area and stream slope is moderately correlated with area, so that relationships involving area A alone, or stream length L alone are often sufficient to describe the regional relationship.

6.2.1. Regional relationships for RORB

The greatest number of derived parameter values and regional relationships are available for the RORB Model. The relationships recommended below are derived from all readily available data. Values of the parameters and the catchments used in deriving the relationships are generally listed in the cited publications. Although the nonlinearity parameter *m* can be varied to improve the hydrograph fit when calibrating the model, most studies have found *m* to lie in the range 0.6 to 1.0, and many studies adopt a constant value of *m* = 0.8 (Hansen et al., 1986; Dyer et al., 1993; Dyer et al., 1995; Pearse et al., 2002). All relationships for k_c given in this section are for a value of *m* = 0.8 except where specifically noted.

Most of the relationships are of similar form and involve only the single catchment variable, area A in km^2, since this has been found to be the dominant variable. To allow comparisons, relationships developed by various researchers are presented, together with the number and size range of the catchments used (where available). The recommended regional relationships for each region are then given in boxes.

6.2.1.1. Queensland

Relationships have been developed by Weeks and Stewart (1978), Morris (1982), Hairsine et al (1983), Weeks (1986) and Titmarsh and Cordery (1991). For 14 catchments (158 to 3430 km^2) generally covering the coast, plus one catchment near Mt. Isa, Weeks and Stewart (1978) obtained:

$$k_c = 0.69A^{0.63} \tag{7.6.2}$$

$$\mathrm{m} = 0.73 \tag{7.6.3}$$

For 25 catchments (56 to 5170 km^2), with parameters adjusted to m = 0.75, Morris (1982) obtained:

$$K_c = 0.35A^{0.71} \tag{7.6.4}$$

For four catchments in the Darling Downs (2.5 to 50 km^2) with m = 0.8, Hairsine et al (1983) obtained:

$$K_c = 0.80A^{0.62} \tag{7.6.5}$$

For nine small catchments in south-east Queensland (0.002 to 50 km^2) with m = 0.8, Titmarsh and Cordery (1991) obtained:

$$K_c = 0.83A^{0.35} \tag{7.6.6}$$

For 88 catchments (2.5 to 16,400 km^2), covering both coastal and inland areas of Queensland, with parameters adjusted to m = 0.80, Weeks (1986) obtained:

$$K_c = 0.88A^{0.53} \tag{7.6.7}$$

Although Equation (7.6.2) to Equation (7.6.8) appear to be quite different, when plotted together, with each relationship covering its range of catchment sizes, they conform to a general trend and can be viewed as different samples from the population of Queensland catchments. The relationship of Weeks (1986), equation Equation (7.6.8), is a good average to all relationships and is recommended. Weeks (1986) also investigated possible variations of K_c within the various regions of the study, and also any effects of catchment slope, but no relationships were found.

The relationship of Weeks (1986), Equation (7.6.8), is a good average to all relationships and is recommended.

$$K_c = 0.88A^{0.53} \tag{7.6.8}$$

6.2.1.2. New South Wales

Relationships have been developed by Kleemola (1987), Sobinoff et al. (1983) and Walsh and Pilgrim (1993). For 26 catchments (0.1 to 4560 km^2) in the Newcastle-Sydney-Wollongong region, with m = 0.8, Sobinoff et al. (1983) obtained:

$$K_c = 1.09A^{0.45} \tag{7.6.9}$$

No regional trends were apparent, except for some lower values of K_c in the Upper Hunter valley. Addition of slope to the regressions did not improve the fitted relationships appreciably. Walsh and Pilgrim (1993) added to the data of Kleemola (1987) and derived relationships for 46 catchments (0.1 to 13,000 km^2). Relationships were derived using area A, stream length L and stream length divided by slope (L/S$^{0.5}$). The fit of these various relationships to the data were similar, and a relationship involving area A was considered to be the most logical one to adopt. The relationships were:

West of Great Dividing Range, upper western slopes and tablelands (12 catchments, 100 to 4770 km^2)

$$K_c = 0.80A^{0.51} \tag{7.6.10}$$

East of Great Dividing Range (34 catchments, 0.1 to 6465 km^2)

$$K_c = 1.18A^{0.47} \tag{7.6.11}$$

Since the relationships are very similar, a combined relationship was derived for all 46 catchments:

NSW catchments

$$K_c = 1.18A^{0.46} \tag{7.6.12}$$

Walsh and Pilgrim (1993) found that most catchments had values of m in the range 0.75 to 1.0 and adopted a fixed value of m = 0.8 for all catchments. No trends for K_c to vary with event size were evident, indicating that the nonlinearity was adequately described by adopting m = 0.8. Weighted average and direct average K_c values were calculated from all events on each catchment, with little difference being apparent.

When Equation (7.6.9) to Equation (7.6.12) are plotted to cover their range of catchment sizes, all equations are very similar, and equationEquation (7.6.12) is recommended for catchments both east and west of the Great Dividing Range.

Equation (7.6.13) is recommended for catchments both east and west of the Great Dividing Range.

NSW catchments

$$K_c = 1.18A^{0.46} \tag{7.6.13}$$

6.2.1.3. Victoria

Regional relationships have been developed by Morris (1982) and Hansen et al. (1986).

Morris (1982) developed relationships for 16 catchments (20 to 1924 km^2) with m = 0.75:

$$K_c = 1.37A^{0.59} \tag{7.6.14}$$

Region with mean annual rainfall greater than 800 mm (19 catchments, 38 to 3910 km^2, mainly the eastern part of Victoria):

$$K_c = 2.57A^{0.45} \tag{7.6.15}$$

Region with mean annual rainfall less than 800 mm (21 catchments, 20 to 1924 km^2, mainly the western part of Victoria):

$$K_c = 0.49A^{0.65} \tag{7.6.16}$$

The relationships of Morris (1982) and Hansen et al. (1986) for RF > 800 mm are reasonably consistent, while the K_c values for the drier part of the state are somewhat lower, particularly for the smaller catchments. Comparing the Hansen et al. (1986) relationships for the eastern and western parts of Victoria, predicted K_c values are similar for catchments greater than about 2,000 km^2, but the eastern region values are approximately double for catchment areas near to 100 km^2.

6.2.1.4. South Australia

Regional relationships have been developed by Lipp (Pilgrim, 1987), Maguire et al. (1986) and Kemp (1993). For the south-east region, corresponding to zone 6 of the ARR design storm temporal patterns, Australian Rainfall and Runoff (Pilgrim, 1987) recommended:

For catchments smaller than 100 km^2:

$$K_c = 0.60A^{0.67} \tag{7.6.17}$$

For catchments larger than 100 km^2, based on limited data:

$$K_c = 1.09A^{0.51} \tag{7.6.18}$$

For the northern and western regions, corresponding to zone 5 of the ARR design storm temporal patterns, Australian Rainfall and Runoff (Pilgrim, 1987) recommended for flat to undulating country:

$$K_c = \mathrm{Coeff.}A^{0.57} \tag{7.6.19}$$

where the Coefficient ranges from 1.2 to 1.7 for equal area stream slopes ranging from 1.0 to 0.2%.

For the northern and western regions, undulating to steep country, with slopes greater than 1%, (Equation (7.6.25)) for the wheatbelt, north-west and Kimberley regions of Western Australia was recommended by ARR 1987 (Pilgrim, 1987). However, the more recent relations developed by Kemp (1993) are now recommended for these arid regions.

Kemp (1993) derived relationships for low and high rainfall areas, using m = 0.8. For average annual rainfall RF less than 320 mm (7 catchments, 170 to 6020 km^2):

$$K_c = 7.06A^{0.71} \quad (\mathrm{RF}/1000)^{2.79} \tag{7.6.20}$$

For average annual rainfall greater than 500 mm (17 catchments, 5 to 690 km^2):

$$K_c = 0.89A^{0.55} \tag{7.6.21}$$

For the higher rainfall south-east region, near Adelaide, There is a good agreement between Equation (7.6.17), Equation (7.6.18) and Equation (7.6.21). Equation (7.6.20) for areas with annual rainfall less than 320 mm also agrees with these equations for RF near to 320, the top of the applicable range. For the drier interior of the state, Equation (7.6.19) predicts higher K_c values, while Equation (7.6.20) predicts lower K_c values. Since the Kemp (1993) study is the most extensive, Equation (7.6.20) and Equation (7.6.21) are recommended for South Australia, but with the note that Equation (7.6.20) predicts quite low K_c values for the drier interior of the state.

6.2.1.5. Western Australia

Regional relationships have been developed by Weeks and Stewart (1978), Morris (1982), Flavell et al. (1983). Netchaef et al. (1985) present some data for the Pilbara region. For 6 catchments in the southwest (67 to 805 km^2), Weeks and Stewart (1978) derived:

$$K_c = 1.23A^{0.91} \tag{7.6.22}$$

The nonlinearity parameter *m* was also calibrated on each catchment, overall being near to *m* = 0.75. K_c values for western Australia were found to be considerably higher than for the eastern states, which they attributed to the observed more sluggish response to rainfall of these catchments. Morris (1982) for 24 catchments (28 to 5950 km^2), using *m* = 0.80 derived:

$$K_c = 2.48A^{0.47} \tag{7.6.23}$$

Flavell et al. (1983) derived relationships for 52 catchments (5 to 6526 km^2) in 4 regions of the state. A non-linearity parameter of *m* = 0.8 was found to give best results for the south-west, and was adopted for the entire state. Variables used in the regressions were catchment area A, main stream length L, main stream equal area slope S_e (m/km), and percentage of land cleared. Generally, regressions involving stream length L were better than those using area A. For the south west (26 catchments, 29 to 3870 km^2) relations for sub regions with different soil types were similar.

The following equation is recommended for all jarrah forest catchments in the south west:

$$K_c = 1.49L^{0.91} \quad (7.6.24)$$

Relationships for the wheatbelt, north-west, and Kimberley regions were similar and the following is recommended, based on 26 catchments (5 to 6526 km^2) :

$$K_c = 1.06L^{0.87} \quad S_e^{-0.46} \quad (7.6.25)$$

For the arid interior of Western Australia, Equation (7.6.25) is recommended.

L was converted to A through the relationship established by Boyd and Bodhinayake (2005) to allow Equation (7.6.24) and Equation (7.6.25)to be plotted against catchment area A. The relationships of Morris (1982) and Flavell et al. (1983) for the south west region are very similar and Equation (7.6.24) is recommended. Equation (7.6.25) for the wheatbelt, north-west and arid regions predicts K_c values which are considerably lower than for the south-west region. The earlier relation of Weeks and Stewart (1978) was based on only sixcatchments and predicts K_c values which are considerably higher than those of Flavell et al. (1983), and is not recommended.

6.2.1.6. Northern Territory

Relationships for K_c have been derived for the northern half of the Northern Territory by the Department of Mines and Energy (Pilgrim, 1987). The Northern Territory was divided into the three zones.

For the humid zone, north of latitude $15°$ S, with equal area slope S_e in m/km the following equation is recommended:

$$K_c = 1.8(A/S_e^{0.5})^{0.55} \quad (7.6.26)$$

For the transition zone, between latitudes $15°$ S and $17.5°$ S:

$$K_c = 0.35A^{0.64} \quad (7.6.27)$$

Equation (7.6.26) and Equation (7.6.27) are similar for catchments greater than 2500 km^2, but with smaller K_c values predicted for smaller catchments in the transition zone compared to the humid zone.

For the arid zone, below latitude $17.5°$ S ARR 1987 (Pilgrim, 1987) recommended the relationship for the northern and western regions of South Australia (Equation (7.6.19)) be used. Equation (7.6.19) predicts higher K_c values than both the humid and transition zones, and as discussed below, is not recommended. Equation (7.6.25), for the wheatbelt, north-west, Kimberley and arid interior of Western Australia, lies within the range of values for the drier interior of south Australia (Equation (7.6.20)) and is close to K_c values derived by Board et al. (1989) for two catchments in the arid zone near to Alice Springs.

Therefore Equation (7.6.25) is recommended for the arid interior of the Northern territory.

6.2.1.7. Arid Region of Central Australia

For the arid region of Central Australia, approximately corresponding to zone 5 of the ARR storm temporal patterns in ARR 1987 (Pilgrim, 1987), and covering the interior of South Australia, Western Australia and the Northern Territory, the data of Flavell et al. (1983), Kemp (1993), Board et al. (1989) may be used to guide selection of K_c values.

Predicted K_c values for the arid regions of South Australia (Equation (7.6.20)), Western Australia (Equation (7.6.25)) and Victoria (Equation (7.6.16)) are lower than K_c values for the higher rainfall areas of these states. Similar trends for lower K_c values in lower rainfall regions have been found by Yu (1990) and Kemp (1993). Equation (7.6.19) appears to be an anomaly since it predicts higher K_c values for the arid zone of the Northern Territory compared to the humid and transition zones.

Equation (7.6.25) for the wheatbelt, north-west and Kimberley region of Western Australia lies within the range of K_c values predicted by Equation (7.6.20) for the arid zone of south Australia. The data of Board et al. (1989) also agrees with equation Equation (7.6.25).

Therefore equation Equation (7.6.25) is recommended for the arid interior of Western Australia and the Northern territory, and equation Equation (7.6.20) for the arid region of South Australia.

6.2.1.8. Tasmania

Morris (1982) developed the following relation for 17 catchments (63 to 1780 km^2) using m = 0.75:

$$K_c = 4.86A^{0.32} \tag{7.6.28}$$

Australian Rainfall and Runoff (Pilgrim, 1987) presents a relation developed by the Tasmanian Hydro Electric Commission for western Tasmania, with m = 0.75:

$$K_c = 0.86A^{0.57} \tag{7.6.29}$$

Equation (7.6.28) and Equation (7.6.29) are in good agreement for catchments near to 1000 km^2 but Equation (7.6.28) predicts larger K_c values for smaller catchments.

In the absence of further data, equation Equation (7.6.29) is recommended for Tasmania.

6.2.2. Regional relationships for RORB using Area-Standardised Lag Parameter

Equation (7.6.2) to Equation (7.6.29) all show that the lag parameter K_c of RORB is strongly correlated with catchment area A raised to a power slightly greater than 0.5. Since stream lengths are also strongly related to catchment area raised to a very similar power, it follows that K_c will be related to stream length or a measure of stream length. One measure of stream length which has been adopted in RORB is the average flow distance d_{av}. McMahon and Muller (McMahon and Muller, 1983; McMahon and Muller, 1986) and Yu (1990) have used these relations to form an area-standardised lag parameter K_c/d_{av}. Kemp (1993) formed a similar area-standardised lag parameter $K_c/A^{0.57}$. Because of the strong relation between K_c and measures of area and stream length, the area-standardised lag parameter should be essentially independent of the catchment size. The area-standardised lag parameter can then be seen as analogous to lag parameters C , B and β in the WBNM, RAFTS and URBS models respectively.

Yu (1990) found that K_c/d_{av} increased as mean annual rainfall increased in Victoria (30 catchments) and western Australia (51 catchments), but not in New South Wales, Queensland or the Timor sea region of the Northern Territory (41 catchments in total). For all 122 catchments, the average value of K_c/d_{av} was found to be 1.09. Kemp (1993) found a similar increase of $K_c/A^{0.57}$ as mean annual rainfall increased for South Australia, Victoria, Western Australia and the Alice Springs region of the Northern territory. The effect appears to be more pronounced in the drier winter rainfall regimes.

Pearse et al. (2002) combined the data of Hansen et al. (1986), Dyer et al. (1995) and Yu (1990), for more than 220 catchments in Queensland, New South Wales, Victoria, Tasmania and Western Australia. The non-linearity parameter was set at m = 0.8. The mean value of K_c/d_{av} was found to range between 0.96 and 1.25, depending on the particular region. The results of Yu (1990) and Pearse et al. (2002) allow K_c to be estimated for any catchment by first calculating the average flow distance dav, then multiplying it by the appropriate area-standardised lag parameter value.

These results also indicate that many of the regional relations across Australia Equation (7.6.2) to Equation (7.6.29)) could be fitted by either a single relation or a small number of similar relations. For example, the data of Hansen et al. (1986) for 30 catchments (20 to 3910 km^2) produces the following relation between dav (km) and A (km^2):

$$d_{av} = 0.98A^{0.54} \tag{7.6.30}$$

Combining equation Equation (7.6.30) with the range of K_c/d_{av} values from 0.96 to 1.25 produces a general relation:

$$K_c = \mathrm{Coeff.}A^{0.54} \tag{7.6.31}$$

where the coefficient ranges from 0.94 to 1.22.

Equation (7.6.31) can be plotted for the mid-range coefficient value 1.08, together with the recommended relationships for the various regions of Australia, and is seen to lie in the middle of these relationships.

While the area-standardised lag parameter can be expected to be essentially independent of catchment size, it may be related to other variables. Dyer et al. (1995) developed regression relationships between K_c/d_{av} and a range of catchment, climatic and RORB model properties for seven groups. The method is also presented in Grayson et al (1996). Catchments were placed into groups based on hydrological similarity, utilising Andrews curves, rather than on geographical region. All values of K_c/d_{av} are for m = 0.8 and using a proportional loss rather than a continuing loss rate model. Data from 72 catchments from the east coast of Australia, Tasmania, the Adelaide Hills, and the south-west of Western Australia were used. The various regression equations, and the variables in them, were not consistent across the seven groups. Slope appeared in the equation for only one group. While reasonably strong regressions could be developed for the catchments in the data set, difficulties in assigning ungauged catchments to a particular group have been found to cause problems in application of the method (Perera, 2000; Pearse et al., 2002).

6.2.3. Regional Relationships for WBNM

In WBNM the non-linearity parameter is recommended to be set at *m* = 0.77, unless there is strong evidence to use another value. This is very close to the widely adopted value of *m* = 0.80 in RORB. With parameter *m* set, only the lag parameter C needs to be evaluated. As noted previously, parameter C is effectively an area-standardised value analogous to K_c/d_{av} in RORB. Therefore parameter C should be independent of catchment size. Additionally, if the value of the non-linearity parameter *m* is correct, parameter C should be independent of the flood size.

Parameter values have been derived for WBNM by Boyd et al. (1979), Boyd et al. (2002), Sobinoff et al. (1983), Bodhinayake (2004) and Boyd and Bodhinayake (2005). Plots of parameter C against catchment area A have shown no trend for C to vary with catchment size, indicate that the power of area A is satisfactory. The lack of dependence between the area-standardised form of the lag parameter in RORB and catchment area has also been noted by Pearse et al. (2002). Additionally, plots of C against the peak discharge of the recorded flood have shown no trend for C to vary with flood size, indicating that the non-linearity parameter *m* = 0.77 is satisfactory.

Bodhinayake (2004) investigated possible trends in parameter C against a range of storm and catchment characteristics. Storm variables which were considered included the peak discharge, rainfall depth, rainfall excess depth, rainfall intensity, location of peak burst within storm, and spatial distribution of rainfall. Catchment variables included area A, stream slope S_e, stream length L, length to centroid L_c, spatial distribution of area L_c/L, catchment shape A/L2, catchment elevation, number of rain days, and mean annual rainfall. The study used 251 storms on 17 catchments in eastern Queensland. While slight trends were apparent in some cases and for some subsets of catchments, there were no strong trends for C to vary with any of these variables. The independence of parameter C from these storm and catchment characteristics indicates that one value applies generally over a wide range of regions. A similar result has been obtained for the area-standardised lag parameter in RORB by Pearse et al. (2002).

Values of the lag parameter C calibrated for south and eastern Australia are:

For 207 storms on 17 coastal catchments in Queensland (164 to 7300 km^2), ranging from the North Johnstone to the Mary River, Bodhinayake (2004) obtained a mean value of parameter C of 1.47.

For ten catchments in the coastal region of NSW (0.4 to 250 km^2), Boyd et al. (1979) obtained a mean lag parameter C of 1.68. For 17 catchments (0.1 to 800 km^2) in the Newcastle, Sydney-Wollongong region Sobinoff et al. (1983) obtained a mean C of 1.16. Recent calibration of WBNM for 205 storms on 19 coastal catchments of NSW (0.2 to 6910 km^2) by Boyd and Bodhinayake (2005) obtained a mean C of 1.74.

For 59 storms on six catchments in Victoria on the coastal side of the Great Dividing Range, ranging from Bairnsdale to Ballarat (0.1 to 153 km^2) plus 45 storms on four catchments inland of the Great Dividing Range near Healseville and Stawell (63 to 259 km^2), Boyd and Bodhinayake (2005) obtained a mean value of C = 1.74.

For 90 storms on eight catchments in the Adelaide Hills near to Adelaide (4 to 176 km^2), Boyd and Bodhinayake (2005) obtained a mean value of C = 1.64.

The small range of these mean parameter values corresponds to the similar small range of the area-standardised parameter K_c/d_{av} in RORB found by Pearse et al. (2002). Boyd and

Bodhinayake (2005) calculated a mean value of parameter C for all 54 catchments in Queensland, New South Wales, Victoria and South Australia of 1.64, and 1.59 when the parameter values were weighted by the number of storms on each catchment.

With no strong regional trends being apparent, and no strong relationships between parameter C and catchment or storm characteristics, an overall mean value of parameter C = 1.60 is recommended for these states of Australia.

6.2.4. Relationships between RORB, WBNM, RAFTS and URBS Lag Parameters

As noted previously, all four runoff-routing models contain area-standardised lag parameters. These are K_c/d_{av}, C, B and β of the RORB, WBNM, RAFTS and URBS models respectively. It could therefore be expected that these parameters will be related to one another. For example, comparing equations for WBNM with equations for RORB reveal that $CA_i^{0.57}$ in WBNM corresponds to $(K_c / d_{av}).L_i$ in RORB. Note that the area term A_i and stream length term L_i in these equations refer to subcatchments and stream segments rather than to complete catchments.

Measures of stream length, such as L and d_{av} (Equation (7.6.30)) are strongly related with catchment area, and it is reasonable to assume that stream segment lengths are also strongly correlated with subcatchment areas. Replacing the L_i term by $A^{0.55}$, it is seen that parameter C of WBNM should be directly proportional to K_c/d_{av} of RORB. From the previous sections the average value of C is close to 1.60 and the average value of K_c/d_{av} is 1.1 (range 0.96 to 1.25). Therefore a relationship between these two parameters is:

$$C = 1.45\,K_c/d_{av} \tag{7.6.32}$$

Similar analysis indicates that parameter β of URBS should be directly proportional to K_c/d_{av}. For RAFTS the proportionality coefficient should be related to K_c/d_{av} but with an adjustment required for the slope term S.

It should be noted that the correspondence between the area-standardised lag parameters of the various models depends slightly on the power to which area A is raised, as well as the non-linearity parameter *m*, however these are not too dissimilar in the four models. The particular ratio between the parameters will depend on the way in which the lag parameter is incorporated into flood routing in the particular modelling platforms, as well as the method adopted for stream channel routing. The ratio 1.45 between RORB and WBNM will not apply to RAFTS and URBS.

6.3. Modelling Urban Catchments

Increased flood discharges in urban or partially urbanised catchments can be attributed to two factors. The increased proportion of paved or impervious surfaces produce greater runoff volumes, and the decreased lag times for the runoff produces higher peak discharges. These increases are not the same for all floods, being more pronounced for the smaller more common events. Data given by Cordery (1976a), Codner et al. (1988) and Mein and Goyen (1988) indicate that for 10% Annual Exceedance Probability events, urban flood peak discharges increase by 2 to 5 times, while for 1% Annual Exceedance Probability events urban peaks increase by less than two times.

Increased runoff volumes from paved surfaces result from decreased rainfall losses.

The decrease in lag time can be attributed to replacement of vegetated overland flow surfaces and natural stream channels by more hydraulically efficient paved surfaces, gutters, pipes and channels. Ratios of lag times in urban compared to otherwise equivalent natural catchments typically range from 0.1 to 0.5 (Cordery, 1976b; Codner et al., 1988; Mein and Goyen, 1988; Boyd et al., 1999; Boyd et al., 2002). Decreases in lag time have been related to the fraction of the catchment which is urbanised by Aitken (1975) and NERC (1975). Other studies by Rao et al. (1972), Crouch and Mein (1978), Desbordes et al. (1978), Schaake et al. (1967) and Espey et al. (1977) relate lag time to the impervious fraction. A survey of these relations is given by Boyd et al. (1999) and Boyd et al. (2002).

All of these studies show a decrease in lag time as the catchment becomes more urbanised. Typically, the lag reduction is expressed in terms of the urban fraction U in the form (1+U)z where z ranges between –1.7 and –2.7, with an average near to –2.0. Equation 5.3.4.19 adopts z = -1.97 for RAFTS, while equation 5.3.4.20 adopts z = -2.0 for URBS. A value of z = -2.0 in this relation produces a lag ratio of 0.25 for a fully urbanised catchment.

The urban fraction urban often does not fully describe the state of urbanisation, since a 100% urban catchment can be residential with typically 30% impervious surfaces, or it can be a high density commercial centre with close to 100% impervious. Typical relationships between the impervious and the urban fraction are given by Boyd et al. (1993) and Boyd et al. (2002). The RAFTS model accounts for this by allocating an equivalent urban fraction U to each level of impervious fraction. For a fully impervious surface, this produces a lag ratio of 0.11.

When a subcatchment is partly urbanised, it can be modelled in a lumped form whereby a single hydrograph is calculated for the combined pervious and impervious surfaces, using a reduced lag time. This is often done in the RORB, RAFTS and URBS models. Alternatively, the subcatchment can be split into separate pervious and impervious surfaces with separate lag times and separate hydrographs calculated for each surface. This is the recommended method for WBNM, where the lag ratio for fully impervious surfaces is set at 0.10, similar to the value recommended in RAFTS. However, all models can be configured to operate in either lumped or split form, and RAFTS has been found to produce good results in the split form for catchments in the ACT (Knee and Bresnam, 1993). Split pervious and impervious modelling is similar to the procedures used in detailed urban drainage modelling platform such as DRAINS.

Urban catchments have other features which need to be considered when setting up a model. During large storms flows may be diverted out of the catchment’s stream network to form new overland flow routes. This can happen particularly when culvert or bridge openings become blocked by debris. The model should be set up to reflect these alternative flow paths. Another feature requiring consideration is that runoff from small development sites, and particularly when onsite detention storage is used to reduce flood peaks, will require routing calculations at small time steps and with small discharges. The stream network runoff-routing models currently used in Australia all have these capabilities.

6.4. References

Aitken, A.P. (1975), Hydrologic investigation and design of urban stormwater drainage systems, AWRC Technical Paper No. 10, Canberra: Australian Government Publishing Service.

Board, R.J., Barlow, F.T.H. and Lawrence, J.R. (1989), Flood and Yield Estimation in the Arid Zone of the Northern Territory. Hydrology and Water Resources Symposium 1989. Christchurch, New Zealand, pp: 367-371

Bodhinayake, N.D. (2004), Influence of hydrological, geomorphological and climatological characteristics of natural catchments on lag parameters, PhD Thesis, School of Civil, Mining and Environmental Engineering, University of Wollongong.

Boyd, M.J. and Bodhinayake, N.D. (2006), WBNM runoff routing parameters for south and eastern Australia, Australian Journal of Water Resources, 10(1), 35-48.

Boyd, M.J., Pilgrim, D.H. and Cordery I. (1979), A storage routing model based on basin geomorphology, Journal of Hydrology, 42: 209-230.

Boyd, M.J., Rigby, E.H., VanDrie, R. and Schymitzek, I. (2002), WBMN 2000 for flood studies on natural and urban catchments, In 'Mathematical Models of Small Watershed Hydrology and Applications', pp: 225-258.

Boyd M.J., Rigby E.H. and van Drie, R. (1999), Modelling urban catchments with WBNM2000, Proceedings of the Water 99 Conference, Institution of Engineers Australia, Brisbane.

Boyd, M.J., Bufill, M.C. and Knee, R.M. (1993), Pervious and impervious runoff in urban catchments. Hydrological Sciences Journal, 38(6), 463-478.

Codner, G.P., Laurenson, E.M. and Mein, R.G. (1988), Hydrologic Effects of Urbanisation: A case study. Hydrology and Water Resources Symposium, ANU Canberra, 1-3 Feb 1988, Institution of Engineers Australia, pp: 201-205.

Cordery, I, Pilgrim, D.H. and Baron, B.C. (1981), Validity of use of small catchment research results for large basins. Instn. Engrs. Australia, Civil Engg. Trans., CE23: 131-137.

Cordery, I. (1976a), Some effects of urbanization on streams, Civil Engineering Transactions, Institution of Engineers, Australia, CE18(1), 7-11.

Cordery, I. (1976b), Some effects of urbanisation on streams. Civil Engineering Transactions, The institution of Engineers, Australia, CE18(1), 7-11.

Crouch, G.I. and Mein R.G. (1978), Application of the Laurenson Runoff Routing Model to Urban Areas, I.E. Aust. Hydrology Symposium, Canberra, pp: 70-74.

Desbordes, M. (1978), Urban runoff and design storm modelling, Proceedings of the First International Conference on Urban Storm Drainage.

Dyer, B., Nathan, R.J., McMahon, T.A. and O'Neill, I.C. (1993), A cautionary note on modelling baseflow in RORB. I.E. Aust. Civil Engin. Trans., CE35(4), 337-340.

Dyer, B.G., Nathan, R.J., McMahon, T.A. and O’Neill, I.C. (1995), Prediction Equations for the RORB Parameter kc Based on Catchment Characteristics, Australian Journal of Water Resources, 1(1), 29-38.

Espey, W.H., Altman, D.G. and Graves, C.B. (1977), Nomograph for 10 minutes unit hydrographs for urban wartersheds, Technical Memo No.32, American Society of Civil Engineers, New York.

Flavell, D.J., Belstead, B.S., Chivers, B. and Walker, M.C. (1983), Runoff routing model parameters for catchments in Western Australia, Hydrology and Water Resources Symposium, Institute of Engineers, Hobart.

Grayson, R.B., Argent, R.M., Nathan, R.J., McMahon, T.A. and Mein, R.G. (1996), Hydrological recipes - Estimation techniques in Australian Hydrology, Cooperative Centre for Catchment Hydrology, Monash University, pp.125.

Hairsine, P.B., Ciesiolka, C.A.A., Marshall J.P. and Smith, R.J. (1983), Runoff routing parameter evaluation for small agricultural catchments, Proceedings of the Hydrology and Water Resources Symposium, Publication 83/13, I.E. Aust. pp: 33-37.

Hansen, W.R., Reed, G.A. and Weinmann, P.E. (1986), Runoff routing parameters for Victorian catchments, Proceedings of the Hydrologoy and Water Resources Symposium, Publication 86/13, Griffith University, pp: 192-197.

Kemp, D.J. (1993), Generalised RORB parameters for Southern, Central and Western Australia, WATERCOMP, The Institution of Engineers, Melbourne.

Kleemola, S. (1987), Estimation of a Runoff Routing Parameter From Catchment Characteristics for Eastern New South Wales, M. Eng. Sci. Thesis, School of Civil Engineering, The University of New South Wales.

Knee, R.M. and Bresnan, S. (1993), Urban Stormwater Design Methods in the ACT, National Conference Publication, Instituion of Engineers Australia, p: 445.

Kneen, T.H. (1982), A runoff routing model for the Yarra river to Warrandyte. Instn. Engrs. Australia, 14 Hydrology and Water Resources Symposium NCP82/3, pp: 115-119.

Loy, A. and Pilgrim, D.H. (1989), Effects of data errors on flood estimates using two runoff routing models. Instn. Engrs. Australia. 19 Hydrology and Water Resources Symposium, NCP 89/19, pp: 176-182.

Maguire, J.C., Kotwicki, V., Purton, C.M. and Schalk, K.S.(1986), Estimation of the RORB parameter Kc for small South Australian catchments. Engineering and Water Supply Department, Adelaide, Jan. 1986, E & WS Library Ref. 86/2, 17 pp.

McMahon, G.M. and Muller, D.K. (1983), Calibration strategies for non-linear runoff-routing models. Instn. Engrs. Australia, 15 Hydrology and Water Resources Symposium, NCP 83/13, pp: 129-135.

McMahon, G.M. and Muller, D.K. (1986), The application of the peak flow parameter indifference curve technique with ungauged catchments. Instn. Engrs. Australia, 17 Hydrol. and Water Resources Symposium, NCP 86/13, pp: 186-191.

Mein, R.G. and Goyen, A.G. (1988), Urban runoff. Civil Engineering Transactions, 30: 225-238.

Morris, W.A. (1982), Runoff routing model parameter evaluation for ungauged catchments. Instn. Engrs. Australia, 14 Hydrology and Water Resources Symposium, NCP 82/3, pp. 110-114.

Natural Environment Research Council (NERC; 1975), Flood Studies Report, 5 Vols, Natural Environment Research Council, Institute of Hydrology, Wallingford, UK.

Netchaef, P., Wood, B. and Franklin, R. (1985), RORB parameters for best fit – catchments in the Pilbara region. Instn. Engrs. Australia, 16 Hydrology and Water Resources Symposium NCP85/2, pp: 53-57.

Pearse, M., Jordan, P. and Collins, Y. (2002), A simplemethod for estimating RORB model parameters for ungauged rural catchments. Instn. Engrs. Australia, 27th Hydrology and Water Resources Symposium, CD_ROM, 7 pp.

Perera, B.J.C. (2000), Evaluation of post-ARR87 flood estimation inputs for ungauged rural catchments in Victoria. Australian Jour. Water Resources, 4(2), 99-110.

Pilgrim, DH (ed) (1987) Australian Rainfall and Runoff - A Guide to Flood Estimation, Institution of Engineers, Australia, Barton, ACT, 1987.

Rao, A.R., Delleur, J.W. and Sarma, P.B.S. (1972), Conceptual hydrologic models for urbanizing basins. ASCE Jour. Hydraulic Engg., 98(7), 1205-1220.

Schaake, J.C., Geyer, J.C. and Knapp, J.C. (1967), Experimental examination of the Rational method. ASCE Jour. Hydraulics Div., 93(6), 353-370.

Sobinoff, P., Pola, J.P. and O'Loughlin, G.G. (1983), Runoff routing parameters for the Newcastle-Sydney-Wollongong region. Instn. Engrs. Australia, 15 Hydrology and Water Resources Symposium, NCP 83/13, pp: 28-32.

Titmarsh, G.W. and Cordery, I. (1991), An examination of linear and non linear flood estimation methods for small agricultural catchments, Australian Civl Engineering Transactions, 33(4), 233-242.

Walsh, M.A. and Pilgrim, D.H. (1993), Re-assessment of some design parameters for flood estimation in New South Wales. Instn. Engrs. Australia. 21 Hydrology and Water Resources Symposium, NCP93/14, pp: 251-256.

Weeks, W.D. (1986), Flood estimation by runoff routing - model applications in Queensland. Instn. Engrs. Australia, Civil Engg. Trans., CE28(2), 159-166.

Weeks, W.D. and Stewart, BA. (1978), Linear and non-linear runoff routing for ungauged catchments. Instn. Engrs. Australia, Hydrology Symposium, NCP 78/9, pp: 124-128.

Wong, T.H.F. (1989), Nonlinearity in catchment flood response. Instn. Engrs. Australia, Civil Engg. Trans., CE31(1), 30-38.

Yu, B. (1990), Regional relationships for the runoff routing model RORB revisited. Instn. Engrs. Australia, Civil Engg. Trans. CE31(4), 186-191.

Chapter 7. Validation and Sensitivity Testing

William Weeks, Erwin Weinmann

Chapter Status	Final
Date last updated	14/5/2019

7.1. Validation

After the catchment modelling system calibration has been finalised, the final step in acceptance of the model is the validation process. The calibration has resulted in model parameters that are suitable for application to the design problem, but validation provides a means of ensuring that the parameters are suitable and that the catchment modelling system can be applied to the design problem required. The validation process is therefore a confirmation that the calibrated catchment modelling system is fit for the required purpose.

Validation can be associated with independent verification of the model parameters. In this process, the calibrated catchment modelling system is tested with an independent data set that was not used in the parameter estimation process. While this does provide additional confirmation that the catchment modelling system is performing adequately, calibrations are usually very limited in the availability of data, and there are usually insufficient events to allow this independent assessment.

Validation therefore is a careful review of the catchment modelling system and its application to the problem at hand, so must consider the suitability of both the catchment modelling system and the parameters.

The first step is to review whether the catchment modelling system applied is appropriate for the application required. The questions are as follows.

- Is the model suitable for the problem being investigated?
- Does the model include sufficient detail in the spatial coverage of flooding?
- Does the model represent the flooding questions with sufficient accuracy to answer the required questions?
- Can the model be extrapolated accurately to rarer (or sometimes smaller) floods from the flood magnitudes used to establish it?
- Can the model be used to represent the range of design conditions (such as developed conditions or flood mitigation options) that are required in the design applications?

In addition to the review of the model and calibration, additional validation can be considered by reconciling the model performance with an alternate independent estimate. For example, for hydrology calculations, rainfall based methods can be reconciled with streamflow based methods or two alternative models may be calibrated separately and the results compared.

A special form of validation is for hydrologic models that are used to estimate probability based design flood characteristics. In these cases the main performance criterion for the model and the adopted parameter set is that the model is able to transform the probability based design inputs (design rainfalls, design losses and baseflows) into probability based flood outputs (flood hydrographs and flood levels) without introducing any probability bias ie. Probability neutral. Here the validation has to be against independent design flood estimates, e.g. from the flood frequency estimation procedures covered in Book 5.

In summary, the validation process is a critical independent review of the model establishment and performance to ensure that it is appropriate for the intended application.

If the model is determined to be appropriate, it can be applied to the required design problem. If the model with the adopted parameter set does not perform satisfactorily, the model establishment should be checked first to ensure that it adequately represents the important characteristics influencing flood behaviour. Only if this is found to be satisfactory should further effort be put into reviewing and adjusting model parameters.

7.2. Sensitivity Testing

Sensitivity testing of model platform parameters, uncertainties in input data and the model's schematisation (resolution) should be a regular part of a practitioners activities, especially for inexperienced practitioners, whilst calibrating a model. It also plays a useful role for establishing the uncertainty of uncalibrated models.

For models that are well-calibrated to a range of flood events and later verified, considerable confidence can be had in the model's ability to reproduce accurate flood levels. This in turn means that factors of safety such as the design freeboard applied to flood planning levels can be kept to a minimum.

However, for uncalibrated or poorly calibrated models less confidence can be had in the model's accuracy, and greater factors of safety (e.g. larger freeboards) should be applied to reflect the greater uncertainty (further discussion on uncertainty can be found in Book 7, Chapter 9). To quantify these uncertainties, sensitivity testing could be carried out where a model's calibration is non-existent or poor.

Examples of sensitivity testing to help quantify a model's uncertainty are:

- Adjust hydraulic roughness parameters values up and down by 20%;
- Adjust lag parameters;
- Increase inflows by 20%;
- For downstream boundaries, not at a receiving water body such as the ocean, vary the stage discharge or water level upwards to check that the water levels in the area of interest are not greatly affected;
- Apply blockages and greater losses to hydraulic structures and inlets; and
- Apply lower discharge coefficients across embankments such as roads.

Other useful sensitivity tests include:

- Making the model's resolution finer to check that results do not demonstrably change; and
- Varying the timestep and other computational parameters.

Sensitivity testing is also a very important part of developing a modeller's knowledge base and should be encouraged wherever possible.

> After a few weeks of pulling their hair out trying to calibrate to a well-defined flood mark in a house (the model was calibrating well elsewhere), the modellers called the owner of the house. After chatting for a while the owner suddenly remembered "my Dad had the house raised after that flood". Once the flood mark was adjusted by how much the house was raised, a good calibration was revealed! The modellers regretted not making that call a few weeks earlier…

Chapter 8. Application to Design

William Weeks, Erwin Weinmann

Chapter Status	Final
Date last updated	14/5/2019

8.1. Overview

Once the practitioner is satisfied with the calibration and validation, the next step in the application of a catchment modelling system is to apply it to the design problem. Australian Rainfall and Runoff is principally concerned with design flood estimation problems where floods of defined probabilities are required, but other applications are required for flood forecasting and warning or for assessment of impacts. A concern is that the calibration and validation processes concentrate on recorded historic flood events, whereas the design applications are more theoretical probabilistic events.

In the analysis of these probabilistic events, an important objective is to transfer probabilistic rainfalls into probabilistic flood levels, through the calculation of flood discharges. These design events are quite different from the historic floods and care must be taken in transferring the catchment modelling system application from the variable historic events into the design results.

As discussed in Book 7, Chapter 7, the parameters selected for application to the design conditions must be appropriate for the required application as well as consistent with the calibration to the available historic data.

There are therefore three conditions where model parameters may be required:

- *Historic Floods* - These are the floods where the data has been used to estimate parameters and to validate the models. Where there is more than one flood event, there may be a variety of conditions represented, with different spatial and temporal rainfall distributions possible. The flood events will sample a limited range of conditions that have applied during the period when data could be collected and these may not necessarily be representative of the conditions where the model must be applied. In addition, catchment conditions may have changed since the historic flood event occurred. Historic floods may also be analysed in the "design" application of models where flood impacts may be required to assess how development would have affected a historic flood for example.

- *Design Applications* - This is the main application where models discussed in Australian Rainfall and Runoff will be required and require results for floods of defined probability to be calculated. This is a more theoretical application than the analysis of historic floods and the parameters need to be established to ensure that the probability is calculated correctly. It is likely that the probabilistic floods calculated will be larger than the historic floods used to estimate the model parameters. The probabilistic design flood estimates must consider the relevant requirements. In some cases, flood peaks may be the only requirement, while flood hydrographs or flood volumes may be relevant at other times. There are different issues for each requirement.

- *Real Time Flood Estimation* - This is the requirement to use the model for a flood forecasting and warning application. This is different from the design application since timing is critical and the parameters must be available to carry out the analysis as the event is occurring. This is a far more complex application than the design situation, and while similar conditions apply in model application, this chapter concentrates on the design conditions.

This chapter concentrates on the probabilistic applications, though there are some similarities with the others.

8.2. Issues with Historical Calibration Floods

8.2.1. Introduction

Where parameters estimated from historical events must be transferred to design applications, a common concern is how representative the historic events are of the design flood events that must be estimated with the catchment modelling system.

8.2.2. Magnitude of the Calibration Events

A common issue is that the historic calibration events are relatively frequent, while the design applications may be needed for floods up to 1% Annual Exceedance Probability or rarer. It is therefore important that the model parameters selected for design application should still be appropriate for analysis of the rarer flood events.

A common assumption is that the model parameters calibrated on relatively frequent events remain constant for all rarer events. This may or may not be correct, so this assumption should be checked with regard to the representativeness of the calibration floods and with the model processes and whether there is a change in response for rarer flood events.

> In general constant parameters are recommended for the range of design events unless there is some evidence otherwise.

8.2.3. Calibration Event Conditions

The calibration events used to determine catchment modelling system parameters are generally all that is available, and the practitioner must apply the data from these events without the luxury of making a selection of the most appropriate events.

During calibration the practitioner must carefully review the properties of the historic floods used for calibration and determine how appropriate these are to be applied to the design problem. For example, the available calibration floods may be localised on a part of the catchment while the design flood event should be a more widely distributed event. On larger catchments, floods may be usually produced from a part of the catchment and the actual contributing section may vary from one event to another. The design case must therefore allow for the different catchment properties while estimating the probabilistic floods correctly. Some of these issues are discussed in Book 4 but there may be an impact on the transfer of the model parameters from the historic calibration events to the design flood events.

The calibrated parameters are for the situation/time when the calibration event occurred. However, there may be significant changes from one event to another. For example, in agricultural regions, the pattern of cropping may be different from one event to another. These varying catchment conditions may be considered in the individual calibration events, but they then need to be generalised for the design application. There are questions concerning how this is implemented. For example, sugar cane agriculture has areas of very high floodplain roughness in some locations and areas of fallow ground in other parts. These patterns vary from year to year and are difficult to determine for historic events. There is a question of the “average” conditions that should apply for the design application. A common approach is to adopt an average value of two very different conditions and then carry out some sensitivity tests to assess the impact of changes in the pattern of agriculture.

Similarly in arid areas, the antecedent conditions may have a major impact on catchment conditions for individual flood events, but these conditions then need to be represented in the design situation.

8.2.4. Applied Parameters

Calibration usually works with historic flood events while the design requirements are for probabilistic events. The parameters calculated for the historic recorded flood events may not be applicable to the design flood events and the results may not be consistent.

It is therefore important to confirm the model performance with probabilistic results. For situations where sufficient streamflow gauging is available, the model parameters can be confirmed using the Flood Frequency Analysis results to confirm that the model is representing the probabilistic flood discharges. Where there is insufficient streamflow records for a Flood Frequency Analysis, the model can be cross checked with a regional flood frequency results (Book 3, Chapter 3). Similarly, the model output can be confirmed with other anecdotal data, to confirm that the parameters are appropriate for the design application.

When applying hydrologic and hydraulic models to design situations, there are additional details that add complexity to the process. Often the historic floods are calibrated to the conditions that apply when the flood event occurred, so there are set values for antecedent conditions, losses, baseflow and the particular conditions that applied in the event, such as spatial or temporal patterns of rainfall. These additional factors are often not a part of the calibration process but must be incorporated into the design conditions.

8.3. The ARR Data Hub

The ARR Data Hub holds the design input data required for the application of ARR for design flood estimation. By inputting catchment centroid or a shape file you can download: the River basin, long duration ARF, Short duration ARF, storm loss value, pre-burst and temporal patterns. Entering the catchment outlet location allows the practitioner to download baseflow factors. As the data underlying ARR will progressively change as new data and techniques are available practitioners are recommended to visit the data hub at the start of each project. It is accessible at http://data.arr-software.org/. The data can be downloaded as a text file and included as an appendix to a project report.

Chapter 9. Uncertainty Determination

Rory Nathan, Erwin Weinmann

Chapter Status	Final
Date last updated	14/5/2019

9.1. Introduction

An overview of the various sources of uncertainty relevant to flood estimation and their treatment is provided in Book 1, Chapter 2. This guidance distinguishes between two broad types of uncertainty, namely:

- Aleatory (or inherent) uncertainty, which refers to uncertainty attributed to natural randomness or natural variability observed in nature; and,
- Epistemic (or knowledge-based) uncertainty, which refers to uncertainty attributed to incomplete/imperfect knowledge of a physical system (and hence its model), and to the inability to measure it precisely if at all.

Practical procedures for dealing with *aleatory uncertainty* are provided in Book 4, Chapter 4, whereas the focus of this chapter is on the assessment of *epistemic uncertainty*. Procedures for dealing with epistemic uncertainty for some specific methods are described elsewhere in ARR, and in particular it is worth noting the rigorous procedures provided for estimates of peak discharges using flood frequency and regional flood prediction methods, as described in Book 3.

It is perhaps a common view amongst practitioners that uncertainty analysis is too difficult to undertake. It is certainly true that assessing uncertainty takes additional time and effort, but there are uncertainty assessment frameworks with generic applicability to a range of practical problems (e.g. Pappenberger and Beven (2006); Pappenberger et al. (2006); Doherty (2016); Kuczera et al. (2006); Palisade Corporation (2015); Vrugt and Braak (2011)). For those with the necessary skills and interests, it is reasonable to assume that the effort required to become proficient with such tools will return benefits across a range of projects. That said, it would appear that specialists who are comfortable with uncertainty analysis tend to underestimate the depth of tacit knowledge required to implement and interpret such schemes, and the entry hurdle for many practitioners is a material one. Regardless, at this point in time it is acknowledged that the available hydrological and hydraulic models commonly used in Australia do not include the capability to assess uncertainty. It is expected that this situation will improve with time.

The intended audience for this Chapter are interested practitioners who do not have specialist training in the application of uncertainty techniques. The procedures described in this Chapter are not intended to cover the steps required to estimate the true uncertainty associated with input data, model parameters, and model structure. Rather, a small number of practical procedures are presented in the hope that these will allow practitioners to better understand (and communicate) the nature of uncertainty associated with selected key aspects of the flood estimates provided.

Book 7, Chapter 9, Section 2 discusses the role of sensitivity analysis in the assessment of uncertainty, and this is followed by a discussion (Book 7, Chapter 9, Section 3) of some simple analytical approaches relevant to error propagation. Book 7, Chapter 9, Section 4 discusses the application of Monte Carlo methods that can be used to assess uncertainty. Each method is supported by illustrative examples of their usage.

9.2. Sensitivity Analysis

Sensitivity analysis is a standard engineering technique that provides information on how model outputs are affected by changes in model inputs. Such analyses do not provide estimates of uncertainty, but they do provide a useful means of identifying which factors have greatest influence on the outcome. This insight, combined with some judgement regarding the relative accuracy of the different factors, can highlight which areas of analysis warrant further investigative effort. Importantly, such analysis can also reduce the dimensionality of subsequent uncertainty analysis so that effort is expended only on the factors of most importance.

There are a variety of ways that the sensitivity of an outcome to uncertainties can be represented, and two simple examples are shown in Figure 7.9.1. The tornado diagram provides a simple summary of the sensitivity of an outcome to reasonable estimates of upper and lower ranges, and the spider plot illustrates the dependence of the outcome on the percentage deviation of the key parameters from their adopted values.

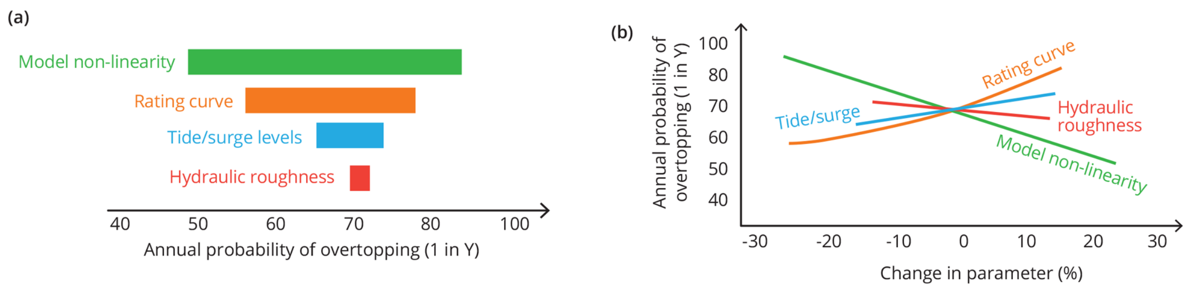


Figure 7.9.1. Representation of Relative Uncertainty of Outcome to Uncertainties Using a (a) Tornado Diagram and (b) Spider Plot

Changes in model input values can affect model outputs in different ways, and any dependency between inputs can mask the manner in which factors combine to influence the outputs. In addition, the nature of the factors which most influence the outcome may well vary with event magnitude. For example, the sensitivity to non-linearity in storage-routing models is dependent on the degree to which estimates are extrapolated beyond the magnitude of floods used for calibration; the reasonable range of estimates of roughness parameters in a hydraulic model may vary with the depths of flow considered. Accordingly, judgement needs to be used when selecting which factors to vary over a particular set of conditions, and care is needed to ensure that the range of values considered takes account of the possible dependencies.

Care also needs to be taken when considering the parameter ranges over which the sensitivity is assessed. The upper and lower limits of parameter values considered should reflect a similar range of notional uncertainty in each, otherwise misleading inferences may be drawn about the sensitivity of their impact on the outcome. More details on the uses and application of sensitivity analyses can be found in Loucks et al. (2005).

9.3. First Order Approximation Method

The purpose of introducing this simplified method of uncertainty analysis is to illustrate the general nature of how errors in model inputs and parameters can propagate through a model to produce errors in the model outputs. The first order error propagation method can be used in a similar fashion to sensitivity analysis to firstly identify the relative importance of different error sources and secondly to assess how the influence of different error sources changes with event magnitude and frequency.

9.3.1. General Approach

The equation for error propagation for a function *f* of independent variables *x, y, z* is (Haan, 2002):

$$s_f = \sqrt{\left(\frac{\partial f}{\partial x}\right)^2 s_x^2 + \left(\frac{\partial f}{\partial y}\right)^2 s_y^2 + \left(\frac{\partial f}{\partial z}\right)^2 s_z^2 + \ldots} \quad (7.9.1)$$

where s_f and s_x, s_y, s_z are respectively the standard deviations of the function and the independent variables. This approximation assumes that the errors are normally distributed and independent.

This error propagation equation can be used to gain an approximate indication of how errors in the independent variables translate into errors in the estimation results. The following example illustrates this and compares the errors in the estimates from three different methods.

9.3.2. Example: Flood Volume of an n-Day Flood Event with AEP 1 in T

A number of approximate methods can be used to estimate the flood volume V_x for a design flood event of given AEP and duration. Here the errors in volume estimates for three different methods are derived and compared. The assumed percentage errors in the inputs to the different methods have been selected somewhat arbitrarily but should be indicative of the expected error magnitude.

A. *Estimate derived using a transposition model*

If an estimate of the flood volume at gauged site *Y* is available from a frequency analysis of flood volumes, the flood volume (*V*) at ungauged site *X* can be estimated from a scaling relationship:

$$V_x = V_y\left(\frac{A_x}{A_y}\right)^m = V_y(R)^m \quad (7.9.2)$$

where *A* denotes the area of each catchment, *m* is a scaling parameter, and the subscripts refer to the individual catchments. Assuming that the estimate of the ratio of the areas of the two catchments (*R*) is error free but that the volume estimate at site *Y* (V_y) and the exponent of the scaling relationship (*m*) have errors s_V and *sm*, respectively, then the relative error in V_x can be calculated as:

$$\frac{s_{Vx}}{V_x} = \sqrt{\left(\frac{s_{Vy}}{V_y}\right)^2 + \left(ln(R) \quad s_m\right)^2} \quad (7.9.3)$$

Estimates of errors in the transposed flood volumes based on Equation (7.9.3) are provided in Table 7.9.1 for a range of representative input errors and parameter values. The results indicate that errors in the flood estimate at the gauged site are directly transferred to the estimate at the ungauged site, so errors increase as AEP decreases. Scaling to different catchment areas introduces little extra error as long as the catchment areas differ by no more than about 20 to 30% and the exponent in the scaling equation can be estimated to within about 10% accuracy. Scaling up and scaling down introduces similar errors.

Table 7.9.1. Errors Flood Volumes Estimated Using a Transposition Model for a Range of Assumptions.

AEP	Variables		Output/ Input	Input Errors		Output Error
R	m	V_x/V_y	s_{V_y}/V_y	s_m/V_y	s_{V_x}/V_x	
0.5 to 0.1	0.8	0.7	0.86	0.10	0.10	0.101
	0.8	0.9	0.82	0.10	0.10	0.102
	0.8	0.7	0.86	0.10	0.20	0.105
	0.5	0.7	0.62	0.10	0.10	0.111
	0.5	0.7	0.62	0.10	0.20	0.139
	1.25	0.7	1.17	0.10	0.10	0.101
	1.25	0.7	1.17	0.10	0.20	0.105
	2.0	0.7	1.62	0.10	0.10	0.111
	2.0	0.7	1.62	0.10	0.20	0.139
0.01	0.8	0.7	0.86	0.20	0.10	0.201
	0.5	0.7	0.62	0.20	0.10	0.206
	0.5	0.7	0.62	0.20	0.20	0.222
0.001	0.8	0.7	0.86	0.40	0.10	0.400
	0.5	0.7	0.62	0.40	0.10	0.403
	0.5	0.7	0.62	0.40	0.20	0.412

B. *Estimate derived using a runoff coefficient model*

This method assumes that a certain percentage of the average design rainfall depth *P* over the catchment for the given duration and AEP 1 in T is converted to a corresponding flood volume at the catchment outlet. The flood volume can thus be computed as

$$V_x = CAP \quad (7.9.4)$$

where *C* is a volumetric runoff coefficient and *A* is the catchment area.

Assuming that all the three variables have estimation errors associated with them, the relative error in the estimated flood volume can be approximated as:

$$\frac{s_{Vx}}{v_x} = \sqrt{\left(\frac{s_C}{C}\right)^2 + \left(\frac{s_A}{A}\right)^2 + \left(\frac{s_P}{P}\right)^2} \quad (7.9.5)$$

where s_{V_x} and s_C, s_A, s_P are respectively the standard deviations of the estimated volume and the three independent variables used in the estimate. Estimates of errors in the flood volumes based on Equation (7.9.5) are provided in Table 7.9.2 for a range of representative input errors and parameter values. The results show that the relatively large error in the volumetric runoff coefficient C dominates the error in the estimated flood volume.

Table 7.9.2. Errors Flood Volumes Estimated Using a Runoff Coefficient Model for a Range of Assumptions

AEP	Input Errors			Output Error
s_C/C	s_A/A	s_P/P	s_{V_x}	
0.5 to 0.1	0.5	0.00	0.1	0.51
	0.5	0.05	0.1	0.51
	0.5	0.05	0.2	0.54
0.01	0.4	0.00	0.2	0.45
	0.4	0.05	0.2	0.45
0.001	0.3	0.00	0.3	0.42
	0.3	0.05	0.3	0.43

C. *Estimate derived using a water balance model*

The flood volume can also be estimated from a water balance equation for the catchment over the duration of interest, in this case expressed in terms of design values for the different terms in the equation, (average depths over the catchment area A, in mm):

$$\frac{v_x}{A} = I - L + BF \tag{7.9.6}$$

where I is the design event rainfall, L is the total loss and BF the total baseflow contribution over the duration of the flood event. The loss and baseflow values are assumed to be invariant with AEP.

The relative error in the estimated flood volume can then be calculated as:

$$\frac{s_{v_x}}{v_x} = \frac{1}{v_x}\sqrt{s_I^2 + s_L^2 + s_{BF}^2} \tag{7.9.7}$$

Estimates of errors in the transposed flood volumes based on Equation (7.9.7) are provided in Table 7.9.3 for a range of representative input errors and parameter values. The results show that for frequent flood events the errors in the loss and baseflow values play an important role in the flood volume estimates, which can have large errors, while for very rare events the errors in estimated flood volumes are dominated by errors in the design rainfalls.

Table 7.9.3. Errors Flood Volumes Estimated Using a Water Balance Model for a Range of Assumptions

AEP	Input Variables			Output
I	L	BF	V_x	
0.5	65	40	20	45
0.1	100	40	20	80
0.01	150	40	20	130

AEP	Input Variables			Output
0.0001	500	40	20	480

Table 7.9.4.

AEP	Input Errors (Relative)			Input Errors (Absolute)				
s_I/I	s_I/L	s_{BF}/BF	s_l	s_I	s_{BF}	s_{V_x}	s_{V_x}/V_x	
0.5	0.2	0.3	0.4	13.0	12.0	8.0	19	0.43
0.5	0.1	0.2	0.3	6.5	8.0	6.0	12	0.27
0.1	0.2	0.3	0.4	20.0	12.0	8.0	25	0.31
0.1	0.1	0.2	0.3	10.0	8.0	6.0	14	0.18
0.01	0.2	0.3	0.4	30.0	12.0	8.0	33	0.26
0.01	0.1	0.2	0.3	15.0	8.0	6.0	18	0.14
0.0001	0.3	0.3	0.4	150.0	12.0	8.0	151	0.31
0.0001	0.2	0.2	0.3	100.0	8.0	6.0	100	0.21

Evaluation of the three models

The comparison of the error estimates from the three methods indicates that for relatively frequent events the transposition model (A) using an estimate based on flood frequency analysis performs best. Method (B) is dominated by relatively large errors in the runoff coefficient for all flood event magnitudes and frequencies. Method (C) performs best for rare to very rare events, where errors in the loss and baseflow play only a minor role.

9.4. Monte Carlo Simulation

9.4.1. General Approach

Monte Carlo simulation provides an alternative practical means for assessing how uncertainties in input parameters propagate through to the results of interest. Book 4, Chapter 4 describes the formulation and implementation of Monte Carlo procedures for the analysis of joint probabilities, and these same procedures may be applied to the assessment of uncertainty; however, rather than sampling from distributions representing natural variability, the stochastic samples are generated from distributions that characterise uncertainty in the inputs.

A general framework for how Monte Carlo simulation may be used to assess uncertainty is illustrated in Figure 7.9.2. The area of light-blue shading in this figure represents the main elements used to consider the joint interaction of the factors that are subject to natural variability (aleatory uncertainty), as discussed in detail in Book 4, Chapter 4 (Figure 4.4.7). The outer loop (in green) represents the additional simulations undertaken in which the parameters are stochastically sampled from distributions representing uncertainty in the inputs. That is, undertaking the inner loop of simulations yields an estimate of exceedance probability that a particular outcome might be exceeded (step D in Figure 7.9.2), and the outer loop provides an estimate of uncertainty of the derived quantile (step E). Of course, this framework could be simplified to provide an assessment of the uncertainty in the magnitude only of the outputs, in which case only the deterministic modelling within the blue shaded area is required (step C). The additional simulations required to consider epistemic uncertainty increases the number of simulations by up to two orders of magnitude. For example, if a stratified sampling scheme used 5000 simulations to derive a frequency curve of outputs, then around 500 000 simulations would be required to derive the corresponding 90% confidence limits.

Details of the simulation procedures required to undertake Monte Carlo simulation are provided in Book 4, Chapter 4. Two examples are provided below which illustrate application of these procedures. One example is used to assess the errors in the transposition of flood volumes (model A, as outlined in the preceding section), and the other extends the worked example presented in Book 4, Chapter 4, Section 4 for the analysis of concurrent tributary flows. The first example just considers the uncertainty in the magnitude of the outcome, the second considers the uncertainty in both its magnitude and frequency.

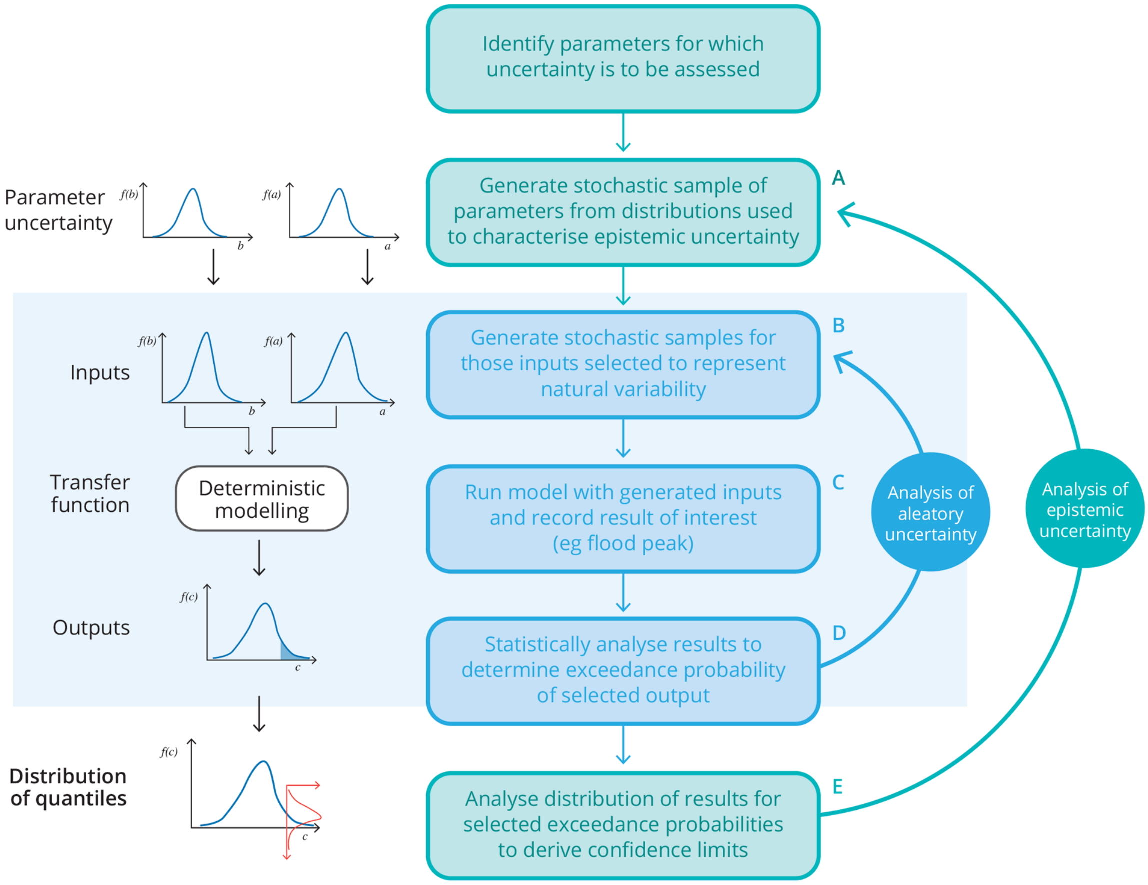


Figure 7.9.2. General Framework for the Analysis of Uncertainty using Monte Carlo Simulation.

9.4.2. Example: Flood Volume of an n-Day Flood Event with AEP 1 in T

A simple Monte Carlo scheme may be implemented in standard spreadsheet software to assess the propagation of uncertainty in the transposition example (Equation (7.9.2)), as shown in Figure 7.9.3. The approach is simply to generate a large number of normally distributed values about the mean estimates of m and V_y,, where the standard deviation of the sample reflects the magnitude of the errors. This is achieved by generating a sample of normally distributed values with a mean of zero and a standard deviation equal to s_m and s_v.

The steps required to do this are described in Book 4, Chapter 4, Section 3. For this example 2000 random numbers uniformly varying between 0 and 1 are generated for each of the variables V_y and m, as shown in columns 2 and 5 of Figure 7.9.3. The relative errors associated with V_y and m are assumed to be 10% and 20%, the ratio of catchment areas (R) is assumed to be 1.25 and the value of the exponent (m) is 0.7. The standard normal variates corresponding to the uniform random numbers are computed (columns 3 and 6), and these are multiplied by the selected variable values and their respective error terms (s_m, s_v) to yield 2000 stochastic values of V_y and m (columns 4 and 7). These steps yield a sample of values with a mean of zero and a standard deviation equal to their respective errors (s_m, s_v). Values of V_x are computed using Equation (7.9.2) for each pair of stochastically generated values of m and V_y (column 8). The standard deviation of these values represents the error about the mean estimate of V_x, and for the sample shown in Figure 7.9.3 this is found to be 3.11; when expressed as a proportion of the mean (0.108), this is similar to the result found by First Order Approximation, as shown in the 6th row of entries in Table 7.9.1.

The sample size is selected by trial and error such that successive estimates of the uncertainty change little with repeated stochastic samples. A sample size of 100 yields estimates of uncertainty that vary by around 10% of the mean value, and that obtained using a sample of 2000 vary by around only 1%.

Col 1	Col 2	Col 3	Col 4	Col 5	Col 6	Col 7	Col 8
No.	Uncertainty in V_y			Uncertainty in m			V_x
	N(0,1)	Zstd	V_y	N(0,1)	Zstd	m	
1	0.87	1.12	27.79	0.58	0.21	0.73	32.71
2	0.68	0.47	26.18	0.65	0.39	0.75	30.98
3	0.52	0.06	25.14	0.52	0.04	0.71	29.43
4	0.06	-1.51	21.21	0.70	0.52	0.77	25.21
5	0.84	1.00	27.51	0.57	0.17	0.72	32.33
6	0.12	-1.16	22.09	0.56	0.15	0.72	25.94
7	0.16	-1.01	22.47	0.00	-4.02	0.14	23.17
8	0.22	-0.78	23.05	0.96	1.73	0.94	28.44
9	0.32	-0.48	23.80	0.85	1.04	0.85	28.75
10	0.39	-0.27	24.32	0.06	-1.59	0.48	27.05
	⋮						⋮
1994	0.69	0.48	26.20	0.53	0.08	0.71	30.71
1995	0.39	-0.28	24.30	0.70	0.52	0.77	28.87
1996	0.20	-0.85	22.87	0.25	-0.69	0.60	26.17
1997	0.86	1.10	27.74	0.47	-0.08	0.69	32.35
1998	0.76	0.69	26.73	0.26	-0.64	0.61	30.64
1999	0.56	0.15	25.39	0.05	-1.66	0.47	28.18
2000	0.80	0.83	27.07	0.30	-0.53	0.63	31.12

R	1.25
m	0.7
s_m	0.2
s_v	0.1
Av(V_x)	28.79
s.d.(V_x)	3.11

Figure 7.9.3. Monte Carlo simulation of transposed flood volumes

9.4.3. Example: Flood level downstream of the confluence between two rivers

This example is based on the case study presented in Book 4, Chapter 4, Section 4. The example involves deriving a level frequency curve for a point below the confluence of two streams, where hydraulic modelling is used to estimate flood levels as a function of the coincident flood maxima.

The analysis presented in Book 4 demonstrates the use of the Total Probability Theorem in combination with a stratified sampling scheme to derive quantiles of flood levels below the confluence. The analysis presented here extends that original analysis, and shows how Monte Carlo simulation can be implemented in spreadsheet software to derive confidence limits for the derived flood levels. In essence, this example follows the framework illustrated in Figure 7.9.2, where the steps analysing aleatory uncertainty (blue shading) are described in Book 4, Chapter 4, Section 4, and those associated with the analysis of epistemic uncertainty are presented below.

The analysis is subject to three sources of uncertainty, namely the errors associated with:

- the parameters of the log-Normal distribution fitted to the flood maxima;
- the estimate of correlation between flood maxima in the two streams; and
- the estimates of the corresponding downstream flood levels from the hydraulic modelling.

The separate treatment of these uncertainties is discussed below.

Uncertainty in parameters of the flood frequency model

The assessment of uncertainty in the log-Normal distributions is undertaken by a parametric bootstrapping method. With this approach, stochastic samples are generated from the log-Normal distribution fitted to historical maxima, and new log-Normal distributions are fitted to each synthetic data set; the quantiles obtained from these synthetic parameters are then used to provide an estimate of uncertainty in the flood quantiles. The steps involved in this approach are:

1. Use the log-Normal distribution obtained from fitting to the N maxima in the historic record to generate a sample of N synthetic flood maxima (using the parametric sampling approach described in Book 4, Chapter 4, Section 3)

2. Fit a log-Normal distribution to this synthetic sample (ie calculate the mean and standard deviations of the logs of this sample)

3. Repeat steps i) and ii) 100 times to obtain 100 sets of log-Normal parameters, where the 90% confidence limits of the parameters are determined simply by calculating the 5% and 95% exceedance percentiles of each sample.

The above steps are applied separately to the flood data available for the mainstream and tributary. The resulting distributions of the parameters are shown in Figure 7.9.4. It is seen that the uncertainties in the tributary parameters are slightly wider than those of the mainstream, which reflects the shorter record length (30 years versus 50).

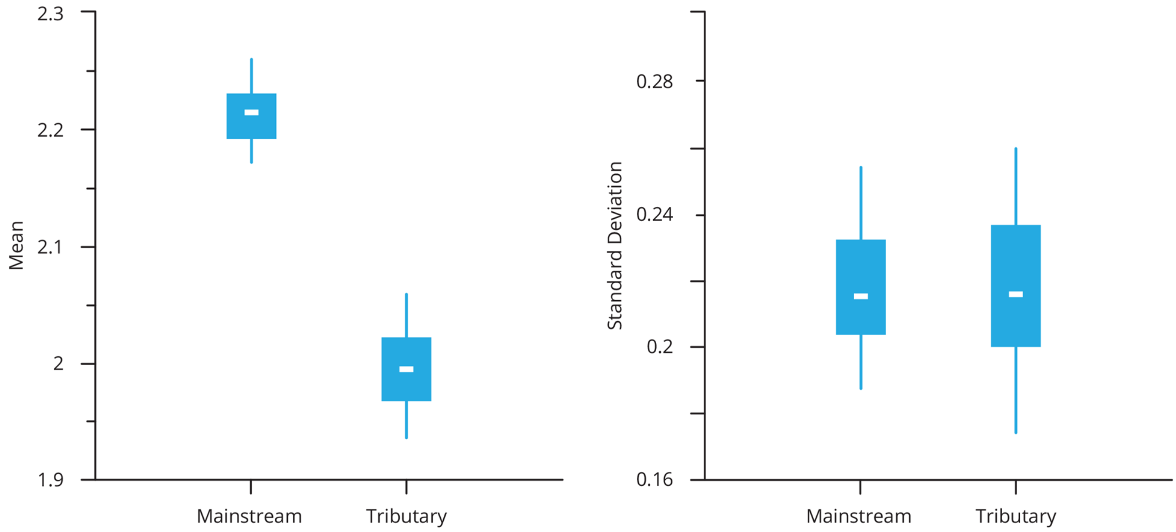


Figure 7.9.4. Uncertainty in parameters of the log-Normal distribution (high and low bars represent 5% and 95% limits, the high and low boxes represent 25% and 75% limits, and the central bar shows the median).

Uncertainty in correlation between flood maxima

The approach used to characterise uncertainty in the degree of correlation (*r*) between flood maxima in the two streams is similar to that used when errors are assumed to be Normally distributed (as described in Book 7, Chapter 9, Section 4). However, an additional transformation step is introduced to better conform to the assumed distribution of errors in estimates of the correlation coefficient (Fisher, 1915). Fisher's transformation of the correlation coefficient is approximately normally distributed with a mean (*r'*) and standard error (se'_r):

$$r' = \frac{1}{2}\ln\left(\frac{1+r}{1-r}\right) \tag{7.9.8}$$

$$se'_r = \frac{1}{\sqrt{N-3}} \tag{7.9.9}$$

The correlation between the log-transformed flood maxima in the two streams (*r*) is calculated to be 0.6, based on 30 years of concurrent data. The Fisher transformed estimates of *r'* and se'_r are thus calculated to be 0.693 and 0.192. With these calculated, the steps to generate a stochastic sample of correlations are as follows:

i. Generate a uniform random variate (*p*) between 0 and 1

ii. Compute the standard normal variate (*zi*) corresponding to *p*

iii. Obtain the quantile (g_i) corresponding to p from the inverse of the transformed normal distribution: $g_i = 0.693 + z_i 0.192$

iv. Apply the inverse of Fisher's transformation to g_i to obtain a stochastic estimate of the correlation coefficient (*ri*), where the inverse transform is calculated from the inverse of Equation (7.9.8):

$$r_i = \frac{e^{(2g_i)} - 1}{e^{(2g_i)} + 1} \tag{7.9.10}$$

v. Repeat steps i) to iv) 100 times to obtain a stochastic sample of correlation coefficients.

In this example, the mean of 100 correlation coefficients generated in this manner is found to be 0.603, where 90% of the sample is found to lie between 0.407 and 0.762.

Uncertainty in flood level estimates

A pragmatic approach is used to account for errors in the relationship between flows in the two streams and downstream flood levels. The approach is based on the simple assumption that the errors are normally distributed and invariant with magnitude, where the adopted standard deviation of the errors is 0.1m. The magnitude of the error term is based on the standard error of the regression relationship developed using hydraulic modelling, but this was increased slightly to reflect the additional uncertainty associated with the hydraulic modelling. The adopted approach could be modified to allow for errors in the slope of the fitted regression line and include dependency on flow magnitude, but this simpler approach provides a useful basis for exploring the sensitivity of the outcome to this source of uncertainty.

The steps involved in this are identical to that used in the preceding example in Book 7, Chapter 9, Section 4, as shown in Figure 7.9.3. An illustration of level estimates derived with the error term included is provided in Figure 7.9.5. The values on the x-axis correspond to the levels derived using the regression equation between upstream flows and levels simulated by the hydraulic model (Figure 4.4.15), and those on the y-axis include the normally distributed errors generated with a standard deviation of 0.1.

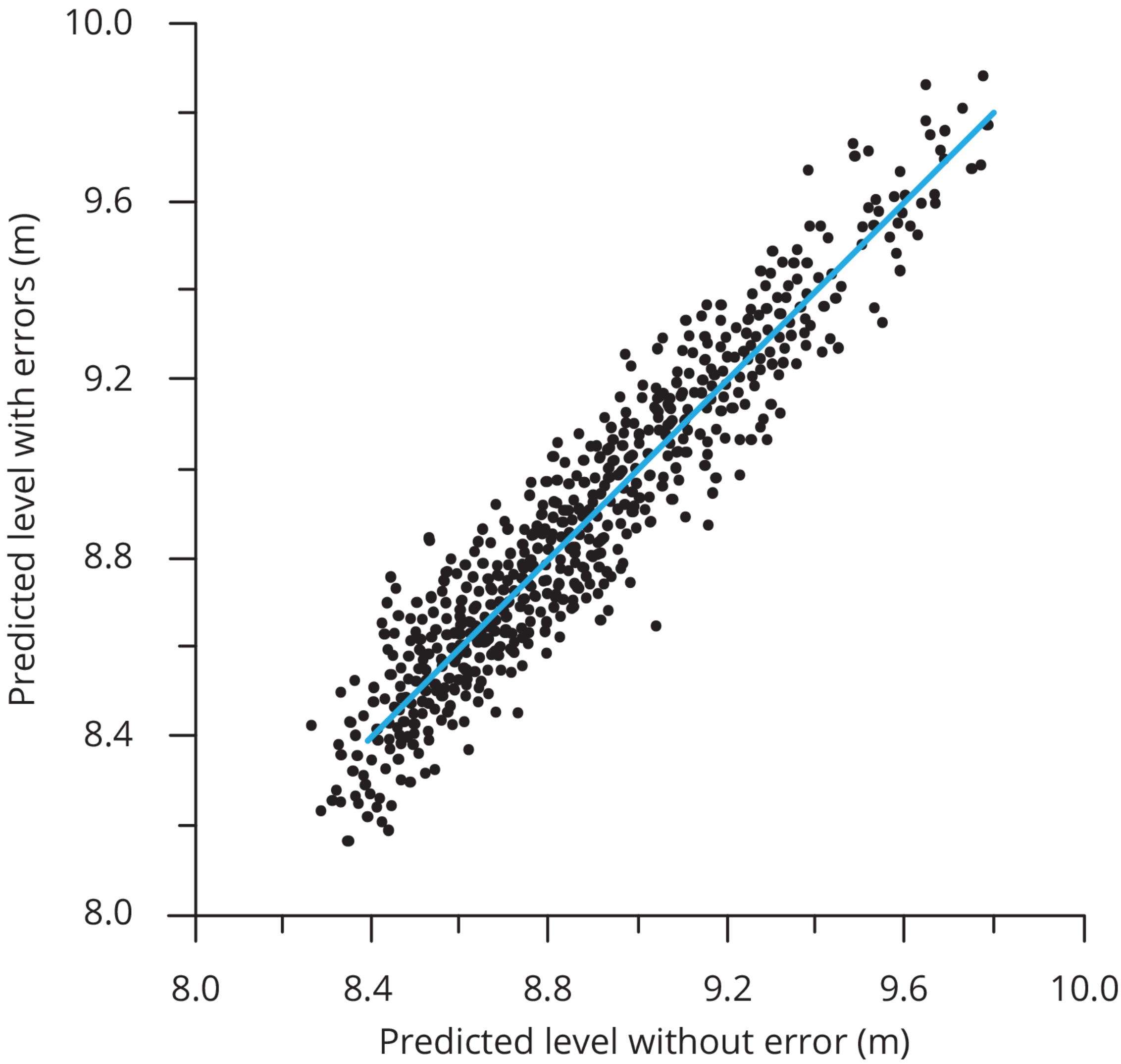


Figure 7.9.5. Uncertainty in levels estimated from the regression equation.

Uncertainty in derived level frequency curve

The final step required to assess the uncertainty in the derived frequency curve is to derive a frequency curve for each set of input parameters derived from the preceding three steps. That is, a flood level frequency curve is derived for the set of stochastic parameters generated in the preceding three steps, and the uncertainty in the design flood levels is obtained from the distribution of results.

The steps implemented to solve this using spreadsheet software are:

1. Generate 100 sets of stochastic parameters for the two log-Normal distributions and the correlation coefficient (this corresponds to step A in Figure 7.9.2)

2. For each set of parameters, generate 1000 stochastic samples of flood maxima in the mainstream and the tributary, using the procedure described in Table 4.4.1, then calculate the corresponding downstream flood levels from the regression relationship with a normally distributed error term added to the level estimates, as described above; these calculations correspond to steps B and C, shown in Figure 7.9.2.

3. Derive a flood level frequency curve by fitting a simple probability model to the 1000 stochastic maxima, as described in Table 4.4.1 (step D, Figure 7.9.2); this is used to estimate design levels for a range of exceedance probabilities.

4. Steps ii) and iii) are repeated for each of the 100 sets of stochastic parameters, which yields 100 estimates of design levels for each of the exceedance probabilities; these levels are ranked, and 90% of the range is used to represent uncertainty (step E, Figure 7.9.2).

The final level frequency curve and confidence limits derived using the above steps are shown in Figure 7.9.6.

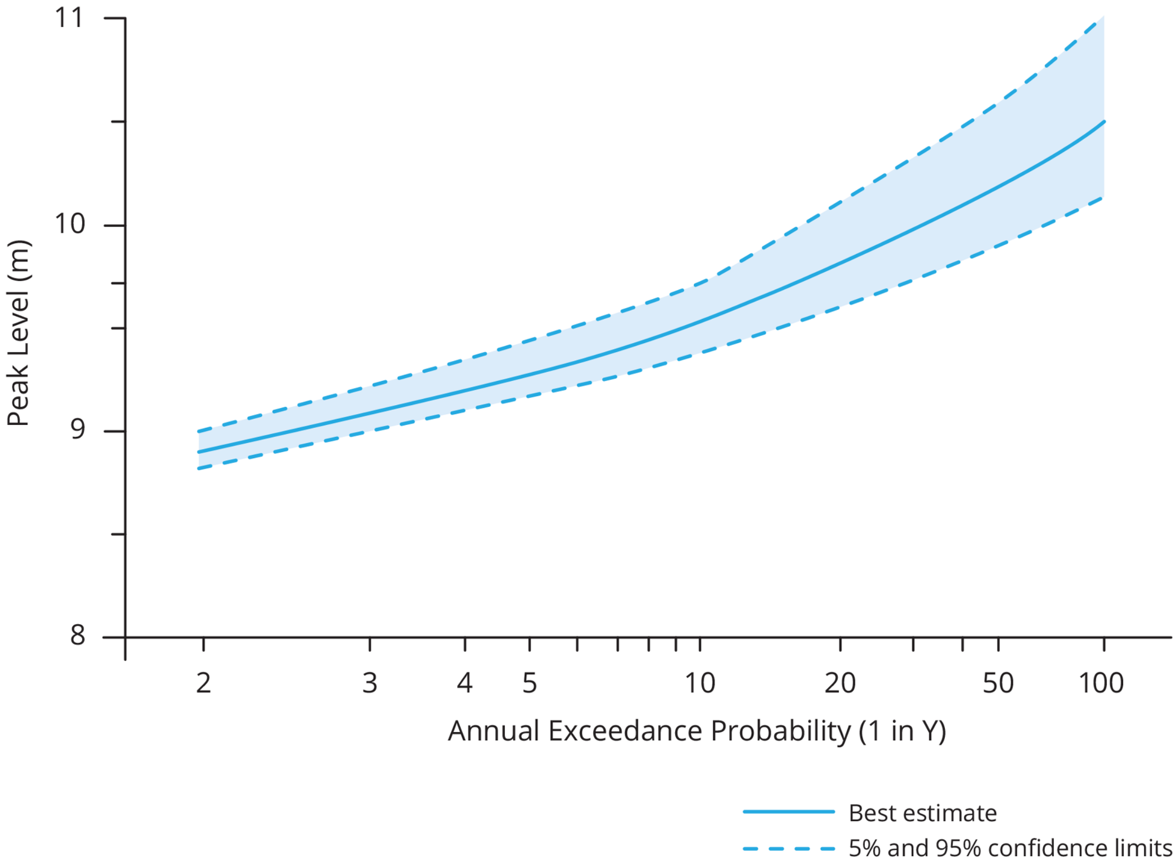


Figure 7.9.6. Confidence limits on the flood level frequency curve determined using the general framework for the analysis of uncertainty using Monte Carlo simulation.

9.5. References

Doherty, J. (2016), PEST, model-independent parameter estimation User Manual, Watermark Numerical Computing, http://www.pesthomepage.org.

Fisher, R.A. (1915). Frequency distribution of the values of the correlation coefficient in samples of an indefinitely large population. Biometrika (Biometrika Trust), 10(4), 507–521

Haan, C.T. (2002). Statistical Methods in Hydrology. The Iowa State University Press.

Kuczera, G., Kavetski, D., Franks, S.W. and Thyer, M. (2006), Towards a Bayesian total error analysis of conceptual rainfall-runoff models: Characterising model error using storm dependent parameters, J Hydrol, 331(1-2), 161-177.

Loucks, D.P., van Beek, E., Stedinger, J.R., Dijkman, J.P.M., Villars, M.T. (2005), Chapter 9 Model Sensitivity and Uncertainty Analysis, In Water Resources Systems Planning and Management: An Introduction to Methods, Models and Applications. Paris: UNESCO. doi:ISBN: 92-3-103998-9.

Palisade Corporation (2015), Risk analysis and simulation add-in for Microsoft Excel, User's Guide, version 7, Palisade Corporation, New York.

Pappenberger, F., Beven, K.J., (2006), Ignorance is bliss: Or seven reasons not to use uncertainty analysis. Water Resources 42, 1–8. doi:10.1029/2005WR004820.

Pappenberger, F., Harvey, H., Beven, K., Hall, J., Romanowicz, R., Smith, P. (2006), Implementation plan for library of tools for uncertainty evaluation. FRMRC Research Report UR2, Flood Risk Management Research Consortium, Manchester.

Vrugt, J.A., Ter Braak, C.J.F. (2011), DREAM(D): An adaptive Markov Chain Monte Carlo simulation algorithm to solve discrete, noncontinuous, and combinatorial posterior parameter estimation problems. Hydrol. Earth Syst. Sci. 15, 3701–3713.

Chapter 10. Documentation, Interpretation and Presentation of Results

Erwin Weinmann, William Weeks

Chapter Status	Final
Date last updated	14/5/2019

10.1. Introduction

Catchment modelling systems for flood estimation are applied to provide information to decision makers and designers on magnitudes and probabilities of flood characteristics, as a basis for decisions on flood-related planning, design and operations. The purpose, scope and required outputs of any flood investigation should be clearly described in the brief or technical specification for the design problem or flood study (NFRAG, 2014; McLuckie and Babister, 2015). It is therefore important that the client who commissions flood investigations should be comprehensive in the preparation of the brief to ensure that all requirements and objectives are covered in detail. The brief should be detailed but should not specify unrealistic objectives for the model performance. Unrealistic objectives may include over-optimistic calibration performance.

The results of any modelling should be documented and presented in a way that satisfies the requirements of the brief. However, even if such a brief or specification is not readily available, it is the responsibility of the modelling team to ensure that the modelling process is well documented and that the results are presented and communicated in a way that will be clearly understood by the target audience and will avoid any misinterpretation or misuse of the information. The documentation may need to cover requirements for several different audiences in particular circumstances, so it must be relevant for these audiences. In some cases, different reports may need to be prepared for these varied audiences.

10.2. Audience Considerations

Depending on the project and the specific requirements of the specification, the documentation should cater for the required audiences. Different audiences could include:

- *Client* - The client is the agency that has commissioned the flood report, and they will be seeking a report that outlines the whole scope of the report, especially covering the main issues required, as well as limitations and comments on accuracy and reliability. This report will be the basis for the client's requirements, whether this is for planning, feasibility or design of infrastructure. The report should also clearly demonstrate the methodology and show that it was appropriate for the requirements, subject to the limitations of the specification. The client will also need to have a report that will be archived in their technical library and be available for reference in the future when the flood study may be reviewed or if later queries arise. All supporting data should also be archived by the client for future reference.

- *Regulatory or Approval Agencies* - Where the client is not itself a regulatory agency, these agencies need to be considered. For example, these may include agencies such as local authorities who need to consider impacts of projects on flood levels outside the project boundary, or environmental agencies who may need to understand any impacts on water quality or fauna movement. The report needs to demonstrate to these agencies that the flood study has been carried out to an acceptable technical standard and that their interests are satisfied.

- *Residents and the Public* - Local residents will take an interest in the findings of flood studies, particularly as they affect their individual interests. To meet their interests, the report should be written in plain English, though still to a high level of technical credibility, and should clearly outline the impacts on the local community and demonstrate that any adverse impacts have been mitigated or, if this proves impossible, demonstrate that all efforts have been made to minimise impacts.

- *Other Stakeholders* - These may include local community or environmental groups, who have no direct regulatory interest but who have a community interest in the results of the flood study. In this case, the report must be written in plain English but it must also be of a high technical standard, since these stakeholders will often have a high level of technical expertise.

10.3. Documentation

10.3.1. General

Documentation should be progressive through the different steps of a flood estimation study. The scope and level of detail of the documentation will depend to some degree on the nature of the modelling application but should be sufficient to provide the basis for an independent review of the modelling process and the results produced (DECC, 2007).

As discussed above the documentation needs to consider the requirements of the audience for the report, noting that there may be more than one audience. The documentation requirements outlined below apply to both the hydrologic and hydraulic modelling phases of a flood study. More detailed guidance on interpretation of the results of hydraulic modelling is provided in the ARR Project 15 report (Babister and Barton, 2016).

10.3.2. Data Collation and Quality Checking

The data used in the model development and study is the basis for the work, and a clear description and documentation of this data is essential for review and understanding of the process as well as for archiving and future reference. This documentation should cover all forms of data used from systematically recorded or surveyed data to informal sources of flood information, including historic records of rainfall, streamflow, flood level, flood extent data, topographic and survey data, as well as photographic and documentary information on floods (see Book 1, Chapter 4). It is recommended that the project report include a copy of the design input data downloaded from the ARR Data Hub (http://data.arr-software.org/) to aid in the reproducibility and review of results.

It is important that the process of data quality checking and the associated decisions are clearly recorded, as well as any assumptions or limitations. The documentation should clearly describe the approach to checking the data and indicate a descriptive understanding of the data quality and the impacts of this quality on the final outcomes of the project. To the extent that data ownership allows, a copy of the original data sets and the finally adopted data sets should be kept.

10.3.3. Model Development and Calibration

The documentation should cover all the stages of the model development, including the selection of the catchment modelling system, the key assumptions made in the model representation of the catchment or the flooded area, the selection of model parameter and design inputs, and the process used to ensure that the model is fit for the intended purpose. Key decisions made in this process should be clearly recorded. Comments on the parameter estimation process and the expected reliability of the results should also be included.

It is now quite common in flood study briefs to include as part of the study deliverables a requirement to provide a copy of the calibrated model (NFRAG, 2014). This should include the relevant information to allow a third party to run the model and review the modelling results. More details are provided in Babister and Barton (2016).

10.3.4. Modelling Results

Records of modelling results should include clear documentation of the scenarios, parameters and design inputs for the model runs. Electronic records of results should be in a format that allows ready processing for summaries and reports.

The modelling results should be supported by maps and graphs which can illustrate the procedures and methodology. Maps are an excellent means of allowing a comprehensive but easily understood interpretation of the results.

10.4. Interpretation of Modelling Results

10.4.1. Model Representation vs Reality

Hydrologic and hydraulic models are simplified representations of reality that are developed to allow assessment of flood problems, the final step in any form of modelling is therefore the interpretation of the modelling results in the light of the assumptions and simplifications made in the model formulation and any other limitations that might affect the modelling results. This can be seen as the reverse of the process of representing the real catchment and floodplain by a simplified, conceptualised model. The practitioner is in the best position to assess the impacts of the simplifications of the real system in terms of the uncertainties and potential bias introduced into the modelling results and it is thus the practitioner's responsibility to communicate the results of this assessment.

10.4.2. Checking of Results

The documentation for studies should describe the checking of results that has been carried out. This checking covers a number of formal and informal processes and must ensure that the client and other readers have confidence in the conclusions and are satisfied that the model and results are as consistent as possible with reality. This checking can also assist clients in model applications and any limitations.

As discussed elsewhere in Australian Rainfall and Runoff, inaccuracies can result from a number of sources including:

- *Data Quality* - The quality of the hydrologic and hydraulic modelling depends on the quality of the local data used in the development and testing of the model.
- *Model Representation* - The model is a theoretical representation of reality and the quality of this representation should be indicated.

- *Model Extrapolation* - The model will be developed using certain available data for calibration or using regional parameter estimates. The application to design situations then requires extrapolation to larger floods or alternative catchment development scenarios. The quality of the model extrapolation into these alternative conditions should be reviewed.

All of these issues should be described in the study documentation.

The process for checking the performance of the model with these concerns will need to focus firstly on the basis of the model development and implementation. Secondary checks, which are equally important should focus on the results, where there are several approaches to checking.

Developing a process for checking that model results are sensible and consistent is a vital quality control measure for the practitioner. The practitioner needs to satisfy themselves that the model results are reasonable prior to publishing them in a report. The following is a checklist that the practitioner should consider when interpreting results:

- *Mass Balance* – errors greater than 1% to 2% should generally be investigated, and the cause of the errors identified and rectified where possible;

- *Runoff Volumes* – the total runoff as a percentage of rainfall volume should be determined and checked against typical runoff coefficients for similar catchments;

- *Runoff Rates* – can be used to check that the runoff rates predicted by the hydrologic model do not significantly diverge from runoff rates predicted by the hydraulic model. If divergence is significant, reason(s) for such should be determined.

- *Continuity* – discharge hydrographs should be obtained at several locations along each flow path, and at locations upstream and downstream of major flow path intersections, to check that the continuity and attenuation of flows is reasonable;

- *Stability* – the results should be checked for signs of instability, such as unrealistic jumps or discontinuities in flow behaviour, oscillations (particularly around structures or boundaries), excessive reductions in time step or iterations required to achieve convergence. Many models will specify criteria based on the Courant number (refer to Book 6) that can be checked to assess model instability;

- *Froude Numbers* – Froude numbers should be checked to identify areas of trans-critical and super-critical flow, and the implications of this flow behaviour on the model results considered. In general, model results in areas of trans-critical flow should be used with extreme caution. Flow over embankments, levees and other hydraulic control structures should be roughly checked with suitable hand calculations, such as the broad-crested weir equation;

- *Model Startup* – many models do not perform well from a completely “dry” start during the initial wetting stage. The practitioner should consider using a suitable “hot-start” condition if such functionality exists, or should exclude results from the very start of the model run from their analysis. This can be particularly important near structures;

- *Structure Head Losses* – head losses through structures such as bridges, culverts, siphons etc should be checked against suitable hand calculations. More discussion on how to deal with structures is presented in Book 6, Chapter 3. In particular, consideration should be made of the amount of expansion/contraction losses that are captured by the two dimensional schematisation, and whether the flow regime is adequately handled by the model; and

- *Steep areas/shallow flow* – it may be difficult to interpolate flow depths where steep shallow flow is occurring, particularly if the flow is not sub-critical. It may be necessary to check results against total energy calculations in such locations.

Results for similar projects in the vicinity should be reviewed to ensure that the results are consistent with these previous analyses. If there are differences, reasons for these differences should be sought and explained. If this is not the case, reconsideration of the model selection or implementation should be considered.

Alternative flood estimation methods, generally a simple regional method should also be considered again to check consistency. Again where there are inconsistencies, these should be investigated and reasons found for the differences.

These checks of results are important and increase confidence in the analysis. The flood study documentation should clearly outline this checking and demonstrate the level of confidence in the results.

10.4.3. Accuracy of Results

Book 1, Chapter 2, Section 8 gives information on the sources of uncertainty and Book 7, Chapter 9 of this book provides guidance on methods for determining uncertainty in modelling results. However, the formal sensitivity or uncertainty analysis will generally only cover the influence of the most important inputs and parameters on the modelling results. The practitioner thus needs to consider the likely magnitude of additional uncertainties introduced by secondary inputs and parameters.

The degree of scatter in results shown up by uncertainty analyses describes the *precision* of the flood estimate. However, the accuracy of modelling results depends also the degree of *bias* in the results (systematic underestimation or overestimation). Inappropriate representation of the real system by the adopted model is likely to introduce model errors (additional uncertainty and bias) into the modelling results, which are not captured by normal uncertainty analysis. The results of uncertainty analyses should thus be regarded as lower bound estimates of uncertainty.

An estimate of the likely model errors can be obtained by comparing results produced by different models of the same system or by comparison of flood estimates obtained by different flood estimation approaches.

The documentation must include sufficient discussion to allow the client and others who read the report (including non-experts) to understand the level of accuracy provided and to ensure that the report is not used to indicate a higher accuracy than can be justified by the model and the particular application of model calibration. To avoid misinterpretation, modelling results should be presented to the number of significant figures implied by accuracy considerations. Where there is uncertainty, this must be described clearly and understandably so that the client and others can make a reasonable decision on the results.

10.5. Presentation of Results

10.5.1. General

Depending on the nature and scope of the flood investigation, the modelling results may be presented in the form of a summary table of flood estimates, graphs, detailed reports, maps, audio-visual presentations or combinations of these elements. In all cases it is important that the form and detail of the presentation is directed at the target audience. Generally different forms and levels of presentation of study findings will be required for different stakeholder groups.

In addition to the summary of results, the documentation should include comments on the accuracy and reliability of the results. It should cover the basic discussion of calibration to historical flood events as well as extrapolation of the model to the design scenarios and to assessments beyond the scope of the calibration.

10.5.2. Qualifications and Caveats

The scenarios used in deriving the flood estimates need to be clearly stated, including the assumptions made with regard to climate and land use conditions, and possibly other system characteristics (e.g. operational conditions).

The modelling will have been developed for a specific application and therefore the model performance for other applications may be limited. This limitation could include the geographical extent as well as the flood magnitudes considered. For example, if the model has been developed for design of major infrastructure, it may be prepared for analysis of large floods, so the calibration may be inappropriate for small in-channel flows which may be required for another application.

The documentation therefore should clearly describe the limitations and the scope where the model results may be appropriate.

10.5.3. Use of Modelling Results in Decision Making

While the main interest of the stakeholders is mainly on ‘best estimates’ of the flood characteristics as the direct basis for flood maps and other regulatory instruments, reporting on the uncertainties attached to these ‘best estimates’ is important as a basis for decision making. This additional information and comments on the interpretation of the modelling results (Book 7, Chapter 9, Section 3) are essential inputs to risk assessment and risk management studies that will use the modelling results (Book 1, Chapter 5).

10.6. References

Babister, M. and Barton, C. (eds) (2016). Australian Rainfall and Runoff Support Document: Two dimensional modelling in urban and rural floodplains. Project 15

DECC (NSW Department of Environment and Climate Change), (2007), Floodplain Risk Management Guideline, Residential Flood Damage, Sydney.

McLuckie, D., Babister, M., (2015), Improving national best practice in flood risk management, Floodplain Management Association National Conference, Brisbane, May 2015.

National Flood Risk Advisory Group (NFRAG) (2014), Guideline for using the national generic brief for flood investigations to develop project-specific specifications, National Flood Risk Advisory Group and Australian Emergency Management Institute, Australian Emergency Handbook Series.

BOOK 8

Estimation of Very Rare to Extreme Floods

Estimation of Very Rare to Extreme Floods

Table of Contents

1. Introduction 1
   1.1. Scope and Intent of the Book 1
   1.2. Applications covered by this Book 1
   1.3. Event Classes 2
      1.3.1. General Limits 2
      1.3.2. Rare Events 3
      1.3.3. Very Rare Events 3
      1.3.4. Extreme Events 5
   1.4. Relationship with Other Sections of Australian Rainfall and Runoff 5
      1.4.1. Specific Focus of Book 8 5
      1.4.2. Terminology 6
      1.4.3. Risk-Based Design 6
   1.5. References 6
2. Procedures for Estimating Very Rare to Extreme Floods 7
   2.1. Overall Design Approach 7
      2.1.1. Flood Frequency Analysis 7
      2.1.2. Rainfall-Based Procedures 7
   2.2. Procedures for Different Categories of Design Floods 8
   2.3. Relevance of Procedures to Specific Applications 12
3. Estimation of Very Rare to Extreme Rainfalls 15
   3.1. General 15
      3.1.1. Overview of Requirements and Sources of Design Rainfall Information 15
      3.1.2. Uncertainty in Design Rainfall Estimates 16
   3.2. Estimation of Very Rare Design Rainfalls 16
   3.3. Estimation of the Probable Maximum Precipitation Depth 16
   3.4. Assigning an Annual Exceedance Probability to the Probable Maximum Precipitation 18
      3.4.1. Background 18
      3.4.2. Regional Recommendations 19
      3.4.3. Site-Specific Estimation 21
   3.5. Estimation of Extreme Rainfalls 22
      3.5.1. General 22
      3.5.2. Interpolation Procedure 23
   3.6. Estimation of Rainfall Depths for Non-Standard Durations 26
      3.6.1. General 26
      3.6.2. Very Rare Rainfalls for Intermediate Durations 26
      3.6.3. Very Rare Event Rainfalls for Short Durations 26
      3.6.4. Rainfalls for Very Rare to Extreme Events of Very Long Durations 27
   3.7. Seasonal Estimates of Rare to Extreme Rainfalls 28
      3.7.1. Theoretical and Practical Issues 28
      3.7.2. Derivation of Seasonal Design Rainfalls 30
   3.8. Temporal Patterns 31
      3.8.1. General 31
      3.8.2. Specific Recommendations 32
   3.9. Spatial Patterns 34
      3.9.1. Basis of Adopted Patterns 34
      3.9.2. Specific Recommendations 34
   3.10. References 36
4. Estimation of Rainfall Excess for Very Rare to Extreme Events 39
   4.1. General Considerations 39

4.1.1. Importance of Design Losses – Very Rare to Extreme Events 39
4.1.2. Losses Associated With Design Storms and Design Bursts 40
4.1.3. Variation of Loss Values with Event Magnitude 40
4.1.4. Variation of Design Losses with Season 41
4.1.5. Consideration of Joint Probabilities 42
4.2. Methods for Derivation of Design Loss Values 42
4.3. Guidelines for Selection of Design Loss Values 42
4.3.1. General 42
4.3.2. Rural Initial Loss Values for Use with Design Bursts 43
4.3.3. Rural Initial Loss Values for Use with Design Storms (IL_s) 44
4.3.4. Rural Continuing Loss Values (CL) for use with Design Bursts and Design Storms 45
4.3.5. Loss Recommendations for Urban Catchments 46
4.4. References 46
5. Selection, Configuration, and Calibration of Hydrograph Models 48
5.1. General 48
5.2. Model Features and Capabilities Required to Estimate Very Rare to Extreme Events 49
5.2.1. Considerations in Model Selection 49
5.2.2. Basic Model Requirements 49
5.2.3. Enhanced Model Capabilities 50
5.3. Model Configuration 50
5.3.1. General Considerations 50
5.3.2. Specific Issues 50
5.4. Determination of Model Parameter Values for Rural Catchments 51
5.4.1. General 51
5.4.2. Parameter Determination for Gauged Catchments 52
5.4.3. Design Parameter Values for Ungauged Catchments 53
5.4.4. Physically-based Extrapolation of Model Parameter Values for Extreme Events 54
5.4.5. Specific Recommendations for Modelling Extreme Events 56
5.5. Model Parameterisation for Urban Catchments 57
5.6. References 59
6. Derivation of Design Floods 60
6.1. Overview 60
6.1.1. Selection of Basic Procedure 60
6.1.2. Design Flood Characteristics 60
6.1.3. Expected Probability Adjustment 61
6.1.4. Applications Requiring Special Considerations 62
6.2. Flood Data Based Estimates 62
6.2.1. General 62
6.2.2. Applications of Results of Flood Frequency Analysis 62
6.2.3. Incorporation of Paleohydrological Estimates 63
6.2.4. Preliminary Estimate of Rare to Extreme Events 63
6.3. Rainfall Based Estimates 65
6.3.1. General 65
6.3.2. Surface Runoff Hydrographs 65
6.3.3. Incorporation of Baseflow 65
6.3.4. Simulation Framework 66
6.3.5. Derivation of Complete Design Flood Frequency Curve 66
6.4. Estimation of the Probable Maximum Flood 68
6.4.1. Design Context 68

6.4.2. General Guidance ... 69
6.4.3. Checks on Upper Limiting Magnitude ... 70
6.4.4. Assessment of Reasonableness for Design Estimates ... 70
6.5. Treatment of Uncertainty ... 72
6.6. References ... 72
7. Special Design Considerations ... 75
7.1. General ... 75
7.2. Derivation of Reservoir Outflow Frequency Curves ... 75
7.2.1. Importance of Reservoir Storage and Initial Drawdown ... 75
7.2.2. Approximate Methods - Representative Initial Storage Volume ... 76
7.2.3. Joint Probability Analysis of Inflow and Initial Storage Volume ... 76
7.2.4. Consideration of Cascade of Storages ... 80
7.3. Concurrent Tributary Flows ... 82
7.3.1. Overview ... 82
7.3.2. Stochastic Simulation ... 83
7.3.3. An Approximate Approach ... 84
7.4. Seasonal Design Floods ... 85
7.4.1. The Need for Seasonal Estimates ... 85
7.4.2. Theoretical and Practical Issues ... 85
7.5. Consideration of Snowmelt ... 86
7.5.1. Overview ... 86
7.5.2. Selection of Snowmelt Model ... 87
7.5.3. Application to Extreme Events ... 87
7.6. Consideration of Long Duration Events ... 88
7.7. Impact of Climate Change ... 88
7.8. References ... 89
8. Worked Examples ... 92
8.1. The Design Problem ... 92
8.1.1. General ... 92
8.1.2. Approach Adopted and Intent ... 92
8.1.3. Nature of Available Data ... 92
8.1.4. Note on Accuracy of Final Results ... 93
8.2. Derivation of Rainfall Frequency Curves ... 93
8.2.1. Estimates of Rare to Very Rare Rainfalls ... 93
8.2.2. Estimates of Extreme Rainfalls ... 94
8.2.3. Interpolation of Rainfall Depths for Intermediate Durations ... 96
8.3. Derivation of Flood Frequency Curve ... 96
8.4. Joint Probability Analysis of Initial Reservoir Level ... 97
8.4.1. System Characteristics ... 97
8.4.2. Laurenson's Analytical Solution ... 99
8.4.3. Monte Carlo Solution ... 102
8.5. Estimation of Concurrent Flows ... 103
8.5.1. Basic Flood Data ... 103
8.5.2. Fitting of log-Normal Distribution ... 104
8.5.3. Estimation of Concurrent Tributary Flows ... 105
8.6. References ... 106

List of Figures

8.1.1. Design Characteristics of Notional Event Classes 3

8.2.1. Qualitative Indication of the Range of Flood Magnitudes and the Relative Degree of Reliability Required for Different Applications 13

8.3.1. Generalised Long-Duration Probable Maximum Precipitation Method Zones (Bureau of Meteorology, 2006) 18

8.3.2. Recommended Regional Estimates for the AEP of PMP 20

8.3.3. Schematic Illustration of Interpolation Procedure 24

8.3.4. Hypothetical Frequency Curves for Seasonal and Annual Design Rainfalls Based on the AEP Assigned to the Annual PMP (adapted from Laurenson and Kuczera (1998), using Four Seasons of Relative Lengths 0.33, 017. 0.33, 0.17 and Relative Seasonal PMP Depths of 1.0, 0.85, 0.6, 0.85, for Summer, Autumn, Winter, and Spring, respectively) 30

8.6.1. Illustration of Derived Frequency Curve Based on Reconciliation with Flood Frequency Quantiles 68

8.6.2. (a) Differential Importance of "Reasonableness" in PMF Assumptions on Dam Safety Decision-Making, and (b) Use of Simple Extrapolation to Infer Degree of Reasonableness 71

8.7.1. Schematic Illustration of the Determination of the Probability Interval of Storage Volume as a Function of Inflow and Outflow 78

8.7.2. Illustration of Monte Carlo Framework to Derive Outflow Frequency Curve 80

8.7.3. Illustration of Conditional Empirical Sampling in Which the Storage Volume in an Upstream Dam is Correlated with the Volume in a Downstream Dam 82

8.7.4. Illustration of the Generation of Variables with a Correlation of 0.7 Based on Normal Distributions 84

8.7.5. Schematic Diagram Illustrating the Conversion of Seasonal Exceedance Probabilities into Annual Estimates 86

8.8.1. Example Rainfall Frequency Curves 95

8.8.2. (a) Simulation Framework used to Generate Floods for Selected Stochastic Inputs and (b) Resulting Flood Frequency Curves 97

8.8.3. Inflow-Outflow-Storage Volume Relationship 98

8.8.4. Probability Distribution of Initial Storage Volume 99

8.8.5. Transition Probabilities between Reservoir Inflow and Outflow Classes 101

8.8.6. Outflow Frequency Curves Obtained using Joint Probability Analysis and a Median Level of Drawdown 102

8.8.7. Simulation Framework to Derive Outflow Frequency Curve Based on Variable Initial Starting Level in Reservoir 103

8.8.8. Calculation of Concurrent Tributary Flows 104

8.8.9. Fitted log-Normal Flood Frequency Curves for Mainstream and Tributary Design Flows 105

List of Tables

8.1.1. Limit of Credible Extrapolation for Different Types of Data in Australia (modified after USBR (1999)) 4
8.2.1. Summary of Procedures to Derive Design Rainfalls 9
8.2.2. Summary of Procedures to Derive Design Floods 11
8.3.1. Subjective Probability Mass Function for Describing Uncertainty in Regional Estimate of the AEP of PMP (Adapted from (Laurenson and Kuczera, 1999)) 20
8.3.2. Growth Curve Factors for Derivation of Sub-Daily Design Rainfalls Standardised by the 1 in 100 AEP Rainfall Depth) 27
8.3.3. Selection of Design Burst Temporal Patterns for Different Regions, Durations and AEPs 33
8.3.4. Selection of Design Spatial Patterns for Different Regions, Durations and AEPs 35
8.4.1. Recommended Loss Rates for Urban Catchments 46
8.8.1. Calculation of Areal Design Rainfalls for Rare to Very Rare Events 93
8.8.2. Parameters Calculated of Areal Design Rainfalls for Very Rare to Extreme Events . 94
8.8.3. Calculation of Areal Design Rainfalls for Very Rare to Extreme Events 95

Chapter 1. Introduction

Rory Nathan, Erwin Weinmann

Chapter Status	Final
Date last updated	14/5/2019

1.1. Scope and Intent of the Book

In the past two decades, there has been an increasing focus on the derivation of floods with very low probabilities of exceedance (often termed very rare flood events). Information on these floods is required in many aspects of civil engineering, including floodplain management and the design of major infrastructure (e.g. roads, bridges, and railways). The assessment of flood risk is of particular importance to the safe design, maintenance, and operation of dams. The safety and economic implications of these flood estimates accentuate the desirability of using similar or compatible procedures by all Australian authorities and designers.

The floods under consideration in this Book are events with an Annual Exceedance Probability (AEP) of between 1 in 100 and 1 in 10^7. The emphasis of this Book is on the estimation of a flood frequency curve between these limits as inputs to risk-based design rather than on the estimation of a design flood of specific AEP and/or magnitude. The absolute upper limit of flood magnitude under consideration is the Probable Maximum Flood (PMF), which is a design concept that cannot be readily assigned an AEP.

These Guidelines are intended to provide a clear statement of what constitutes best practice in sufficient detail to enable the procedures to be applied to practical problems. Best practice in this field is constantly evolving, and thus this Book focuses on the general principles that should be considered when estimating extreme floods rather on the detailed description of prescriptive procedures. It should be noted that this Book is aimed at practitioners with some experience in the field, rather than at people new to extreme flood estimation. Worked examples are provided to illustrate some of the design concepts involved, but overall the thrust is to state what best practice is, not to explain in detail how to achieve it.

1.2. Applications covered by this Book

Applications of estimates of the very rare to extreme floods considered here include:

- *Spillways*: the Australian National Committee on Large Dams (ANCOLD, 2000) provides recommendations on most aspects of spillway provision and safety levels for all potentially hazardous structures which store water or other liquids, including flood retarding basins, service basins and tailings dams. The recommended design floods range from 1 in 100 to 1 in 10^7 AEP. The ANCOLD guidelines focus on design considerations of such structures, and refer to ARR for the hydrologic procedures involved.

- *Detention Basins*: large structures of this type may fall within one of the ANCOLD (2000) referable dam categories and thus be subject to its recommendations. Even when this does not apply, it may be desirable to check the performance of a detention basin for the consequences of a very rare or extreme flood, where the structure is located in a populated area and if failure could endanger lives or property. This may apply particularly where a series of structures is constructed on a watercourse and progressive failure could occur. Detention basins are discussed further in Book 9.

- *Urban Trunk Drainage*: while these drains generally are not designed to carry extreme floods, good practice requires that the effects of an extreme flood should be checked where lives and property could be endangered, as discussed in Book 9.

- *Floodplain Management*: for floodplain management or flood protection schemes it may be necessary to consider the potential flood damage arising from very rare to extreme floods, either as an input to risk-based design or as a check on the upper limiting magnitude of potential inundation for planning and emergency management.

- *Major Bridges*: the Australian Standards relevant to the hydraulic design of bridges (AS 5100.2-2004;(Standards Australia, 2004)) have adopted a limit state approach. For the Ultimate Limit State Floods, it is necessary to assess flood loading up to and including the 1 in 2000 AEP event.

- *Other Major Works*: in some cases, it may be desirable to at least check the effects of extreme floods, even if a smaller flood is used for design. Examples include portals for tunnels associated with major infrastructure, water supply intakes and sewage treatment plants where flood damage could cause severe disruption to a community, flat roofs where blockage of roof drains could cause collapse, or floodplain management studies where national heritage buildings or other irreplaceable items are endangered.

An overview of the applicability of different parts of these Guidelines to specific investigations or design tasks is provided in Book 8, Chapter 2, Section 2.

1.3. Event Classes

1.3.1. General Limits

The procedures recommended herein are based on the recognition that the uncertainties involved with the flood estimation process increase with increasing size of the flood (or reducing AEP). The type of available information, and hence the nature of procedure that can be used in the analysis and the degree of uncertainty associated with it, varies with flood magnitude. The notional event classes of most relevance to Book 8 are summarised in Figure 8.1.1, which represent the more extreme classes of events discussed in Book 1, Chapter 3. This figure broadly divides the floods and rainfalls of interest into Rare, Very Rare, and Extreme ranges, but the adopted classes represent a continuum of increasing uncertainty rather than discrete intervals. The estimation procedures and design data discussed in Book 8 relate specifically to the Very Rare and Extreme event classes.

These Guidelines are intended to provide practitioners with an approach that yields estimates in the mid-range of the notional uncertainty band indicated in Figure 8.1.1, and it is recommended that this ‘best estimate’ be adopted rather than a value at the limits of the uncertainty band. Nevertheless, if there are significant differences in flood consequences within the range of uncertainty, then the likely range of outcomes must be explicitly considered in a risk management framework when developing flood management strategies.

The procedures presented herein have been reviewed by experienced designers and academics from around Australia. They therefore constitute recommended best practice. Innovation and trialling of new techniques based on additional research (with peer review) is strongly encouraged for the estimation of Very Rare to Extreme floods but the pragmatic nature of procedures requires an increasing level of prescription as estimates extend beyond the credible limit of extrapolation. Details concerning the characteristics of each event class are provided below.

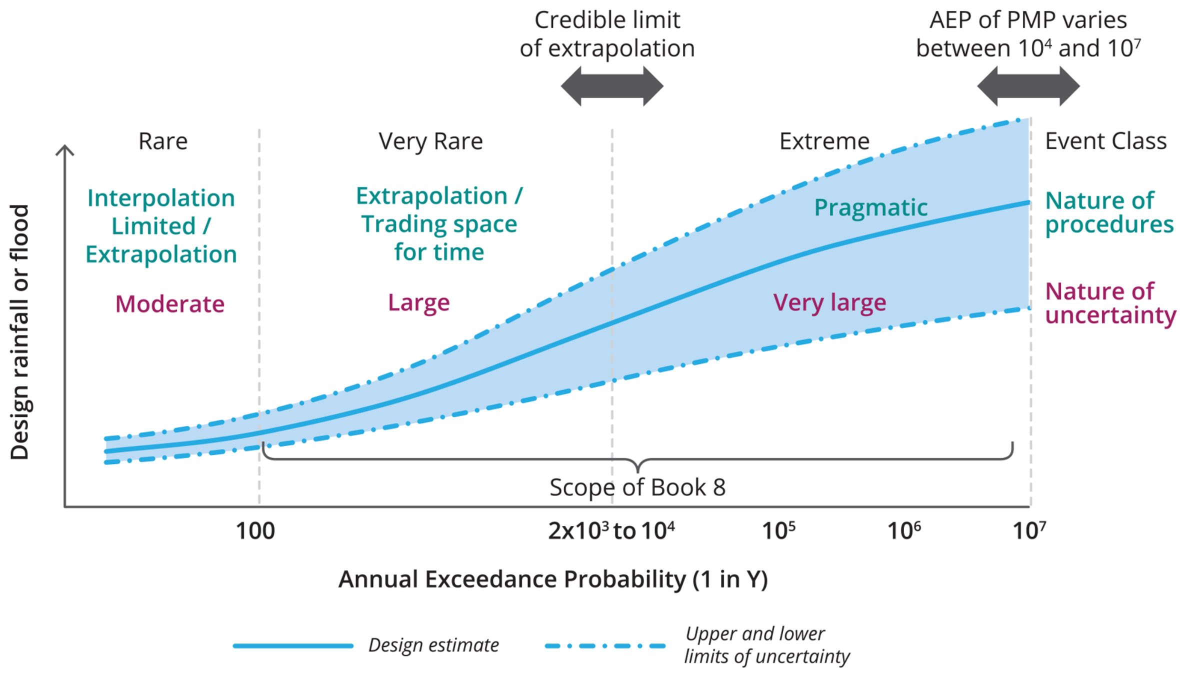


Figure 8.1.1. Design Characteristics of Notional Event Classes

1.3.2. Rare Events

The class of Rare events is intended to represent those events for which direct observations relevant to the site of interest are generally available. The most common sources of information for this range of floods are the systematic records of rainfalls or streamflow (available either at the site of interest or else transposed from similar catchments), though they include historic information for notable events that occurred prior to the beginning of continuous gauged records. Accessible records in general only extend back to the past 100 years, and thus notionally the AEPs corresponding to this category are limited to events more frequent than 1 in 100 AEP. Given that guidance provided elsewhere in ARR is generally restricted to events with AEPs equal to or more frequent than 1 in 100 AEP, for convenience the lower limit of AEP associated with Rare floods is assumed to be 1 in 100 AEP.

The procedures relevant to the analysis of this type of information are largely covered by the Books related to rainfall and Flood Frequency Analysis, and rainfall-runoff routing (Book 2; Book 4; and Book 5). However, given that this range of events is often used to 'anchor' the lower end of frequency curves used to extrapolate to extreme events, some mention of their estimation is retained in this Book.

The analyses are based on deriving design flood estimates that lie within the upper range of direct observations, and thus generally involve some degree of extrapolation. A large body of experience and a great variety of procedures are available to help the practitioner derive flood estimates within this range, and the associated degree of uncertainty in the estimates can be readily quantified.

1.3.3. Very Rare Events

Very Rare floods represent the range of events between the largest direct observations and the **credible limit of extrapolation**. With reference to the latter concept, it is worth noting that the term:

- 'credible' is used to represent the justifiable limit of extrapolation without the use of other confirming information from an essentially independent source; and,
- 'extrapolation' is used to denote estimates that are made outside the range of observations available at a single site.

The credible limit of extrapolation is dependent upon the nature of available data that can be obtained at and/or transposed to the site of interest. Procedures are often used which are based on “trading space for time”, in which data from several sites are used to help inform the estimation of exceedance probabilities at a single site. The defensibility of this form of extrapolation depends on the strength of the assumptions made, particularly those relating to the assumed degree of similarity between the sites used in the regional pooling. It is important to realise that in any given region the credible limit of rainfall extrapolation may well differ from the limit applicable to floods.

The notional credible limits of extrapolation for a range of data types in Australia are shown in Table 8.1.1 (modified after USBR (1999)). This table indicates the lower AEP bound corresponding to both typical and the most optimistic situations, though in most cases the credible AEP limits are likely to be considerably closer to the typical estimates than the most optimistic bounds. At present in Australia rainfall regionalisation procedures yield credible limits of extrapolation of around 1 in 2000 AEP (Green et al., 2016), though when larger regions are considered within a joint probability framework, the limit can be extended (with considerable uncertainty) out to limits that are one to two orders of magnitude rarer (Nathan et al., 2015).

The analyses required to extrapolate estimates to the credible limit require substantial resources and a high level of specialist expertise, and they are thus generally beyond the level of resources available to a single study. Practitioners will usually need to rely on processed information prepared specifically for the region of interest. There is considerable scope for innovation and trialling of new estimation techniques for this class of events to reduce the uncertainty of the estimates and perhaps extend the limit of extrapolation. However, adoption of new estimation approaches will depend on the outcome of detailed peer review.

Table 8.1.1. Limit of Credible Extrapolation for Different Types of Data in Australia (modified after USBR (1999))

Type of Data Used for Frequency Analysis	Credible Limit of Extrapolation (AEP)	
	Typical	Most Optimistic
At-site Gauged Flood Data	1 in 50	1 in 200
At-site Gauged Rainfall Data	1 in 100	1 in 200
t-site/Regional Gauged Flood Data	1 in 200	1 in 500
At-site Gauged and Paleoflood Data	1 in 5 000	1 in 10 000
Regional Rainfall Data	1 in 2 000	1 in 10 000
Regional Gauged and Paleoflood Data	1 in 15 000	1 in 40 000
Large Scale Regional Rainfall Data	1 in 10 000	1 in 1 000 000

1.3.4. Extreme Events

Extreme floods, the third class, represent the range of floods which borders on the 'unknowable', where even a high level of expertise cannot reduce the level of uncertainty substantially. Estimates of such events lie beyond the credible limit of extrapolation, but are hopefully based on our broadest understanding of the hydrometeorological and catchment processes governing flood production, including their physical limits. It should be recognised that our understanding of catchment processes is largely based on observations of relatively frequent floods, and it is possible that a catchment may change its behaviour when subjected to extreme rainfalls.

Any extensions beyond the credible limit of extrapolation should employ a consensus approach that provides consistent and reasonable values for pragmatic design. The procedures relating to this range of estimates should be regarded as inherently prescriptive, as without empirical evidence or scientific justification there can be no rational basis for departing from the consensus approach.

The level of uncertainty of these estimates can only be reduced by long-term fundamental research. Accordingly, it is important that the procedures related to this class of floods be reviewed periodically to ensure that any advances in our understanding of extreme hydrological and hydrometeorological processes are incorporated into design practice.

1.4. Relationship with Other Sections of Australian Rainfall and Runoff

1.4.1. Specific Focus of Book 8

The main focus of this Book is on the estimation of floods rarer than 1 in 100 AEP; its intention is to *supplement* the design information provided in other Books rather than to replace it. Specifically, the following aspects of procedures are generally intended for the estimation of events less frequent than 1 in 100 AEP, but they may assist estimation of more frequent events when only limited data are available:

- *Hydrograph modelling considerations:* the discussion presented in Book 8, Chapter 5, Section 2 and Book 8, Chapter 5, Section 3 regarding the use of hydrograph models is generally focused on those issues that most need to be considered when extrapolating to conditions well beyond those encountered during calibration to observed floods. While in general these considerations encourage a sound understanding of model features, they are less important if the models are applied to event magnitudes and conditions similar to those applicable during calibration.

- *Additional design considerations:* there are a number of additional design considerations discussed in this Book that are potentially applicable to floods with AEPs more frequent than 1 in 100 AEP. These issues (mostly presented in Book 8, Chapter 7) include the derivation of seasonal design floods, the joint probability treatment of initial reservoir drawdown and concurrent tributary flows, and the treatment of uncertainty. Most of these considerations are not specifically addressed elsewhere in ARR, and thus are not duplicating recommendations provided elsewhere.

1.4.2. Terminology

The terminology used in this book is generally in accordance with Book 1, Chapter 1, Section 2. However, for convenience and consistency of expression in this Book, all AEPs in the range of Very Rare to Extreme events are expressed in the form of 1 in Y.

1.4.3. Risk-Based Design

The guidance presented herein represents a major revision to Book 6 of ARR (Nathan and Weinmann, 2000), which was in turn based on the scope and procedures presented in Chapter 13 of the ARR 1987 (Pilgrim and Rowbottom, 1987). Book 6 represented a shift in emphasis away from a standards-based design approach to a risk-based one, in which the focus of interest is on deriving design flood estimates of specified Annual Exceedance Probability (AEP) rather than estimates of flood magnitude. This approach is also generally recommended for Very Rare to Extreme flood events. However, as the PMF event cannot be assigned a specific AEP, design for the PMF does not lend itself to a risk-based approach, and a standards based approach must be adopted instead. This is discussed further in Book 8, Chapter 6, Section 4, where an objective basis for the concept of ‘reasonableness’ is advocated.

1.5. References

ANCOLD (Australian National Committee on Large Dams) (2000), Selection of acceptable flood capacity for dams, March

Green, J.H., Beesley, C., The, C., Podger, S. and Frost, A. (2016), Comparing CRC-FORGE estimates and the New Rare Design Rainfalls. Proc. ANCOLD Conference, Adelaide.

Nathan, R.J. and Weinmann, P.E. (2000), Book VI:Estimation of large and extreme floods, In Australian Rainfall and Runoff: A Guide to Flood Estimation, Engineers Australia, Canberra.

Nathan, R.J., Scorah, M., Jordan, P., Lang, S., Kuczera, M., Schaefer, M. and Weinmann, P.E. (2015), A Tail of Two Models: Estimating the Annual Exceedance Probability of Probable Maximum Precipitation. Proc.Hydrology and Water Resources Symposium, Hobart.

Pilgrim, D.H. and Rowbottom, I.A. (1987), Chapter 13 - Estimation of large and extreme floods, In Pilgrim, D.H. (ed.) Australian Rainfall and Runoff: A Guide to Flood Estimation, I.E. Aust., Canberra.

Standards Australia (2004), Bridge Design, Part 2 - Design Loads. AS 5100.2 AP-G15.2/04. Standards Australia, Sydney.

U.S. Bureau of Reclamation (USBR) (1999), A framework for developing extreme flood inputs for dam safety risk assessments, Part I - Framework, Draft, February 1999.

Chapter 2. Procedures for Estimating Very Rare to Extreme Floods

Rory Nathan, Erwin Weinmann

Chapter Status	Final
Date last updated	14/5/2019

2.1. Overall Design Approach

The overall emphasis of this Book is on the estimation of a flood frequency curve for Very Rare to Extreme floods rather than on the estimation of a design flood of specific magnitude. The procedures employed are generally based on Flood Frequency Analysis and rainfall-based simulation techniques. While general guidance on these techniques should be sought elsewhere in ARR (Book 3, Chapter 2), this Book gives consideration to a number of issues specific to the estimation of Very Rare to Extreme design floods.

The procedures are generally based on the assumption that final design estimates should incorporate the best and most relevant information available. As emphasised in Book 1 the use of new procedures and design information is encouraged, especially where these can be shown to be more appropriate than the guidance provided here.

2.1.1. Flood Frequency Analysis

The procedures recommended in Book 3 are directly relevant to the estimation of Rare floods, and with the incorporation of regional information at-site Flood Frequency Analysis can also be used to estimate Very Rare floods. Special consideration is provided here on the benefits of incorporating paleo-hydrological information, (Book 3, Chapter 2, Section 3) as this type of information has the potential to considerably extend the credible limit of extrapolation.

2.1.2. Rainfall-Based Procedures

The procedures provided in Book 5 provide general guidance on the use of rainfall-based simulation methods. For risk-based design it is necessary to transform design rainfalls into floods in a fashion that minimises bias in the resulting exceedance probabilities; that is, we wish to ensure that the 1 in Y AEP design rainfalls are converted to the corresponding 1 in Y AEP floods (i.e. Probability neutrality). For those inputs and model parameters with a small impact on flood discharge it is usually sufficient to adopt a single representative value from the central range of observations; often either the mean or median is adopted. However, the most appropriate value depends on the degree of non-linearity in the transformation between rainfall and runoff and in the cumulative probability distribution. If one or both of these forms of non-linearity is great, it is desirable to adopt a joint probability approach in which the inputs and model parameters are characterised by their probability distributions rather than by a single value. This is particularly so for those inputs and model parameters with a large impact on flood magnitude.

The adoption of a simplified probability neutral approach is accepted practice for Frequent to Rare floods, where it is usually possible to derive independent estimates of the design floods to check that no bias has been introduced into the transformation between rainfall and runoff. Where such independent information is available, simple event based approaches, i.e. those involving the deterministic application of models based on linear and non-linear routing with representative values of inputs and model parameters, should be adequate for many practical purposes. However, it becomes increasingly difficult to obtain independent estimates of floods in the Very Rare to Extreme range, and thus there is an increased need to explicitly consider the joint probabilities involved. This is particularly so when considering the impacts of changing factors (such as revised operating conditions on reservoir levels) whose variability may be characterised with reasonable confidence but whose influences may not be reflected in the observed record. Ensemble event approaches have the potential to mitigate this bias, but these are only likely to be defensible for those problems (linearly) influenced by a single dominant factor in addition to rainfall. Monte Carlo simulation methods provide a more flexible and rigorous means of resolving these difficulties, but the defensibility of these estimates rests upon the representativeness of the inputs and the correct treatment of correlations which may be present.

2.2. Procedures for Different Categories of Design Floods

The procedures for deriving design estimates for flood classes of most relevance to extreme floods can be summarised in the following three main categories depending on the probability of the flood to be estimated:

i. Floods with AEPs approaching 1 in 100 (Rare floods):

   Estimates should be based on a combination of approaches that consider (where possible) at-site flood frequency analyses (Book 2, Chapter 2), regional flood methods (Book 2, Chapter 3), and rainfall-based event modelling (Book 3, Chapter 3, Section 2). The comparison of results obtained using different methods yields insights about errors or assumptions that might otherwise be missed, and the process of reconciling the different assessments provides valuable information that aids adoption of a final “best estimate”. As discussed in Book 1, Chapter 3, the adoption of a single best estimate is ideally achieved by weighting estimates obtained from different methods by their uncertainty, or through the process of reconciliation in which selected factors are varied within their expected range to achieve the desired level of consistency.

   While there is scope for considering the use of continuous simulation approaches, their use for estimation of Vary Rare to Extreme events should only be considered for systems which are strongly dependent on flood volume in a manner not easily handled by event-based procedures; this might be the case for the design of tailings dams with small catchment areas, or cascade systems of storages involving complex interaction of joint probabilities. The advantages and limitations of continuous simulation approaches are broadly discussed in Book 1, Chapter 3, but in the context of the estimation of extreme floods, it is worth noting that their use will require careful generation of stochastic rainfall inputs that are consistent with design rainfall information provided in Book 2. If the exceedance probabilities of interest lie in the Very Rare to Extreme event range then there is little point using a different approach for estimation of Rare floods.

ii. Floods with AEPs beyond 1 in 100 AEP to the credible limit of extrapolation (Very Rare floods):

   Estimates should be primarily based on rainfall-based simulation methods, with rainfalls derived using methods described in Book 8, Chapter 3, Section 1, or else on flood frequency estimates derived using historical and paleo flood information (Book 8, Chapter 6, Section 2). Such estimates necessarily involve significant extrapolation, and their defensibility will partly depend on the ease with which different estimates of Rare floods can be reconciled.

iii. Floods with AEPs beyond the credible limit of extrapolation (Extreme floods, including the Probable Maximum Precipitation Flood):

These flood estimates may be required for direct use in design situations of high risk, either in terms of risk to human life or economic losses, or where social or political considerations require a very high level of safety. Estimates should be based on the use of a flood event model with design rainfalls obtained by interpolation between the credible limit of extrapolated rainfalls and the Probable Maximum Precipitation (PMP). To avoid confusion with the Probable Maximum Flood (refer to iv), the flood derived from the PMP using probability neutral assumptions is here termed the “PMP Flood”.

An additional category of procedures differs from the above in its design objective:

iv. Probable Maximum Flood (the limiting value of flood that could reasonably be expected to occur):

This may be required for comparison with estimates derived from previous studies or for some other design objective that usually requires a notional upper limiting value of flood without an associated AEP. In practice, the magnitude of the PMF will generally be greater than the magnitude of the flood derived from the PMP using probability neutral assumptions (the PMP Flood).

A brief summary of the recommended procedures and references to the relevant sections are presented in Table 8.2.1 and Table 8.2.2. It should be recognised that these tables represent a summary of procedures that are described in detail in later sections; they are not intended to be self-explanatory.

Table 8.2.1. Summary of Procedures to Derive Design Rainfalls

Design Consideration	**Design Category**			**Comments**
	Rare **Towards 1 in 100 AEP**	**Very Rare** **Beyond 1 in 100 AEP to 1 in 2000 AEP**	**Extreme** **Rarer than 1 in 2000 AEP**	***The credible limit of extrapolation generally ranges from 1 in 100 to 1 in 5000 AEP***
Point Design Rainfall Depths up to the PMP	Generalised information on design rainfall bursts, as described in Book 2, Chapter 3.		Not applicable	• *Point design rainfall depths for non-standard durations from procedures in* Book 8, Chapter 3, Section 6 • *For information on seasonal estimates see* Book 8, Chapter 3, Section 7
Areal Design Rainfall Depths up to the PMP	Derived from point design rainfalls by application of Areal Reduction Factors		Interpolation to areal PMP estimate based on	• *See* Book 2, Chapter 4 *for*

Design Consideration	Design Category			Comments
	Rare Towards 1 in 100 AEP	Very Rare Beyond 1 in 100 AEP to 1 in 2000 AEP	Extreme Rarer than 1 in 2000 AEP	***The credible limit of extrapolation generally ranges from 1 in 100 to 1 in 5000 AEP***
	(ARFs), selected as a function of storm area, duration, and AEP (Book 2, Chapter 4).		two parameter parabolic function (Book 8, Chapter 3, Section 6)	*discussion on Areal Reduction Factors*
Probable Maximum Precipitation (PMP)	Not applicable		• GSDM for short durations and small areas • GSAM for South-East Australia region • GTSM for tropical region	• *See* Book 8, Chapter 3, Section 3 *for areas covered by different generalised methods* • *PMP estimates are areal depths* • *AEP of PMP based solely on catchment area,* Book 8, Chapter 3, Section 4
Temporal Patterns	Areal patterns (for bursts and complete storms) as described in Book 2, Chapter 5.	• GSAM & GTSM-R areal patterns for all long durations • GSDM point patterns for short durations • Both GSDM and GSAM/ GTSM-R patterns for intermediate durations		• *See* Table 8.3.1 *for summary of temporal pattern selection* • *Pre-burst temporal patterns may be used in conjunction with storm losses (*Book 8, Chapter 3, Section 8*)*
		Use of a mix of selected at-site and generalised PMP patterns in an ensemble for Rare and Very Rare events may be required to provide a smooth transition across different sources of data		
Spatial Patterns	Regional information on design rainfall spatial patterns as described in Book 2, Chapter 4	Use either GSDM, GSAM, or GTSMR spatial patterns as appropriate for the location and duration		• *See* Book 8, Chapter 3, Section 9 *for discussion on need for incorporation of spatial trend* • *See* Table 8.3.2 *for summary of spatial pattern selection*

Table 8.2.2. Summary of Procedures to Derive Design Floods

Design Consideration	**Design Category**			**Comments**
	Rare **Towards 1 in 100 AEP**	**Very Rare** **Beyond 1 in 100 AEP to the credible limit of extrapolation**	**Extreme** **Beyond the credible limit of extrapolation**	***The credible limit of extrapolation generally ranges from 1 in 100 to 1 in 5000 AEP***
Losses • For complete design storms	Use "complete storm" initial and continuing losses for all durations and all AEPs (a probability distributed model may be warranted for south-west Australia) consistent with Book 5, Chapter 3			• *See* Book 8, Chapter 4, Section 1 *for general recommendations*
Hydrograph model • Selection and configuration • Calibration	• Non-linear storage-routing models (or equivalent) • Calibration to range of flood magnitudes, including reconciliation with design flood estimates derived from at-site/regional flood frequency and paleohydrological procedures • For Rare to Extreme events non-linearity in $S = kQ^m$ relation generally assumed to be in range 0.8 to 0.9 (depending on catchment characteristics)			• *See* Book 8, Chapter 5, Section 2 *and* Book 8, Chapter 5, Section 3 • *See* Book 8, Chapter 5, Section 4 *and* Book 8, Chapter 5, Section 4 • *See* Book 8, Chapter 5, Section 4
Baseflow	Adopt baseflow recommendations as discussed in Book 5, Chapter 4	Baseflow to be varied gradually between that adopted for the 1 in 100 AEP and the PMP Flood	Adopt constant value 20% to 50% higher than maximum observed	• *See* Book 8, Chapter 6, Section 3
Design flood frequency curve	Derive rainfall-based estimates for range of design rainfall durations and AEPs, and adopt highest peak discharge from the range of durations for each AEP			• *See* Book 8, Chapter 6, Section 3 *for general guidance and* Book 8, Chapter 7, Section 4 *for guidance on seasonal floods*
Probable Maximum Flood (PMF)	Not applicable		Defined as the limiting value of flood that could reasonably be expected to occur (Book 8, Chapter 6, Section 4)	*There are no established procedures to assign an AEP to the PMF*

Design Consideration	Design Category			Comments
	Rare **Towards 1 in 100 AEP**	**Very Rare** **Beyond 1 in 100 AEP to the credible limit of extrapolation**	**Extreme** **Beyond the credible limit of extrapolation**	***The credible limit of extrapolation generally ranges from 1 in 100 to 1 in 5000 AEP***
Additional design considerations • Reservoir outflows • Concurrent flooding • Seasonal floods • Snowmelt floods • Long duration events	Probability neutral procedures are recommended to ensure that any bias in the AEP of the transformation between rainfall and runoff is minimised. A range of procedures from the simple to the rigorous are provided			• *See* Book 8, Chapter 7 *for general guidance*
Preliminary design flood estimates	Regional procedures and Flood Frequency Analysis (Book 4	Simple log Normal interpolation may be used to determine Very Rare preliminary	Regional information may be used to estimate the PMP Flood (conservatively assumed to equal the PMF)	• *See* Book 8, Chapter 6, Section 2

2.3. Relevance of Procedures to Specific Applications

The procedures outlined in this Book are intended to cover the broad spectrum of applications listed in Book 8, Chapter 1, Section 2 and the flood classes described in Book 8, Chapter 1, Section 3. However, when deriving flood estimates for a specific application, the practitioner's interest may be focused on a more limited range of procedures. This section provides some guidance on the applicability of different parts of the guidelines to specific investigations or design tasks.

Figure 8.2.1 gives a qualitative indication of the range of flood magnitudes and the relative degree of reliability required for different applications. An extension of the flood range of interest is associated with a greater degree of extrapolation and thus larger uncertainty (lower reliability). The level of expertise and effort required for deriving design floods increases with increasing level of reliability required.

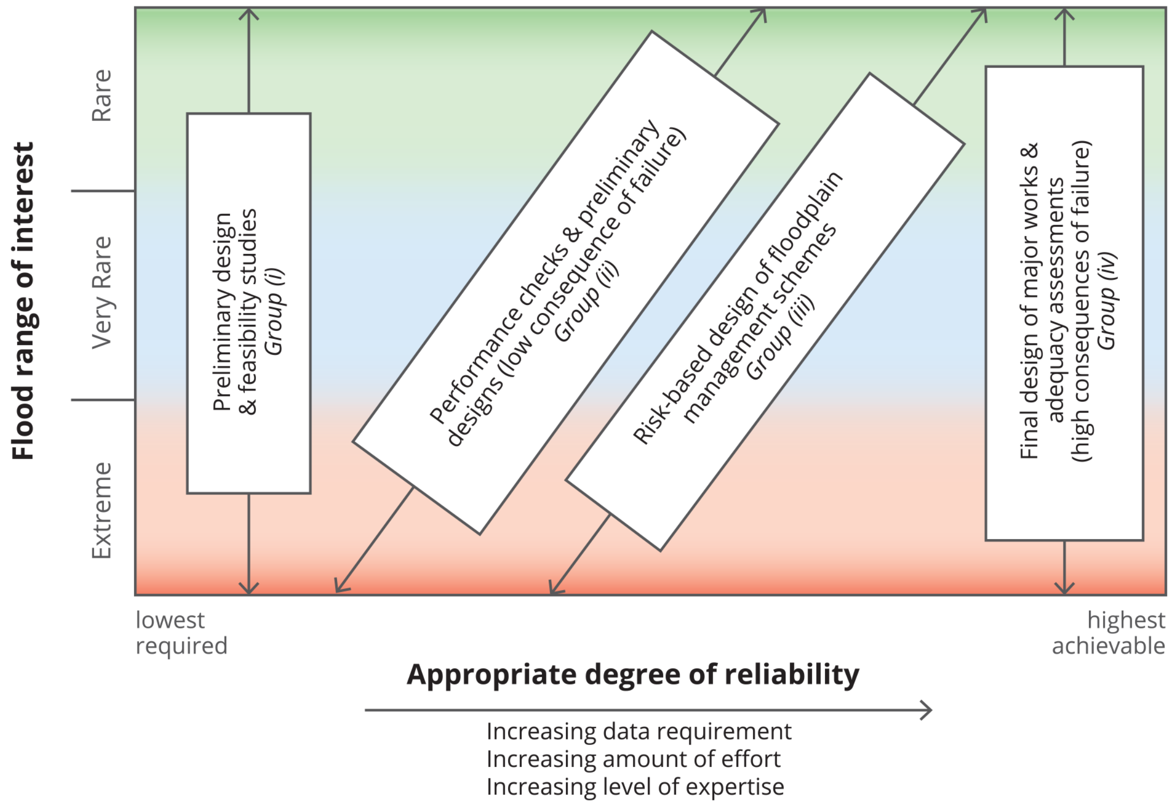


Figure 8.2.1. Qualitative Indication of the Range of Flood Magnitudes and the Relative Degree of Reliability Required for Different Applications

As shown in Figure 8.2.1, the qualitative indication of the range of flood magnitudes and associated relative degree of reliability can be divided into four groups of applications. For the first three groups of applications, approximate or simplified flood estimation procedures may be applicable; however, the practitioner has to apply engineering judgement in deciding on the degree of detail and accuracy required for a specific application. The fourth group of applications demands the most accurate estimates and hence the greatest level of effort.

(i) *Planning and feasibility studies, initial screening of options, preliminary designs:*

For these types of applications, where decisions based on the flood estimates are only moderately sensitive to estimation uncertainties, approximate design flood estimates can be derived by preliminary methods (refer to Book 8, Chapter 6, Section 2).

(ii) *Performance checks or preliminary designs of structures:*

Many design codes require safety checks for conditions exceeding the design objective. Approximate estimates of floods from 1 in 100 AEP to an absolute limit of 1 in 2000 AEP can be obtained by use of design rainfalls (Book 2, Chapter 3) in combination with a flood event model configured using regional estimates of losses and routing parameters. The rainfall frequency curve may be extended to the AEP of the PMP by deriving site-specific estimates of the PMP. Flood estimates based on use of regional parameters without calibration or additional confirmatory estimates are subject to considerable uncertainty, and correspondingly greater responsibility then rests with the practitioner to ensure that the estimates are consistent with any relevant flood estimates for the region.

(iii) *Design of floodplain management or flood protection schemes based on risk management principles:*

While risk-based design requires the assessment of the contribution of Rare to Extreme floods to the total expected flood damage figure, the low probability of floods in this range means that the contribution from this range of floods to the total expected flood damage is relatively low in most situations. A lower degree of reliability of flood estimates in the Rare to Extreme range is therefore acceptable for these applications.

(iv) *Final design of major works and assessment of the adequacy of existing infrastructure, where failure would result in serious consequences or possible of life:*

For this group of applications, efforts should be made to reduce uncertainties in design flood estimates to the minimum possible. The further the AEP range of interest extends beyond the Rare floods, the more important it is for the practitioner to consider in detail the guidelines in Book 8, Chapter 4 and Book 8, Chapter 5 on extrapolation of hydrograph model characteristics. For all applications where the range of interest extends to extreme floods, and where large uncertainty in flood estimates would impact significantly on design decisions, detailed flood studies are justified.

Chapter 3. Estimation of Very Rare to Extreme Rainfalls

Rory Nathan, Erwin Weinmann

Chapter Status	Final
Date last updated	14/5/2019

3.1. General

3.1.1. Overview of Requirements and Sources of Design Rainfall Information

In general, estimates of Very Rare to Extreme floods are derived using *rainfall-based flood estimation methods* (possible exceptions to this are discussed in Book 8, Chapter 2, Section 2). Information is required on the average depth of rainfall over the catchment for a range of rainfall event durations, its distribution in space (spatial pattern) and its distribution in time during the event (temporal pattern). Design floods are generally calculated separately for each duration, including routing through any reservoirs or other storages, to determine the critical rainfall durations that produce the maxima for the flood characteristics of interest (peak inflow/outflow, flood volume or possibly duration of flooding). Short duration design rainfalls may be required even on large catchments to check that their occurrence on only part of the catchment area does not produce a critical flood, and to check that the magnitudes of the calculated floods vary in a regular manner as the duration of the rainfall increases.

Book 2, Chapter 3 provides details of *design rainfall depths at a grid of points* over the whole of Australia for the range of AEPs and durations of interest for the estimation of Very Rare to Extreme floods. Except for the PMP, these design rainfall depths are point rainfalls at the grid point location; they need to be converted to *average catchment rainfalls* by application of the *areal reduction factors (ARFs)* provided in Book 2, Chapter 4.

General guidance on *design spatial rainfall patterns* is provided in Book 2, Chapter 6. This guidance applies to design spatial patterns for the estimation of Very Rare to Extreme floods, with some more specific guidance provided in Book 8, Chapter 3, Section 9. The limited information available on spatial patterns of extreme storm events generally precludes the application of an ensemble of spatial patterns; spatial patterns can be sampled in an ensemble fashion in stochastic simulation frameworks, but generally a single representative pattern derived from design rainfall fields (or observed storms) in a larger region is sufficient for most design purposes.

The guidance provided in Book 2, Chapter 5 for the selection of *design temporal rainfall patterns* also generally applies to the range of Very Rare to Extreme floods, with more specific guidance provided in Book 8, Chapter 3, Section 8. Given the sensitivity of flood estimates to the high degree of natural variability in the temporal patterns of actual storms, it is recommended that an ensemble of temporal patterns rather than a single 'representative' temporal pattern is applied.

3.1.2. Uncertainty in Design Rainfall Estimates

There is limited detailed information on the uncertainty associated with the rainfall estimates provided in Book 2, Chapter 3. It should be recognised that the magnitude of the uncertainty in design rainfall estimates increases with decreasing AEP (refer to Book 1, Chapter 2), as model uncertainty plays an increasingly important part in the estimation of extreme events, including the PMP. Some information is available on uncertainty estimates of Very Rare to Extreme rainfalls (e.g. McConachy et al. (1997); Nandakumar et al. (1997)), though until formal estimates of uncertainties are available probably the most pragmatic approach to characterizing uncertainty is to use a parametric bootstrapping approach with at-site maxima (e.g. Kyselý (2008)), where the effective size of the sample is adjusted to represent the degree of pooling used. Estimates of very rare rainfalls were derived using a region of influence approach based on a minimum of 2000 years of station data (Green et al., 2016) and analyses by Lang et al. (2016) indicate that the effective number of independent years in such data sets varies between around 300 to 500 years, depending on gauge density. The uncertainty of design rainfall estimates reflects the level of information available, with significantly increased uncertainty in areas of sparse rain gauge coverage, such as mountainous areas.

3.2. Estimation of Very Rare Design Rainfalls

Design rainfall depths for Very Rare events are derived by regional estimation methods, such as the regional L-moment method described in Book 2, Chapter 3, Section 4. These methods pool data from large rainfall events in a region that satisfies basic homogeneity criteria. By using a ‘space for time trade-off’, these methods allow estimation of rarer events than would be possible by using data from an individual site only.

The AEP range covered by regional estimates, referred to as the ‘credible range of extrapolation’, depends on the number of stations in a region and the length and quality of their records. For the relatively well gauged parts of Australia this range has been taken as extending to the 1 in 2000 AEP.

Practitioners should recognise that making available design rainfall estimates for a dense grid covering the whole of Australia has been achieved at the cost of potentially reduced accuracy at locations for which long and reliable rainfall records are available. However, the results of frequency analysis of local records should only be used to fine-tune regional design rainfall estimates if there is strong evidence confirmed by peer review. In such a situation the shape of the rainfall frequency curve in the range of Very Rare events should closely follow the shape indicated by the regional estimate.

3.3. Estimation of the Probable Maximum Precipitation Depth

The theoretical definition of the Probable Maximum Precipitation (PMP) is “the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location at a certain time of year” (World Meteorological Organisation, 1986). Estimates are derived using generalised methods that are based on the analysis of data over a wide region, as described in Book 2, Chapter 3, Section 7.

Estimates of PMP rainfall data have been developed by the Bureau of Meteorology. There are three generalised methods appropriate for different locations and durations:

i. the Generalised Short Duration Method (GSDM) is applicable for durations up to six hours and areas up to 1000 km^2 (Bureau of Meteorology, 2003a);

ii. the Generalised Tropical Storm Method (GTSMR) is used to estimate PMPs for durations up to 120 hours and areas up to 150 000 km^2 in the region of Australia where tropical storms are the source of the greatest depths of rainfall (Bureau of Meteorology, 2003b); and

iii. the Generalised Southeast Australia Method (GSAM) is used for durations up to 96 hours and areas up to 100 000 km^2 for the region of Australia where tropical storms are not the source of the greatest depths of rainfall (Bureau of Meteorology, 2006).

The zones of application for the GTSM and GSAM methods are shown in Figure 8.3.1. For the west coast of Tasmania, data constraints and the size of region have prevented the development of a generalised method, and thus site-specific advice should be sought from the Bureau of Meteorology. It should be noted that the PMP estimates provided by the Bureau of Meteorology are for design rainfall bursts rather than complete storm events, though these can be adjusted to include likely pre-burst rainfalls using information provided by Minty and Meighen (1999) and Book 2, Chapter 5.

All PMP estimates are based on a set of simplifying assumptions applied when extrapolating from the hydrometeorological conditions of observed large events to “maximised conditions”. They thus represent operational estimates of the PMP and should not be interpreted as being equivalent to a theoretical upper limit on rainfall for that location i.e., there is a very small, but finite, probability that the estimates may be exceeded (Book 8, Chapter 3, Section 4).

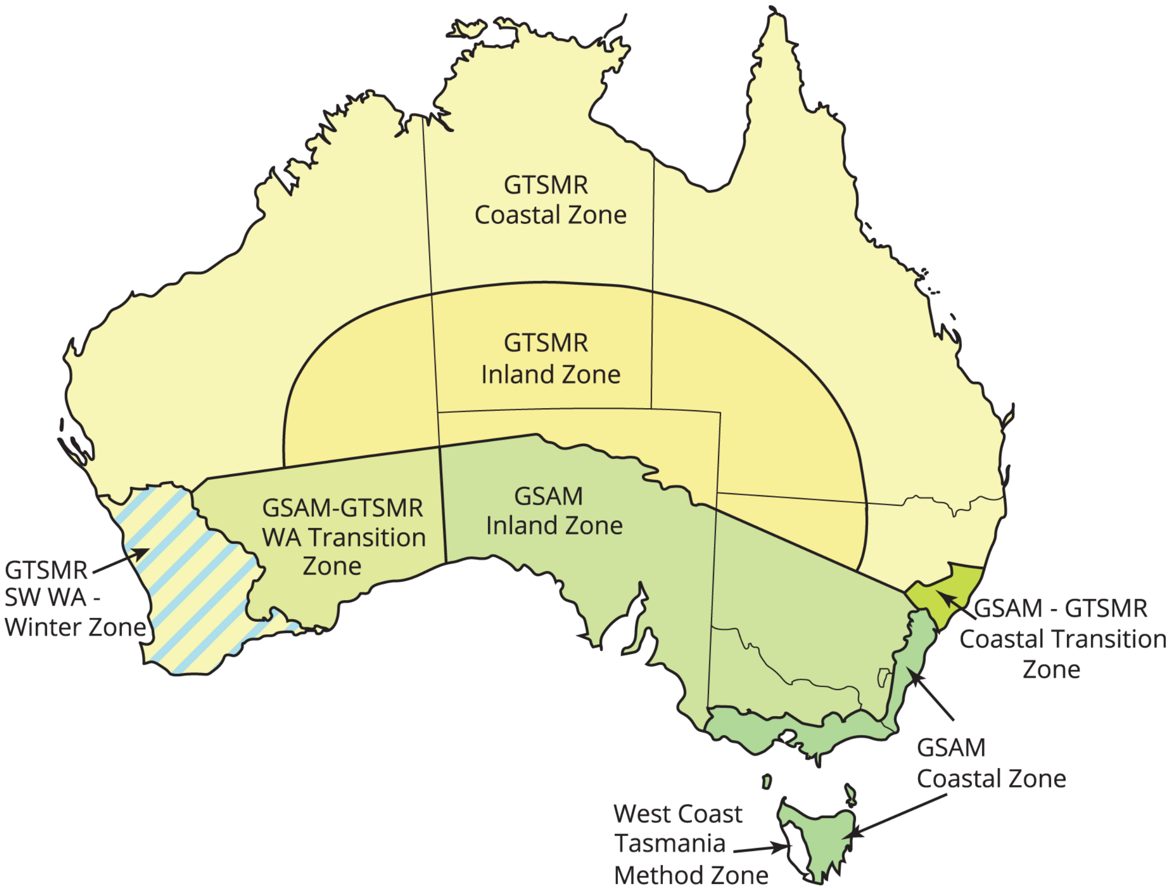


Figure 8.3.1. Generalised Long-Duration Probable Maximum Precipitation Method Zones (Bureau of Meteorology, 2006)

3.4. Assigning an Annual Exceedance Probability to the Probable Maximum Precipitation

3.4.1. Background

Assigning an AEP to the PMP is consistent with the concept of operational PMP estimates (as described Book 8, Chapter 3, Section 3 and in Book 2, Chapter 3, Section 7), which should not be regarded as theoretical upper limits of rainfall, as they may conceivably be exceeded.

The method proposed to assign an AEP to the PMP is based on the review by Laurenson and Kuczera (1999) of the procedures recommended in the 1987 edition of ARR and subsequent work conducted in both Australia and overseas. More recent research into regional estimates for the inland zone of south-east Australia (Nathan et al., 2015) provides some evidence to suggest that the Laurenson-Kuczera recommendations might be slightly conservative, though the authors concluded that there is insufficient justification to consider changing either the best estimate or the inferred width of the confidence intervals.

Overall it is considered that recommendations provided below represent a reasonable basis for design, and that the associated confidence intervals do reflect the true uncertainty involved. It should be recognised that this is an area of ongoing research and practitioners should take advantage of revised guidance where shown to be more appropriate by independent peer review.

3.4.2. Regional Recommendations

The AEP of PMP estimates are considered to vary solely as a function of catchment area, and are similar to the recommendations of Kennedy and Hart (1984). These recommendations had been adopted as the basis of the guidance provided in ARR 1987, and are consistent with the more recent estimates of Pearse and Laurenson (1997) and Nathan et al. (1999). The relationship recommended by Laurenson and Kuczera (1999) is shown graphically in Book 8, Chapter 3, Section 7. It should be noted that these AEP estimates indicate the probability of a PMP event in any part of the year (annual PMP). The question of the AEP of a PMP event occurring in a specific season (seasonal PMP) is addressed in Book 8, Chapter 3, Section 7.

It should be recognised that there is considerable uncertainty surrounding these recommendations as they are for events beyond the realm of experience and are based on a limited body of information. The estimates should be interpreted as follows:

- the recommended AEP values plus or minus two orders of magnitude of AEP should be regarded as the notional upper and lower limits for the true AEP;
- the recommended AEP values plus or minus one order of magnitude of AEP should be regarded as the confidence limits with about 75% subjective probability that the true AEP lies within these limits; and
- the recommended AEP values should be regarded as the best estimates of the AEPs.

The notional 75% confidence and upper and lower limits are shown on Figure 8.3.2. While the recommended error bands are undoubtedly wider than is desirable, they are regarded as a realistic assessment of the true uncertainty.

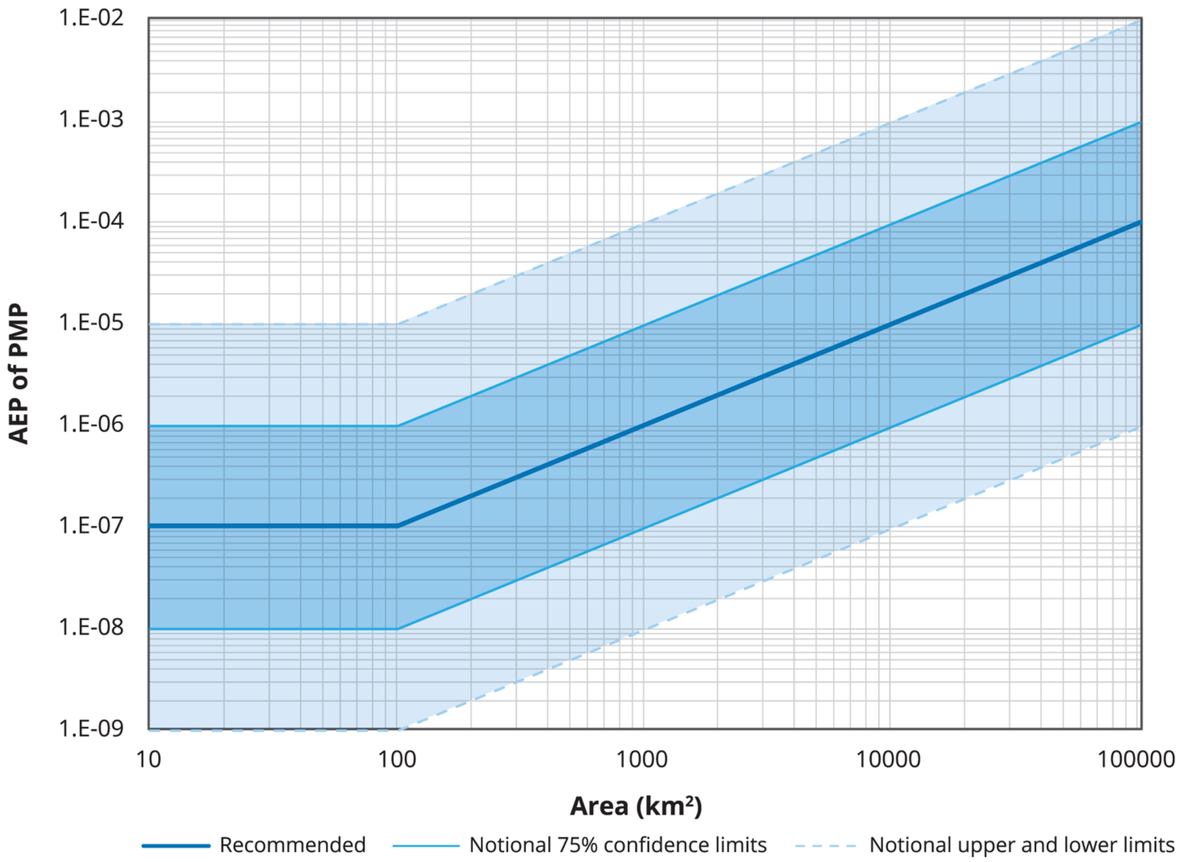


Figure 8.3.2. Recommended Regional Estimates for the AEP of PMP

Table 8.3.1. Subjective Probability Mass Function for Describing Uncertainty in Regional Estimate of the AEP of PMP (Adapted from (Laurenson and Kuczera, 1999))

Class Interval ($\log_{10}$(AEP) - $\log_{10}$(Recommended AEP))		Subjective probability mass in class interval
Class bounds	Mid-point	
-2.00		
	-1.875	0.010
-1.75		
	-1.625	0.022
-1.50		
	-1.375	0.038
-1.25		
	-1.125	0.055
-1.00		
	-0.875	0.073
-0.75		
	-0.625	0.090
-0.50		

Class Interval ($\log_{10}$(AEP) - $\log_{10}$(Recommended AEP))		Subjective probability mass in class interval
Class bounds	Mid-point	
	-0.375	0.102
-0.25		
	-0.125	0.110
0.00		
	0.125	0.110
0.25		
	0.375	0.102
0.50		
	0.625	0.090
0.75		
	0.875	0.073
1.00		
	1.125	0.055
1.25		
	1.375	0.038
1.50		
	1.625	0.022
1.75		
	1.875	0.010
2.00		

In order to incorporate this uncertainty into a risk analysis, Laurenson and Kuczera (1999) recommend the construction of a probability mass function that provides a 75% chance that the true AEP lies within one-order-of-magnitude of the recommended AEP, and a 100% chance that the true AEP lies within two-orders-of-magnitude of the recommended AEP. Table 8.3.1 presents an example of a probability mass function which meets these requirements. For example, if the recommended AEP were 1 in 10^6, then there is an 11.0% chance that the true AEP lies between 1 in 10^6 and 1 in $10^{5.75}$, and there is a 42.4% chance that it lies between 1 in $10^{5.5}$ and 1 in $10^{6.5}$; the first example corresponds simply to a single probability interval adjacent to the mid-point of 0.00 in Table 8.3.1, and the second example corresponds to the central four probability intervals. Although the probabilities are subjective, they do reflect the considerable uncertainty in the AEP estimates. The uncertainty can be directly incorporated into a risk analysis by performing an assessment for each of the AEPs in Table 8.3.1 and weighting the results using the associated subjective probability.

3.4.3. Site-Specific Estimation

Laurenson and Kuczera (1999) included a review of appropriate approaches, and they concluded that the most promising avenues of research were based on total probability approaches developed and applied by the National Research Council (1988), Fontaine and Potter (1989) and Wilson and Foufoula-Georgiou (1990). Another promising method was demonstrated by Klemes (1993) who developed a combinatorial approach that considered the joint distributions of the independent components that combined to produce PMP, and this was applied to the coastal GSAM methodology by Pearse and Laurenson (1997).

More recently (Nathan et al., 2015) described the development and application of two largely independent methods for deriving site-specific estimates of the AEP of PMP. One method uses the total probability theorem to combine the probabilities of extreme storms occurring in the transposition region with the likelihood that they were positioned in a manner that would equal or exceed the estimated target depth on the catchment for the specified duration. The other method involved the development of a stochastic regression model to estimate catchment rainfalls from point rainfalls at the key sites, and is based on an approach developed and applied by Schaefer over a number of years (e.g. MGS Engineering and Applied Climate Services (2014)).

These studies are mentioned to make the point that methods are available to derive site-specific estimates that are potentially more defensible than the regional recommendations described in the preceding section. While there remain a number of research questions which, if resolved, may increase confidence in such estimates, the undertaking of site-specific studies does merit practical consideration. Until the required methodology is more mature such studies would need to be undertaken by specialists with peer review. It is expected that this option is of most relevance to a minority of cases which involve the design of infrastructure on large catchments (> 1000 km^2) with high potential consequences of failure.

3.5. Estimation of Extreme Rainfalls

3.5.1. General

The previous sections provide recommendations on deriving catchment rainfall estimates for Very Rare events to the credible limit of extrapolation and for the PMP. In order to derive a complete areal rainfall frequency curve it is necessary to interpolate between these two limits. The interpolation is necessarily pragmatic as it attempts to link estimates based on conceptually different methods and different data sets. As there are no independently estimated design rainfalls for this range of AEPs, any specific interpolation procedure cannot be supported by direct evidence but it must be able to produce plausible and consistent estimates. The practical implication of this is that design rainfall estimates for Extreme events have a greater level of uncertainty than the events within the credible limit of extrapolation.

Estimates of rainfalls for Extreme events beyond the credible limit of extrapolation are predicated on the following two design rainfall characteristics, namely:

(i) the magnitude and AEP of the PMP; and,

(ii) the rainfall depth and slope of the rainfall frequency curve at the credible limit of extrapolation.

As discussed above, estimates of the AEP of the PMP are subject to a high degree of uncertainty and are based on the interpretation of the PMP values as operational estimates that can be exceeded, rather than upper limiting values of rainfall. This interpretation of the PMP implies that the frequency curve should not be asymptotic to the horizontal at the estimated PMP, but rather extend through the PMP at a slope consistent with the shape of the lower sections of the frequency curve.

3.5.2. Interpolation Procedure

3.5.2.1. Basis of Interpolation Procedure

Siriwardena and Weinmann (1998) developed a procedure suited to the interpolation between regional estimates of Very Rare rainfalls and the PMP. The procedure was developed and tested on Victorian data using design information from the CRC-FORGE (Cooperative Research Centre - Focussed Rainfall Growth Estimation) procedure. While it is possible that other procedures may be developed for other regions, the procedure developed by Siriwardena and Weinmann is described here as it is considered to have generic applicability.

The procedure is applicable to 'gaps' of different ranges corresponding to differences in both the AEP of the credible limit of extrapolation and to the assigned AEP of the PMP. The procedure involves the fitting of a 2-parameter parabolic function in log-log space to ensure a smooth, well-behaved function when design rainfalls are plotted against AEP on logarithmic scales. The following boundary conditions are adopted:

- at the starting point of interpolation, the slope of the interpolated curve matches the slope defined by design estimates from the upper segment of the frequency curve bounded at the upper end by the credible limit of extrapolation; and,

- the slope of the interpolated curve through the PMP estimate is not constrained to the horizontal but is determined by the shape of the frequency curve at AEPs more frequent than that assigned to the PMP.

It needs to be emphasised that the interpolation is entirely determined by estimates of the conditions at the two end points; no additional information is introduced in fitting the curve. Details on the derivation of the procedure can be found in Siriwardena and Weinmann (1998).

3.5.2.2. Detailed Steps in Interpolation Procedure

Application of the procedure is quite straightforward, and design estimates over the interpolated range can be easily computed, as described below.

With reference to Figure 8.3.3, the AEP of 1 in Y_2 represents the starting point of the interpolation (the credible limit of extrapolation), and the AEP of 1 in Y_1 represents a lower value such that between 1 in Y_1 and 1 in Y_2 the frequency curve can be assumed to be linear in the log-log domain. X_{Y1} and X_{Y2} represent the design rainfalls with AEPs of 1 in Y_1 and 1 in Y_2. The slope of the frequency curve at the commencement of the transition, S_{gc}, is determined by the slope between the two design values at AEPs of 1 in Y_1 and 1 in Y_2.

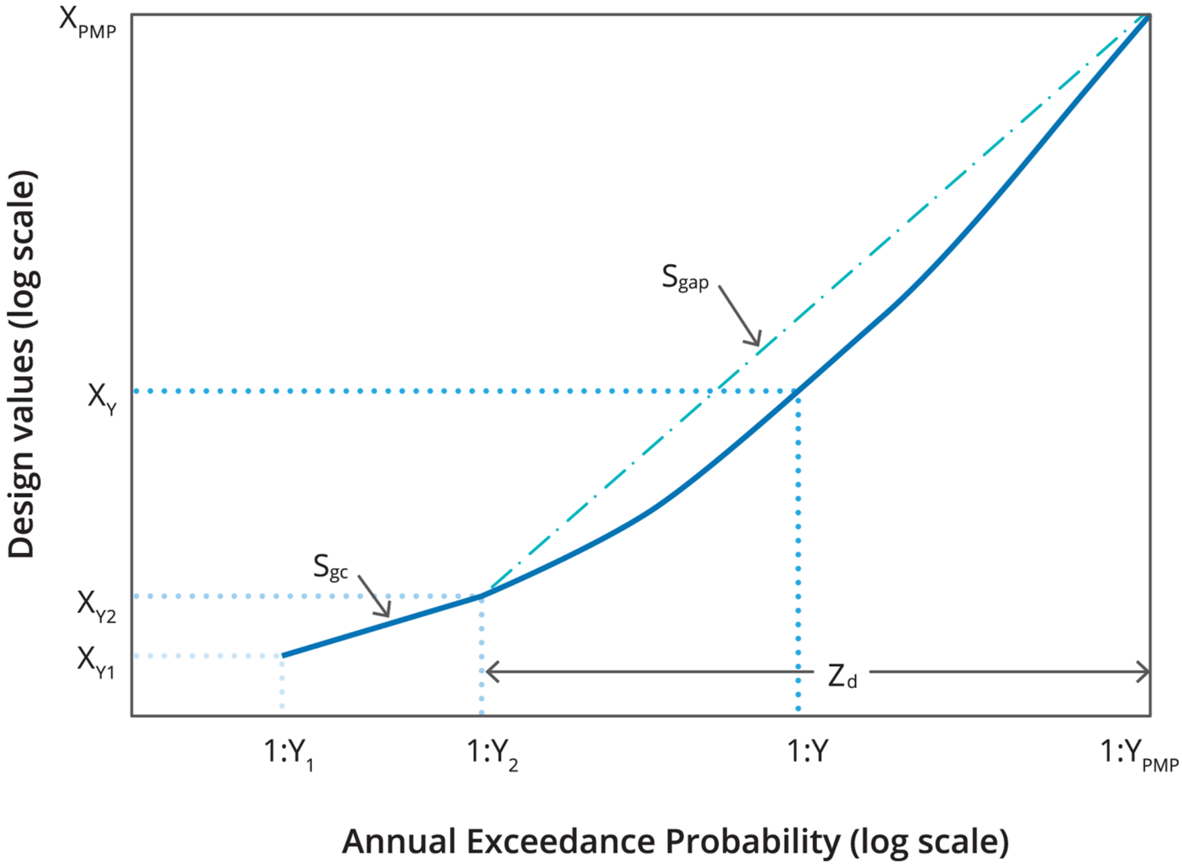


Figure 8.3.3. Schematic Illustration of Interpolation Procedure

The end point of the interpolation is the AEP of the PMP, which is denoted 1 in Y_{PMP}. For consistency of nomenclature, the magnitude of the PMP is here denoted as X_{PMP}.

A design rainfall estimate of 1 in Y AEP (denoted X_Y) can be estimated using:

$$X_Y = 10^{R_Y \log(X_{Y2})} \tag{8.3.1}$$

where R_Y is defined by the parabola fitted to the coordinates of the two end points (ie between X_{Y2} , Y_2 and X_{PMP}, Y_{PMP}) and the slope of the lower end of the frequency curve (ie the straight line between X_{Y1}, Y_1 and X_{Y2}, Y_2).

$$R_Y = 1 + S_{gc} z_d g_Y + \left(S_{gap} - S_{gc}\right) z_d g_Y^2 \tag{8.3.2}$$

$$z_d = \log\left(\frac{Y_{PMP}}{Y_2}\right) \tag{8.3.3}$$

$$g_y = \frac{\log\left(\frac{Y}{Y_2}\right)}{z_d} \tag{8.3.4}$$

$$S_{\mathrm{gc}} = \frac{\log\left(\frac{X_{\mathrm{Y1}}}{X_{\mathrm{Y2}}}\right)}{\log(X_{\mathrm{Y2}})\log\left(\frac{Y_1}{Y_2}\right)} \tag{8.3.5}$$

$$S_{\mathrm{gap}} = \frac{\frac{\log(X_{\mathrm{PMP}})}{\log(X_{\mathrm{Y2}})} - 1}{z_d} \tag{8.3.6}$$

Siriwardena and Weinmann (1998) recommend that the slope of the frequency curve at the commencement of the interpolation should be defined by the 1 in 1000 AEP and 1 in 2000 AEP events, i.e. Y_1 = 1000 and Y_2 = 2000. Thus the start point of interpolation is the credible limit of extrapolation obtained from the upper limit of design rainfalls obtained from the regional LH-moments approach (Book 2, Chapter 3).

An example describing the application of the above interpolation procedure is provided in Book 8, Chapter 8, Section 2.

3.5.2.3. Range of Application

Siriwardena and Weinmann (1998) have shown that the procedure performs satisfactorily over a range of design situations that specifically include:

- different starting points for interpolation (i.e. the AEP of the credible limit of extrapolation may vary);
- different AEPs assigned to the PMP (ranging from 10^{-4} to 10^{-7}, as discussed in Book 8, Chapter 3, Section 4); and,
- different 'shape parameters' defined by the ratio of the slope of the upper end of the directly determined frequency growth curve, S_{gc}, and the slope between the two end points of the 'gap', S_{gap} (the 'shape parameter' S_{gc}/S_{gap} ranges between 0.25 to 2.0).

The above concepts are schematically illustrated in Figure 8.3.3. Siriwardena and Weinmann (1998) have tested the above interpolation procedure on 25 catchments ranging in size from 25 to 15000 km^2 with diverse characteristics. The resultant frequency curves were shown to be plausible and well behaved for all test catchments. However, it is worth noting that Hill et al. (2000) reported that the above interpolation approach did not yield plausible results for GSAM-derived storms for 13 small catchments in South Australia. They observed that inconsistencies in the relationship between rainfall depth and catchment size for short- and long-duration events resulted in physically infeasible frequency curves (i.e. values of S_{gc}/S_{gap} exceeded 2.0). This problem was largely obviated by undertaking the above interpolation procedure in the log-Normal domain (i.e. using the standard normal variate of the exceedance probabilities rather than the log of the inverse of AEP); in a few cases it was also necessary to slightly increase the estimate of the AEP of the PMP, but the degree of change was well less than the notional uncertainty involved. Thus, while the recommended interpolation procedure has been found to generally yield plausible results, it may be necessary to make pragmatic adjustments to the method where dictated by circumstances.

3.6. Estimation of Rainfall Depths for Non-Standard Durations

3.6.1. General

The application of generalised methods yields design rainfall depths for a range of standard durations. The design rainfalls presented in Book 2 ensure that rainfall depths can be derived for a consistent set of durations for standard AEPs, though there are a minority of circumstances where approximate procedures may be required to derive estimates for non-standard combinations.

There are three broad design categories for which non-standard durations may be required:

- Very Rare event rainfalls for durations intermediate to multiples of 24 hour periods;
- Very Rare event design rainfalls for durations less than 24 hours; and
- Design rainfalls for very long durations (ie durations longer than those obtainable from any design rainfall database).

Guidance on the above three categories is provided within this section.

3.6.2. Very Rare Rainfalls for Intermediate Durations

Over a limited range of storm burst durations, the variation of point rainfall depth with duration can be closely approximated by a power function relationship. Weinmann et al. (1998) thus propose that design rainfalls for intermediate durations may be estimated by linear interpolation between log-transformed rainfall depths and the log-transformed interval between adjacent standard durations (e.g. 24 and 48 hours).

3.6.3. Very Rare Event Rainfalls for Short Durations

Sites with daily rainfall records provide a considerably denser spatial coverage and longer period of record than is available from the pluviograph network. The majority of research effort to date has been focussed on the derivation of Very Rare design rainfalls for burst durations of 24 hours and longer (as provided in Book 2), and by comparison the availability of design information for shorter duration rainfalls is rather limited.

The Bureau of Meteorology is scheduled to produce very rare rainfalls for durations less than 24 hours and when available these should be used to estimate rainfall up to the credible limit. Until this information is released, the growth factors derived by Jordan et al. (2005) should be used. These estimates are based on the analysis of data from ten pluviograph sites around Australia. Melbourne had the longest period of record at 130 years. Five of the stations used (Darwin, Sydney, Hobart, Adelaide, and Perth) had over 80 years of record each. The frequency analysis was undertaken using the simple “station year” method as the data satisfied the required assumptions of independence and homogeneity (the storms were largely derived from thunderstorm or deeply convective events). For the ten stations analysed, this pooled data set represents a sequence of around 800 station years. Non-dimensional frequency curves were derived for eight durations varying between 0.5 and 12 hours. The mean growth curve obtained from these distributions fell well within the 90% confidence limits (refer to Table 8.3.2).

Pending the outcome of more comprehensive analyses, it is recommended that the growth curve factors in Table 8.3.2 be used for design purposes. Rainfall depths for durations between 0.5 and 12 hours can be obtained by simply multiplying the relevant 1 in 100 AEP design rainfall by the frequency factors shown in Table 8.3.2. It should be noted that these factors represent the characteristics of events that are associated with thunderstorms, or deeply convective, storm activity and are derived from analyses that are largely independent of the data and procedures described in Book 2, Chapter 3. Accordingly, in some locations there may be the potential for significant discontinuity in growth factors between the values in Table 8.3.2 and those for longer duration events (24 hours and longer). If this is the case then it may be necessary to smooth the growth factors to ensure that the tails of the frequency curves do not cross, and that the rainfall depths vary in a consistent manner across storm duration and exceedance probability.

Table 8.3.2. Growth Curve Factors for Derivation of Sub-Daily Design Rainfalls Standardised by the 1 in 100 AEP Rainfall Depth)

AEP (1 in Y)	100	200	500	1000	2000
Growth Factor	1.00	1.140	1.344	1.513	1.698

3.6.4. Rainfalls for Very Rare to Extreme Events of Very Long Durations

For dams with very large storage volumes relative to the volumes of inflow floods or dams with little or no spillway provision, or for some very large catchments, it is possible that the critical duration of interest may be longer than available from the generalised design rainfall information. The longest available storm duration using Book 2 procedures is 168 hours (7 days). This duration generally relates to the meteorological limits associated with single storm events, and thus longer duration design events involve the consideration of storm sequences.

The approach to solving design problems involving long critical durations is in essence a joint probability problem. In special circumstances the problem may involve the assessment of joint probabilities of extreme storm sequences, but when considering issues associated with reservoir outflow floods, the issue of storm sequences over extended periods may be implicitly solved by undertaking a joint probability analysis of inflow floods and initial reservoir volume (Book 8, Chapter 7, Section 2).

3.6.4.1. Storm Sequences in South-Eastern Australia

Analysis of storm data in south-eastern Australia (Minty and Meighen, 1999) indicates that about 40% of large storms are preceded by a rainfall event in the 15 days prior to the storm. Based on their magnitude, these antecedent rainfall events appear to comprise two different populations: most (32% of all large storms) had accumulated rainfall totals of less than 30% of the subsequent large storm, but a small proportion (8% of all large storms) had accumulated rainfall totals of between 30% and 80% of the subsequent large storm.

In addition, Scorah et al. (2015) undertook an analysis of areal antecedent rainfalls in the inland GSAM region for periods ranging between 7 days and 24 months using 113 years of gridded data. The analysis was undertaken for storm areas of 750 km^2 and 1860 km^2. They concluded that there is no correlation between pre-storm rainfalls and storm severity for the extremes considered, and thus the two processes could be treated independently in joint probability analyses.

3.6.4.2. Storm Sequences in Tropical Regions

The nature of rainfall sequences for Rare to Extreme events in the tropical region is not as well understood. Limited evidence on the dependence of antecedent rainfalls is provided by Scorah et al. (2015) based on an analysis of areal rainfalls in the coastal GSTMR region. Scorah et al. (2015) undertook the analysis for a storm area of 750 km^2 and found that total rainfalls in the three months prior to the most extreme maxima on record were larger than the 20% percentile values. However, little correlation was found between the severity of the event and 7 day maxima within the preceding three months. Overall, it might be expected that the conditions prior to Extreme events are typically wetter than more frequent events, but further analysis would be required for specification of dependencies on initial reservoir level.

3.7. Seasonal Estimates of Rare to Extreme Rainfalls

3.7.1. Theoretical and Practical Issues

The derivation of the design rainfall data as discussed in Book 8, Chapter 3, Section 2 to Book 8, Chapter 3, Section 6 is based on the assumption that rainfall is independent of the season in which it occurs, or at least that its seasonal variation does not have a significant influence on the design outcome. However, there are situations where the seasonal variation of rainfall characteristics is significant and may need to be taken into account in design flood estimation. As an example, severe thunderstorms may occur predominantly during the summer season. Where other design factors (such as initial loss or initial reservoir level) are also characterised by significant seasonal variation, it may be necessary to combine seasonal design rainfalls with the corresponding seasonal values of these other design factors, rather than with their average annual values. For this purpose, a season is defined as a period of one to several months during which the rainfall conditions (and other design factors) can be assumed to be the same.

Book 8, Chapter 3, Section 7 describes typical design situations where seasonal estimates of design rainfalls for Rare to Extreme events may be required. While it would appear sensible in these design situations to deal explicitly with seasonal effects, there are a number of practical and theoretical issues that are not easily resolved. Some of the issues related to the derivation of seasonal design rainfalls are discussed.

3.7.1.1. Seasonal Rainfall Estimates for Rare Events

Seasonal design rainfalls for AEPs equal to or more frequent than 1 in 100 AEP cannot be obtained directly from information provided in Book 2. One approach is to extract seasonal maxima from rainfall records at a particular site, and then undertake a frequency analysis to derive seasonal rainfalls (the Bureau of Meteorology can provide these estimates if required). However, this approach can provide inconsistent seasonal estimates for the rarer events because of the inherent uncertainties in fitting the tails of the distribution to observed data, though theoretically this could be overcome by developing a fitting procedure that jointly fits all the seasonal distributions. In addition, the seasonal design rainfalls derived from the at-site data will need to be adjusted to ensure that the annual design rainfalls are consistent with the Book 2 estimates.

3.7.1.2. Seasonal Rainfall Estimates for Very Rare events

At present regional frequency estimates of seasonal rainfalls for AEPs rarer than 1 in 100 AEP are only available for regions in Western Australia (Durrant and Bowman, 2004; Durrant et al., 2006). Similar analyses could be undertaken using seasonally censored data for other hydrometeorologically homogeneous regions, where the likelihood of rainfalls occurring in different seasons could then be applied to the Very Rare design rainfalls which have been derived on an annual basis (as described in Book 8, Chapter 3, Section 7).

3.7.1.3. Seasonal Probable Maximum Precipitation Estimates

The PMP definition quoted in Book 8, Chapter 3, Section 3 allows different PMP estimates to be derived for different parts of the year, i.e. seasonal PMPs. A procedure for estimating seasonal PMPs for short duration storms on areas up to 1000 km^2 in southern Australia is given in the GSDM method (Durrant et al., 2006). Approximate seasonal estimates for four seasons are available for longer duration events in south-east Australia using the GSAM procedure, but it should be recognised that these estimates are based on a biased seasonal sample as the storms were selected on the basis of magnitude rather than season. Seasonal PMP estimates for longer duration storms in tropical areas (i.e. GTSMR estimates) are available for summer and winter seasons.

3.7.1.4. Annual Exceedance Probability of the Seasonal Probable Maximum Precipitation

At present there is no generally accepted procedure for assigning an AEP to a seasonal PMP. Laurenson and Kuczera (1998) give two alternative approaches to derive the AEP of seasonal PMP estimates.

The first approach is based on the assumption that factors other than dew point (and factors deriving from that) affecting the value of PMP do not significantly vary with season. This is consistent with the Bureau of Meteorology's assumption that each season has its own PMP; in other words, the magnitude of the seasonal PMP is different for different months of the year. It also follows that the probability of experiencing a PMP event of different magnitude in any month of the year is equal.

This interpretation means that the exceedance probability of a PMP event occurring *in a specific season* of the year is proportional to the fraction of the year occupied by that season, but it does not yield directly an estimate of the exceedance probability of a seasonal PMP. The additional constraint to be considered follows from an argument based on extreme value theory, namely that the sum of the exceedance probabilities of events of *the same magnitude* for the different seasons should add to the AEP of the annual rainfall event of that magnitude.

In practice, an iterative approach needs to be adopted, using the product of the AEP of the annual PMP and the fraction of the year occupied by the season as an initial (lower bound) estimate of the exceedance probability of the seasonal PMP estimates. These initial estimates are shown as hollow circles in Figure 8.3.4. A segment of the complete design rainfall frequency curve for each season then needs to be drawn between the rainfall depths of the largest and smallest seasonal PMP estimates (indicated by broken lines in Figure 8.3.4). Over the upper range of the seasonal rainfall magnitudes, the curve segments can be assumed to be parallel to the annual frequency curve. The addition of the AEPs corresponding to the annual PMP estimated from each seasonal rainfall curve will generally yield an AEP less than the AEP assigned to the annual PMP. The ratio of these two AEP estimates defines the correction factor (> 1.0) that needs to be applied to each of the initially estimated AEPs of the seasonal PMP. This correction is indicated by arrows in Figure 8.3.4, and the final AEP estimates of seasonal PMPs are shown as filled in circles.

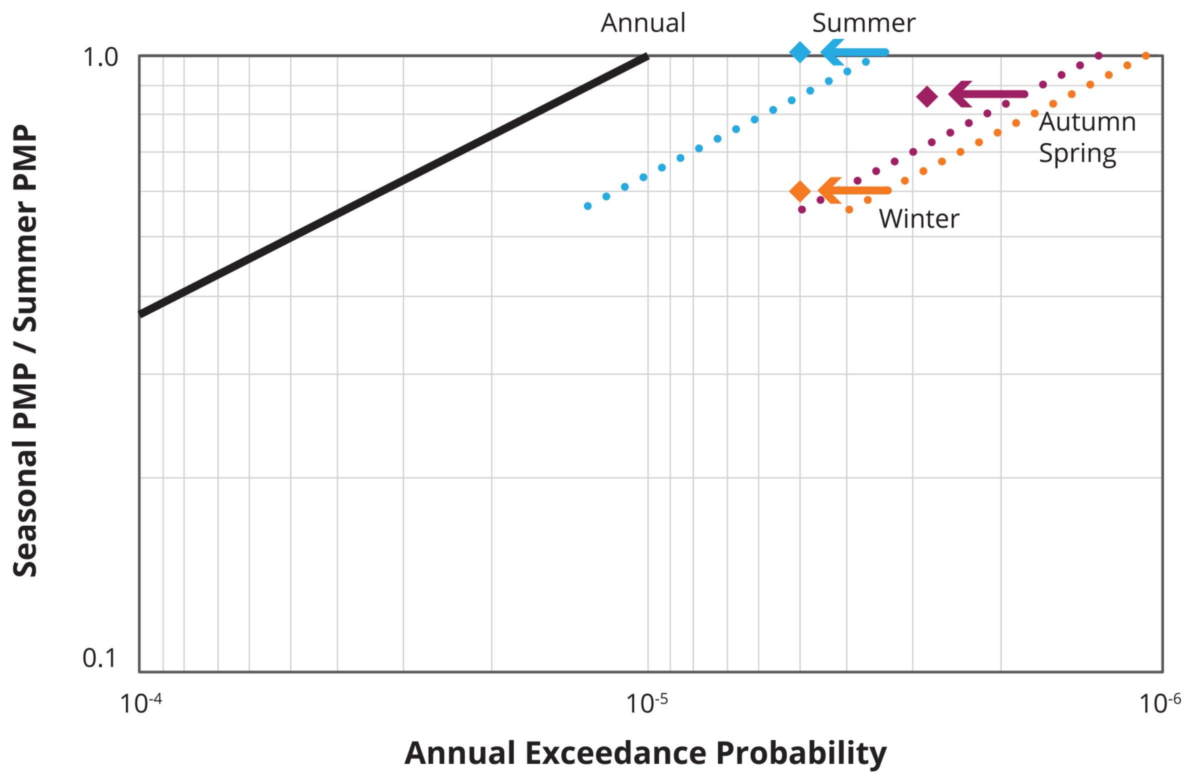


Figure 8.3.4. Hypothetical Frequency Curves for Seasonal and Annual Design Rainfalls Based on the AEP Assigned to the Annual PMP (adapted from Laurenson and Kuczera (1998), using Four Seasons of Relative Lengths 0.33, 017. 0.33, 0.17 and Relative Seasonal PMP Depths of 1.0, 0.85, 0.6, 0.85, for Summer, Autumn, Winter, and Spring, respectively)

The second approach proposed by Laurenson and Kuczera (1998), which is not fully developed at this stage, does not use the upper limit concept, but recommends the derivation of separate extreme rainfall frequency curves for each season, using the joint probability method (Pearse and Laurenson, 1997).

3.7.2. Derivation of Seasonal Design Rainfalls

The procedure required to derive a complete seasonal frequency curve of design rainfalls is not straightforward, and is subject to differences in interpretation, particularly in respect to assigning an AEP to the seasonal PMP. The basic criterion to be satisfied by any procedure for estimating seasonal rainfall frequencies is that, for any given rainfall magnitude, the seasonal frequencies over all seasons should add up to the AEP of the rainfall magnitude determined from the analysis of annual rainfalls.

In the absence of better design information, and noting the foregoing discussion, the following recommendations should prove adequate for most design problems where seasonal effects are important.

3.7.2.1. Rare Events

Both seasonal and annual frequency analyses should be undertaken using rainfall data obtained from sites relevant to the study area. When applied in conjunction with seasonal PMP estimates, the adopted seasons should correspond to the seasons used in the derivation of the seasonal PMP depths, and the seasonal rainfall estimates should be expressed as fractions of the annual estimates. The seasonal fractions can then be converted to design rainfall depths by multiplying by the (annual) design rainfall values obtained from the standard information provided in Book 2. Note that the inherent uncertainties in fitting the tails of the distributions to observed seasonal data may mean that for a given rainfall magnitude the sum of the seasonal exceedance probabilities do not equal the annual exceedance probability. If this problem occurs one or more of the seasonal frequency curves will need to be adjusted to ensure that the seasonal and annual exceedance probabilities are consistent.

3.7.2.2. Very Rare Events

Unless specific regional estimates are available, the seasonal fractions corresponding to design rainfalls at the credible limit of extrapolation may be obtained by an interpolation procedure similar to that used for losses (e.g. linear interpolation on a log-log frequency plot – Equation (8.4.1)). The lower and upper end points used in the interpolation are defined, respectively, by the seasonal fractions derived for the 1 in 100 AEP and PMP design rainfalls. Once the seasonal fractions have been obtained by interpolation, seasonal design rainfalls for Very Rare events are derived by multiplying the fractions by the (annual) design rainfall values at the credible limit of extrapolation.

3.7.2.3. Probable Maximum Precipitation Events

Seasonal estimates of the PMP should be obtained from the Bureau of Meteorology. When plotted with Very Rare event seasonal design rainfalls for the corresponding season, the seasonal PMP estimates should be assigned an AEP equal to the product of the AEP of the annual PMP (from Figure 8.6.1) and the fraction of the year occupied by the season. These AEPs need to be adjusted to ensure that the sum of the exceedance probabilities of events of *the same magnitude* for the different seasons add to the AEP of the annual rainfall event of that magnitude (as discussed in Book 8, Chapter 3, Section 7, and illustrated in Figure 8.6.1). It is important to recognise that the uncertainty associated with the currently available seasonal PMP estimates is higher than that for the annual estimates.

3.8. Temporal Patterns

3.8.1. General

The temporal patterns provided in Book 2 relate to the time distribution of design rainfall depths within rainfall bursts. Additional rainfall occurring immediately before the start of the burst, as part of a complete storm event, can be accounted for by ‘pre-burst’ temporal patterns.

The concept of a single ‘representative’ temporal pattern that allows a probability neutral transformation of design rainfall inputs to flood outputs of the same AEP is basically flawed, as this transformation is quite sensitive to the routing characteristics of the catchment. This sensitivity can best be allowed for by applying an ensemble of typical temporal patterns rather than a single design temporal pattern, as can be done in the Ensemble Event and Monte Carlo Event approaches.

3.8.2. Specific Recommendations

3.8.2.1. Selection of Patterns for Design Bursts

Table 8.3.3 summarises the recommended application of different temporal patterns for design rainfall bursts in the range of Very Rare to Extreme events. There are three main sources of design information:

- short duration point rainfall patterns from GSDM (Bureau of Meteorology, 2003a; Jordan et al., 2005);
- long duration rainfall patterns for use across Australia (Book 2, Chapter 5); and
- areal temporal patterns developed for the generalised GSAM and GTSM-R PMP methods.

Ensemble sets of areal temporal patterns are available for the latter two sources of data and there are advantages and limitations to using both sets. The prime advantage to using areal temporal patterns derived for use with PMP estimates is that they are based on careful hydrometeorological analysis of storms that are known to be the most extreme in the historical record. The disadvantage of them is that more extreme storms may have occurred since development of the methods, and – particularly in the inland GSAM region – there are a disparate number of patterns in the different combinations of storm areas and durations. Conversely, as described by Podger et al. (2016) the areal patterns provided in Book 2, Chapter 5 are based on the largest storms that have occurred in eleven regions across Australia. The limitation of these patterns with respect to extreme events is that they were selected on the basis of the depth (rather than rarity) of their associated rainfalls, and also that they were derived for smaller regions than used in development of the PMP methods. As such, it is likely that these patterns are associated with events that are less severe than those considered in the PMP analyses. Conversely, their main advantage is that they may have included extreme events that have occurred since completion of the PMP analyses, and also that they provide a consistent set of ten patterns for a more comprehensive range of storm area and duration combinations.

Further analysis of the efficacy of these different data sets for application to Very Rare and Extreme event is warranted, but at present it is recommended that the PMP method areal temporal patterns be used to derive all design events rarer than 1 in 100 AEP. That said, it may be appropriate to use the Book 2, Chapter 5 areal patterns in lieu of the PMP patterns to reconcile flood estimates with independently derived design flood estimates (as described in Book 5, Chapter 3, Section 4). If so, it may be prudent to adopt a changing mix of areal temporal patterns from both sources of data over the Very Rare range to ensure a smooth transition in flood response over this range of exceedance probabilities. Also, where there is a paucity of information on areal temporal patterns – such as for some durations in the inland GSAM zone – it may be necessary to supplement the adopted ensembles using the patterns provided in Book 5, Chapter 3, Section 4. At present, the best source of ensemble temporal patterns for use with short duration Very Rare to Extreme events are those derived by Jordan et al. (2005); these patterns were derived specifically from storms associated with thunderstorm or deeply convective events.

An issue requiring specific mention is the absence of temporal patterns in south-eastern Australia for use with storm durations between the upper limit of GSDM (3 or 6 hours) and the lower limit of the GSAM method (generally 24 hours). A pragmatic solution to this problem is to apply both sets of temporal patterns and to adopt a weighted average peak flow, where the weighting is based on storm duration. The weighted average peak flow is then used to scale the hydrograph obtained using the most relevant generalised method;

weighting all the ordinates of the hydrograph is not recommended as the resulting hydrograph may exhibit a lower peak than either of the individual hydrographs.

In the transition zone between the GTSMR and GSAM regions, temporal patterns from both the GSAM and GTSMR methods should be applied separately (in conjunction with the corresponding spatial patterns), and the largest flood adopted.

3.8.2.2. Patterns for Complete Storms

The design information required to define the design rainfall depths and temporal patterns for complete Very Rare and Extreme storms is available nationally (Book 2, Chapter 5). These pre-burst patterns might be suitable for scaling to more extreme events, but it should be noted that the patterns provided are for point not areal storms, and will need censoring to ensure that the patterns selected are from appropriately rare events. Book 2, Chapter 5 outlines the principles for constructing complete storms from design bursts using dimensionless pre-burst temporal patterns. Guidance on the determination of rainfall excess for complete storms is provided in Book 8, Chapter 4, Section 3.

3.8.2.3. Dealing with Inconsistencies and Smoothing of Results

In practice, the simplistic use of single design temporal patterns for different durations and AEPs can yield flood estimates that do not vary in a consistent manner. In extreme cases, this can result in design flood magnitudes that decrease with decreasing AEP. More typically, the patterns may result in critical storm durations that vary inconsistently with AEP; such a variation will impact upon the volume of design hydrographs which, when routed through a reservoir, may produce inconsistent results. The judicious use of simulation results using ensembles of temporal patters will largely avoid such inconsistencies. Problems are more likely to occur with the transition between temporal patterns for more frequent events and those derived for PMP events. If problems arise consideration should be given to filtering out (or excluding) embedded bursts of lower AEP by re-distributing rainfalls of high intensity to other time increments proportionally to their magnitude (e.g. (Herron et al., 2011)). Where significant inconsistencies remain, practitioners will need to apply judicious smoothing of results for different durations and AEPs.

Table 8.3.3. Selection of Design Burst Temporal Patterns for Different Regions, Durations and AEPs

Descriptive Event Class	Range of AEP	Storm Duration and Source of Design Information				
		Short Durations for Whole of Australia	Long Durations in South-East Australia		Long Durations in Tropical Regions	
		Up to 3 or 6 hours duration	Intermediate durations	24 hours and longer	Intermediate durations	24 hours and longer
Very Rare	Beyond 1 in 100 up to 1 in 2000 AEP	Use patterns as for Extreme range below, though consideration should be given to including areal temporal patterns as described in Book 2, Chapter 5 and Podger et al. (2016). These areal patterns are likely to be from more frequent storms than those available from the PMP methods. As such, they may be more suited to reconciliation with other independently derived design flood estimates (as described				

Descriptive Event Class	Range of AEP	Storm Duration and Source of Design Information				
		Short Durations for Whole of Australia	Long Durations in South-East Australia		Long Durations in Tropical Regions	
		Up to 3 or 6 hours duration	Intermediate durations	24 hours and longer	Intermediate durations	24 hours and longer
		in Book 5, Chapter 3, Section 4) rather than for deriving design estimates.				
Extreme	**Rarer than 1 in 2000 AEP**	Deterministic areal patterns from GSDM method, ensemble patterns from Jordan et al. (2005)	24 hour GSAM and longest duration GSDM areal rainfall patterns	GSAM areal rainfall patterns (single and ensemble)	Both 24 hour GTSMR areal patterns and longest duration GSDM patterns	GTSMR areal patterns (single and ensemble)

3.9. Spatial Patterns

3.9.1. Basis of Adopted Patterns

Design spatial rainfall patterns are also required to fully define design rainfall events, and general guidance on this is provided in Book 2. The source of spatial patterns as a function of burst duration and AEP is broadly similar to that adopted for temporal patterns (except for the Very Rare rainfall category), and is summarised in Table 8.3.4. As discussed in Book 8, Chapter 3, Section 9 there are four main sources of design information: patterns based on the spatial distribution of design rainfalls for Very Rare events, spatial patterns for use in south-eastern Australia (Minty et al., 1996), GSDM thunderstorm patterns (Bureau of Meteorology, 2003a), and tropical storm patterns (Bureau of Meteorology, 2003b). As with temporal patterns, the last three sets of patterns were originally derived for application to PMP events, but in the absence of any more relevant information they are applied to the range of Very Rare to Extreme events.

Except for catchments with marked rainfall gradients, the spatial distribution of rainfall generally has less influence on the shape and size of the resulting hydrograph than temporal patterns. Thunderstorm and tropical patterns can have an appreciable effect on flood magnitude, particularly if the catchment contains extensive drowned reaches resulting from reservoir inundation. For such catchments, small variations in the spatial distribution of design rainfall may have a marked impact on the magnitude of the flood peak. It is worth assessing the sensitivity of the catchment floods to variations in spatial patterns, and if this is not easily resolved then it would be necessary to include spatial patterns as an ensemble in Monte Carlo analyses.

3.9.2. Specific Recommendations

i. *Very Rare events.*- Spatial rainfall trends may be characterised by dividing the catchment into two or more sub-catchments, and deriving design rainfalls separately for each.

ii. *Extreme short duration events.*- The GSDM thunderstorm patterns (Bureau of Meteorology, 2003a) should be used. The spatial pattern should generally be centred over the catchment and orientated in such a way as to overlap the catchment boundary with the smallest possible ellipse.

iii. *Extreme long duration events in south-eastern Australia.*- The spatial patterns provided with GSAM estimates (Minty et al., 1996) should be applied to all Very Rare to Extreme events. The spatial patterns are based on modified 72 hour 50 year ARI intensity fields of design rainfalls from Book 2, and they incorporate the combined effect of variations in elevation, slope, aspect and geographical location. These patterns should not be rotated or translated.

iv. *Extreme long duration events in tropical regions.*-The spatial patterns provided with GTSMR estimates (Bureau of Meteorology, 2003b) should be applied to all Very Rare to Extreme events. The spatial pattern should be positioned to maximise the rainfall depth within the catchment.

v. *Extreme long duration events in the transition zone.* In the transition zone between the GSAM and GTSMR regions, both sets of spatial patterns should be used (in conjunction with the corresponding temporal patterns) and the highest resulting flood should be adopted.

Table 8.3.4. Selection of Design Spatial Patterns for Different Regions, Durations and AEPs

Descriptive event class	Range of AEP	Storm duration and source of design information			
		Short durations for Wwhole of Australia (GSDM)	Long durations in southeast Australia (GSAM method)		Long durations in tropical regions (GTSMR method)
		Up to 3 or 6 hours duration	Intermediate durations between GSDM and GSAM	24 hours and longer	Longer than 6 hours
Very Rare	**Beyond 1 in 100 to the credible limit of extrapolation**	Based on design rainfalls for Very Rare events derived separately for each sub-catchment			
Extreme	**Beyond the credible limit of extrapolation**	GSDM spatial patterns	Both GSAM and GSDM spatial patterns	GSAM spatial patterns	GTSMR spatial patterns

3.10. References

Bureau of Meteorology (2006), Guidebook to the Estimation of Probable Maximum Precipitation: Generalised Southeast Australia Method. Hydrometeorological Advisory Service, Bureau of Meteorology.

Bureau of Meteorology (2003a), The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method. Hydrometeorological Advisory Service, Bureau of Meteorology.

Bureau of Meteorology (2003b), Guidebook of the Estimation of Probable Maximum Precipitation: Generalised Tropical Storm Method. Hydrometeorological Advisory Service, Bureau of Meteorology.

Durrant, J.M. and Bowman, S. (2004), Estimation of Rare Design Rainfalls for Western Australia: Application of the CRC-FORGE Method, Surface Water Hydrology Report Series Report No. HY17, Department of Environment, Government of Western Australia.

Durrant, J.M, Nandakumar, N. and Weinmann, P.E. (2006), Forging Ahead: Incorporating Seasonality into Extreme Rainfall Estimation for Western Australia. Australian Journal of Water Resources, 10(2), 195-205.

Fontaine, T.A. and Potter, K.W. (1989), Estimating Probabilities of Extreme Rainfalls, Journal of Hydraulic Engineering, Volume 115, pp. 1562-1575, American Society of Civil Engineers.

Green, J.H., Beesley, C., The, C., Podger, S. and Frost, A. (2016), Comparing CRC-FORGE estimates and the New Rare Design Rainfalls. Proc. ANCOLD Conference, Adelaide.

Herron, A., Stephens, D., Nathan, R. and Jayatilaka, L. (2011), Monte Carlo Temporal Patterns for Melbourne. In: Proc 34th World Congress of the International Association for Hydro- Environment Research and Engineering: 33rd Hydrology and Water Resources Symposium and 10th Conference on Hydraulics in Water Engineering. Barton, A.C.T.: Engineers Australia, pp: 186-193.

Hill, P.I., Nathan, R.J., Rahman, A., Lee, B.C., Crowe, P. and Weinmann, P.E. (2000), Estimation of extreme design rainfalls for South Australia using the CRC-FORGE method. In: Proceedings of 3rd international hydrology and water resources symposium interactive hydrology, IE Aust., Perth, Western Australia, 1: 558-563, 20-23 Nov 2000.

Jordan, P., Nathan, R., Mittiga, L. and Taylor, B. (2005),'Growth Curves and Temporal Patterns for Application to Short Duration Extreme Events'. Aust J Water Resour, 9(1), 69-80.

Kennedy, M.R and Hart, T.L. (1984), The estimation of probable maximum precipitation in Australia. Civ. Engg Trans, Inst Engrs Aust, CE26: 29-36.

Klemes, V. (1993), Probability of extreme hydrometeorological events - a different perspective. Extreme hydrological events, precipitation, floods, and droughts. Yokohama Symposium., Yokohama, IAHS Publication No. 213. pp: 167-188.

Kyselý, J. (2008), A Cautionary Note on the Use of Nonparametric Bootstrap for Estimating Uncertainties in Extreme-Value Models. J. Appl. Meteor. Climatol., 47: 3236-3251.

Lang, S., Zhang, J., Scorah, M., and Nathan, R. (2016), Characterising the location and rarity of annual maxima rainfall in the GTSM-R coastal zone. Proc. Hydrology and Water Resources Symposium, Melbourne.

Laurenson, E.M. and Kuczera, G.A. (1999), Annual exceedance probability of probable maximum precipitation, Aus. J. Water Resour., 3(2), 189-198.

Laurenson, E.M. and Kuczera, G.A. (1998), Annual exceedance probability of probable maximum precipitation - report on a review and recommendations for practice. Report prepared for the NSW Department of Land and Water Conservation and the Snowy Mountains Hydro-Electric Authority.

MGS Engineering and Applied Climate Services (2014), Stochastic modelling of floods for the Bridge River system. Consulting report prepared for BC Hydro.

McConachy, F.L.N., Weinman, P.E., Nathan, R.J. and Mein, R.G. (1997), Confidence limits for rainfall frequency curves in the extreme rainfall range. Proceedings, 24th Hydrology and Water Resources Symposium, Auckland, New Zealand, pp: 333-338.

Minty, L.J. and Meighen, J. (1999), Development of temporal distributions of rainfall antecedent to large and extreme design bursts over southeast Australia, Hydrology Report Series HRS Report No.6, Hydrometeorological Advisory Service, Bureau of Meteorology.

Minty, L., Meighen, J. and Kennedy, M.R. (1996), Development of the Generalised Southeast Australia Method for estimating Probable Maximum Precipitation, Hydrology Report Series No.4, Hydrology Unit, Bureau of Meteorology, Melbourne.

Nandakumar, N., Weinmann, P.E., Mein, R.G. and Nathan, R.J. (1997), Estimation of extreme rainfalls for Victoria using the CRC-FORGE method (for rainfall durations 24 to 72 hours). CRC Research Report 97/4, Melbourne, Cooperative Research Centre for Catchment Hydrology.

Nathan, R.J., Scorah, M., Jordan, P., Lang, S., Kuczera, M., Schaefer, M. and Weinmann, P.E. (2015), A Tail of Two Models: Estimating the Annual Exceedance Probability of Probable Maximum Precipitation. Proc.Hydrology and Water Resources Symposium, Hobart.

Nathan, R.J., Weinmann, P.E. and Minty, L. (1999), Estimation of the Annual Exceedance Probability of PMP Events in Southeast Australia, Aus. J. Water Resour., 3(1), 143-154.

National Research Council, Committee on Techniques for Estimating Probabilities of Extreme Floods (1988), Estimating Probabilities of Extreme Floods, Methods and Recommended Research, National Acadamy Press, Washington D.C.

Pearse, M.A. and Laurenson, E.M. (1997), Real probabilities for real floods. Proc., ANCOLD 1997 Conference on Dams. ANCOLD.

Podger, S, Babister, M. and Brady, P. (2016), Deriving temporal patterns for areal rainfall bursts. Proc. 37th Hydrology and Water Resources Symposium 2016: Water, Infrastructure and the Environment. Barton, ACT: Engineers Australia.

Scorah, M., Lang, S. and Nathan, R. (2015), Utilising AWAP gridded rainfall dataset to enhance hydrology studies. Proc. Hydrology and Water Resources Symposium, Hobart.

Siriwardena, L. and Weinmann, P.E. (1998), A technique to interpolate frequency curves between frequent events and probable maximum events. CRCCH Research Report 98/9, Cooperative Research Centre for Catchment Hydrology, December 1998.

Weinmann, P.E., Rahman, A., Hoang, T., Laurenson, E.M. and Nathan, R.J. (1998), A new modelling framework for design flood estimation. 'HydraStorm'98', Adelaide, Australia, Institution of Engineers, Australia, pp: 393-398.

Wilson, L.L. and Foufoula-Georgiou, E. (1990), Regional Rainfall Frequency Analysis via Stochastic Storm Transposition, J Hydraul. Engin., 116(7), 859-880, American Society of Civil Engineers.

World Meteorological Organisation (1986), Manual for estimation of probable maximum precipitation, Second Edition, Operational Hydrology Report No.1, WMO - No.332, Geneva.

Chapter 4. Estimation of Rainfall Excess for Very Rare to Extreme Events

Rory Nathan, Erwin Weinmann

Chapter Status	Final
Date last updated	14/5/2019

4.1. General Considerations

A loss model is needed to partition the design rainfall input into rainfall excess (runoff) and loss. General guidance on loss modelling for the types of loss models used in common practice is provided in Book 5, Chapter 3. The following considerations and guidelines focus specifically on aspects of loss estimation related to the estimation of Very Rare to Extreme floods. The guidance is applicable to simulations undertaken using both deterministic and stochastic frameworks.

The specific recommendations in Book 8, Chapter 4, Section 3 apply to loss parameters for the Initial Loss – Continuing Loss (IL-CL) model, as a large body of relevant experience has been accumulated over many years. However, other loss models may be used if they can be shown to be more appropriate in the specific situation.

4.1.1. Importance of Design Losses – Very Rare to Extreme Events

Like temporal patterns of rainfall, design losses are highly variable and can have an appreciable impact on both the peak flow and volume of the resulting flood. A given rainfall occurring on a dry catchment produces a significantly smaller flood than the same rainfall occurring on a wet catchment. For more frequent events, loss may be the most important factor. Joint probability approaches (e.g. Weinmann et al. (1998)) are able to deal with the high variability of design losses better than the design event approach, as they use a probability distribution of loss values, rather than a single 'representative value'. ('Representative' means that the selected design loss values should ensure a 'probability neutral' transformation of the design rainfall input of a given AEP into a design flood output of the same AEP). However, the impact of the inter-event variability of losses and the relative importance of losses diminishes with decreasing AEP, and for Extreme events it is likely that losses are of lesser importance than temporal patterns. For the estimation of Very Rare to Extreme floods, the use of single-value representative design losses may be adequate, though when simulating long duration events for volume-dependent problems it may be appropriate to adopt stochastic approaches as discussed in Book 8, Chapter 4, Section 1.

For extreme rains and floods, a much greater proportion of a catchment may become saturated during the event than is the case for most floods in the observed range. Also, during extreme rainfalls, vegetation may be stripped from the catchment, thus resulting in an increase in the volume and speed of the overland flow component of runoff (Kemp and Daniell, 1997). Any evidence relevant to the changed behaviour of the catchment under extreme rainfall conditions should be considered when estimating design losses and the resulting design floods.

4.1.2. Losses Associated With Design Storms and Design Bursts

When considering the adoption of design losses it is necessary to understand the distinction between *design bursts* of rainfall, and *design storms*. The difference between the two concepts and the implications of the two concepts for the estimation of design losses are explained in Book 5, Chapter 3, Section 3. The selection of design loss values must take into consideration the manner in which the design information was derived, and whether the losses are to be applied to design storms or design bursts. More specifically, there is a significant difference between the initial loss values applicable to design storms and design bursts, and how these loss values can be expected to vary with event magnitude (see Book 8, Chapter 4, Section 1).

4.1.3. Variation of Loss Values with Event Magnitude

Different loss models will behave differently when extrapolated to Extreme events, as they introduce differing degrees of non-linearity into the transfer between design rainfalls and the resulting hydrograph. Thus, even if different loss models are able to reproduce calibration events equally well, adoption of the same loss parameters for derivation of Extreme design floods may produce significantly different design flood hydrographs. For example, a specific set of loss parameters for the Initial Loss – Continuing Loss (IL-CL) and Initial Loss – Proportional Loss (IL-PL) models may yield similar flood peaks for the 1 in 100 AEP design event, but if the same parameters were retained to derive the 1 in 10^6 AEP flood, the different loss models would produce markedly different design flood hydrographs. The impact of model structure on the extrapolation of loss parameters for application to Very Rare to Extreme events must thus be carefully considered.

The discussion in Book 5, Chapter 3, Section 4 makes it clear that Storm Initial Loss (IL_s) and Burst Initial Loss (IL_b) are expected to show a different degree of variation with event magnitude. The two types of initial loss for rural catchments are therefore treated separately in Book 8, Chapter 4, Section 1 and Book 8, Chapter 4, Section 1.

The interpretation of *Proportional Loss* (PL) as the unsaturated proportion of the catchment implies that with larger storm events the unsaturated proportion of the catchment is reducing and thus the proportional loss also reduces. As it is difficult to extrapolate the rate of this reduction to Extreme events, the proportional loss model is generally considered less appropriate for estimating Very Rare to Extreme floods. On the other hand, the *Continuing Loss* (CL) is expected to approach a limiting value for saturated catchment conditions, and this limiting value is the appropriate design loss rate for all events for which the saturation threshold has been exceeded. More detailed discussion of the variation of CL with event magnitude for rural catchments is given in Book 8, Chapter 4, Section 1.

4.1.3.1. Rural Catchments

4.1.3.1.1. Storm Initial Loss (IL_s)

The available evidence to support the conceptual interpretation of loss variation includes the results obtained by Hill et al. (1996a); these indicated little or no variation of design losses with rainfall severity for events more frequent than 1 in 100 AEP. For IL_s, this finding implies little or no correlation between the magnitudes of pre-storm rainfall (producing the storm antecedent conditions) and storm event rainfall for events more frequent than 1 in 100 AEP.

An analysis of the rainfall conditions prior to the largest storms on record in the GSAM region of south-eastern Australia (Minty and Meighen, 1999) indicated qualitatively no propensity for "greater than normal" rainfall in the 15 days immediately preceding these large storms. The analysis by (Minty and Meighen, 1999) shows that about 75% of the largest storms on record in south-eastern Australia were preceded by rainfall totals of less than 10% of the depth of the storm. Further, the analysis revealed that the average length of the dry period between pre-storm rainfall and the storms was about 8 days.

The available evidence thus suggests that there is no need to vary IL_s with event magnitudes up to the largest event on record. Further research is desirable to confirm the applicability of these findings of little or no variation of IL_s with event magnitude to regions outside south-eastern Australia.

4.1.3.1.2. Burst Initial Loss (IL_b)

The pre-burst rainfall (the rainfall from the beginning of the complete storm to the start of the design rainfall burst), rather than the pre-storm rainfall, is the key determinant of Burst Initial Loss, IL_b, as it results in different degrees of catchment saturation. IL_b is thus systematically smaller than IL_s; the difference decreasing with increasing burst duration, reflecting a tendency for long duration bursts to represent complete storms. As an increasing storm magnitude is generally also associated with larger pre-burst rainfall, IL_b tends to further decrease with increasing event magnitude.

4.1.3.1.3. Continuing Loss (CL)

For events of increasing duration and intensity of rainfall, an increasing proportion of the catchment is expected to become saturated, resulting in a reduced catchment average value of Continuing Loss, CL. However, the available evidence from Hill et al. (1996a), based on catchments located in Victoria and the ACT, indicates no systematic differences in CL for observed events between 1 in 2 to 1 in 100 AEP. This can be interpreted to mean that, except in catchments with highly pervious soils, catchment saturation is approached already during moderate to large storm events. Nevertheless, it should be conservatively assumed that only the CL values associated with the largest observed events are representative of design loss rates for Very Rare to Extreme floods.

4.1.3.2. Losses for Urban Catchments

There is little empirical evidence available on loss values in urban areas that is relevant to the estimation of Very Rare to Extreme floods. As discussed in Book 5, Chapter 3 it is appropriate to conceptualise urban catchments as consisting of Effective Impervious Areas (EIA), 'other areas' and Pervious Areas. Recognising the dearth of information available, it is considered prudent to recommend loss rates at the lower range of that described in Book 5, Chapter 3 for Very Rare to Extreme floods.

4.1.4. Variation of Design Losses with Season

There is clear evidence that initial loss values vary seasonally in some regions of Australia (e.g. Laurenson and Pilgrim (1963), and Hill et al. (1998)), and this can be readily explained by differences in the likelihood of pre-storm rainfall for different seasons. However, the interpretation of the observed seasonal differences in continuing losses is more difficult. Little published information is available on seasonal loss values suitable for design, and efforts should be made to seek out relevant regional information where available. Where there is clear evidence of seasonal differences in losses, and where the seasonal variation of other design factors is being allowed for, the loss values from the appropriate season should be applied.

4.1.5. Consideration of Joint Probabilities

Where losses are considered to have an important influence on the design floods of interest, it is recommended that they be simulated using joint probability approaches to minimise bias in the transformation of rainfalls to floods. In the extreme range of floods it would be expected that losses are generally less important than temporal patterns, and hence where volume is not important it may well be sufficient to model losses in a deterministic fashion.

The recommendations presented below may be applied in either deterministic or stochastic simulation frameworks. If the former, then the recommendations as outlined in Book 8, Chapter 4, Section 3 are adopted as single values for all required simulations. If a stochastic approach is adopted, then the recommendations provided in Book 8, Chapter 4, Section 3 represent the central tendency (either the median or the mean, as appropriate to the method adopted); this “location” parameter is then used to scale the adopted distribution for stochastic simulation, as described in Book 5, Chapter 3, Section 6.

4.2. Methods for Derivation of Design Loss Values

The estimation of design loss values for use with Very Rare to Extreme design events has to be based on observed rainfall and flood hydrograph data for the site or region of interest. Where available, flood frequency data can be used to validate the derived design loss values. The different approaches for estimating design loss values are described in Book 5, Chapter 3, Section 3. In applying these methods to the estimation of Very Rare to Extreme design floods, most weight should be given to largest observed events and corresponding flood frequency estimates.

4.3. Guidelines for Selection of Design Loss Values

4.3.1. General

The recommendations in this section relate specifically to the IL-CL model. However, there is no intention to restrict the application of other loss models, provided appropriate loss parameters are selected in line with the general considerations outlined in Book 5, Chapter 3.

The recommendations provided in Book 8, Chapter 4, Section 3 to Book 8, Chapter 4, Section 3 relate to rural catchments, and guidance for urban catchments is discussed in Book 8, Chapter 4, Section 3.

The selection of loss parameters for Very Rare to Extreme design events should allow for the following factors:

- type of design rainfall data, i.e. design storm or design burst (Book 8, Chapter 4, Section 1);
- design event magnitude and duration (Book 8, Chapter 4, Section 1);
- season(s) of interest in design (Book 8, Chapter 4, Section 1);
- catchment characteristics for design situation (Book 8, Chapter 4, Section 1)

Specific recommendations are given for the selection of design initial loss in (Book 8, Chapter 4, Section 3 for design bursts and Book 8, Chapter 4, Section 3 for design storms) and design continuing loss (Book 8, Chapter 4, Section 3). Except for storm initial loss, different design situations are distinguished depending on the event magnitude. Where appropriate, different recommendations are given for specific geographic regions, consistent with the availability of design information for different parts of Australia.

Beyond the credible limit of flood extrapolation, it is not possible to check the appropriateness of the adopted loss values against independent flood estimates, and thus it is necessary to adopt a more prescriptive, conservative approach. The recommendations in Book 8, Chapter 4, Section 3 and Book 8, Chapter 4, Section 3 reflect this philosophy.

4.3.2. Rural Initial Loss Values for Use with Design Bursts

The selection of initial loss for use with design bursts of rainfall is problematic as the depth of rainfall antecedent to the burst varies with both storm duration and event magnitude. Traditionally, it has been assumed that the net bias resulting when storm losses obtained from calibration are applied with design bursts is negligible. However, the available evidence for flood events more frequent than the 1 in 100 AEP event suggests that the losses obtained from calibration to large historic floods are too low (e.g. (Walsh et al., 1991), and (Hill et al., 1996b)).

The expected reduction of IL_b with reducing burst duration and increasing event magnitude means that the following recommendations have to differentiate between event magnitudes.

4.3.2.1. Rare to Very Rare Events

IL_b values suitable for derivation of floods more frequent than 1 in 100 AEP should be based on recommendations contained in Book 5, Chapter 3, or other relevant design data for the region, as deemed appropriate.

Where possible, reconciliation with independently derived design flood estimates should also be attempted, as described in Book 5, Chapter 3, Section 3.

4.3.2.2. Extreme Events

IL_b values should be varied gradually between the values adopted for Very Rare and PMP events. In the absence of any scientific justification, it is suggested that losses between the two limits are determined from a simple interpolation procedure. For example, if the initial loss value for the 1 in 100 AEP event is 10 mm and that for the most Extreme design event (with an AEP of 1 in 10^6) is 0 mm, then Extreme loss values can be interpolated from a line drawn on log-Normal probability paper between 10 mm at 1 in 100 AEP and, say, 0.1 mm at 1 in 10^6 AEP (the initial loss of 0.1 mm is an approximation of 0 mm in the logarithmic domain).

Alternatively, it may be assumed that the losses vary linearly on a log-log plot of losses versus AEP; this assumption is more consistent with the interpolation procedure used for design rainfalls (Book 8, Chapter 3, Section 5), and is also more amenable to calculation. For example, if initial loss values L_1 and L_2 were assigned, respectively, to events of 1 in Y_1 and 1 in Y_2 AEP, then the loss value to be used in conjunction with a design burst of intermediate 1 in Y AEP could be interpolated using the following equation:

$$\log(L_Y) = \log(L_1) + \{\log(Y) - \log(Y_1)\}\frac{\log(L_2) - \log(L_1)}{\log(Y_2) - \log(Y_1)} \quad (8.4.1)$$

A zero loss value is again to be approximated by a small value, say 0.1 mm. The practical difference between the use of Equation (8.4.1) and the assumption of log-Normal variation is negligible given the uncertainty of loss rate variation.

4.3.2.3. Probable Maximum Precipitation Flood

Very low values of IL_b are recommended as it is assumed that the pre-burst rainfalls associated with the PMP design burst will either partly (longer duration bursts) or fully (short duration bursts) satisfy soil moisture deficits. In conformity with the adopted policy of aiming for reasonable conservatism in the absence of more relevant information, conservatively low estimates are generally recommended. For PMP design burst durations approaching the duration of the observed storms, the IL_b value for use with the PMP should be equal to or possibly a little less than the minimum IL_b value in large floods observed on the catchment. For significantly shorter burst durations, a zero value for IL_b is recommended.

In this context of selecting a design loss, some care and interpretation may be required in assessing the minimum value in observed floods. Sometimes an apparently anomalous value occurs that is appreciably lower than all other derived values. As this could have resulted from the effects of data errors, it may be desirable to neglect the anomalously low value in selecting the minimum value.

Recommendations for specific regions are provided below:

- *Humid and sub-humid regions of south-eastern Australia*: For long duration rainfalls in this region, temporal patterns of pre-burst rainfall are available (Jordan et al, 2005; Minty et al, 1999), and thus the procedures provided in Book 8, Chapter 4, Section 3 for design storms should be used. If PMP design bursts are used directly, and for shorter duration design bursts, an IL_b value of zero should be selected.
- *Tasmania*: For western Tasmania, catchments are likely to be saturated, and 100% runoff (i.e. IL_b=0) is appropriate for design. Loss values for south-eastern Australia should apply to eastern Tasmania.
- *Arid and Semi-Arid regions*: The few data available indicate that no initial loss should be deducted from the PMP.
- *Western Australia*: For the forested south-west region, the following values of IL_b are recommended:
  - Winter: IL_b = 0
  - Summer: IL_b = 200 mm from the high absorbing gravels and sands of the lateritic uplands and zero from the remainder of the catchment.

  For the remainder of the State, IL_b = 0.

4.3.3. Rural Initial Loss Values for Use with Design Storms (IL_s)

Pre-burst temporal patterns are available for the whole of Australia, and their use to construct complete design storm events provides a more logical basis for the derivation of hyetographs of rainfall excess.

Unless specific evidence of significant variation of initial loss with event magnitude or duration has been found in the region of interest, the storm initial loss values derived by the procedures in Book 5, Chapter 3, as representative (median) values from large events, are applicable to flood estimation over the whole range, from Infrequent floods to the PMP Flood, and for all durations.

4.3.4. Rural Continuing Loss Values (CL) for use with Design Bursts and Design Storms

4.3.4.1. Rare to Very Rare Events

The CL values derived by the procedures in Book 8, Chapter 4, Section 2 are based on the analysis of moderate to large events and are thus directly applicable to events in that range. For CL values determined by reconciliation with independently estimated flood estimates (Book 5, Chapter 3, Section 4), the range of application depends on the credible limit of extrapolation of floods for the particular design situation.

4.3.4.2. Extreme Events

CL values in the range from Very Rare events to the PMP Flood should vary gradually in the same manner as for initial loss. Equation (8.4.1) can be applied to estimate the loss rate for the 1 in Y AEP within this range.

4.3.4.3. Probable Maximum Precipitation Flood

General guidelines regarding the CL values to be used with PMP design bursts are given for various regions of Australia, based on published data or local experience. With the general nature of the recommendations, it is not appropriate to delineate precise boundaries of the regions. Where possible, greatest reliance should be placed on values derived from several large observed floods on the catchment of interest, as discussed previously (Book 5, Chapter 3, Section 3). Given the tendency of events of greater rainfall intensity to saturate greater proportions of the catchment, the largest events are expected to be associated with the smallest loss rates. Similarly, long duration events can be expected to be associated with lower CL values than short duration events. However, any anomalously low values, thought to result from the effects of data errors in the volume balance computations, should be neglected.

For short duration events, losses are very small compared with depths of precipitation, and variations in the value adopted will have little effect on the magnitude of the resulting flood. For longer storms, the rate of loss and the form of loss adopted can have a considerable effect on estimated floods, particularly on flood volumes, and greater care is needed in their selection. An example of the variation of maximum pond level with loss values is given by Brown (1982).

Recommendations for specific regions are provided below:

- *Humid and sub-humid regions of south-eastern Australia:* For catchments considered similar to the humid and sub-humid regions of south-eastern Australia, CL values would be unlikely to be greater than 1 or 2 mm/h for use with PMP design bursts. A design value of 1 mm/h seems reasonable where no other data are available. A value of zero is generally too conservative.

- *Humid and sub-humid regions of north-eastern and northern Australia:* higher CL values than for south-eastern Australia may be appropriate, but values greater than 3 mm/h would be unusual.

- *Tasmania:* For western Tasmania, catchments are likely to be saturated, and zero continuing loss is considered appropriate for design. Loss rates for south-eastern Australia should apply to eastern Tasmania.

- *Arid and semi-arid regions:* The few data available indicate that a slightly higher value of loss rate may be appropriate than for more humid regions in the south-east of the continent. It is unlikely that this value would be greater than 3 mm/h.

- *Western Australia:* For the forested south west region, losses should be estimated using a variable proportional loss model based on catchment storage, as described in Book 5, Chapter 3 and Pearce (2011). For the remainder of the State, it is considered unlikely that CL would be greater than 3 mm/h.

4.3.5. Loss Recommendations for Urban Catchments

Following the advice provided in Book 5, Chapter 3, Section 5, it is considered reasonable to apply the loss values recommended for rural catchments to pervious areas. For effective impervious areas it is recommended that the lower bound identified by Phillips et al. (2014) be used, which equates to a storm and burst initial losses of 1 mm and 0 mm, respectively, and a continuing loss of 0 mm. For the 'other area' which represents the remaining impervious area and pervious area connected with the impervious area, it is recommended that loss values be selected from the lower range of values adopted for rural catchments. This guidance is summarised in Table 8.4.1.

Table 8.4.1. Recommended Loss Rates for Urban Catchments

Area Class	Storm Initial Loss (mm)	Burst Initial Loss (mm)	Continuing Loss (mm/hr)
Effective Impervious	1	0	0
Other	At the lower range of values adopted for rural catchments		
Pervious	As for rural catchments		

4.4. References

Brown, J.A.H. (1982), A review of flood estimation procedures. Proc. of the Workshop on Spillway Design, Dept of Natl Devel. and Energy, AWRC Conf. Series No.6, pp: 84-108.

Hill, P.I., Mein, R.G. and Siriwardena, L. (1998), How much rainfall becomes runoff? Loss modelling for flood estimation. CRC for Catchment Hydrology Industry Report 98/5, Dept. of Civil Eng., Monash University.

Hill, P.I., Maheepala, U.K., Mein, R.G., Weinmann, P.E. (1996a), Empirical analysis of data to derive losses for design flood estimation in south-eastern Australia. CRC for Catchment Hydrology Report 96/5, Dept. of Civil Eng., Monash University.

Hill, P.I., Mein, R.G., Weinmann, P.E. (1996b), Testing of Improved Inputs for Design Flood Estimation in South-Eastern Australia. CRC for Catchment Hydrology Report 96/6, Dept. of Civil Eng., Monash University.

Jordan P, Nathan R., Mittiga L and Taylor B (2005): “Growth Curves and Temporal Patterns for Application to Short Duration Extreme Events”. Aust J Water Resour 9(1), 69-80.

Kemp, D. and Daniell, T.M. (1997), The Olary Floods - February 1997, Aqua Australis, 1(3), 3-7.

Laurenson, E.M. and Pilgrim, D.H. (1963), Loss rates for Australian catchments and their significance. I.E. Aust., The Journal, Jan.-Feb. 1963, pp: 9-24.

Minty, L.J. and Meighen, J. (1999), Development of temporal distributions of rainfall antecedent to large and extreme design bursts over southeast Australia, Hydrology Report Series HRS Report No.6, Hydrometeorological Advisory Service, Bureau of Meteorology.

Minty, L.J. and Meighen, J. 1999. “Development of temporal distributions of rainfall antecedent to large and extreme design bursts over southeast Australia

Pearce, L. J. (2011), Regional runoff coefficients for summer and winter design flood events in south-west Western Australia. In Proceedings of the 33rd Hydrology and Water Resources Symposium, Barton, A.C.T., Engineers Australia, pp: 331-338.

Phillips, B., Goyen, A., Thomson, R., Pathiraja, S. and Pomeroy, L. (2014), Project 6 - Loss models for catchment simulation - urban catchments, Stage 2 Report. Report prepared for Australian Rainfall and Runoff Revision, Engineers Australia.

Walsh, M.A., Pilgrim, D.H. and Cordery, I. (1991), Initial losses for design flood estimation in New South Wales. Proc. International Hydrology and Water Resources Symposium, Perth, I.E. Aust., Nat. Conf. Publ. (91/19), 283-288.

Weinmann, P.E., Rahman, A., Hoang, T., Laurenson, E.M. and Nathan, R.J. (1998), A new modelling framework for design flood estimation. 'HydraStorm'98', Adelaide, Australia, Institution of Engineers, Australia, pp: 393-398.

Chapter 5. Selection, Configuration, and Calibration of Hydrograph Models

Rory Nathan, Erwin Weinmann

Chapter Status	Final
Date last updated	14/5/2019

5.1. General

In Australia, both unit hydrograph models and runoff-routing models have traditionally been applied for event-based flood hydrograph estimation but over the last decade there has been a shift to almost exclusive use of runoff-routing models. In recent times attention has also been given to the use of "rain-on-grid" approaches with two dimensional (2D) hydraulic models. Discussion on the general catchment modelling concepts and the application of hydrograph and catchment models to the estimation of design floods is provided in Book 4, Book 5 and Book 7.

The following discussion is focussed on the application of event-based runoff-routing models to estimate Very Rare to Extreme floods, i.e. those design situations in which models are used to estimate floods well beyond the range for which they can be calibrated or their performance tested against observed floods. The principal purpose of these flood estimates is to support risk-based or standards-based design decisions. In some situations, such as floodplain management, extreme floods are estimated to provide a notional upper bound of the flood extent or as a performance check, and it is likely that the more rigorous considerations provided in this book are not justified.

Guidance on the use of rain-on-grid approaches for estimation of Very Rare to Extreme floods is not provided here for two reasons: firstly, as discussed in Book 5, Book 6 and Book 7, the techniques have not been well researched or validated at this point in time and their use to simulate overland flow routing raises a number of difficulties which are likely to be exacerbated under extreme event conditions; secondly, such models are generally focussed on applications in floodplain management where the design risks of interest are at the lower range of events relevant to the guidance presented in this Book. However, the use of hydraulic models to simulate extreme floods does have some theoretical merit, and it is hoped that with further research guidelines can be developed that better integrate the benefits of these two approaches.

For event-based models, the quality of the modelled flood hydrographs depends on three components of the modelling process: (i) the basic model capabilities and constraints, (ii) the quality of the catchment representation in the model, and (iii) the appropriateness of the selected parameter values in the flood range of interest. General recommendations for these three components in the context of estimating Very Rare to Extreme floods are provided separately in Book 8, Chapter 5, Section 2 to Book 8, Chapter 5, Section 5, but it should be recognised that the components are closely linked. The theoretical advantages of a more flexible model that allows a more accurate representation of the important catchment features can only be realised if suitable data or design information exists to identify appropriate model parameter values.

In the application of runoff-routing models, a distinction needs to be made between essentially rural catchments and substantially urbanised catchments. Book 8, Chapter 5, Section 4 deals with the determination of model parameters for essentially rural catchments, while the special aspects of model parameterisation for urban catchments are dealt with in Book 8, Chapter 5, Section 5.

5.2. Model Features and Capabilities Required to Estimate Very Rare to Extreme Events

5.2.1. Considerations in Model Selection

The functionalities of a hydrograph modelling package for estimating Very Rare to Extreme floods can be divided into basic and enhanced modelling capabilities. The basic capabilities indicated in Book 8, Chapter 5, Section 2 are regarded as essential for a modelling package that will allow satisfactory reproduction of runoff response characteristics over a range of catchments with different features for use in final design applications. Small catchments and catchments with reasonably uniform characteristics are less demanding in their basic model requirements. The enhanced model capabilities discussed in Book 8, Chapter 5, Section 2 represent desirable model extensions required for applications in situations where the complexity of the catchment or the importance of the results warrants more detailed modelling. The enhanced models form a sounder basis for extrapolation to extreme events. The importance of these modelling capabilities is somewhat dependent upon catchment size, and judgement is required to determine the extent to which the following issues need to be considered.

5.2.2. Basic Model Requirements

5.2.2.1. Representation of Catchment Routing Elements

Significant variation of routing characteristics over the catchment, particularly in larger catchments, will require at least a semi-distributed representation of routing elements in the catchment (refer to Book 5). The model should have the ability to reflect changes in the routing response of specific elements resulting from modification of catchment, channel, or storage components.

While there is evidence of non-linear routing response over the range of observed floods in most natural catchments, it is unclear to what extent this effect persists to the range of Very Rare to Extreme floods. In this range of flood magnitudes the routing response depends on how the efficiency of flow and the available flood storage change with increasing flood magnitude (or reducing flood frequency). The recommended procedures in Book 8, Chapter 5, Section 4 are based on this assumption. The degree of non-linearity of catchment behaviour and its effects are discussed by Pilgrim (1986), together with the background to the recommended procedures.

5.2.2.2. Spatial Variation of Rainfall Excess

Where it is necessary to apply design rainfalls non-uniformly across the catchment (refer to Book 8, Chapter 3, Section 9) the model should be able to represent spatial variations in rainfall inputs. A semi-distributed rather than a lumped model is thus required in most cases.

5.2.2.3. Distributed Output

Flood estimates are often required at different points of interest within a catchment. The model should thus adequately represent the progressive routing effects through the catchment, i.e. it should be internally consistent to allow matching of observed hydrographs at the catchment outlet and at required internal points. It should be noted that some of the simple hydrograph models in current use only provide an adequate representation of internal flows for locations near the catchment outlet. For other internal locations it may be necessary to increase the degree of catchment sub-division (and re-calibrate the model) to conform with the recommendations for the minimum number of upstream sub-areas (Book 5).

5.2.3. Enhanced Model Capabilities

5.2.3.1. Separation of Routing Elements with Different Non-Linearities

Different catchment elements (e.g. overland flow, well-defined stream/channel flow, floodplain and concentrated storage elements) may be characterised by different non-linearities in their routing response. A model structure that allows the separate representation of routing elements with different non-linearity characteristics (e.g. (Kemp, 1998)) offers distinct advantages, as extrapolation of the routing characteristics for individual elements to model more extreme events can be achieved in a more controlled fashion than for the lumped response of a combination of different elements.

5.2.3.2. Distributed Modelling of Baseflow

Baseflow would generally only make a minor contribution to Very Rare to Extreme floods (refer to Book 8, Chapter 6, Section 3); nevertheless the capability to define baseflow contributions at subcatchment level, for subsequent combined routing with surface runoff to the point(s) of interest, is desirable. Further comments regarding this issue can be found in Dyer et al. (1993).

5.3. Model Configuration

5.3.1. General Considerations

Most hydrograph models are highly conceptual in nature; in setting up a model representation of the catchment, the modeller should therefore try to define conceptual model elements that match the routing response of the main components of the real catchment, without necessarily attempting to exactly match all physical catchment features (e.g. individual drainage lines, drainage divides) in detail. How this can best be achieved will depend on the specific features of the selected model. However, the most important factor determining the quality of the modelling results is the modeller’s understanding of the routing functions incorporated in the modelling package and the practitioner's appreciation of the catchment response under major to extreme flood conditions. More specific guidance on selected model configuration issues is provided in Book 8, Chapter 5, Section 3

5.3.2. Specific Issues

5.3.2.1. Degree of Catchment Homogeneity

The model should be subdivided into as many separate subcatchments as required to represent the broad variation in flood response resulting from differences in topographic, drainage system, land cover and land-use attributes (refer to recommendations on the minimum number of subcatchments provided in Book 7). For the estimation of Very Rare to

Extreme floods, the variation in parameter sets for the different subcatchments should, as far as possible, be directly related to differences in measurable catchment characteristics.

5.3.2.2. Representation of Significant Catchment Features

Major catchment features may have a significant influence on catchment flood response, and may exhibit significantly different routing characteristics compared to the rest of the catchment, particularly when extrapolated to extreme events. All the significant natural storage areas (e.g. swamps, extensive flood plains, off-channel storage areas) and distributary or effluent channel systems should be identified and adequately represented. Consideration should also be given to the modelling of anthropogenic features, such as the specification of diversion channel capacities, or road/rail crossings that may act as retarding basins during extreme events.

5.3.2.3. Representation of Catchment Areas Close to a Reservoir

In the vicinity of a reservoir, the routing response varies from near zero delay for rainfall on the inundated areas, to significant delays for rainfall excess from the less directly connected areas draining to the storage. The modelled hydrographs and the calibrated model parameters can be quite sensitive to the representation of these areas, particularly when the inundated area constitutes a large part of the total catchment. Considerable care should be exercised in ensuring that the routing characteristics of the inundated parts of the catchment and the areas close to it have been realistically represented.

5.3.2.4. Modelling of Changed Catchment Conditions

The effects of likely changes to the catchment during the design life of the structure need to be considered. Urbanisation and removal of vegetation by clearing or fire may reduce the response time of the catchment and increase the peak flow, while soil conservation measures over a large portion of a catchment may have the opposite effect. The impacts of these changed catchment conditions on the formation and propagation of extreme floods is currently not well researched. Generally, a rather arbitrary allowance must be made for these effects. Construction of a reservoir may inundate appreciable lengths of streams in the catchment and can lead to large decreases in travel time and increases in flood peaks, despite the attenuation resulting from the routing effect of the reservoir. This effect is discussed and examples are given by Weeks and Stewart (1982), Brown (1982) and Watson (1982). The last two references give examples where the inflow flood peak is increased by 85% by the construction of a dam. It is therefore important to consider this effect when using a model to derive design floods for a dam if it has been calibrated to pre-dam conditions.

In general, allowance for different catchment conditions can be made more easily by runoff--routing than by unit hydrograph models. In runoff-routing models the different routing characteristics of existing or future catchment conditions can be incorporated by the judicious selection of parameters and the types of routing elements.

5.4. Determination of Model Parameter Values for Rural Catchments

5.4.1. General

The provisions of the 1987 edition of ARR addressed the question of hydrograph model parameter selection for Large to Extreme events, based on the research results and experience available at that time (Pilgrim, 1986). There has been limited research since then to resolve the issue of the appropriate degree of model non-linearity for the estimation of Extreme flood events. Given the lack of strong research evidence, specific design recommendations for this range of events must be based on a consensus approach. The following considerations and recommendations are based on the broad range of views expressed by different groups of practitioners and current practice in Australia.

The key factor to be considered when selecting parameters for modelling Very Rare to Extreme events is that the parameters found from calibration to a relatively narrow range of observed flood events cannot be assumed to apply to the range of more extreme events. As the magnitude of the event to be modelled increases significantly beyond the range of the largest observed events, the parameter selection process has to be guided more strongly by physical/hydraulic consideration of how the response of the catchment is expected to change when exposed to more extreme rainfall events. This will depend on the physical characteristics of hillslopes and on-stream and floodplain characteristics such as breakout points, threshold levels and the availability of significant off-stream storage areas in the lower part of the catchment.

It is necessary to provide recommendations for design situations in which suitable streamflow information may or may not be available. Accordingly, guidance on both these situations is provided in the following two sections. Subsequent sections provide specific guidance on other aspects of parameter determination, and are aimed at minimising the uncertainties in the selection of design parameter values. Guidance on the selection of parameter values for estimation of Very Rare to Extreme design floods in ungauged catchments is provided in Book 8, Chapter 5, Section 4.

5.4.2. Parameter Determination for Gauged Catchments

5.4.2.1. Calibration to the Largest Observed Flood Events

The simplified conceptual representation of catchment response in the commonly used flood hydrograph models means that these models rely heavily on appropriately calibrated parameter values. While calibration of a model provides valuable information on the flood response of a catchment within the range of observations, caution is needed when applying the model to estimate design floods of much larger magnitude. Extrapolation of model parameter values beyond the range of calibration events will introduce considerable uncertainty into flood estimates.

The model should thus be calibrated to events over a range of flood magnitudes up to the largest observed event, and the results analysed for the presence of any trends. If appropriate data are not available at the site of interest, consideration should be given to transferring parameters from a calibrated model of a nearby catchment with similar characteristics, with appropriate adjustments for differences in catchment size and characteristics.

The examination of log-log plots of storage versus discharge for particular routing elements may be helpful in the assessment of calibration results and in identifying parameter variation with flood magnitude. In assessing the calibration results, it should be borne in mind that the calibrated parameter values for individual events reflect not only the catchment response to actual rainfall, but also any errors in the estimated catchment rainfall, in the rating curve used to establish the observed flood hydrograph, and in the adopted baseflow separation procedure. The first two types of errors tend to increase with event magnitude.

Before applying any calibrated parameter values to modelling of more extreme events, they should be checked for consistency with the recommendations as discussed Book 8, Chapter 5, Section 4.

5.4.2.2. Adjusted Parameter Values from Reconciliation with Flood Frequency Estimates

In catchments where a long series of at-site or regional flood data is available, the comparison of rainfall-based estimates with flood frequency based estimates can provide important information on the variation of flood response characteristics with flood magnitude. With this approach, in accordance with the general guidance provided in Book 7, the initial model parameter values found from calibration are adjusted to achieve reasonable agreement between the rainfall based estimate for a selected AEP and the flood frequency analysis based estimate of corresponding AEP. Adjustment of hydrograph model parameters is only necessary if satisfactory agreement of flood estimates from the two methods cannot be achieved by varying loss parameters within reasonable limits. It may also be required, if the comparison indicates that the rainfall based method cannot satisfactorily reproduce the slope of the flood frequency curve. In that case, adjustment of the non-linearity parameter of the selected model would be appropriate.

The approach is particularly suited to catchments with a good flood peaks record but only limited hydrograph information. It can also be applied to reconcile rainfall based flood estimates with flood estimates obtained from paleohydrological procedures (Book 8, Chapter 6, Section 2). Before applying any adjusted parameter values to modelling of more extreme events, they should be checked for consistency with the recommendations in Book 8, Chapter 5, Section 4.

5.4.2.3. Evidence From Very Rare Floods in Similar Catchments

The lack of data on very large floods in the catchment of interest could be partly compensated by analysis of flood observations for very large events in catchments with similar characteristics. The interpretation of calibration results from such catchments should be guided by the considerations in Book 8, Chapter 5, Section 4.

There are relatively few published hydrograph modelling studies of very large Australian flood events (Wong and Laurenson, 1983; Pilgrim, 1986; Sriwongsitanon et al., 1998). The available evidence points towards reducing non-linearity of catchment response for very large events (Pilgrim, 1986; Zhang and Cordery, 1999), indicating relatively more catchment storage for increasing discharge and thus greater attenuation of flood peaks. However, this tendency may not continue to the range of Extreme events, if flow efficiency also increases substantially for these events. The conclusion from these studies might also be affected by the high degree of uncertainty in estimated flow rates for Very Rare to Extreme events: the apparent tendency towards linearity could alternatively be explained by underestimation of the true peak flow rate.

The available studies cover only a limited range of catchment conditions, and care should thus be taken in applying the study results to other catchments. Detailed analysis of other large flood events and publication of results is highly desirable.

5.4.3. Design Parameter Values for Ungauged Catchments

General guidance on the selection of parameter values for the estimation of design floods in ungauged catchments may be found in Book 7. In transferring parameter values from gauged to ungauged catchments for modelling of Very Rare to Extreme events, particular emphasis should be placed on assessing the similarity of catchment characteristics relevant to this flood range.

Before applying any regional parameter values to modelling of more extreme events, they should be checked for consistency with the recommendations provided in the following section.

5.4.4. Physically-based Extrapolation of Model Parameter Values for Extreme Events

5.4.4.1. Background

The most commonly applied runoff-routing models in Australia use conceptual storage elements to represent the hydrograph formation process in response to distributed inputs of rainfall excess. These conceptual storages represent the routing response of all catchment components, from hillslopes to river channels and floodplains. Each storage element is represented by a power-function relationship between Storage *S* and flow rate Q, with coefficient k and exponent m (Book 5). The exponent m expresses the degree of non-linearity in the catchment response; it typically varies between 0.6 and 1.0, where a value of 1.0 corresponds to a linear response. Within a limited range of S and Q, different combinations of k and m can produce similar S-Q relationships, and the modelled flood outputs are not overly sensitive to the selection of a particular combination.

While this simplified representation of the relationship between storage and discharge has been shown to produce satisfactory results over a limited range of flood magnitudes, it is well known that it fails to adequately represent the variations of flow conditions over a much wider range of flood magnitudes. As an example, it has been shown that the S-Q relationship for the transitional stages between in-bank and fully developed floodplain flow is much more complex (Wong and Laurenson, 1983; Bates and Pilgrim, 1983; Pilgrim, 1986). The failure of the power-function relationship between S and Q to account for these complexities expresses itself in different calibrated pairs of k and m values for different flow ranges.

The available flood data provide good evidence for the nature of non-linearity in stream-channel and floodplain flow for Rare floods and possibly even Very Rare events. However, relatively little evidence is available to assess the nature of the S-Q relationship for flows on hillslopes beyond the range of relatively frequent events, or for Extreme floods in stream-channel and floodplain systems.

In this situation of limited reliable evidence from very large flood events, the extrapolation of model parameter values for application to extreme events must be guided by the consideration of specific catchment topography and hydraulic factors. These factors are further discussed in Book 8, Chapter 5, Section 4.

Hydraulic models may be used to better define the representation of flow behaviour in complex environments, and their use for this purposes is discussed in Book 8, Chapter 5, Section 5 .

5.4.4.2. Consideration of Catchment Topography and Hydraulic Factors

It is evident that the relationship between catchment storage and flood flow rate for Extreme events is determined by catchment topographic and hydraulic factors. An analysis of these factors for the different parts of the catchment may thus provide valuable information on the general form of the S-Q relationship. As an example, the hydraulic analysis of channel and floodplain flow characteristics may shed some light on the nature of non-linearity in the streamflow routing elements in the extreme flood range. Similar analyses may be undertaken for hillslope segments but the results will necessarily be associated with a greater degree of uncertainty.

The interpretation of calibration results can be guided by consideration of special cases of the relationships between storage and discharge. For the case of steady, uniform flow in a prismatic channel, the analysis using Manning's equation produces a power law relationship between S and Q with m-values ranging from 0.6, for a wide rectangular channel, to 0.75 for a triangular channel (Mein et al., 1974). This assumes that the cross-sectional areas contributing to flow and to storage are identical, and that a *constant Manning's n* applies to all magnitudes of flow. It implies that the average flow velocity is increasing with increasing event magnitude. Another special case applies when a *constant average flow velocity* can be assumed over the range of flood magnitudes, and flow and storage areas are again identical. This case corresponds to a power law relationship between S and Q with an m-value of 1.0, i.e. a linear relationship.

The following factors are considered to be responsible for variations of actual S-Q relationships between the above special cases:

i. *Factors increasing the relative efficiency of flow with increasing event magnitude (and thus decreasing the effective value of m):* With increasing event magnitude, there is a tendency in hill-flow segments for concentration of flow in relatively efficient flow paths. The increasing depth of flow may reduce the effective roughness of vegetation and other flow resistance elements. Similarly, the removal and stripping of vegetation during rare flood flows will tend to decrease the effective value of m. Some short-circuiting of the more sinuous flow path taken during more frequent flood events is also likely to occur. When compared to transitional stream channel and floodplain flow in Very Rare to Extreme flood events, fully developed floodplain flow during Very Rare to Extreme events can be expected to be more efficient.

ii. *Factors reducing the relative efficiency of flow with increasing event magnitude (and thus increasing the effective value of m):* Extreme flood events can be expected to produce significant changes to the catchment, stream and floodplain morphology. The erosive surface changes and sediment transport processes require significant inputs of flow energy, resulting in an increase of effective flow resistance. In stream/floodplain systems, an increase in flood magnitude is generally associated with more complex flow patterns and increased turbulence, also resulting in an increase of effective flow resistance. The question to be resolved for extrapolation to Extreme events is to what extent the increasing resistance will be offset by more efficient flow paths.

iii. *Factors increasing or reducing the effects of catchment storage (and thus increasing or reducing, respectively, the value of m compared to calibration events):* In catchments with extensive floodplains, increasingly larger flood events will mobilise additional storage areas that may not contribute significantly to flood flow conveyance. The question to be addressed in extrapolation of calibration results to Extreme events is, to what extent these areas will still contribute mainly to storage, and to what extent they will become effective conveyance areas. In heavily vegetated catchments, flood debris may create temporary pondage areas and thus additional catchment storage.

In extrapolating model parameter calibration results to Very Rare and Extreme events, the above factors should be carefully balanced.

It is recognised that in many cases the constraints on the study budget will limit the extent to which the above factors can be evaluated. It will thus be necessary to place a greater reliance on experience gained from earlier studies and to introduce a margin of conservatism into the selection of parameter values. Book 8, Chapter 5, Section 4 gives recommendations for parameter selection based on these considerations.

5.4.5. Specific Recommendations for Modelling Extreme Events

5.4.5.1. General

The model parameter values for design flood estimates in the range of Extreme events should be selected on the basis of the available evidence for the catchment of interest, as described in Book 8, Chapter 5, Section 4. Where the available information for the catchment is limited essentially to the range of Rare events, it should be supplemented by information from other catchments, and/or by consideration of catchment topography and hydraulic factors (Book 8, Chapter 5, Section 4).

5.4.5.2. Gauged Catchments

As discussed by Pilgrim (1986), and on the balance of the factors in Book 8, Chapter 5, Section 4, a value of the exponent m in the power law storage-discharge relation ($S = k Q^m$) of less than 0.8 is generally conservative, in that Extreme floods tend to be overestimated. The recommended procedure described below for parameters associated with Extreme events and the PMP Flood applies directly to this form of the storage-routing relation as most published information relates to this form of model.

i. Where most of the valleys in the catchment are V-shaped with only small floodplains:

  - if the available model calibration results for the catchment of interest include Very Rare events and the calibrated m is in the range from 0.8 to 0.9 inclusive, adopt the calibrated value;
  - if the available model calibration results for the catchment of interest include Very Rare events and the calibrated m is outside the range from 0.8 to 0.9, select an appropriate value, guided by the additional information and considerations in Book 8, Chapter 5, Section 4;
  - if the range of available model calibration results for the catchment of interest is limited to Rare events, select an appropriate value of m in the range from 0.8 to 0.85, guided by the additional information and considerations in Book 8, Chapter 5, Section 4;
  - if neither Very Rare event calibration data nor the appropriate expertise for the considerations in Book 8, Chapter 5, Section 4 are available, adopt a conservatively low value of m = 0.8.

ii. Where many of the valleys in the catchment have appreciable floodplains:

  - if the available model calibration results for the catchment of interest include Rare events and the calibrated m is in the range from 0.85 to 0.9 inclusive, adopt the calibrated value;
  - if the available model calibration results for the catchment of interest include Very Rare events and the calibrated m is outside the range from 0.85 to 0.9, select an appropriate

value, guided by the additional information and considerations in Book 8, Chapter 5, Section 4;

- if the range of available model calibration results for the catchment of interest is limited to Rare events, select an appropriate value of m in the range from 0.85 to 0.9, guided by the additional information and considerations in Book 8, Chapter 5, Section 4;
- if neither Very Rare event calibration data nor the appropriate expertise for the considerations in Book 8, Chapter 5, Section 4 are available, adopt a conservatively low value of m = 0.85.

It should be noted that in the context of the above recommendations the term “Very Rare event” should be interpreted as floods that are *clearly beyond the transition between within-bank and floodplain flow*, i.e. fully developed floodplain flows of appreciable depth.

The recommendations for m relate to all floods beyond the credible limit of extrapolation. If the value of m selected for extreme floods differs from the value of m for floods of lesser magnitude, then the coefficient k in the power law storage-discharge relation (Book 5) should be adjusted to ensure that the magnitude of flow at the credible limit of extrapolation is unchanged when used with the new value of m. An initial estimate of the required value of k can be obtained by means of Equation (Book 5).

5.4.5.3. Ungauged Catchments

For ungauged catchments, the model parameter values must be estimated from calibration on nearby catchments or from regional relationships (refer to Book 7). The regional relationships for the catchment routing parameters (k) are generally given for an m of 0.8, and they will thus be directly applicable to catchments with small flood plains. For catchments with appreciable flood plains, it may be possible to increase m and adjust the value of k from a regional relationship by means of equation 3.19, (Pilgrim, 1998). An estimate would be necessary of the magnitude of the floods used in deriving the data on which the regional relationship was based (this estimate represents the credible limit of extrapolation associated with the derived regional relationship). If possible, the designer should check the magnitudes of the floods from which the regional relationship is derived as a guide to the likely conservatism of the estimate.

5.5. Model Parameterisation for Urban Catchments

Floods in urban catchments are the product of more complex interactions of hydrometeorological, hydrologic and hydraulic factors than in rural catchments. Severe floods can result from short duration intense rainfall over relatively small areas. The hydrologic response to heavy rainfalls is affected by changes to the natural runoff characteristics by reducing infiltration and increasing impervious surface areas. The drainage characteristics are changed by provision of more efficient flow paths in storm drain systems and channelised sections of streams. The hydraulic characteristics of drainage systems are also made more complex by the presence of bridges, culverts, floodways and detention basins.

The degree of complexity required when modelling an urban system is largely dictated by the design context. If the main focus is on sizing trunk drainage capacities then it may be sufficient to use non-linear storage routing models, where appropriate attention is given to characterising the shorter relative delay times associated with urbanisation of the natural drainage paths. Many hydraulic controls that influence flood response in urban catchments become drowned out under extreme conditions, and the complexities required to model the performance of these systems under Very Frequent to Rare conditions may not be required for more extreme events.

In complex systems it may not be possible to predict the changing nature of flow paths with event magnitude, or adequately characterise the influence of major floodplain features. In such cases it would be expected that flood behaviour is best assessed using hydraulic models, as described in Book 6 and Book 7. However, while the use of such models better resolves the influence of hydraulic controls, they introduce additional complexity associated with the need to interface with the hydrologic models used to derive input hydrographs. The need for such an interface might be avoided by inputting rainfall directly onto the hydraulic model grid, but this is only possible for catchments where the model covers the whole contributing area. While this potentially provides a more realistic representation of catchment controls, the approach is not well validated at this point in time and is subject to additional uncertainties, as discussed in Book 7.

The joint use of hydrologic and hydraulic routing models involves some explicit trade-offs in modelling complexity. On one hand hydrologic models are easily run within a joint probability framework and are thus able to explicitly solve the joint probabilities involved in the production of flood runoff to yield unbiased estimates of flood risk. On the other, they are ill-suited to representing the influence of complex hydraulic controls that might arise in an urban environment under Extreme conditions.

One means of balancing this trade-off is to use a hydraulic model to define the characteristics of a storage-discharge relationship. With this approach, a selection of flood hydrographs spanning the range of conditions of interest are input into the hydraulic model, and the outputs are then used to derive a relationship between storage, discharge and/or level, as relevant to the design problem of interest. This relationship can then be incorporated into a joint probability framework and then used to derive the flood characteristics without further need for hydraulic modelling. The advantage of the approach is that it combines the benefits of hydraulic modelling with stochastic simulation of flood processes but without impractical computational burden. The limitations of the approach is that it assumes that the derived storage-discharge relationship is adequate for all combinations of inputs, a situation that is only likely to be valid when considering one or two dominant mechanisms of flood loading. An example of this approach is provided by Sih et al. (2012), who used the hydrologic model to resolve the joint probabilities involved in reservoir drawdown and the concurrence of flood inflows from two major tributaries, and a hydraulic model to relate tributary inflows and tide levels to peak water levels at locations within a complex urbanised floodplain.

For more complex environments it will be necessary to rely directly on a hydraulic model to provide a realistic representation of flow behaviour. At present it is usually impractical to consider running complex hydraulic models in a stochastic simulation scheme, though it is expected that this approach will become increasingly feasible as parallel and distributed computing capabilities improve and become more easily implemented. The simplest way of trading off the potential for bias associated with rainfall-runoff modelling and the need for accurate representation of hydraulic behaviour is by careful selection of deterministic hydrologic inputs. For example, estimates of the concurrent peak design floods may be obtained through ensemble or Monte Carlo approaches, and these may be used to scale representative hydrographs for input to deterministic simulation in a hydraulic model. At its simplest, single runs of hydraulic models may be undertaken for each combination of storm duration and event severity, but this can be extended to ensemble hydraulic runs for a more representative range of flood inputs. The success of either approach rests on the selection of inputs that minimise bias in the transformation between rainfall exceedance probability and the flood level (or outflow) of interest, and sensitivity analyses will assist the identification of dominant influences and the selection of representative scenarios to be modelled.

5.6. References

Bates, B.C. and Pilgrim, D.H. (1983), Investigation of storage-discharge relations for river reaches and runoff-routing models. Civ. Eng. Trans., I.E. Aust., CE25: 153-161.

Brown, J.A.H. (1982), A review of flood estimation procedures. Proc. of the Workshop on Spillway Design, Dept of Natl Devel. and Energy, AWRC Conf. Series No.6, pp: 84-108.

Dyer, B., Nathan, R.J., McMahon, T.A. and O'Neill, I.C. (1993), A cautionary note on modelling baseflow in RORB. I.E. Aust. Civil Engin. Trans., CE35(4), 337-340.

Kemp, D. (1998), Flood hydrology modelling of Keswick Ck using the RRR model. Hydrastorm 98, 3rd International Symposium on Stormwater Management, Adelaide, September 1998, pp: 349-354.

Mein, R.G., Laurenson, E.M. and McMahon, T.A. (1974), Simple nonlinear model for flood estimation. ASCE J. Hydraulics Div., 100(HY11), 1507-1518.

Pilgrim, D.H. (1986), Estimation of large and extreme floods. Civ. Engg Trans, Inst. Engrs Aust., CE28: 62-73.

Pilgrim, D. (1998), Runoff routing methods, Book V Section 3, in Australian Rainfall and Runoff, A Guide to Flood Estimation, Volume 1, D.H.Pilgrim (ed), National Committee of Water Engineering, Institution of Engineers, Australia, reprint.

Sih, K., Hill, P.I., Nathan, R. and Mirfenderesk, H. (2012), Sampling in Time and Space - Inclusion of Rainfall Spatial Patterns and Tidal Influences in a Joint Probability Framework Proceedings of the 34th Hydrology & Water Resources Symposium. ISBN 978-1-922107-62-6. Engineers Australia.

Sriwongsitanon, N., Cordery, I. And Ball, J.E. (1998), Improvement of storage-discharge relationships for river reaches and runoff-routing models. Aus. J. Water Resour., 2(2), 77-87.

Watson, B. (1982), The estimation of spillway design floods - The Hydro Electric Commission, Tasmania. Proc. of the Workshop on Spillway Design, Dept of Natl Devel. and Energy, AWRC Conf. (6), 141-156.

Weeks, W.D. and Stewart, B.J. (1982), Aspects of design flood estimation using runoff routing. Proc. of the Workshop on Spillway Design, Dept of Natl Devel. and Energy, AWRC Conf. (6), 109-118.

Wong, T.H.F. and Laurenson, E.M. (1983), Wave speed-discharge relations in natural channels. Water Resour. Res., 19(3), 701-706.

Zhang, S. and Cordery, I. (1999), the catchment storage-discharge relationship: non-linear or linear? Aus J Water Resources, 3(1), 155-165.

Chapter 6. Derivation of Design Floods

Rory Nathan, Erwin Weinmann

Chapter Status	Final
Date last updated	14/5/2019

6.1. Overview

6.1.1. Selection of Basic Procedure

The available procedures can be divided into two main groups: those based on fitting a frequency curve to flood maxima, and those based on design rainfalls. Flood frequency methods (Book 3, Chapter 2) are used to provide estimates of peak discharge, but perhaps their most valuable role in the context of this Book is to provide information that can be used to validate, or even calibrate, rainfall-based procedures. The limit of credible extrapolation for Flood Frequency Analysis based on regional gauged data is perhaps 1 in 500 AEP (Table 8.1.1), though paleoflood analysis (Book 8, Chapter 6, Section 2) can be used to considerably extend this limit. The credible limit of flood frequency analysis that can be typically obtained using at-site data is perhaps only 1 in 100 AEP (Table 8.1.1).

Rainfall-based procedures use loss models and hydrograph models to transform design rainfall inputs into design flood estimates. Final design estimates of Very Rare to Extreme floods, beyond the credible limit of extrapolation (of either rainfall or floods), should be derived using rainfall-based procedures. The design details in the following sections relate mainly to rainfall-based procedures. As discussed in Book 1, Chapter 3, Section 4, event-based approaches are generally more applicable to the estimation of Very Rare to Extreme floods than are approaches based on continuous simulation; accordingly, the procedures as outlined in Book 4 are generally applicable to the estimation of design floods for Very Rare and Extreme events.

Book 8, Chapter 6, Section 1 briefly discusses issues related to the specification of design flood characteristics. Book 8, Chapter 6, Section 1 introduces a number of special design considerations that are covered in more detail in Book 8, Chapter 7. Subsequent sections (Book 8, Chapter 6, Section 2 to Book 8, Chapter 6, Section 4) provide guidance on final design procedures, while Book 8, Chapter 6, Section 5 discusses the treatment of uncertainties associated with flood estimates derived by these procedures.

6.1.2. Design Flood Characteristics

In many cases, the flood hydrograph is required as well as the peak discharge and in some cases may be more important. For the design of a dam spillway or a detention basin, floods calculated from a range of design rainfall durations should be routed through the storage for a variety of combinations of spillway and gate configurations, operating procedures and dam crest heights to determine the optimum design. Different durations of design rainfalls may be critical for different configurations and combinations of conditions, which should all conform with the recommendations of ANCOLD (2000). The complete hydrograph of the design flood is also required for flood studies where flow profiles in natural or constructed channels are to be calculated by unsteady flow procedures.

6.1.3. Expected Probability Adjustment

The concept of 'expected probability' and its application in practical design problems is explained in Book 3, Chapter 2, Section 5. It denotes the expected value of the AEP of a given flood magnitude. The expected probability is influenced by the uncertainty in parameters used to estimate the flood magnitude. Where the expected probability has not been implicitly determined in the Flood Frequency Analysis, an 'expected probability adjustment' to estimates of AEP can be applied subsequently to correct for any systematic bias in the estimated risk arising from parameter uncertainty.

In principle, the issue of expected probability is of considerable importance to the estimation of Very Rare to Extreme floods, as the magnitude of the adjustment is greatest for design flood magnitudes that involve significant extrapolation beyond the flood sizes in the sample being analysed. Furthermore, in many applications, the interest is directly on the AEP of a given flood magnitude (e.g. the spillway capacity of an existing dam), rather than on the flood magnitude for a given design AEP (relevant to design of new structures with a standards-based approach).

However, as indicated in Book 3, Chapter 2, the question of when it is appropriate to apply an expected probability adjustment is a complex one, and the decision depends on a number of theoretical and practical considerations. In general, the estimates of risk for Rare to Extreme floods are not used in an absolute sense, but to allow comparison with established levels of acceptable risk, or to establish risk-based economic costs for comparative evaluation of options.

It should be noted that use of the Total Probability Theorem to derive flood quantiles using Monte Carlo Event procedures as described in Book 4 yields expected probability quantiles. This approach ensures probability neutrality, at least for the set of hydrologic inputs used in its application. If estimates are derived from a blend of approaches, comments on the likely magnitude and importance of expected probability adjustments for different flood ranges are as follows:

- *Rare Floods* – significant extrapolation may be involved where these floods are estimated from frequency analysis of at-site data. The recommendations on the adjustment of Book 3, Chapter 2, Section 5 should thus be followed.

- *Very Rare Floods* – design rainfalls are estimated from regional analysis of large data sets. Generally this does not involve extrapolation beyond the probability plotting positions of the largest events, and any expected probability adjustment would thus be relatively minor for the rainfall frequency distribution. However, there is usually significant extrapolation of the rainfall-runoff model. In such cases, parameter uncertainty in rainfall-runoff model parameters may lead to a significant expected probability adjustment. At present, there are no accepted methods for making this adjustment.

- *Extreme Floods* – these are derived by methods of interpolation from other estimates for which an adjustment may have been made. Therefore separate adjustment is not required.

- *PMP Flood* – adjustment for expected probability may be appropriate in principle. However, current methods for estimation of the AEP of the PMP involve large uncertainties and are not sufficiently well developed to meaningfully apply expected probability.

6.1.4. Applications Requiring Special Considerations

The recommendations in this section apply only to the direct estimation of floods for the most common design applications without the consideration of other complicating factors. However, there may be some cases where some other set of circumstances may be critical for design. One example is dams with very large storages where it is necessary to take explicit account of initial storage level or rainfall sequences over very long durations. Another is the need to assess the likely rate of lake level rise to assist planning for emergency response purposes. The assessment of consequences on communities downstream of a dam may require consideration of concurrent floods in adjacent catchments. Also, a series of dams along a given stream requires special consideration, as failure of an upstream dam could impose more severe conditions on a downstream dam than its normal design flood. It is the responsibility of the practitioner to consider all circumstances that are critical for design. A number of issues related to these and other special design considerations are discussed in Book 8, Chapter 7.

6.2. Flood Data Based Estimates

6.2.1. General

The recommended methods for frequency analysis of Australian flood data are outlined in Book 3, Chapter 2. When selecting suitable data and methods for flood frequency analyses in the context of this Book, it should be kept in mind that the specific interest is on the upper tail of flood frequency distributions. This requires careful scrutiny of the accuracy of the largest flood observations in relation to possible data problems, in particular the accuracy of rating curves in the extrapolated range. Analysis of annual flood series would generally be more appropriate than partial series and, where available, data on large historical floods should be incorporated in the analysis. Similarly, the value of limited flood data at the site of interest may be enhanced by combining the results of at-site Flood Frequency Analysis and Regional Flood Frequency Estimation, using data from a number of sites within a homogeneous region. The special case of incorporating paleoflood data in the analysis is discussed briefly in Book 8, Chapter 6, Section 2.

In principle, it is also possible to derive design flood estimates for a site downstream of a reservoir directly from Flood Frequency Analysis of flood data available at that site. However, for reservoirs with large storage capacity compared to typical flood volumes, and with significant inter-event variability of reservoir flood storage, a much longer data series is required to adequately sample the combined effects of inflow and storage content variability. In these cases, the scope for extrapolation of directly determined flood frequency curves to the range of Rare to Very Rare events is severely limited. Book 8, Chapter 7, Section 2 gives further guidance on the derivation of reservoir outflow frequency curves.

6.2.2. Applications of Results of Flood Frequency Analysis

There are four different cases of how flood estimates based directly on Flood Frequency Analysis can contribute to the estimation of Rare to Extreme design floods:

i. *as direct basis for estimating Rare floods for final design applications:* where the range of AEPs is limited to 1 in 100 (perhaps 1 in 200 AEP for analysis of at-site and regional data);

ii. *as direct basis for estimating Rare to Very Rare floods for preliminary design or performance checks:* where the lowest AEP of interest is around 1 in 200 (for analysis of at-site flood data only) or perhaps 1 in 500 (for analysis of at-site and regional flood data);

iii. *as a basis for determining the lower end of a complete flood frequency curve:* where an estimate of the PMP Flood is available but no rainfall-based estimates of Rare to Very Rare floods;

iv. *as basis for independent checking of rainfall-based design flood estimates and possible adjustment of model parameters:* where rainfall-based design floods are to be determined for the full range of design floods, from Rare to Extreme.

Cases (i) and (ii) only involve extension of the flood frequency curve to the credible limit of extrapolation, but case (iii) requires the estimation of the PMP Flood and the application of an interpolation procedure for intermediate events (Book 8, Chapter 6, Section 3). Case (iv) requires detailed consideration of how the flood estimates from different sources can best be reconciled (Book 8, Chapter 5, Section 4).

6.2.3. Incorporation of Paleohydrological Estimates

Paleohydrological estimates of floods are based on the study of the geomorphic and stratigraphic record of past floods, as well as evidence of past floods and streamflow derived from historical, archeological, dendrochronologic, or other sources. The advantage of paleohydrologic data (USBR, 1999) is that it is often possible to develop records that are 10 to 100 times longer than conventional or historical records from other data sources. This information thus has the potential to provide estimates of Very Rare flood peaks that are independent of rainfall-based procedures. Such information can provide estimates of design floods directly, or else can be used to help select probability neutral design inputs for rainfall-based procedures (Book 8, Chapter 6, Section 3).

Overall, it is recognised that paleohydrological techniques have received little attention in Australia to date, but their potential for providing useful information on Very Rare floods has been demonstrated in Australia (Lam et al (2017a), and Lam et al. (2017b)). In view of the potential benefits, it is recommended that the use of paleoflood data should be considered where expenditure of the additional resources can be justified. Further information on the incorporation of paleoflood data in flood frequency analysis is provided in Book 3, Chapter 2, Section 3.

6.2.4. Preliminary Estimate of Rare to Extreme Events

There are some design situations where it is desirable to derive approximate design flood estimates by applying a “quick” method. Examples of situations where preliminary estimates are desirable include:

- flood estimates for preliminary assessment of spillway adequacy of existing dams;
- determination of priorities for the undertaking of detailed studies;
- estimation of concurrent floods of minor importance for the analysis of incremental consequences arising from dam failure;
- preliminary evaluation of different dam sites for planning studies; and,
- determination of hydrologic loads in a portfolio risk analysis of a group of storages.

The overall requirement for these types of analyses is that estimates can be derived quickly, and that given the large uncertainty the flood estimates should be biased towards conservatively high values.

Preliminary estimates should not be used for final design purposes, nor should the results be relied upon for making decisions about long term levels of acceptable risk. Practitioners are encouraged to use any information and methods that they consider appropriate, and the following recommendations are provided for general guidance only.

Generally two types of preliminary design estimates are required:

- *Peak discharge*: estimates of peak discharge are directly suitable for the preliminary design of bridge waterways and spillways for those storages where it can be conservatively assumed that only minor attenuation of the inflow hydrograph occurs;

- *Flood hydrograph*: estimates of the hydrograph are required where it is necessary to obtain an estimate of flood volume as well as peak discharge, for example the sizing of detention basins or the assessment of spillway adequacy for storages which appreciably attenuate the inflow hydrograph.

6.2.4.1. Preliminary Estimates of Peak Discharge

One possible approach to deriving a frequency curve of peak discharges is to derive preliminary estimates of Rare events from Flood Frequency Analysis and regional methods (Book 3, Chapter 2 and Book 3, Chapter 3), and for the PMP Flood. Preliminary estimates of the PMP Flood can be conservatively approximated by estimates of the PMF. Regional prediction equations for the PMF are available for some regions (e.g. Nathan et al. (1994), Pearce (2011), Malone (2011), Smythe and Cox (2006), and Watt et al. (2018)), though envelope curves for world floods may also provide useful information (Herschy, 2003). The preliminary estimates of the PMP Flood are plotted at the relevant AEP of PMP for the catchment using the recommendations provided in Book 8, Chapter 3, Section 4. These flood estimates can then be used to construct a frequency curve based on a log-Normal approximation, i.e. by fitting a straight line through the flood peaks in the logarithmic domain and probability as a standardised normal variate. Previous guidance (Nathan et al., 1999) recommended use of shape factors to define intermediate quantiles, but use of a simple log-Normal relationship should be sufficient as long as due regard is given to the large uncertainties involved.

6.2.4.2. Preliminary Estimates of Design Hydrographs

Estimates of the complete design hydrograph can also be obtained in a variety of ways. Such estimates generally require more time and effort in application than estimates of peak discharge, particularly as the estimated inflow hydrographs often need to be routed through a structure to assess the degree of attenuation.

Estimates of the volume of the hydrographs can easily be determined from estimates of design rainfalls and losses. The volume of the hydrograph can simply be determined as the average depth of rainfall excess over the catchment multiplied by the catchment area. Appropriate hydrograph shapes can be derived by scaling hydrographs obtained from either detailed studies on similar catchments or from suitable at-site records, though hydrographs obtained from rainfall-based models (using regional parameters) can be scaled to suit the preliminary peaks derived in Book 8, Chapter 6, Section 2.

6.3. Rainfall Based Estimates

6.3.1. General

General guidance on the estimation of design flood hydrographs using rainfall-based procedures is provided in Book 4, Chapter 2 and Book 4, Chapter 3. The following sub-sections provide guidance of specific relevance to the estimation of Very Rare to Extreme design floods, and should be read in conjunction with the guidance provided in Book 4.

6.3.2. Surface Runoff Hydrographs

The key input to the procedures is the appropriate design rainfall information from Book 8, Chapter 3. Rainfall excess must be estimated from the design rainfalls after due allowance is given to catchment losses (Book 8, Chapter 4). A rainfall-runoff model must then be used to convert the rainfall excess into the design hydrograph of direct runoff (Book 8, Chapter 5).

Where suitable rainfall and runoff data are available, the model selected should be calibrated using observed floods on the catchment of interest and, where appropriate, the parameter values should be adjusted to help reconcile differences between design values derived from Flood Frequency Analysis and rainfall-based methods (Book 8, Chapter 5, Section 4). In other cases, design values for the model parameters must be estimated from calibration on adjacent gauged catchments, regional relationships, or other relevant information. Where a concentrated storage, such as a reservoir or lake, can have a significant impact on the catchment response to rainfall, allowance must be made for its effect (Book 8, Chapter 5, Section 3). Design hydrographs usually need to be estimated for a range of design rainfall durations and AEPs in order to derive a complete flood frequency curve, and this is discussed in Book 8, Chapter 6, Section 3.

The rainfall-based procedure described above provides estimates of design floods that are comprised solely of direct runoff, i.e. that portion of the hydrograph that is derived from event-based rainfall excess. To derive design floods that reflect the total volume of the hydrographs, it is necessary to add baseflow (Book 8, Chapter 6, Section 3).

6.3.3. Incorporation of Baseflow

The hydrograph models generally only give the direct storm runoff, and some baseflow must be added to obtain the total hydrograph. While the proportion of baseflow is generally small compared with direct runoff, especially for Very Rare to Extreme floods, it may be of significance when simulating long duration events in volume-dependent problems (e.g. dam outflows).

Baseflow estimates for Rare events should be based on procedures described in Book 5, Chapter 4. Where there is clear evidence that initial baseflow increases with flood magnitude a constant baseflow 20% to 50% greater than the maximum value estimated in observed floods may be appropriate for Extreme events. If the difference between these two baseflow values is of minor importance then a representative, fixed value could be used for all intermediate AEPs. However, if deemed appropriate, the magnitude of the baseflow could be varied linearly on a plot of baseflow versus log(AEP) between the value adopted for the 1 in 100 AEP event and that adopted for the flood resulting from the PMP (alternatively Equation (8.4.1) could be used).

6.3.4. Simulation Framework

As discussed in Book 1, Chapter 3, Section 3, event-based models can be applied in a deterministic fashion ("simple event" simulation), where key inputs are fixed at representative values that minimise the probability bias in the transformation of rainfall into runoff. Alternatively, stochastic techniques can be used to explicitly resolve the joint probabilities of key hydrologic interactions; ensemble techniques provide simple (and approximate) means of minimising the bias associated with a single hydrologic variable, whereas Monte Carlo techniques represent a more rigorous solution that can be expanded to consider interactions from a range of natural and anthropogenic factors.

There seems little justification for use of *simple event* approaches for the estimation of Very Rare to Extreme floods as the dominant source of natural variability that influences flood magnitude for this class of events (other than rainfall depth) is typically the temporal pattern of incident rainfall. The *ensemble event* method (Book 3, Chapter 3, Section 2) represents a modest increase in computational requirements, whereby a representative sample of temporal patterns is used to provide a centrally tended estimate (either the arithmetic mean or the median) of the peak flow associated with the AEP of the input rainfall. A representative hydrograph from the ensemble can be scaled to match the derived peak for design purposes.

Monte Carlo event approaches provide the additional attraction that losses can be sampled (where designs are sensitive to long-duration events), along with other factors which may have a significant influence on the design outcome (such as reservoir drawdown, or spatial patterns of rainfall).

The general issues involved in the selection of the simulation framework are discussed in Book 4, though it should be noted that the estimation of extreme events can involve more significant degrees of non-linearity than present in the estimation of more frequent floods. For example, use of an ensemble event method to assess the influence of initial reservoir level on outflow floods is likely to provide highly biased estimates, which is avoided if a Monte Carlo scheme based on the Total Probability Theorem (or similar) is used (Book 4).

6.3.5. Derivation of Complete Design Flood Frequency Curve

The shape of the complete flood frequency curve in the Rare to Extreme event is largely determined by the shape of the design rainfall frequency curve described in Book 8, Chapter 3. Design rainfall inputs for specified AEPs are then converted to flood outputs in a probability neutral fashion, as discussed above (Book 8, Chapter 6, Section 1). For each AEP, flood outputs for a range of different durations have to be determined, and the one that gives the highest peak discharge (corresponding to the critical duration) is generally adopted. Minor adjustment of design inputs or smoothing of derived design floods for different critical durations may be required to obtain a smooth flood frequency curve. It is expected that estimates of Rare to Very Rare floods represent the "best estimate" obtained from multiple methods, as described in the preceding section (Book 8, Chapter 6, Section 3).

While the focus of the guidance in this Book is on Very Rare to Extreme flood events, it is important to check that the models yield estimates that are consistent with available evidence. Estimates of Rare floods provide the "anchor point" for derivation of more extreme events, and it is advisable to select a best estimate by weighting the estimates obtained from different methods by their uncertainty. In practice, however, the information required to do this is limited and it is recommended that where possible rainfall-based estimates are reconciled with independent estimates from Flood Frequency Analysis or regional flood method estimates (Book 3, Chapter 2 and Book 3, Chapter 3).

An example of such reconciliation is illustrated in Figure 8.6.1. In this example the independent estimates are obtained from Flood Frequency Analysis of observed annual maxima; the initial rainfall-based estimates were obtained from calibration of model parameters to historical floods (dashed blue line, Figure 8.6.1), and the loss parameters were then adjusted within their expected range to better align with the results obtained from Flood Frequency Analysis (solid blue line). As discussed in Book 8, Chapter 5, Section 4, reconciliation is best achieved by adjustment of the loss parameters within reasonable limits where routing parameters are obtained from fitting to historical floods. Ideally, the loss parameters should be reconciled jointly with quantiles based on flood peaks as well as flood volumes, where the duration over which flood volumes are calculated correspond to the critical duration of interest (e.g. the duration of a storm that yields maximum levels in a storage). The objective of such reconciliation is to adjust loss parameters within reasonable bounds to achieve a result that is reasonably consistent with both flood peak and flood volume quantiles, allowing for uncertainty in these estimates and the final best estimates based on consideration of both approaches should reflect the relative weight given to each approach for the range of AEPs of interest.

In reconciling differences in flood estimates from rainfall–based and flood frequency procedures, the assumptions behind each procedure should be carefully examined. For example with rainfall-based procedures, there is very little known about the manner in which non-linearity changes with flood magnitude, and the differences between design flood estimates may easily be explained by different assumptions regarding non-linearity. Similarly, certain assumptions will be inherent in the available period of flood record and quality of the rating curve. Ideally, the uncertainties should be explicitly evaluated to determine confidence limits, but in practice, sensitivity analysis of design inputs/parameters within expected limits will need to suffice.

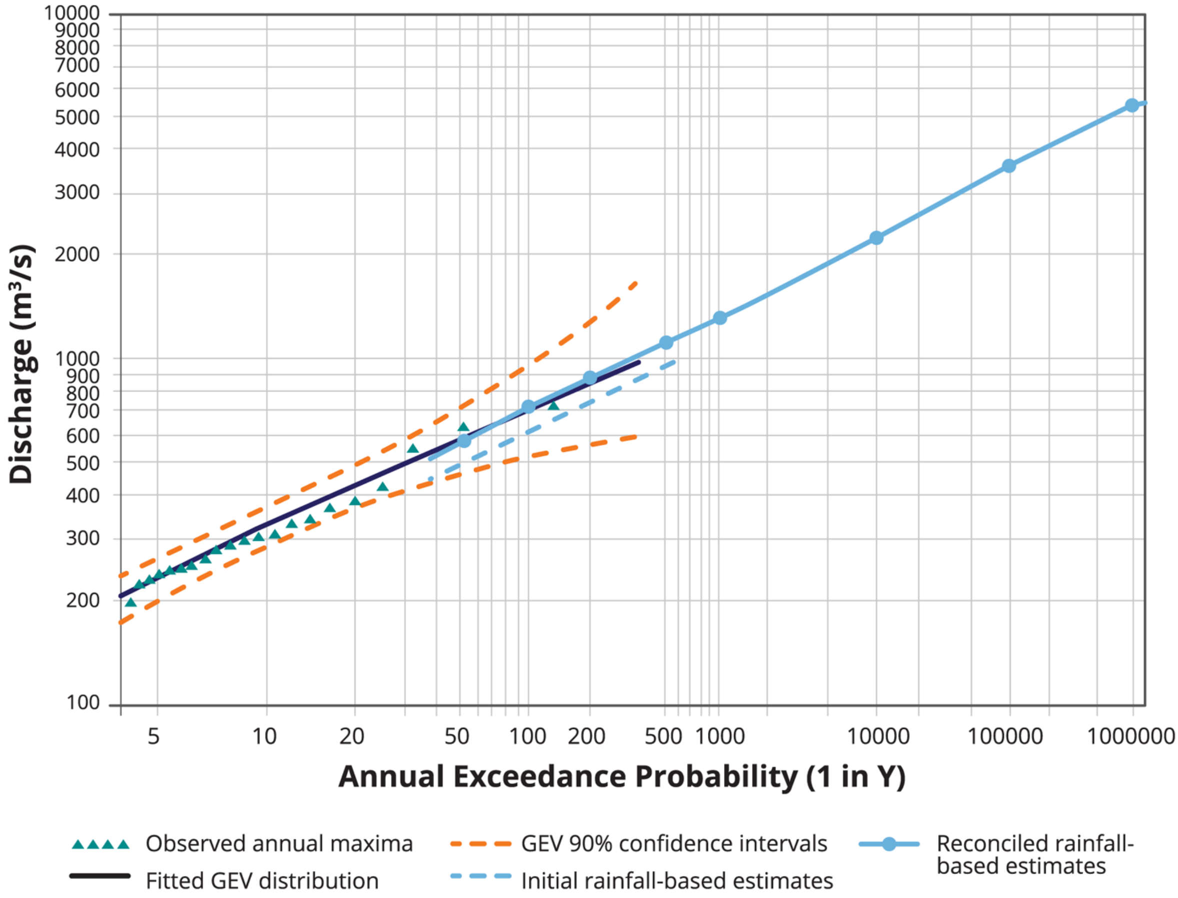


Figure 8.6.1. Illustration of Derived Frequency Curve Based on Reconciliation with Flood Frequency Quantiles

6.4. Estimation of the Probable Maximum Flood

6.4.1. Design Context

The Probable Maximum Flood (PMF) is a hypothetical flood estimate relevant to a specific catchment whose magnitude is such that there is negligible chance of it being exceeded. It represents a notional upper limit of flood magnitude and no attempt is made to assign a probability of exceedance to such an event. The concept of the PMF has been an important element in design flood standards for dams in the United States and Australia over the past 60 to 70 years (Myers, 1967; Brown, 1982; ANCOLD, 2000). It is commonly used in many other countries (ICOLD, 1991), though there are some countries, such as Russia, with little experience of the method and where preference is given to probabilistic methods (Zhirkevich and Asarin, 2010).

The PMF is also used to define the extent of flood-prone land (AEMI, 2014). The extent, nature and potential consequences of flooding associated with a range of events up to and including the PMF event is considered in some floodplain management studies. The PMF causes the largest scale of flood emergency and is also therefore often used for emergency management planning (AEMI, 2014). Guidance relevant to these purposes is provided in Book 8, Chapter 6, Section 4.

Pilgrim and Rowbottom (1987) defined the PMF as the limiting value of flood that could *reasonably* be expected to occur. Superimposing risks of very low probabilities was not considered reasonable, but it was considered prudent to incorporate some degree of conservatism. While it is possible to estimate an upper limiting value of flood magnitude, the estimation of its AEP is subject to even greater uncertainty than that of the PMP. Conservatively estimated (reasonably possible) values of the factors involved in the transformation of the PMP to the PMF introduce a shift in probability but, because the phrase “reasonably possible” is a qualitative description of probability, the AEP of the resulting flood varies depending on the degree of conservatism adopted. In practice, the magnitude of the PMF will be greater than the magnitude of the flood derived from the PMP using a transformation based on probability neutral objectives, but its AEP will be smaller.

Concerns around the difficulties of estimating the PMF in a consistent manner have been recognised for a long time (eg, (Newton, 1982; Barker et al., 1996; Nathan et al., 2011)). While the notion of a “probable maximum” flood standard appears a simple enough concept, in practice its estimation is confounded by a number of key problems (Nathan and Weinmann, 2004), namely:

- The lack of established criteria to determine the “reasonableness” with which to combine the various flood producing factors;
- The level of subjectivity inherent in assigning limiting maxima;
- Limited understanding of physical factors that constrain extrapolation of flood producing processes and their representation in models;
- Differential availability of relevant design information across the country; and,
- Poor selection of model structure and calibration of model parameters.

Accordingly, the intention of the recommendations herein is to retain the concept that the PMF represents the limiting value of flood that could reasonably be expected to occur, but to provide additional considerations that reduce the scope for inconsistency.

6.4.2. General Guidance

In the derivation of the PMF, the probability neutral objective for selection of design inputs is explicitly rejected in favour of adopting conservatively high estimates. With regard to losses, the general recommendations provided in Book 8, Chapter 4, Section 3 should be adopted, i.e. losses should be equal to or possibly a little less than the minimum value in large floods observed on the catchment. In all cases, losses are likely to be low; in many regions of Australia a burst initial loss value of zero and a continuing loss rate of 1 mm/hr will be appropriate. If pre-burst temporal patterns are used to represent complete storms, then it would be expected that the storm initial losses would be greater than zero, but at the lower end of the range of losses adopted for estimation of the PMP Flood.

The temporal patterns used to derive the PMF should be selected from an ensemble of patterns appropriate for use with the Generalised PMP (Book 8, Chapter 3, Section 8). Rearrangement of rainfall intensities within the patterns to give the highest possible flood peak may yield rainfall patterns with implausible serial correlation structure and is at variance with the objective of deriving a limiting value of flood that could reasonably occur. An estimate of a reasonable upper limiting value of floods may be derived by using the temporal (or spatially varying temporal (space-time) pattern from the available ensemble that yields the maximum flood characteristic of interest. It should be recognised that temporal and space-time patterns of rainfall based on historical events (Book 2, Chapter 4) are usually based on a limited number of pluviometers; when scaled to PMP storms over large catchments such patterns may yield embedded bursts of rainfall that are quite unrealistic. Accordingly, the characteristics of the PMF derived using a single temporal (or space-time) pattern should be checked against the results obtained from other patterns in the available ensemble. If the difference between the maximum adopted pattern and other results is anomalously large, then it may be appropriate to adopt a less severe pattern so as not to superimpose inputs of very low probabilities.

The hydrograph models used to transform the PMP to the PMF should follow the general recommendations provided in Book 8, Chapter 5, Section 2 and Book 8, Chapter 5, Section 3. Parameter values should be selected in accordance with the recommendations provided in Book 8, Chapter 5, Section 4. The selection of other design inputs, such as initial reservoir level or snowpack depth, should be representative of the more extreme conditions that could reasonably be expected to occur.

6.4.3. Checks on Upper Limiting Magnitude

Estimates of the PMF may be required as a check on the upper limiting magnitude of potential inundation in floodplains for planning and emergency management. Given that these estimates are used for planning rather than design purposes it is appropriate to adopt simpler considerations than those discussed in the following section. Accordingly, for this class of estimates it is considered sufficient to estimate the PMF based on the following simple deterministic assumptions:

- 0 mm burst initial and 1 mm/hr continuing loss rates (or higher as justified for such regions as the south west of Western Australia);
- the temporal (or space-time) pattern from a sample of ten that yields the highest magnitude flow;
- the storage levels in any upstream impoundments are assumed to be initially full; and
- other inputs influencing the design estimate should be set at their notional maximum.

It is considered reasonable that estimates required for such upper limiting checks be derived for the critical location at a single location representative of the planning focus of interest. It is accepted that the above assumptions may be considered unreasonably conservative compared to the more detailed assessment described in the following section, but that this considered reasonable given the planning context for which such estimates are required.

6.4.4. Assessment of Reasonableness for Design Estimates

For PMF estimates used for detailed design purposes, such as for the assessment of dam safety, it is recommended that more careful consideration be given to the reasonableness of the underlying assumptions than is required for upper limiting checks, as described in the preceding section.

The cost implications of upgrading dams may well be very sensitive to the degree of conservatism adopted by practitioners when assessing the “reasonableness” of assumptions used to derive the PMF. For example, as illustrated in Figure 8.6.2a (dashed curve), the costs of providing additional flood capacity may increase monotonically with flood magnitude; under such a scenario there is no obvious point where the upgrade costs increase disproportionally with the degree of conservatism adopted. However, if there is a step function involved in the relationship between flood capacity and cost – for example if an additional spillway is required because the practical limit of extending a wave wall has been reached (solid curve in Figure 8.6.2a) – then a small difference in subjective judgement may have a significant impact on the costs and feasibility of an upgrade.

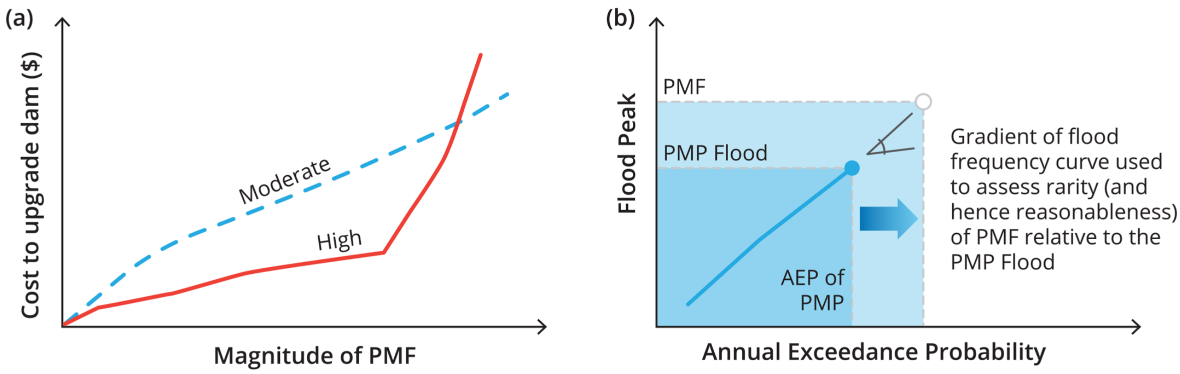


Figure 8.6.2. (a) Differential Importance of "Reasonableness" in PMF Assumptions on Dam Safety Decision-Making, and (b) Use of Simple Extrapolation to Infer Degree of Reasonableness

It is clearly undesirable that small differences in subjective assessments of “reasonableness” might have a large impact on design costs. Accordingly, in some situations it will be prudent to explicitly examine the impact of any subjective hydrological judgement, and the final decision regarding the appropriate level of conservatism should be made in consultation with the wider dam safety engineering team involved.

To this end, the following steps may be warranted when providing an estimate of the PMF:

- Derive the estimate of the PMP Flood under probability neutral assumptions
- Then, derive a deterministic estimate of the PMF using:
  - 0 mm burst initial and 1 mm/hr continuing loss rates (or higher as justified for such regions as the south west of Western Australia);
  - the temporal (or space-time) pattern from a sample of ten that yields the highest magnitude flow;
  - if the design is for a dam, then adopt an initial storage at Full Supply Level.
- Estimate the shift in AEP associated with the difference in magnitude between the PMP Flood and the PMF (by simple extrapolation as shown in Figure 8.6.2b).
- If a deterministic modelling framework is used to estimate the PMP Flood, then undertake a number of simulations using inputs selected from a plausible range of values to understand the catchment specific impacts of the PMF assumptions made.
- If a Monte Carlo framework is used to estimate the PMP Flood, then also calculate the proportion of samples in which the PMF is exceeded given the PMP depth as input. If the shift in AEP (as shown in Figure 8.6.2b) is greater than one order of magnitude, or the conditional probability that the PMF is exceeded is less than around 10% to 1%, then revisit assumptions used to derive the PMF and relax as appropriate.
- Finally, check the sensitivity of any decisions that are to be based on the PMF estimate – if there is a marked difference in outcome within a range of estimates that could be

considered to be based on a “reasonable” set of assumptions, then reach agreement with the wider engineering team on the appropriate degree of conservatism to adopt.

It is expected that the above steps will only be required in a small proportion of cases in which design and or mitigation costs increase disproportionally with the degree of conservatism adopted.

6.5. Treatment of Uncertainty

Uncertainties in the estimation of extreme floods have important economic and social consequences, and thus recognition of the impacts of uncertainty should be incorporated into advice given to management and political decision makers. If there are significant differences in outcome within the range of uncertainty then the likely range of consequences should be explicitly considered when developing mitigation strategies and advice. An under-estimate of the flood magnitude will lead to the infrastructure being under-designed, thus potentially resulting in increased flood damage costs and possible loss of life. Conversely, an over-estimate of the flood magnitude will lead to extra costs from the over-design of the infrastructure.

General guidance on techniques for characterising uncertainty is presented in Book 7. In the context of this Book, it should be stressed that the uncertainty or flood estimation error increases with increasing size of flood (or reducing AEP) and the relative impacts of different sources of uncertainty also change with flood magnitude. Uncertainty in the AEP of the PMP becomes increasingly important beyond the credible limit of extrapolation, and so does the epistemic uncertainty associated with the increasing lack of evidence to support the process descriptions of salient factors (e.g. temporal and spatial characteristics of rainfall) and model structure (e.g. degree of non-linearity in flood behaviour).

The estimation of Very Rare to Extreme floods is a region where “the computation of hydrologic probabilities is based on arbitrary assumptions about the probabilistic behaviour of hydrologic processes rather than on empirical evidence or theoretical knowledge and understanding of these processes” (Klemes, 1993). Improving the consistency of the manner in which such assumptions are applied in practice will thus minimise the potential for differences in the results obtained by different hydrologists. The main strategy available for reducing the impact of this form of uncertainty is to ensure that the practitioners undertaking the work are appropriately qualified and supervised. In addition, prescriptive procedures relating to the estimation of floods beyond the credible limit of extrapolation are justifiable as without empirical evidence or scientific justification there can be little rational basis for departing from a consensus approach.

6.6. References

Australian Emergency Management Institute. (2014). Managing the floodplain: a guide to best practice in flood risk management in Australia. Attorney-General's Department, Canberra.

ANCOLD (Australian National Committee on Large Dams) (2000), Selection of acceptable flood capacity for dams, March

Barker, B., Schaefer, M., Mumford, J. and Swain, R. (1996), A Monte Carlo Approach to Determine the Variability of PMF Estimates. In Proc. Conf. American Society Dam Safety Officials.

Brown, J.A.H. (1982), A review of flood estimation procedures. Proc. of the Workshop on Spillway Design, Dept of Natl Devel. and Energy, AWRC Conf. Series No.6, pp: 84-108.

Herschy, R. (2003), World catalogue of maximum observed floods. IAHS Publ No. 284, pp. 285

ICOLD (International Commission on Large Dams). (1991) Vienna.

Klemes, V. (1993), Probability of extreme hydrometeorological events - a different perspective. Extreme hydrological events, precipitation, floods, and droughts. Yokohama Symposium., Yokohama, IAHS Publication No. 213. pp: 167-188.

Lam, D., Thompson, C., Croke, J., Sharma, A. and Macklin, M. (2017a), Reducing uncertainty with flood frequency analysis: the contribution of palaeoflood and historical flood information. Water Resour. Res. 53: 2312-2327.

Lam, D., Croke, J., Thompson, C. and Sharma, A. (2017b), Beyond the gorge: Palaeoflood reconstruction from slackwater deposits in a range of physiographic settings in subtropical Australia. Geomorphology, 292: 164-177.

Malone, T. (2011), Extreme design floods for Seqwater storages. In: Proc. 33rd Hydrology and Water Resources Symposium (Brisbane). Barton, A.C.T.: Engineers Australia, pp: 323-330.

Myers, V.A. (1967), 'The Estimation of Extreme Precipitation as the Basis for Design Floods, Resume of Practice in the United States', Proceedings of the Leningrad Symposium, Floods and Their Computation, International Association of Scientific Hydrology.

Nathan, R.J. and Weinmann, P.E. (2004), Towards increasing objectivity in the Probable Maximum Flood. Proc. ANCOLD/NZSOLD Conference, 14-17 Nov 2004.

Nathan, R.J., Weinmann, P.E., and Gato, S. (1994), A quick method for estimation of the probable maximum flood in South East Australia. Int. Hydrology and Water Resources Symposium: Water Down Under, November, Adelaide, I.E. Aust. Natl. Conf. Publ. No.94, pp: 229-234.

Nathan, R.J., Weinmann, P.E. and Minty, L. (1999), Estimation of the Annual Exceedance Probability of PMP Events in Southeast Australia, Aus. J. Water Resour., 3(1), 143-154.

Nathan, R., Hill, P, and Weinmann, E. (2011), Achieving consistency in derivation of the Probable Maximum Flood, ANCOLD 2011 Conference on Dams. Melbourne.

Newton, D. W. and Herrin, J. C. (1982), Assessment of commonly used methods of estimating flood frequency. Transportation Research Record, p: 896.

Pearce, L.J. (2011), Regional Flood Estimation for Small Catchments in South-West Western Australia. In: Proc. 33rd Hydrology and Water Resources Symposium (Brisbane). Barton, A.C.T., Engineers Australia, pp: 323-330.

Pilgrim, D.H. and Rowbottom, I.A. (1987), Chapter 13 - Estimation of large and extreme floods, In Pilgrim, D.H. (ed.) Australian Rainfall and Runoff: A Guide to Flood Estimation, I.E. Aust., Canberra.

Smythe C. and Cox G. (2006), A regional flood frequency method for Tasmania, Proc. Hydrology and Water Resources Symposium, Launceston.

U.S. Bureau of Reclamation (USBR) (1999), A framework for developing extreme flood inputs for dam safety risk assessments, Part I - Framework, Draft, February 1999.

Watt, S., Sciacca, D., Hughes, M., Pedruco, P. (2018), A quick method for estimating the Probable Maximum Flood in the Coastal Zone. Proc. Hydrology and Water Resources Symposium, Melbourne.

Zhirkevich, A., Asarin, A. (2010), Probable maximum flood (PMF): basic information and problems with the procedure used for its calculation in Russia, Power Technology and Engineering, 44(3), 195-201.

Chapter 7. Special Design Considerations

Rory Nathan, Erwin Weinmann

Chapter Status	Final
Date last updated	14/5/2019
Minor edits	27/08/2024

7.1. General

There are a number of special considerations that are relevant to some design situations and the following sections detail some of the more common issues that may need to be considered. The importance of these considerations, and hence the complexity of the techniques required to adequately address the issues, is very much dependent on the characteristics of the specific design problem. For example, where the storage volume of a reservoir is large compared to the volume of catchment runoff, the choice of initial starting levels in the reservoir is likely to have a more significant impact on the outcome of the study than the selection of runoff-routing parameter values.

One design objective of general importance is the derivation of floods of specified AEP. Satisfying this objective generally requires the adoption of probability neutral inputs i.e. the selection and/or treatment of design inputs to ensure that any bias in the AEP of the transformation between rainfall and runoff is minimised. The issues considered in this section are generally aimed at the more rigorous treatment of the joint probabilities involved in the selection of design inputs. However, as discussed in Book 8, Chapter 2, Section 1, it should be recognised that the defensibility of these estimates rests upon the representativeness of the selected inputs and the correct treatment of correlations which may be present.

The appropriate level of complexity to be adopted is dependent upon the sensitivity of the design outcome to the input. Accordingly it is not possible to provide recommendations that are applicable to all design situations. The procedures recommended here are relevant to many situations, but they should be regarded as providing only a general guide to recommended practice. The practitioner is thus encouraged to adopt different procedures if they have a sound theoretical basis.

7.2. Derivation of Reservoir Outflow Frequency Curves

7.2.1. Importance of Reservoir Storage and Initial Drawdown

The attenuation of an inflow hydrograph as it passes through a reservoir or another natural or artificial storage depends mainly on the available storage volume relative to the flood volume, and to a lesser degree on the spillway capacity and the degree of regulation of outflows by spillway gates or other outflow control structures. More specifically, the total storage available to mitigate floods can be divided into two parts: the storage above the normal full supply level (flood storage) and the drawdown below full supply level at the onset of a flood (initial drawdown, or air-space). The flood storage for a given inflow hydrograph is a fixed system characteristic determined by the adopted spillway and freeboard characteristics of the storage, but the initial drawdown or initial reservoir level is a stochastic variable.

The selection of an appropriate initial reservoir level is of considerable importance in determination of spillway adequacy. In particular, it is an important consideration in the determination of criteria related to the flood capacity of the dam, such as the Dam Crest Flood and the Imminent Failure Flood (ANCOLD, 2000). In many cases it may be appropriate to adopt a full reservoir level, but if there is a reasonable chance that the reservoir may be drawn down, and if the volume of drawdown is significant compared to the volume of the inflow floods of interest, then it will be desirable to analyse in more detail the effect on estimates of the frequencies of a particular peak outflow of the variation in storage volume. Where there is a strongly seasonal variation of storage volume, it may be necessary to undertake a seasonal analysis of storage impacts on outflow floods.

7.2.2. Approximate Methods - Representative Initial Storage Volume

For preliminary analyses it may be sufficient to adopt a mean or median storage volume, or else compute the mean or median storage volume associated with, say, the top 10% of inflow floods. In general, adoption of a mean or median value will not provide a probability neutral transformation as the relationship between inflow and outflow floods is highly non-linear. Accordingly, for detailed design estimates, it is prudent to determine the probability of the outflow hydrograph by the joint probabilities of the inflow and initial storage volume, and by the deterministic relationship that governs the conversion of an inflow hydrograph of given duration and magnitude into an outflow hydrograph for different storage volumes.

7.2.3. Joint Probability Analysis of Inflow and Initial Storage Volume

7.2.3.1. Background

Laurenson (1974) developed a method for the analysis of systems which incorporate both stochastic and deterministic components (in this context, the joint probabilities of the inflow and initial storage volume represent the stochastic component, and the relationship between the magnitudes of inflow and outflow floods represent the deterministic component). Laurenson’s method provides a rigorous means of solving the joint probabilities involved, though it is not easily automated and is not well suited to accommodating correlations that may exist between the stochastic components.

The analysis of the joint probabilities of storage volume and inflows is just one example of the more generic solution offered by Monte Carlo methods. Accordingly, if the rainfall-runoff modelling is undertaken in a Monte Carlo framework, then this is easily extended to consider reservoir outflows.

Application of either method is straightforward as long as the probabilities of all the inputs can be appropriately defined; some care is required to ensure that the distributions are representative of the design conditions of interest, though in most situations where it is worthwhile undertaking the analysis the required information can usually be derived. The guidance in this section first covers specification of the input distributions as this is common to both methods, and this is followed by a description of the different solution schemes.

7.2.3.2. Representation of Input Distributions

The selection of class intervals for the approximate representation of continuous probability distributions by discrete ones represents a compromise between efficiency and accuracy of computations. A total of around 20 to 30 class intervals is generally sufficient, but they need to be well distributed over the range of possible variate values to ensure accuracy in the most important part of the range. Each interval is then represented by the variate value at the mid-point of the interval and by the width of the interval on the probability scale. The total probability of all the intervals must add up to unity. It is worth considering the following issues when discretising the distributions:

- *Discrete probability distribution of flood inflows:* It is desirable to discretise the probability distribution of flood inflows (Book 8, Chapter 6, Section 3) so as to have most of the classes representing Rare to Extreme floods; classes do not need to cover equal probability or flow ranges. One pragmatic approach is to discretise using *N* intervals uniformly spaced over the standardised normal probability domain. For example for an *N* of 20, the probability domain for AEPs over the range 0.5 to 1.0^{-6} equate to standard normal variates ("*z* scores") of 0.0 and 4.75, thus 20 inflows ranges can be computed for 19 intervals of width z = 0.25. Equal intervals in the standard domain equate to unequal intervals between AEPs, and are preferred as inflows are approximately log-Normally distributed, and intervals of equal probability would lead to the selection of most of the classes encompassing flows of little concern.

- *Probability distribution of initial storage volume.* The analysis of a time series of storage level or storage volume is used to define the probability distribution of initial storage volume. The time series of reservoir storage volume could be derived directly from the historical record, but in most cases a synthetic time series of storage volume, derived from simulation (behaviour analysis) studies, would be more appropriate. In the latter approach the current operating rules can be applied to the historic climatic sequence, thus providing a long stationary series relevant to the system under consideration. The usual time interval for behaviour analyses is one month, which allows the within-season variation of storage volume to be taken into account in the frequency analysis.

- *Dependence between flood inflows and storage volume.* The historical (or synthetic) time series should be checked to see if there is a strong dependence between initial reservoir level and flood inflows. If such dependence exists, then it would be necessary to derive conditional probabilities of initial storage volume that correspond to different ranges of flood inflows. To this end, it would be necessary to divide the inflow magnitudes into a small number, say, three flow ranges (corresponding to low, average and high flows), and derive separate distributions of initial storage for each. Care needs to be taken when inferring correlations for extreme conditions based on a short period of historic (or simulated) record, and distributions based on empirical analyses may need pragmatic adjustment to ensure that they are representative of extreme conditions. Analysis of regional rainfall information for relevant critical durations within a meteorologically homogeneous region can provide information to help condition such relationships, and an example of this using standardisation to trade space for time is provided by Scorah et al. (2015).

7.2.3.3. Laurenson's Analytical Solution

The analytical solution proposed by Laurenson (1974) involves the convolution of the conditional probability distribution of outflows with the distribution of the conditioning event. In principle, the conditioning event may be either the reservoir inflow or the initial storage volume, but reservoir inflow is adopted in most applications. In practice the convolution is achieved by approximate numerical methods, based on discrete approximations to the continuous probability distributions of the inflows and the outflows. To this end, the total range of inflows and outflows has to be divided into a finite number of class intervals.

The conditional probability of a specified outflow event occurring, given that the conditioning event is in a specific class interval, can be determined using a deterministic relationship between inflows, outflows, and storage volume (the I-S-Q relationship). The I-S-Q relationship has to be determined for a range of peak inflows (corresponding to a range of design rainfalls for selected exceedance probabilities) and for a set of initial storage values. The process of computing the conditional probability of a specified outflow event is illustrated in Figure 8.7.1 for the case where the reservoir inflow was chosen as the conditioning event and the initial storage volume as the secondary variable. From Figure 8.7.1 it is seen that the conditional probability of a specified outflow event is evaluated as the width of the storage volume probability interval ($P[Q_j|I_i]$) that translates an inflow in the interval I_i into an outflow in the interval Q_j.

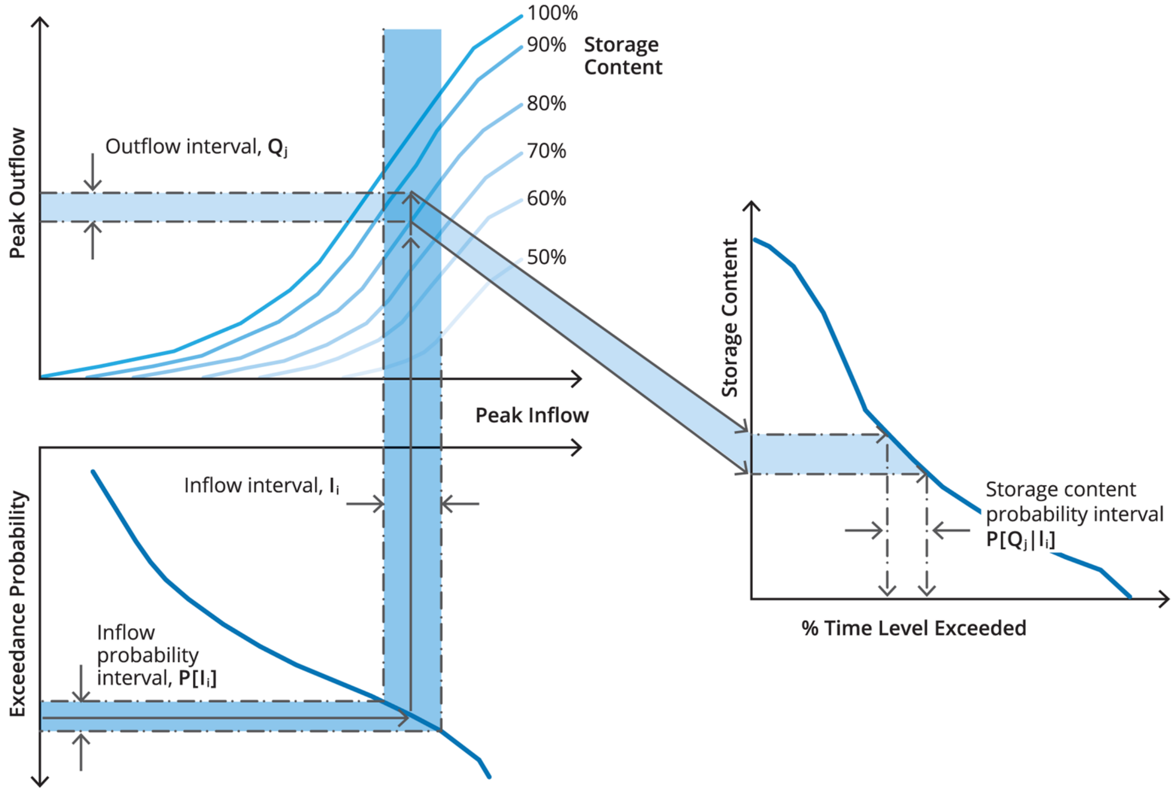


Figure 8.7.1. Schematic Illustration of the Determination of the Probability Interval of Storage Volume as a Function of Inflow and Outflow

As different design rainfall durations result in different I-S-Q relationships, the computed value of the storage volume probability interval will also depend on the rainfall duration used. The critical rainfall duration to be used in the analysis is the one that translates into the highest outflow; this also produces the largest estimate of conditional outflow probability. Unfortunately, the critical rainfall duration varies with reservoir drawdown, and in some cases it is necessary to compute separate I-S-Q relationships for different durations, and to derive an outflow frequency curve as the envelope of frequency curves derived for different durations.

Another complication is that the above formulation assumes that the two distributions of storage volume and inflows are independent. This may not be the case, and if such correlation is found to be significant then the calculations must be based on the appropriate conditional selection of input variables.

The evaluation of the I-S-Q relationship is the most time consuming element of the process. Many tens of individual runs are required to define the I-S-Q relationship in sufficient detail, though it is possible to automate the processing of different initial starting levels. The computation of the conditional probabilities is readily undertaken using spreadsheet software and is not resource intensive.

The derivation of the outflow frequency curve by Laurenson (1974) joint probability approach involves the calculation of a transition probability matrix. Each element in this matrix represents the conditional probability of an inflow within the given inflow interval resulting in an outflow in a specified interval. Depending on the degree of non-linearity of the spillway rating curve, outflows may be discretised into class intervals of equal magnitude, or else intervals can be selected to provide more accuracy in the region of interest (e.g. for flows just above and below the spillway capacity). The total probability of an outflow in that interval can then be obtained as the sum of the probabilities over all the inflow intervals, i.e. all the inflow and initial storage combinations that produce an outflow in the specified range. Outflow AEPs are then computed as the cumulative probability over all outflow ranges exceeding the flood magnitude of interest.

An example of the application of this approach is given in Book 8, Chapter 8, Section 4.

7.2.3.4. Monte Carlo Analysis

An outflow frequency curve can be derived using Monte Carlo techniques as a straightforward extension of the framework described in Book 4. The concept for this is shown in Figure 8.7.2, where in this example the distribution of drawdown is based on a simple non-parametric relationship between drawdown and the proportion of time that it is not exceeded (solid line, lower left panel). It should be noted that it is not necessary to define the extreme tails of the drawdown distribution as the largest outflows are primarily driven by extreme rainfalls, where initial reservoir levels are most likely to be within the central range of exceedance. If the distribution of initial drawdown is assumed independent of extreme event rainfalls then it will only be necessary to sample from the one relationship; however, the dashed blue curve in the lower left panel of Figure 8.7.2 illustrates that a different drawdown distribution could be used for more extreme events, and thus different distributions can be selected conditional upon rainfall depth.

The outflow frequency curve can be derived either by direct frequency analysis of the outflow peaks, or by application of the Total Probability Theorem. The latter approach is suited to stratified sampling schemes, as would generally be required for estimation of Extreme events. A description of the different methods available to derive a frequency curve based on Monte Carlo sampling is provided in Book 4, and an example calculation for sampling from an empirical frequency curve is provided in Book 4.

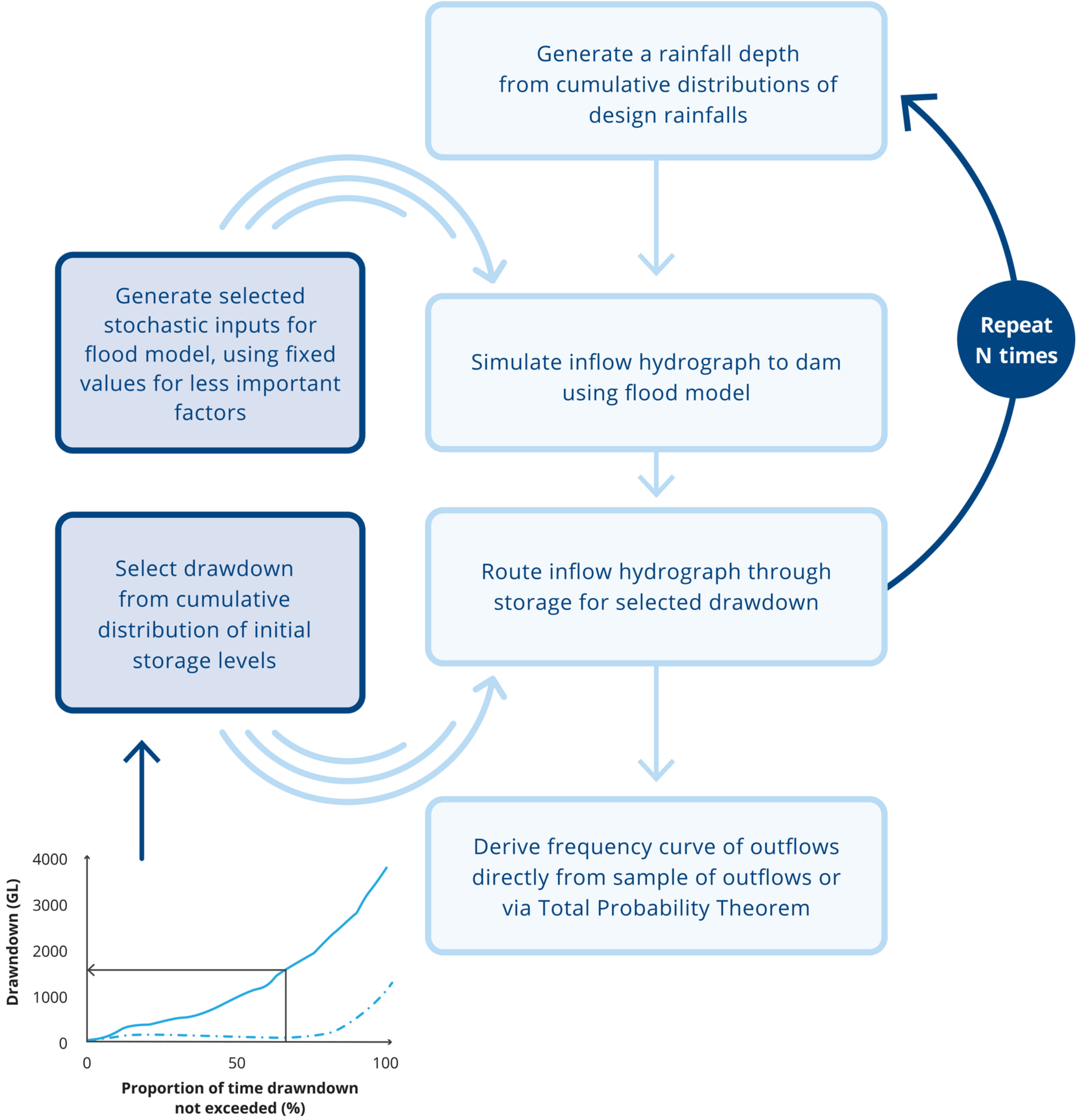


Figure 8.7.2. Illustration of Monte Carlo Framework to Derive Outflow Frequency Curve

7.2.4. Consideration of Cascade of Storages

It is sometimes necessary to derive a flood frequency curve for a location downstream of several dams. This situation most commonly occurs with hydropower schemes, but also arises with storages used for water supply. The complexity of analysis required depends on the size of the upstream storages and the degree of inter-dependence in their operation. For the simplest cases it may be sufficient to represent drawdowns in the smaller storages as fixed values and derive outflow frequency curve in the storage most sensitive to initial conditions as described in Book 8, Chapter 7, Section 2 Where initial levels in one reservoir are correlated with levels in another, then a conditional sampling approach can be adopted.

The nature of dependence in storage contents is shown by the large diamond symbols in Figure 8.7.3, which is derived from the behaviour of two reservoirs located in south-eastern Australia. Such data is difficult to normalise or fit to (bivariate) probability distributions, and thus an empirical sampling approach can be used. The approach to stochastically sample from such a data set can be described as follows:

1. Identify the “primary” variable that is most important to the problem of interest, and prepare a scatter plot of the two variables with the primary variable plotted on the x-axis (as shown in Figure 8.7.3).

2. Divide the primary variable into a number of ranges such that variation of the dependent variable (plotted on the y-axis) within each range is reasonably similar; in the example shown in Figure 8.7.3 a total of seven intervals has been adopted as being adequate. This provides samples of the secondary variable that are conditional on the value of the primary variable.

3. Stochastically generate data for the primary variable using an empirical sampling approach as described in Book 8, Chapter 8, Section 3.

4. Derive an empirical distribution of the dependent data for each of the conditional samples identified in Step 2 above; thus, for the example shown in Figure 8.7.3 a total of seven separate empirical distributions of upstream storage levels are prepared (these are shown as separate curves on the inset panel in Figure 8.7.3).

5. For each generated value of the primary variable, stochastically sample from the conditional distribution corresponding to the interval that it falls within; for example, if a downstream storage level of 1500 ML was generated in Step 3 above, then the corresponding conditional distribution (E) is used.

The results from application of the above procedure are illustrated in Figure 8.7.3 for 2000 stochastic samples (shown by the blue “+” symbols). It is seen that the correlation structure in the observed data set is preserved reasonably well by this procedure.

While the above approach can be extended to multiple storages, obviously this becomes progressively more tedious to implement. At some point the dependencies are better modelled using continuous simulation as the system will be largely dependent on the sequences of flood volumes.

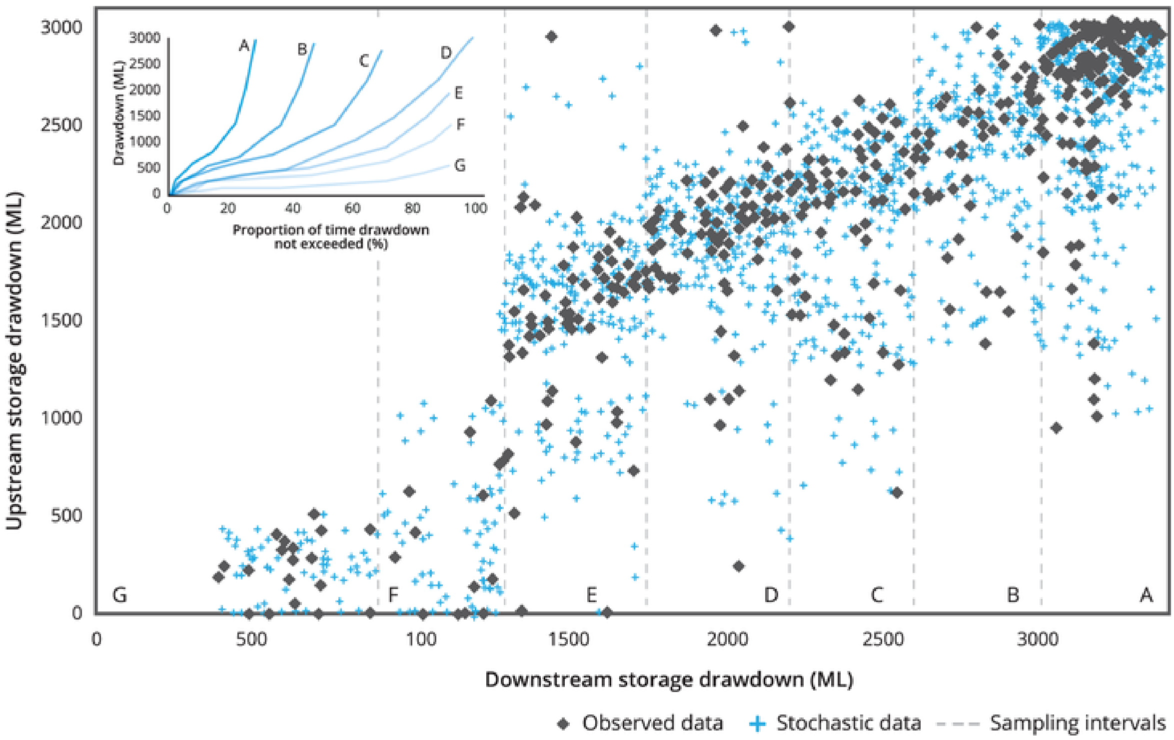


Figure 8.7.3. Illustration of Conditional Empirical Sampling in Which the Storage Volume in an Upstream Dam is Correlated with the Volume in a Downstream Dam

7.3. Concurrent Tributary Flows

7.3.1. Overview

In some design situations it is desirable to determine the flow in an adjacent catchment that is likely to coincide with design floods in the stream of interest. The most common requirement for this is the assessment of the incremental impact of dam failure, where it is desirable to identify separately the inundation due to the direct consequences of dam failure and the floods generated from adjacent catchments.

There are a number of methods available for the assessment of concurrent flows (refer to for example, Book 4, Chapter 4). In the context of risk analysis it is important to focus on those methods that yield probability neutral estimates. In essence, the issue of concurrent flooding is another joint probability problem, and the method of Laurenson (1974) described in Book 8, Chapter 7, Section 2 can be applied directly to the joint occurrence of floods in tributaries and adjacent catchments. With the analysis of concurrent flows, the deterministic I-S-Q relationship referred to in Book 8, Chapter 7, Section 2 is replaced by the relationship between total flows downstream of the confluence and the joint occurrence of upstream flows of differing magnitudes, and the marginal distribution of storage volume is replaced by the probability distribution of flows in the adjacent tributary. Careful consideration needs to be given to the specification of the marginal distribution of tributary inflows as the two flow distributions will be correlated. Also, peak discharges are unlikely to coincide. The worked example provided on this approach provided in Book 8, Chapter 8, Section 3 is directly applicable to this situation and can be applied if desired.

Monte Carlo techniques also provide a rigorous solution to the problem. If space-time patterns of rainfall are used in the modelling then an unbiased estimated of the frequency distribution of tributary inflows can be obtained by application of the Total Probability Theorem as described in Book 4. An example of this approach is described by Jordan et al. (2005). However, it may be that the tributary inflows are located well downstream of the catchment being modelled, and if this is the case then it may be easier to estimate concurrent flows using a more explicit scheme, as described Book 8, Chapter 7, Section 3 and Book 8, Chapter 7, Section 3.

7.3.2. Stochastic Simulation

The generation of tributary flows can be simulated using a stochastic approach in which the correlation structure of the inputs is explicitly preserved. A simple means of generating correlated variables is described by Saucier (2000). The approach is based on rotational transformation and the steps involved in generation of normally distributed variates can be stated as follows:

1. Independently generate two normal random variates with a mean of zero and a standard deviation of 1: *X= N(0,1)* and *Z= N(0,1)*
2. Set $Y = \rho X + Z\sqrt{1-\rho^2}$

   where ρ is the required correlation between *X* and *Z*
3. Return:

   $x = \mu_x + X\sigma_x$

   $y = \mu_y + X\sigma_y$

   where and are the means of the two distributions and and are the required standard deviations.

For application to catchment rainfalls, *X* and *Y* could represent the log-transformed values of rainfall maxima, in which case the above scheme would represent the generation of a bivariate log-Normal distribution of rainfalls which has been found to provide a satisfactory approximation over the range of AEPs of interest (Nandakumar et al., 1997). The stochastic rainfalls could be used in conjunction with a rainfall-based method to provide concurrent flood hydrographs. Estimates of suitable correlations can be obtained from the analysis of observed rainfall data, or else using the generalised correlation-distance relationships reported in Nathan et al. (1999). Ideally, however, such correlations would be determined using areal rainfall estimates based on site-specific analysis of gridded data (e.g. Jones et al. (2009)).

Application of the above algorithm is illustrated in Figure 8.7.4. The input parameters to this example are ρ=-0.7, μ_x=70 and σ_x=10, and μ_y=50 and σ_y=10, and as before a total of 2000 correlated variates are generated. Any distribution could be used in lieu of the Normal distribution, or else the variates of interest could be transformed into the normal domain.

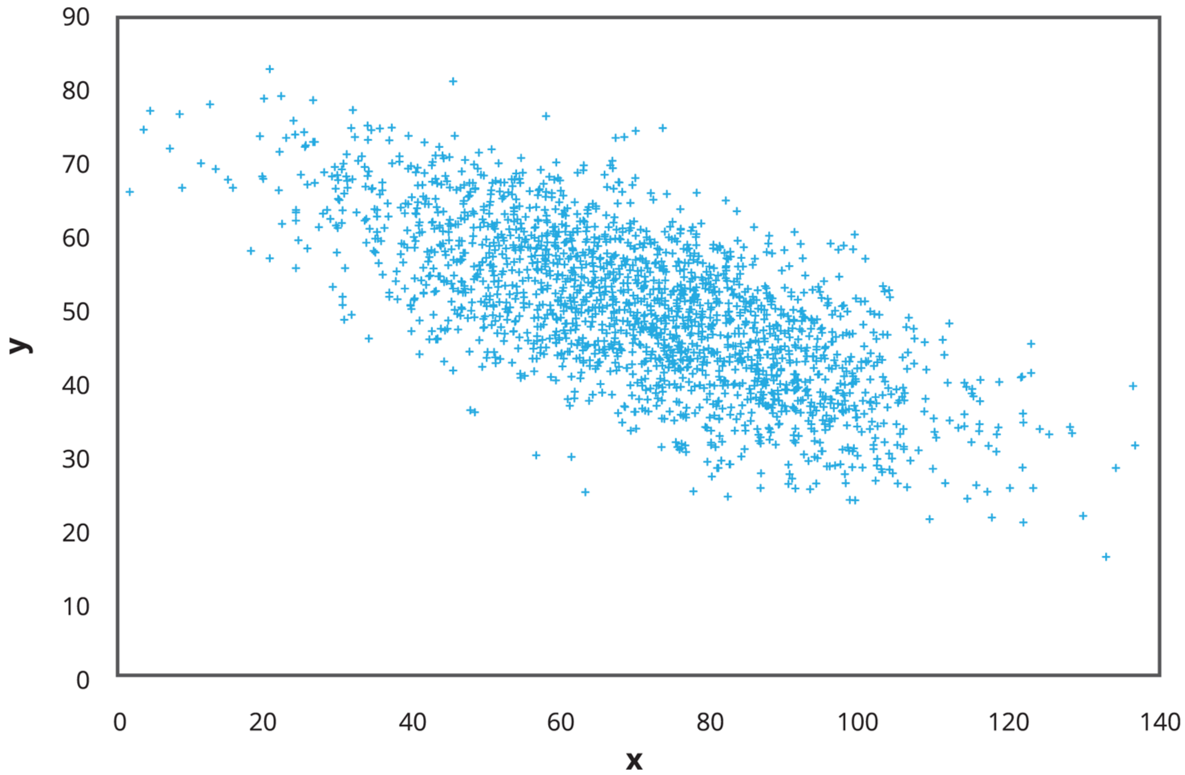


Figure 8.7.4. Illustration of the Generation of Variables with a Correlation of 0.7 Based on Normal Distributions

7.3.3. An Approximate Approach

The following method provides one example of an approximate approach which may be suited to those applications where the contribution of tributary flows is small compared to the mainstream flows of most interest. The basis of the approach is to assume that the joint distribution of the concurrent flows at two sites can be characterised by a bivariate log-Normal distribution. The magnitude of the *average* concurrent flow in one tributary ($\mu_{(y|x)}$), given a flow of magnitude *x* in the other, can be approximated by:

$$\mu_{(y|x)} = \mu_x + \rho \frac{\sigma_y}{\sigma_x}(x - \mu_x) \quad (8.7.1)$$

where μ and σ signify the mean and standard deviation of the marginal distributions, ρ is the correlation between the two variates, and *x* and *y* represent design flows at the two sites; note that all flows need to be transformed into the logarithmic domain.

The correlation ρ can be obtained from an analysis of large historic events, and the other parameter values can be found by fitting log-Normal distributions to both the mainstream and tributary streamflow data. The mean and standard deviation can be determined by fitting a line of best fit (either graphically or analytically) through the available design flood estimates in the log-Normal domain. Usually a number of design flood estimates will be available for the mainstream flows as a complete frequency curve will have been derived (Book 8, Chapter 6, Section 3), but design flood estimates for the tributary flow may be derived using the approximate procedures provided in Book 8, Chapter 6, Section 2.

Given the uncertainty of the correlation structure over the range of magnitudes of interest, it is considered that the above approximations are appropriate for those design situations in which the magnitude of the tributary flows are minor compared to the mainstream flows, and the correlation between the two flows is small or modest. It is worth noting that the magnitudes of the tributary floods are very sensitive to the strength of the correlation, and thus careful attention should be given to the nature and selection of the events used to derive the correlation value. It is also perhaps worth noting that the tributary distribution of interest is the flow value coinciding with the peak flows in the mainstream; the use of the peak flow distribution for the tributaries is an additional approximation.

A worked example illustrating some of the above concepts is presented in Book 8, Chapter 8, Section 5.

7.4. Seasonal Design Floods

7.4.1. The Need for Seasonal Estimates

In some situations Rare to Extreme design floods may be required for specific seasons within the year. Seasonal estimates may need to be investigated if it is suspected that the design factors of interest do not have an equal chance of occurring throughout the year, and that certain combinations of factors are unlikely to occur in the same season. For example, seasonal estimates may be required to assess the consequences of dam failure when the population at risk may be dependent on the time of year (e.g. summer holidays). The likelihood of snowmelt is an obvious example, though this will only need to be considered if a large proportion of the catchment lies above the snowline. Perhaps the most commonly encountered example is related to the evaluation of spillway adequacy, where the largest seasonal floods may coincide with the largest expected drawdown in the reservoir (Nathan and Bowles, 1997).

As discussed in Book 8, Chapter 3, Section 7, there are a number of conceptual and theoretical problems associated with the derivation of seasonal design rainfalls. Accordingly, seasonal design floods should only be derived if preliminary investigations indicate that the seasonal factors of interest have an appreciable impact on the required design outcome.

7.4.2. Theoretical and Practical Issues

Seasonal frequency curves can be derived using similar procedures to those required for annual frequency curves, though careful consideration needs to be given to the determination of losses and the manner in which design flood estimates are validated.

Given a set of seasonal frequency curves, care needs to be given to converting the seasonal exceedance probabilities to annual estimates. The AEP of a specific event (e.g. a dam overtopping event, Q_0) which is not conditional on the time of year can be approximated by summing the seasonal exceedance probabilities of the selected event.

As an example, if the year was divided into two seasons, then two separate events could be considered: a summer event Q_s ($Q>Q_0$) and a winter event Q_w ($Q>Q_0$). If these events are regarded as being independent (and if their exceedance probabilities are less than, say, 1 in 10 AEP), then the unconditional AEP of an event $Q>Q_0$, i.e. of Q_s or Q_w, can be computed as:

$$AEP[Q_o] = SEP_s[Q_o] + SEP_w[Q_o] \tag{8.7.2}$$

where $SEP_s[Q_0]$ and $SEP_w[Q_0]$ are respectively the summer and winter Seasonal Exceedance Probabilities (SEP) of the selected event, and $AEP[Q_0]$ represents the probability of one or more events of magnitude Q $\geq Q_0$ occurring in a single year. The computation of the AEPs from seasonal distributions for more than two seasons is analogous, and is illustrated in Figure 8.7.5. The SEPs can be simply added to give AEPs, if the seasons are defined such as to form an exhaustive set of mutually exclusive events (i.e. they are non-overlapping and cover the whole year).

It is important to note here that the event whose AEP is being analysed needs to be clearly defined in terms of a *magnitude* (e.g. Q $\geq$100 m^3/s) rather than in terms of a concept (e.g. "PMP") that does not directly relate to a magnitude. This means that the Equation (8.7.2) cannot be directly applied to PMPs for different seasons but only to rainfalls or floods of a specified magnitude occurring in different seasons.

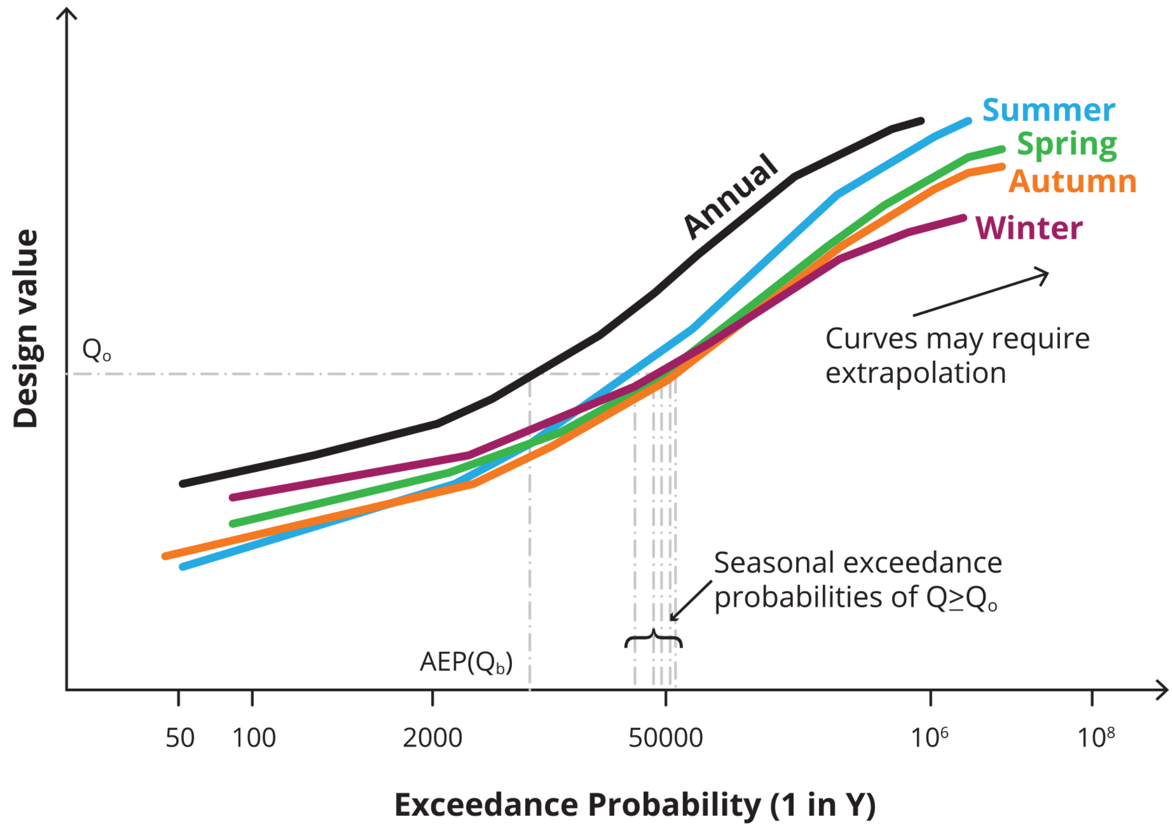


Figure 8.7.5. Schematic Diagram Illustrating the Conversion of Seasonal Exceedance Probabilities into Annual Estimates

7.5. Consideration of Snowmelt

7.5.1. Overview

Snowmelt can have an appreciable impact on the timing and magnitude of floods, though there are only a small number of areas in Australia where it needs to be considered. A large number of different methods are available for estimating snowmelt. The variety of available methods reflects the different purposes for which they have been developed, and the different data resources available for their use. While there is a considerable body of literature concerned with the simulation and quantification of snowmelt processes, there is unfortunately little guidance on estimating the snowmelt component of design floods.

The snowmelt algorithms used in the established flood event models can be broadly divided into two groups. One group of models is based on a *temperature index approach* in which temperature alone is used as a surrogate for the energy available for snowmelt. Another group of snowmelt algorithms is based on an *energy balance approach* in which energy fluxes are calculated explicitly using physically-based process equations. The results of an international comparison of snowmelt runoff models (World Meteorological Organisation, 1986) indicate that the temperature index approach has an accuracy comparable to more complex energy budget formulations. Unfortunately, however, the method does not lend itself to hourly computations (which are required for flood event estimation purposes) because it is the radiation component which is mainly responsible for the hour-to-hour variations (Rango and Martinec, 1995).

7.5.2. Selection of Snowmelt Model

The selection of an appropriate method for snowmelt estimation is subject to the following two conflicting requirements: (i) the need to model as accurately as possible the snowmelt process; and, (ii) the need to adopt a parsimonious model for use in design. The resolution of these two conflicting requirements is a common problem in engineering hydrology, and the accepted philosophy of approach is to match model complexity with the nature of the available data. While the adoption of a complex, physically-based model may appear theoretically appropriate, in practice without the data to confirm component processes such models may perform no better than over-parameterised conceptual models. Parsimony in design snowmelt estimation is particularly important because, compared to rainfall-only flood event models, there is a considerable increase in the number of factors that influence the transfer from rainfall to runoff. The salient factors depend on the nature of the transfer function used, but in general it is necessary to consider carefully the inputs related to initial depth and density of the snowpack, the nature and duration of antecedent conditions prior to the rainfall event, windspeed, and the temperature sequence.

The most appropriate method to use for the derivation of snowmelt design floods will depend largely on the nature of the available data. Practitioners are encouraged to review carefully the type of data that can be obtained for the site of interest, and to select a model that is commensurate with the complexity of the available data. A number of suitable models are commercially available (e.g. USACE (1990)), though there is little documented experience with their application to Australian conditions.

7.5.3. Application to Extreme Events

It is general international practice to maximise all salient factors contributing to rain-on-snow runoff (e.g. (USACE, 1960; NERC, 1975; Bergström, 1996)). Typically, the antecedent snowpack is set equal to the depth and areal extent corresponding to an extreme event of around 1 in 100 AEP, and the wind speed and temperature sequences are selected to maximise runoff. However, such approaches are not consistent with the probability neutral approach, and thus careful consideration needs to be given to the selection of inputs to ensure that no probability bias is introduced into the transformation between rainfall and runoff. The magnitude of snowmelt floods is particularly sensitive to initial snowpack conditions, and accordingly it is likely that a joint probability approach would be required to satisfy probability neutral requirements.

Nathan and Bowles (1997) provide one example of a study in which a joint probability approach was adopted for the derivation of snowmelt design floods. They incorporated the Snow Compaction Procedure (USBR, 1966) into a modified version of the RORB model. This procedure uses a water budget approach which is based on the concept of snow compaction and a threshold density, where the maximum potential rate of snowmelt is derived using the sub-daily application of the US Corps of Engineers degree-day snowmelt equations (USACE, 1960). A simplified approach was taken to sample antecedent snowpack conditions, but this would be better implemented within a Monte Carlo framework.

7.6. Consideration of Long Duration Events

As discussed in Book 8, Chapter 3, Section 6, there are some design situations in which it appears that the critical duration of interest may be longer than the durations for which generalised design rainfall information are available (168 hours or 7 day). The longest available design storm durations generally relate to the meteorological limits associated with single storm events, and thus longer duration design events will involve the consideration of storm sequences.

While it may be necessary to consider the likelihood of storm sequences in tropical regions, it is reasonably clear that long duration design events (one to several days) in south-eastern Australia are unlikely to be preceded by significant antecedent rainfalls (Book 8, Chapter 3, Section 6). Accordingly, the issue of storm sequences over extended periods may be implicitly solved by undertaking a joint probability analysis of inflow floods and reservoir volume, as described in Book 8, Chapter 7, Section 2.

There are other design situations (such as tailings dams) in which the design objective is to ensure that the risk of spills from the storage is negligible. These types of problems can generally be handled by undertaking mass balance calculations of all operational inflows and outflows for very long hydroclimatic sequences. It is usually not necessary to use a hydrograph model to route the rainfall excess as the surface area of the storage may be large compared to the contributing catchment area; it thus may be sufficient to allow for a freeboard in the storage that fully accommodates the volume of runoff corresponding to the required AEP of rainfall. This type of problem does not lend itself to event-based joint probability analyses but requires water balance computations over extended periods. Generally, it is desirable to generate the long hydroclimatic sequences by stochastic data generation techniques (refer to (McMahon and Mein, 1986), and an example of this approach used for spillway design is provided by Kinkela and Pearce (2014). The required security against overtopping can be achieved by using sequences of different lengths, as described for example in Grayson et al (1996), Book 8, Chapter 5, Section 2).

One of the major practical and theoretical problems with the application of stochastic data generation techniques – particularly when used in the assessment of the Very Rare to Extreme risks – is the characterisation of statistical extremes. This difficulty relates both to the tail of the distribution, as well as to the definition of the correlation between the stochastic inputs over a range of event magnitudes. These issues require careful consideration and should only be undertaken by practitioners with specialist experience.

7.7. Impact of Climate Change

Estimates of Very Rare to Extreme rainfalls (and the resulting floods) are subject to change as our understanding of the governing physical processes increases, and as more data becomes available for analysis. The estimates are also subject to change due to long-term climatic variations, such as would result from changes in atmospheric concentration of greenhouse gases.

General guidance on assessing the impact of climate change in estimates of the PMP is provided in Book 1, Chapter 6[1]. The Bureau of Meteorology completed an analysis of a storm database covering the period 1893 to 2001 and concluded that there is little evidence to support the notion that tropical cyclones (connected to major rainfall events) are penetrating further south or have become more frequent (Jakob et al., 2008). At time of writing the Bureau of Meteorology are not intending to revise PMP estimates or methodology to account for effects of climate change. Similarly, in North America standard procedures do not presently allow for climate change adjustments, (Micovic et al, 2015) however, climate model simulations and analysis of conceptual models of relevant meteorological systems would suggest that PMP estimates will increase in the future (Kunkel et al, 2013; Stratz and Hossain, 2014). This is an area of active ongoing research and it might be expected that guidance will evolve in the future as better information becomes available.

There are other factors apart from rainfall intensities that can be considered when assessing the impact of climate change. In the context of Very Rare to Extreme events, Fowler et al. (2010) considered the impacts on two additional factors on the assessment of spillway adequacy, namely catchment losses and the distribution of water levels. The change in catchment losses was assessed by use of a continuous simulation model to derive streamflow sequences corresponding to current-day and changed-climate conditions; design losses were altered to achieve a match between quantiles of 4-day flood volumes obtained from Monte Carlo analysis and the frequency analysis of the derived maxima. Similarly, an altered distribution of drawdown conditions was obtained from a model that simulated altered irrigation demands and streamflow sequences. While that study found an overall reduction in flood risk due to the downward shift in distribution of initial storage levels, it would be expected that outcomes will vary depending on the characteristics of the system being modelled.

Until better information becomes available it is considered that assessments of the impact of climate change on Very Rare to Extreme flood risks are likely to be speculative and most suited to sensitivity analyses.

7.8. References

ANCOLD (Australian National Committee on Large Dams) (2000), Selection of acceptable flood capacity for dams, March

Bergström, S., Harlin, J. and Lindström, G. (1996), Spillway design floods in Sweden: I. New guidelines, Hydrol. Sci. J. 37(5), 505-519.

Fowler, K., Hill, P.I., Jordan, P.W., Nathan, R.J., Sih, K. (2010), Application of Available Climate Science to Assess the Impact of Climate Change on Spillway Adequacy. ANCOLD 2010 Conference on Dams. Hobart.

Grayson, R.B., Argent, R.M., Nathan, R.J., McMahon, T.A. and Mein, R.G. (1996), Hydrological recipes - Estimation techniques in Australian Hydrology, Cooperative Centre for Catchment Hydrology, Monash University, pp.125.

Jakob, D., Smalley, R., Meighen, J., Taylor, B., Xuereb, K., Box, G. P. O. and Vic, M. (2008), Climate change and Probable Maximum Precipitation. In Proc. Hydrology and Water Resources Symposium, Water Down Under, pp: 109-120.

Jones, D., Wang, W. and Fawcett, R. (2009), High-quality spatial climate data-sets for Australia, Australian Meteorological and Oceanographic Journal, 58: 233-248.

[1] This section was written before the latest climate change guidance in Book 1, Chapter 6(2024). A minor change to the text has been made to reflect the change in guidance.

Jordan, P., Nathan, R., Mittiga, L. and Taylor, B. (2005),'Growth Curves and Temporal Patterns for Application to Short Duration Extreme Events'. Aust J Water Resour, 9(1), 69-80.

Kinkela, K. and Pearce, L. (2014), No ordinary dam - rainfall and flood hydrology for the Ord River dam. In: Hydrology and Water Resources Symposium 2014. Barton, ACT: Engineers Australia, pp: 342-349.

Kunkel, K., Karl, T.R., Easterling, D.R. and Redmond, K. (2013), Probable maximum precipitation and climate change. Geophysical Research Letters, 40: 1402-1408.

Laurenson, E.M. (1974), Modelling of stochastic-deterministic hydrologic systems. Water Resources Research, 10(5), 955-961.

McMahon, T.A. and Mein, R.G. (1986), River and reservoir yield. Water Resources Publications, Colorado, U.S.A., p: 368

Micovic, Z., Schaefer, M. G., and Taylor, G. H. (2015). Uncertainty analysis for Probable Maximum Precipitation estimates. J. Hydrology, 521(2015), 360-373. doi:10.1016/j.jhydrol. 2014.12.033

Natural Environment Research Council (NERC; 1975), Flood Studies Report, 5 Vols, Natural Environment Research Council, Institute of Hydrology, Wallingford, UK.

Nandakumar, N., Weinmann, P.E., Mein, R.G. and Nathan, R.J. (1997), Estimation of extreme rainfalls for Victoria using the CRC-FORGE method (for rainfall durations 24 to 72 hours). CRC Research Report 97/4, Melbourne, Cooperative Research Centre for Catchment Hydrology.

Nathan, R.J. and Bowles, D.S. (1997), A probability-neutral approach to the estimation of design snowmelt floods. Proceedings of the Hydrology and Water Resources Symposium: Wai-Whenua, November 1997, Auckland, pp: 125-130.

Nathan, R.J., Weinmann, P.E. and Minty, L. (1999), Estimation of the Annual Exceedance Probability of PMP Events in Southeast Australia, Aus. J. Water Resour., 3(1), 143-154.

Rango, A. and Martinec, J. (1995), Revisiting the degree-day method for snowmelt computations'Water Resour. Bull., 31(4), 657-669.

Saucier, R. (2000), Computer generation of statistical distributions. Army Research Laboratory ARL-TR-2168.

Scorah, M., Lang, S. and Nathan, R. (2015), Utilising AWAP gridded rainfall dataset to enhance hydrology studies. Proc. Hydrology and Water Resources Symposium, Hobart.

Stratz, S.M. and Hossain, F.M. (2014), Probable Maximum Precipitation in a Changing Climate: Implications for Dam Design. Journal of Hydrologic Engineering, pp: 1-7. zoi: 10.1061/(ASCE)HE.1943-5584.0001021.

United States Army Corps of Engineers (USACE) (1990), HEC-1 Flood hydrograph Package, User's Manual. Report CPD-1A, Version 4.0., Hydrologic Engineering Centre, U.S. Army Corps of Engineers.

United States Army Corps of Engineers (USACE) (1960), Runoff from Snowmelt, Manual EM 1110-2-1406, U.S. Government Printing Office, Washington, D.C. 75pp.

U.S. Bureau of Reclamation (USBR) (1966), Effect of Snow Compaction on Runoff from Rain on Snow, Engineering Monograph No.35, Denver, Colorado. P: 45.

World Meteorological Organisation (1986), Intercomparison of models of snowmelt runoff. Operational Hydrology Report No.23, WMO, Geneva.

Chapter 8. Worked Examples

Rory Nathan, Erwin Weinmann

Chapter Status	Final
Date last updated	14/5/2019

8.1. The Design Problem

8.1.1. General

In order to illustrate the application of some of the procedures described in the previous sections, flood frequency curves are derived for a hypothetical 439 km^2 catchment located in south-eastern Australia. It is assumed that a reservoir is located at the outlet of the catchment, and a streamflow recording gauge is located just upstream of the reservoir.

Flood frequency curves are derived for both inflows to the reservoir, as well as for reservoir outflows. As the volume of the reservoir is large compared to the volume of runoff, and it is likely that the reservoir is drawn down below full supply level, the derivation of the outflow frequency curve requires consideration of the joint probabilities of both inflows and storage volume. A tributary enters the mainstream just below the reservoir, and estimates of concurrent tributary flows are required for a range of AEPs in order to help determine the component of incremental damages that could be attributed to dam failure.

8.1.2. Approach Adopted and Intent

The following worked examples illustrate application of the various procedures to different design situations. The one hypothetical problem is used for convenience throughout. While somewhat didactic, the examples are not meant to provide detailed tutorials on the implementation of best practice, and thus some relevant experience will be required to fully understand the context and nature of the procedures. The examples illustrate application of the procedures using (largely) “real-world” data.

8.1.3. Nature of Available Data

The examples are in part based on data derived for an actual catchment, though some changes were introduced to better illustrate application of the range of procedures considered.

A summary of the data available for the catchment is as follows:

- a set of calibration results obtained by fitting a flood event model to several large observed floods;
- a series of annual instantaneous maximum flood peaks at the streamflow gauge;
- a synthetic monthly time series of reservoir volume obtained from a system simulation model;
- design rainfalls between 1 in 50 and 1 in 2000 AEP from Book 2 procedures; and,

- GSAM estimates of the PMP for a range of standard durations obtained from the Bureau of Meteorology.

8.1.4. Note on Accuracy of Final Results

It should be noted that the number of significant figures used to present the results of the worked examples are generally higher than can be justified. In most cases the accuracy of the final results is probably limited to only two significant figures, but greater accuracy is adopted merely to facilitate checking of the calculations.

8.2. Derivation of Rainfall Frequency Curves

Rainfall frequency curves are derived for three durations (12, 24 and 48 hours) for rainfall event classes between Rare and Extreme.

8.2.1. Estimates of Rare to Very Rare Rainfalls

Estimates of point rainfall depths for Rare rainfalls are obtained from the procedures provided in Book 2, as made available online at www.bom.gov.au [http://www.bom.gov.au]. The design rainfalls for the selected durations are shown in bold typeface in the first two rows of Table 8.8.1.

For Very Rare rainfalls, point estimates for 24 and 48 hour durations are also obtained from Book 2 procedures, as made available online at www.bom.gov.au [http://www.bom.gov.au]. Estimates of Very Rare rainfalls for the 12 hour event are obtained from the growth factors provided in Table 8.3.2, multiplied by the 1 in 100 AEP point rainfall depth. For example, the 1 in 2000 AEP 12 hour depth is simply estimated as $111.4 \times 1.698 = 189.2mm$.

To obtain areal design rainfalls, the point rainfall estimates are multiplied by the Areal Reduction Factors (ARFs) provided in Book 2, Chapter 4 For the long-duration rainfalls the ARF for this location is estimated as a function of rainfall duration (D, hrs), catchment area (A, km^2), and 1 in Y AEP as follows:

$$ARF = \min\{1.00, [1.00 - 0.4(A^{0.14} - 0.7\log_{10}D)D^{-0.48} + 0.0002(A)^{0.4}D^{0.41}(0.3 + \log_{10}(Y))]\} \quad (8.8.1)$$

where the area of the catchment is 439 km^2. For the short duration rainfalls, the appropriate Areal Reduction Factors is independent of AEP and can be estimated from:

$$ARF = \min\{1.00, [1.00 - 0.1(A^{0.14} - 0.879) - 0.029(A^{0.233})(1.255 - \log_{10}(D))]\} \quad (8.8.2)$$

The areal rainfalls obtained by applying the above equations are show in the last three columns of Table 8.8.1.

Table 8.8.1. Calculation of Areal Design Rainfalls for Rare to Very Rare Events

AEP (1 in Y)	Point Rainfall (mm)			Areal Reduction factors			Areal Rainfall (mm)		
	12 hour	24 hour	48 hour	12 hour	24 hour	48 hour	12 hour	24 hour	48 hour
50	99.4	135.8	181.9	0.870	0.925	0.940	86.5	125.6	170.9
100	111.4	153.5	205.8	0.864	0.923	0.938	96.3	141.7	193.0

AEP (1 in Y)	Point Rainfall (mm)			Areal Reduction factors			Areal Rainfall (mm)		
200	127.0	172.4	231.2	0.858	0.922	0.936	109.0	158.9	216.5
500	149.7	199.7	268.2	0.850	0.919	0.934	127.3	183.6	250.5
1000	168.6	222.1	298.7	0.844	0.918	0.932	142.3	203.8	278.5
2000	189.2	246.4	332.0	0.838	0.916	0.931	158.5	225.7	308.9

8.2.2. Estimates of Extreme Rainfalls

Estimates of Extreme rainfalls (i.e. rainfalls between an AEP of 1 in 2000 and the AEP of the PMP) are derived using the procedure presented in Book 8, Chapter 3, Section 5. The areal rainfall estimates listed in Table 8.8.1 are extrapolated between 1 in 2000 AEP and the AEP of the PMP using the procedure developed by Siriwardena and Weinmann (Book 8, Chapter 3, Section 5). Table 8.8.2 lists the input design rainfalls (in the 2nd and 3rd rows), where, with reference to Equation (8.3.1) to Equation (8.3.6), the values used in the procedure are as follows:

Lower end point of linear segment:

P_{Y1}- 1 in 1000 AEP areal rainfall depth

Y_1- 1000

Starting point of interpolation:

P_{Y2}- 1 in 2000 AEP areal rainfall depth

Y_2- 2000

Upper end point of interpolation:

P_{PMP}- PMP depth

Y_{PMP}- 2.28×10^6

Z_d , S_{gc} and S_{gap} are calculated from Equation (8.3.3), Equation (8.3.5) and Equation (8.3.6), respectively, and their values are shown in the upper panel of Table 8.8.2. Parameter g_Y varies with AEP and R_Y varies with both AEP and duration, and their values (using Equation (8.3.4) and Equation (8.3.2)) are shown in the lower panel of Figure 8.8.5. Design rainfalls for intermediate AEPs are calculated using Equation (8.3.1), and these are shown in the last three columns of the lower panel of Table 8.8.2.

Table 8.8.2. Parameters Calculated of Areal Design Rainfalls for Very Rare to Extreme Events

Parameter	12 hour	24 hour	48 hour
z_d	3.057	3.057	3.057
S_{gap}	0.071	0.062	0.060
S_{gc}	0.075	0.062	0.055

Table 8.8.3. Calculation of Areal Design Rainfalls for Very Rare to Extreme Events

AEP (1 in Y)	Z_{std}	g_Y	R_Y			Areal Rainfall (mm)		
			12 hour	24 hour	48 hour	12 hour	24 hour	48 hour
1000	3.090					140.3	190.4	268.0
2000	3.291					157.5	210.7	296.7
10000	3.719	0.229	1.050	1.044	1.041	203.1	266.1	375.2
50000	4.108	0.457	1.102	1.087	1.081	263.8	335.7	470.0
100000	4.265	0.556	1.125	1.106	1.097	296.0	370.9	516.4
500000	4.611	0.784	1.179	1.149	1.135	388.7	467.3	638.2
2280000	4.917					510.0	630.0	810.0

To check that the derived rainfall frequency curves are well behaved, it is worth plotting the results on Normal probability paper. Rainfall may be displayed on either arithmetic or logarithmic scales, and if a suitable probability scale is not available then probabilities can be expressed as standard normal variate (i.e. the "*z score*", the inverse of the standard normal cumulative distribution) and then plotted on an arithmetic axis. The *z scores* for the rarer AEPs are shown in the 2nd column of Table 8.8.2. The frequency plot of the derived rainfall frequency curves are shown in Figure 8.8.1. Alternatively, the results could be plotted on log-log scales: while this would not as clearly illustrate the behaviour of extremes, it would be sufficient to check for inconsistencies.

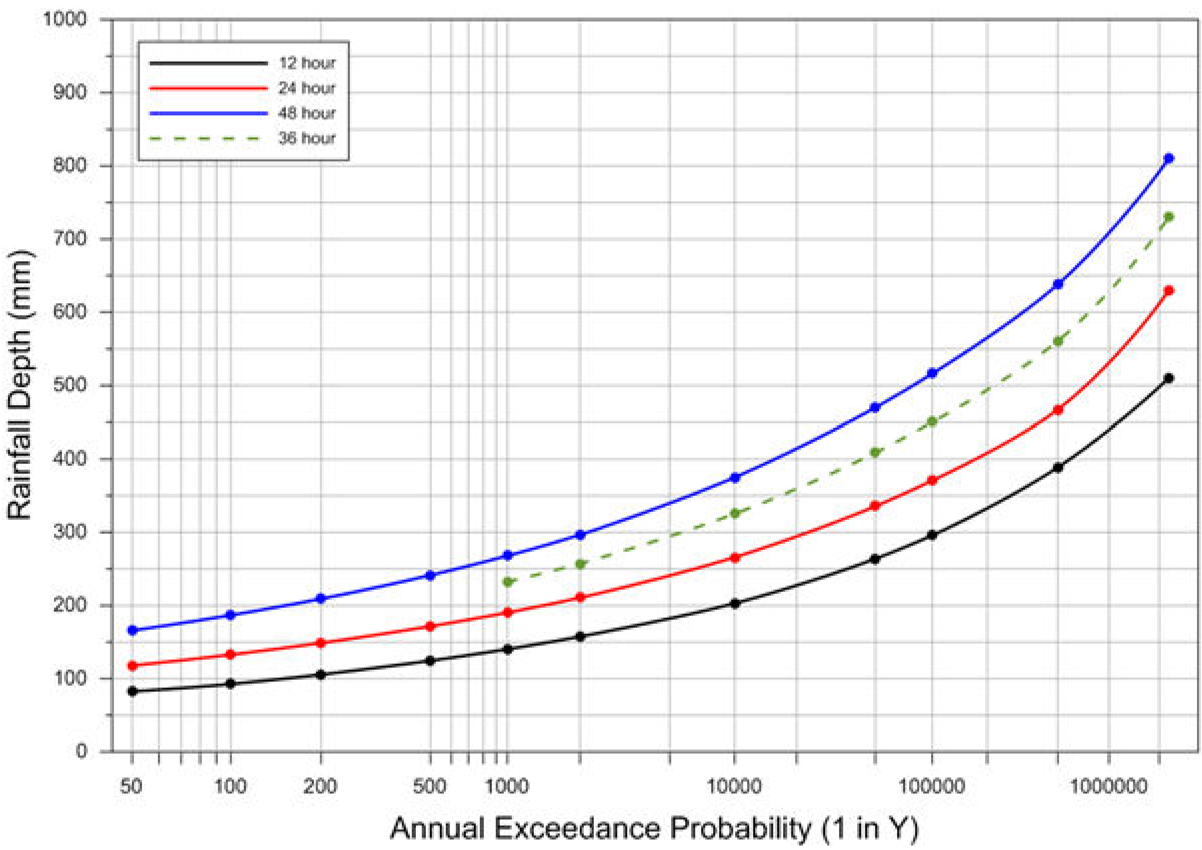


Figure 8.8.1. Example Rainfall Frequency Curves

8.2.3. Interpolation of Rainfall Depths for Intermediate Durations

The results presented in Table 8.8.2 only relate to the standard durations for which rainfall estimates are directly available. It is sometimes desirable to derive frequency curves for non-standard durations, and this can be done for Very Rare to Extreme events by interpolating in the logarithmic domain between rainfall depth and duration for each required AEP.

To illustrate the derivation of a 36 hour rainfall frequency curve, the rainfall depth for an event of 1 in 1000 AEP is calculated from logarithmic interpolation as:

$$\log(36 \text{ hr } 1 \text{ in } 100 \text{ AEP depth}) = \frac{\log(36) - \log(24)}{\log(48) - \log(36)} \times (\log(268.0) - \log(190.4)) + \log(190.4) = 2.367 \quad (8.8.3)$$

where the values for the 24 and 48 hour rainfall depths are obtained from Table 8.8.2. The resulting rainfall depth is computed as $10^{2.367}$ = 232.5 mm. The above steps are repeated for the 1 in 2000 AEP and PMP depths, and the intermediate AEPs are then obtained from the interpolation procedure as described in Book 8, Chapter 8, Section 2. The resulting 36 hour rainfall frequency curve is shown as a dashed line in Figure 8.8.1.

8.3. Derivation of Flood Frequency Curve

The rainfall frequency curves obtained in Book 8, Chapter 8, Section 2 can be used to derive a set of flood frequency curves. It is assumed that the routing parameters of the flood event model have been calibrated to historic flood events, and that the design losses have been adopted after reconciliation with flood frequency curves, as illustrated in Book 8, Chapter 6, Section 3. To illustrate the points below, the event model is run within a variable Monte Carlo framework, as shown in Figure 8.8.2(a). Temporal patterns are selected randomly from a fixed set of ensemble patterns (or from a conditional set based on season, if relevant), and seasonality and losses are sampled from non-parametric distributions, as described in Book 4. Seasonality is most easily accommodated by sampling from a distribution of the relative likelihood that the annual maximum event occurs in the different seasons, and this is expected to vary with AEP (e.g. in southern Australia it is more likely that the annual maximum occurs in winter for frequent events and in summer for more extreme events). Once the season has been selected, then stochastic values of losses (and reservoir drawdown, if relevant) are then sampled from their corresponding seasonal distributions. Inputs not stochastically sampled are fixed using representative values from the central tendency of their distribution. To minimise the number of simulations, a stratified sampling scheme is used in which the rainfall probability domain is divided into 20 intervals, and the expected probabilities of selected flood magnitudes are derived using the Total Probability Theorem (as described in Book 4).

Figure 8.8.2 (b) illustrates the impact of successively introducing variability into the flood estimates. The black curve represents the frequency curve obtained using a simple design event approach in which all inputs (except for rainfall) are held at fixed values. The curve represents the envelope of all durations trialled. When losses are allowed to vary stochastically with season, it is seen from the light blue curve in Figure 8.8.2(b) that the flood peaks beyond 1 in 50 AEP are lower; this result arises as the seasonal distribution of losses is slightly out of phase with that of rainfalls. Next, when an ensemble of temporal patterns is stochastically sampled, it is seen (darker blue curve) that the flood peaks are higher than if a fixed temporal distribution of rainfall is adopted. This reflects the highly non-linear runoff response to variability in temporal patterns. When all the inputs are allowed to vary stochastically it is seen (red curve) that the final result is slightly higher than if deterministic assumptions were adopted.

It should be stressed that the results shown in Figure 8.8.2(b) simply illustrate the manner in which the probability neutral assumptions of flood producing factors can be examined and combined. The magnitudes of differences between deterministic and joint probability approaches are very site-specific, and depend largely on the sensitivity of the system to the dominant hydrometeorological inputs.

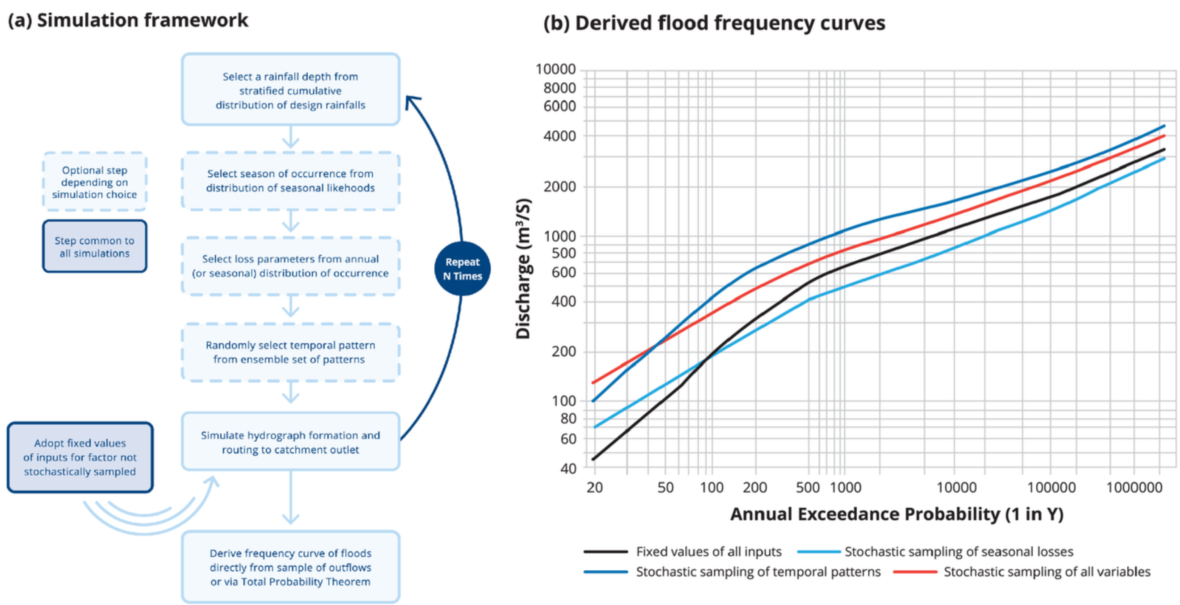


Figure 8.8.2. (a) Simulation Framework used to Generate Floods for Selected Stochastic Inputs and (b) Resulting Flood Frequency Curves

8.4. Joint Probability Analysis of Initial Reservoir Level

The examples provided here illustrate analytic and numeric schemes to derive a frequency curve of outflows from a reservoir under conditions of variable drawdown. It is assumed that the following information has been derived for the reservoir:

(i) inflow frequency curve;

(ii) the relationship between inflows and outflows, for different initial reservoir levels; and,

(iii) the frequency distribution of storage volume.

The analytical approach is based on the method developed by Laurenson (1974), and the numerical approach is based on Monte Carlo simulation.

8.4.1. System Characteristics

The inflow curve of interest is that which yields the maximum outflow peak from the reservoir. In many cases the critical duration of interest varies with reservoir drawdown and AEP, and thus it may be necessary to undertake the analysis for several different durations and to construct an outflow frequency curve that envelopes the results. In most design situations, however, it is sufficient to select the duration that is most relevant to the design objective (say, the determination of the AEP of the overtopping flood) at a typical drawdown.

If a single duration is adopted it is recommended that a sensitivity analysis be undertaken to determine the impact of rainfall duration on the results.

For this example, the inflow frequency curve is that derived in Book 8, Chapter 8, Section 3 based on the stochastic sampling of seasonal losses and temporal patterns (red curve, Figure 8.8.2). The relationship between inflows, outflows, and initial reservoir level (I-O-S relationship) is shown in Figure 8.8.3. The frequency distribution of storage volume is assumed to have been derived from the simulation results of long-term reservoir behaviour, and is shown in Figure 8.8.4.

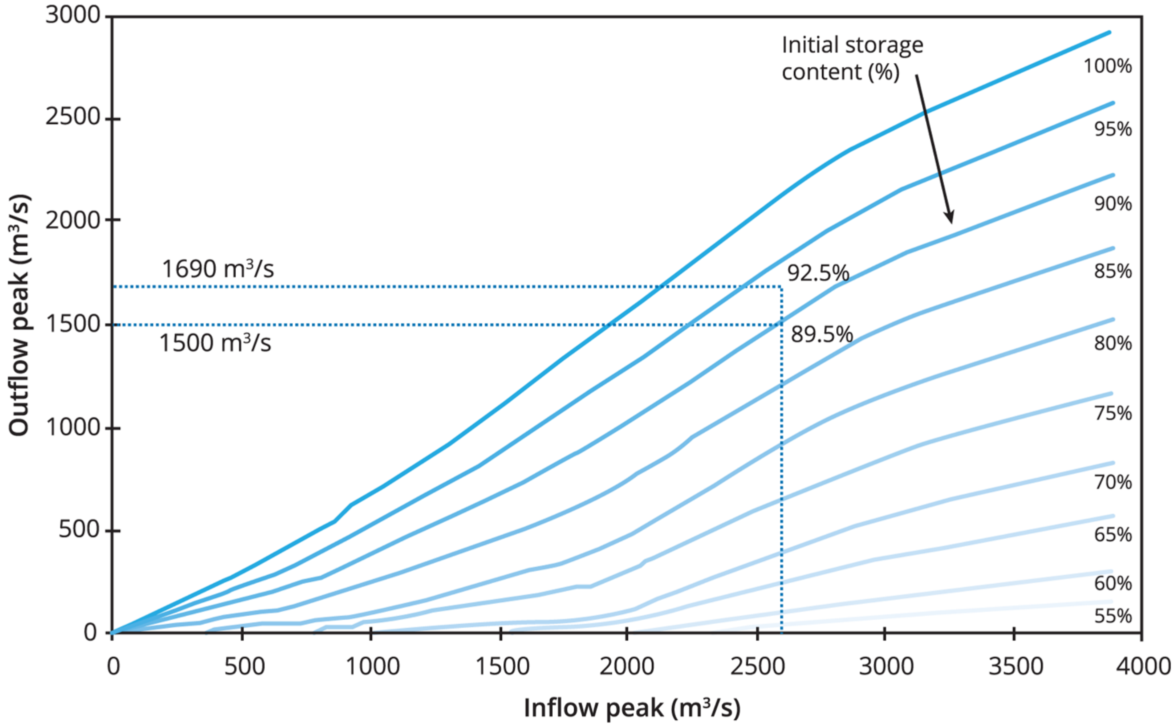


Figure 8.8.3. Inflow-Outflow-Storage Volume Relationship

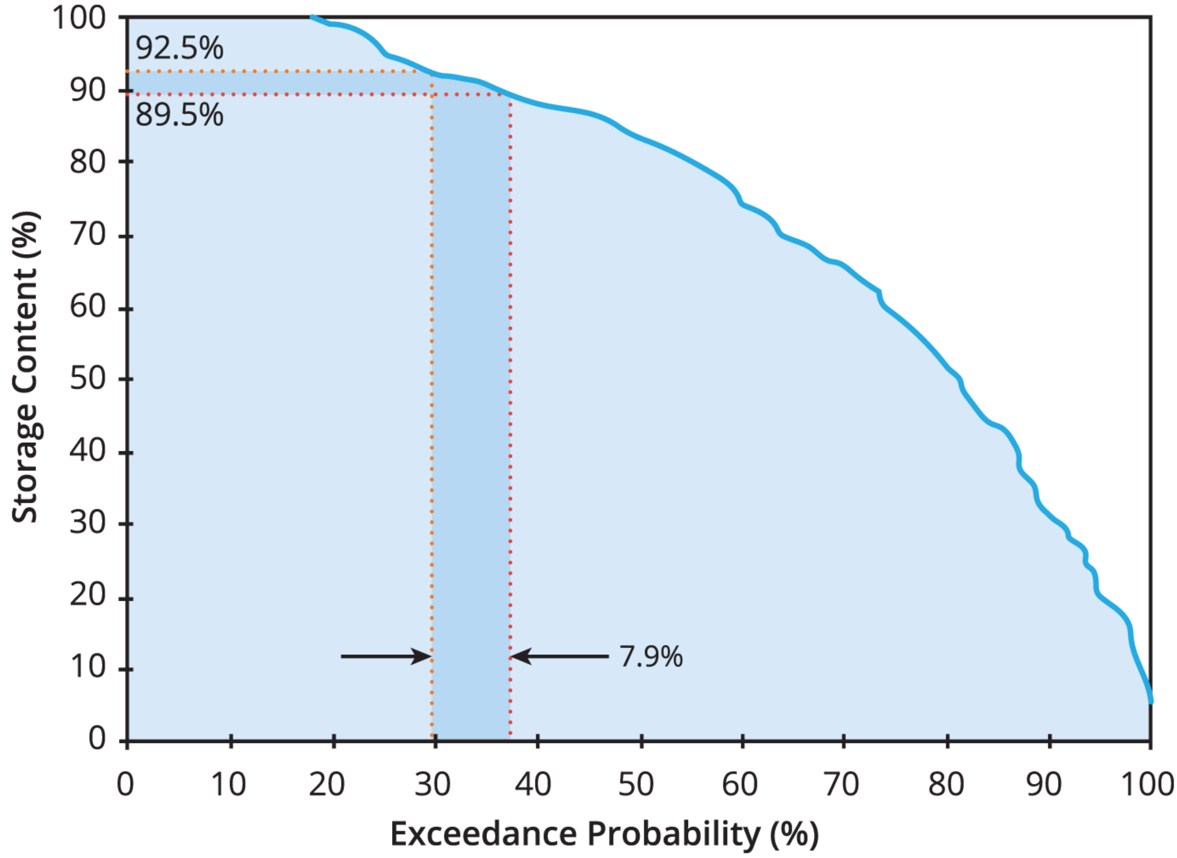


Figure 8.8.4. Probability Distribution of Initial Storage Volume

8.4.2. Laurenson's Analytical Solution

To apply the technique, the frequency distribution of inflows is divided into 8 class intervals, as indicated in the top row of Figure 8.8.5. In practice a larger number of intervals would be preferred, but a small number has been adopted in this example for clarity. The probabilities of occurrence within each class interval are provided in the second row of Figure 8.8.5; these are calculated simply as the difference between the exceedance probabilities corresponding to the class intervals.

The whole range of peak outflows is divided into 20 class intervals, as indicated in the first column of Figure 8.8.5. The elements of the table are then evaluated for each inflow class interval. The numerical values represent the conditional probability (in percentage points) for which an inflow peak in the given class interval produces an outflow peak falling in the selected outflow class interval. The sum of the values in each inflow class interval (i.e. the sum of each column) is 100. It is worth noting that the values provided in Figure 8.8.5 have been computed using specialist software; the numerical accuracy used in the calculations are greater than that which could be achieved using graphical techniques, but the procedural steps are identical.

The derivation of a particular element is described as follows. Consider the inflow class interval of 2200 m^3/s to 3000 m^3/s and represent it by its mid-point of 2600 m^3/s. Consider the outflow class interval 1500 m^3/s to 1690 m^3/s. From Figure 8.8.3, the initial storage volume which produce peak outflows of 1500 m^3/s to 1690 m^3/s from a peak inflow of 2600 m^3/s are respectively 89.5% and 92.5% of full storage. From Figure 8.8.4 the probabilities that the actual storage volume will be greater than the above are respectively 29.5% and 37.4%, so the probability that the initial volume will be between 89.5% and 92.5% of full storage volume is (37.4-29.5)= 7.9%. Thus the probability that a peak inflow between 2200 m^3/s to 3000 m^3/s would lead to an outflow between 1500 m^3/s to 1690 m^3/s is 7.9%. This value is inserted in the appropriate position in the table and other values are computed in a similar manner.

The distribution of peak outflows is evaluated by multiplying each element of the table by the corresponding probability of occurrence of the inflow interval, and the resulting products are summed horizontally (and divided by 100) to give the values in the second last column of Figure 8.8.5. For example, the outflow element corresponding to the outflow range of 1500 m^3/s to 1690 m^3/s is obtained from the following calculation:

$$\frac{(18.8973 \text{ x } 0.00234 + 7.8759 \text{ x } 0.00068 + 4.1623 \text{ x } 0.00013)}{100} = 0.000501\% \qquad (8.8.4)$$

Finally, the values are added for all outflow intervals which exceed the outflow magnitude of interest to give the probabilities of exceedance, as listed in the last column. For this example, the AEP of Q=1500 m^3/s is found to be 0.000757% or about 1 in 130 000.

The calculated outflow points from Figure 8.8.5 are plotted and a curve fitted to define the frequency distribution of peak outflows, as shown in Figure 8.8.6. Note that if a sufficient number of intervals are used to discretise the inflow and outflow frequency curves then it is probably not necessary to fit a curve as the points generally follow a smooth curve in the log-Normal probability domain.

For comparison purposes, outflows are also derived for an initial storage volume fixed at the median level of drawdown, which is 81.3% of the full supply storage (Figure 8.8.6). The corresponding outflow curve is plotted in Figure 8.8.3, and it is seen that this simplistic approach yields an outflow frequency curve that is significantly lower than that obtained using the more accurate joint probability approach.

Inflow Class Interval (m^3/s)	<500	700	1000	1300	1700	2200	3000	4000	Outflow Class Probability (%)	Outflow Cumulative Probability (%)
Inflow Class Probability (%)	99.51341	0.28355	0.15983	0.03106	0.00896	0.00234	0.00068	0.00013		
Outflow Class Intervals (m^3/s)										
0										99.999969
	100	78.4716	69.3027	54.243	47.9156	41.6945	34.404	27.1748	99.86908	
350										0.130893
		21.5284	2.3793	2.2986	1.0037	0.8095	0.9611	0.5138	0.065678	
380										0.065216
			2.9773	3.4824	1.6977	1.3187	1.0253	0.7559	0.006031	
420										0.059185
			2.621	4.5351	1.8148	1.4459	0.7783	1.0328	0.005801	
480										0.053384
			3.1509	4.369	2.0795	1.5345	1.3152	1.4447	0.006626	
530										0.046758
			19.5688	4.1312	4.5458	2.0523	1.4027	1.982	0.033027	
600										0.01373
				2.4953	4.0726	2.2	0.7413	1.764	0.001199	
670										0.012531
				2.9152	5.4847	1.8765	1.0761	1.5613	0.00145	
750										0.011081
				21.5302	4.6796	4.3988	2.0627	1.2286	0.007226	
850										0.003855
					2.8491	4.9704	1.8965	2.0188	0.000387	
950										0.003468
					3.269	5.67	3.05	0.9686	0.000448	
1060										0.003021
					20.5878	5.0081	3.2303	1.4521	0.001986	
1190										0.001035
						3.5337	4.9378	2.3873	0.00119	
1340										0.000916
						4.5897	6.8184	3.5258	0.000158	
1500										0.000757
						18.8973	7.8759	4.1623	0.000501	
1690										0.000256
							4.5785	6.3883	0.00039	
1890										0.000217
							23.8459	9.147	0.000174	
2120										0.000043
								7.3367	0.00001	
2380										0.000033
								5.553	0.000007	
2670										0.000026
								19.6024	0.000026	
3000										0

Figure 8.8.5. Transition Probabilities between Reservoir Inflow and Outflow Classes

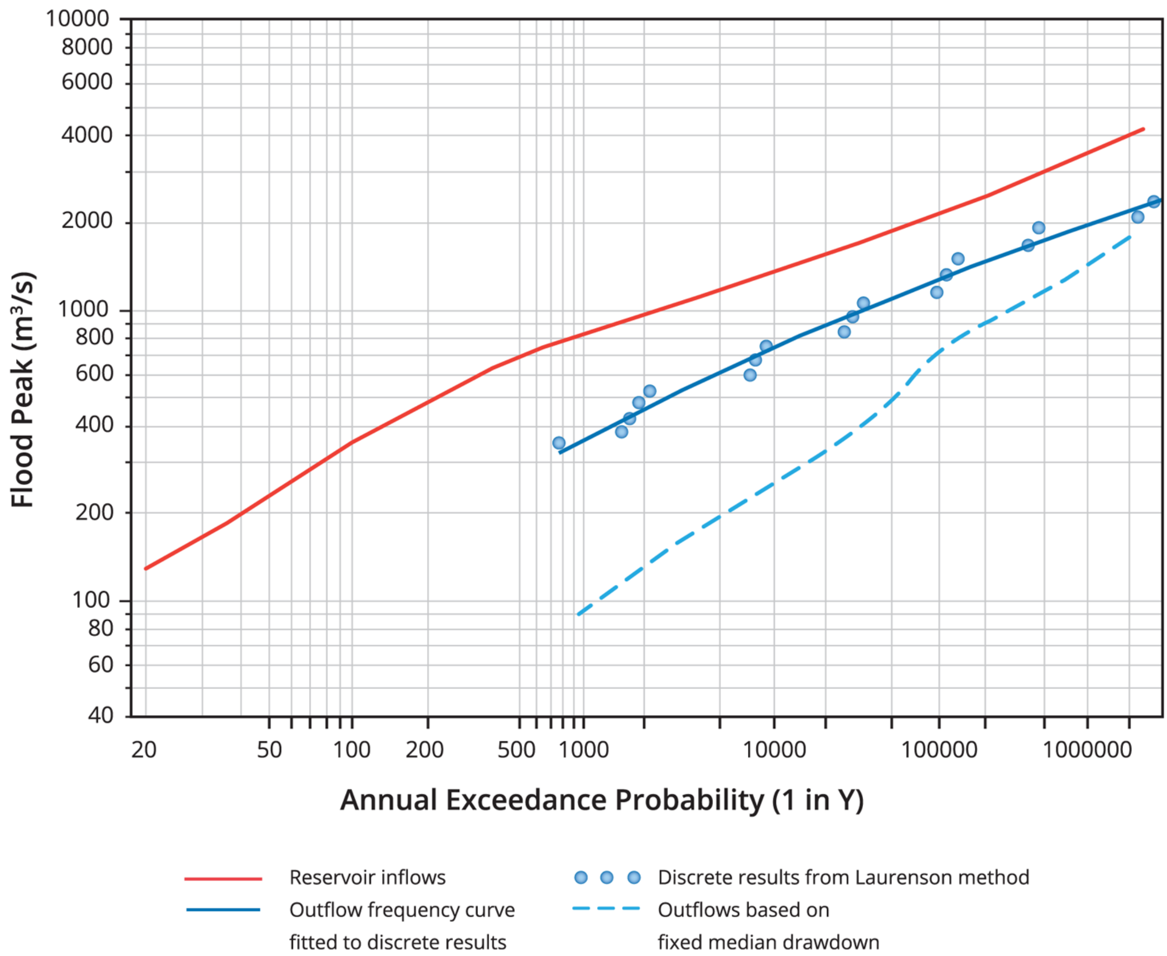


Figure 8.8.6. Outflow Frequency Curves Obtained using Joint Probability Analysis and a Median Level of Drawdown

8.4.3. Monte Carlo Solution

A Monte Carlo scheme can be easily extended to include the consideration of joint probabilities in reservoir drawdown. A framework suited to this is shown in Figure 8.8.7. This framework is in essence identical to the approach used to derive the frequency curves shown in Figure 8.8.2, the only additional step is the (stochastic) sampling of initial reservoir level and subsequent (deterministic) routing of the inflow hydrograph through the storage. The initial reservoir level is best sampled in a non-parametric fashion from the cumulative distribution of drawdown (e.g. the distribution as shown in Figure 8.8.4) using the approach as described in Book 4.

If the distribution of initial reservoir levels is found to vary with event severity (as illustrated by the insert in Figure 8.7.3) then the same framework as shown in Figure 8.8.7 can still be used, the only difference being that a different drawdown distribution is selected depending on the magnitude of the inflow flood (or causative rainfall). The drawdown distribution selected can also vary with season to account for marked differences in seasonal water levels, and again the framework as shown in Figure 8.8.7 is directly applicable if distributions are selected according to season.

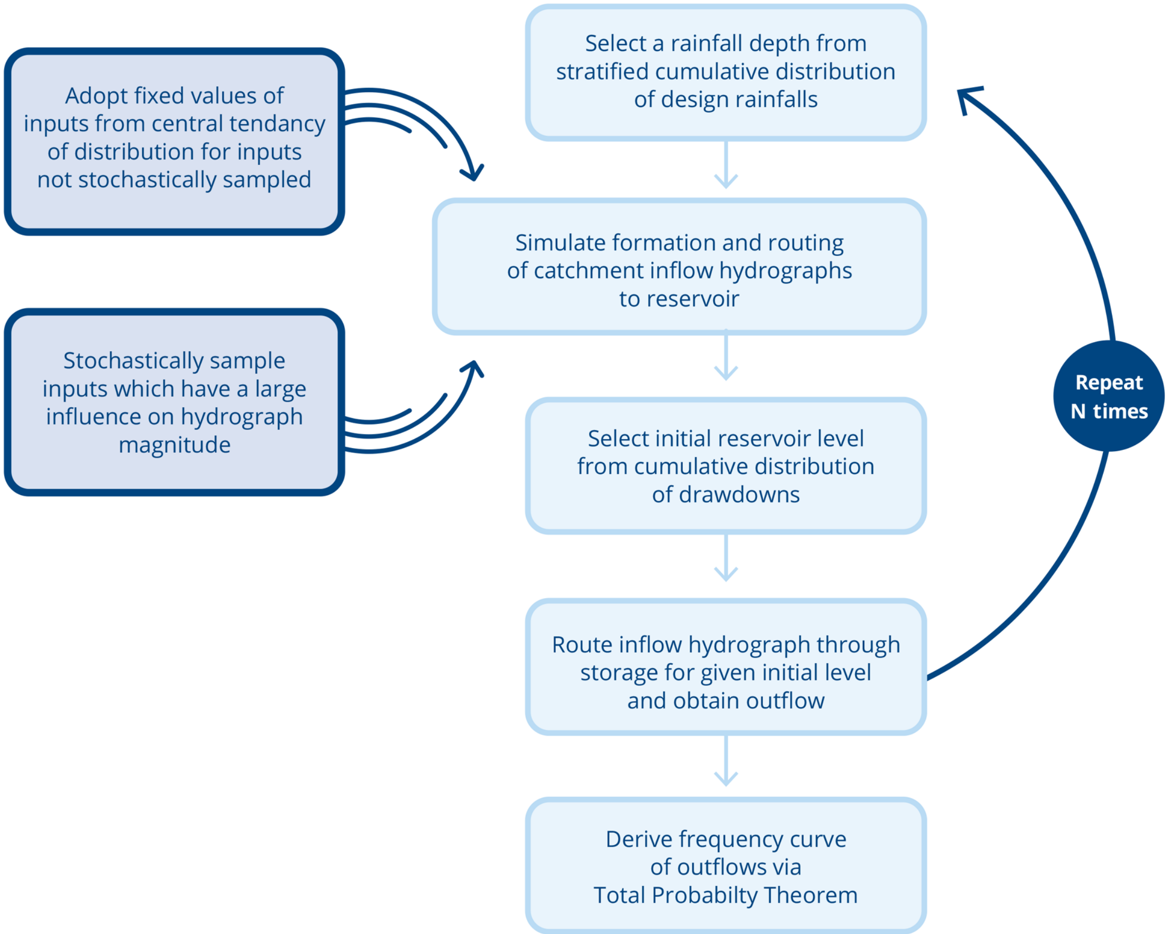


Figure 8.8.7. Simulation Framework to Derive Outflow Frequency Curve Based on Variable Initial Starting Level in Reservoir

8.5. Estimation of Concurrent Flows

For this example it is assumed that it is required to derive the concurrent tributary inflows originating from a 60 km^2 catchment located just downstream of the reservoir. A township is located below the confluence and the concurrent tributary inflows are required to help determine the component of incremental damages that could be attributed to dam failure.

8.5.1. Basic Flood Data

The design floods for the point on the mainstream are shown for a range of AEPs in Figure 8.8.8 (columns 1 and 4), where flood estimates for the mainstream were derived as described in Book 8, Chapter 8, Section 3. Floods flows in the tributary are assumed to be minor compared to that in the mainstream, and it may be assumed that preliminary design flood estimates were derived using regional procedures, as discussed in Book 8, Chapter 6, Section 2. Tributary flood estimates were only obtained for AEPs of 1 in 50, 1 in 100 and 1 in 10^7, and these are shown in column 6, Figure 8.8.8.

Annual Exceedance Probability			Design Quantiles				Bivariate log-Normal Estimates				Concurrent Tributary flood		
1 in Y	%	Standardised Norm. Variate	Mainstream Flood		Tributary		Mainstream Flood		Tributary		Flood		AEP
(1)	(2)	(3)	(m^3/s) (4)	Log (m^3/s) (5)	(m^3/s) (6)	Log (m^3/s) (7)	Log (m^3/s) (8)	(m^3/s) (9)	Log (m^3/s) (10)	(m^3/s) (11)	Log (m^3/s) (12)	(m^3/s) (13)	(1 in Y) (14)
50	2	2.054	344	2.537	105	2.020	2.540	347	2.024	106	1.638	43	7
100	1	2.326	443	2.646	135	2.130	2.639	436	2.127	134	1.689	49	8
500	0.2	2.878	703	2.847			2.839	690	2.334	216	1.793	62	13
1000	0.1	3.090	826	2.917			2.915	823	2.414	259	1.833	68	16
2000	0.05	3.290	969	2.986			2.988	973	2.489	308	1.870	74	20
10000	0.01	3.719	1351	3.131			3.143	1390	2.651	447	1.951	89	32
50000	0.002	4.101	1860	3.270			3.291	1910	2.794	622	2.023	105	50
500000	0.0002	4.611	2910	3.464			3.466	2923	2.986	968	2.119	131	95
2280000	0.0004	4.917	3898	3.591			3.577	3772	3.101	1262	2.176	150	143
10000000	0.00001	5.199			1610	3.207	3.679	4771	3.207	1611	2.229	170	214
		Average (Intercept) =		1.797		1.251					ρ =	0.5	
		Std Dev (slope) =		0.362		0.376							

Figure 8.8.8. Calculation of Concurrent Tributary Flows

8.5.2. Fitting of log-Normal Distribution

In order to calculate the parameters of the marginal log-Normal distributions, the flow data are first converted into the logarithmic domain (columns 5 and 7), and the AEPs are linearised by calculating the corresponding standard normal variates (column 3). The latter can be obtained from normal probability tables, or else using the in-built functions available in spreadsheet software (note that the standardised normal variate obtained using some spreadsheet software may be incorrect at very low probabilities and correct values should be checked against published information (e.g. Abramowitz and Stegun (1964)).

The parameters of the log-Normal distribution can then most easily be calculated by simply fitting a linear regression line through the transformed data (i.e. columns 3 and 5, and columns 3 and 7). The intercept of the fitted line is equivalent to the mean of the distribution (as the standardised variate of the mean of a log-Normal distribution is zero), and the slope is equivalent to the standard deviation. The fitted parameters are listed below columns 5 and 7, and may be obtained either graphically, or by using standard spreadsheet functions. The design flood estimates and the fitted log-Normal distributions are shown in Figure 8.8.9. The log-Normal estimate (x) may be calculated from the relevant sample mean (m), standard deviation (s), and standardised variate (z) as follows:

$$x = m + sz \tag{8.8.5}$$

For example, to calculate the 1 in 100 AEP design flood in the mainstream:

$$\begin{aligned} x &= 1.797 + 0.362 \times 2.326 \\ &= 2.639 \log\,(\mathrm{m^3/s}) \\ &\approx 440\ \mathrm{m^3/s} \end{aligned} \tag{8.8.6}$$

The computed design flood estimates from the fitted distribution are shown in columns 8 and 10; these are then back-transformed into the arithmetic domain, as shown in columns 9 and 11 of Figure 8.8.8.

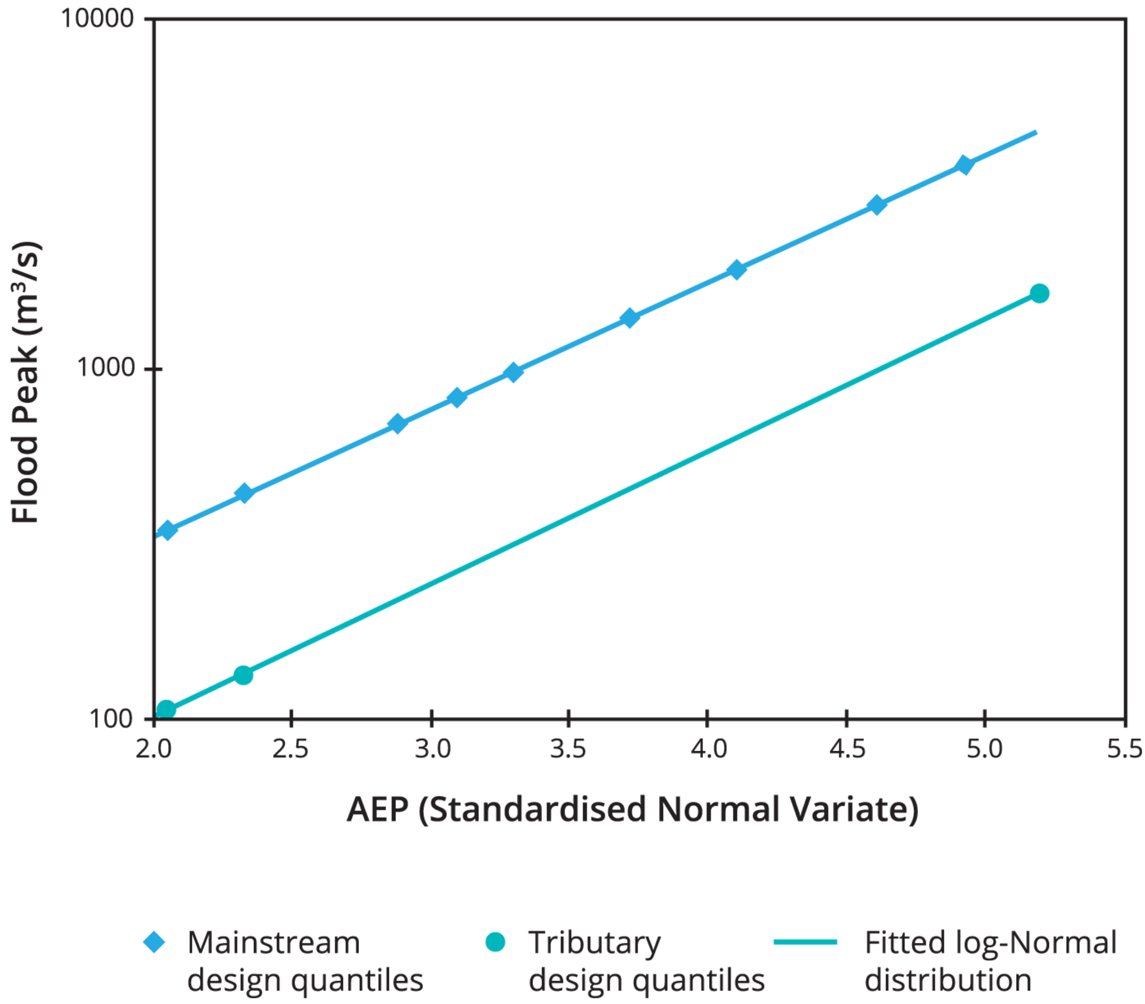


Figure 8.8.9. Fitted log-Normal Flood Frequency Curves for Mainstream and Tributary Design Flows

8.5.3. Estimation of Concurrent Tributary Flows

Computation of the average concurrent flow in the tributary ($m_{(y|x)}$) for varying design floods in the mainstream (x) are determined from Equation (8.7.1) in (Book 8, Chapter 7, Section 3), as follows:

$$m_{(y|x)} = 1.251 + 0.5\frac{0.376}{0.362}(x - 1.797) \tag{8.8.7}$$

where 0.5 represents the correlation between the log-transformed flows calculated for the largest floods on record. The average concurrent flow in the tributary corresponding to a 1 in 50000 AEP event in the mainstream is thus calculated by:

$$
\begin{aligned}
m_{(y|x)} &= 1.251 + 0.5\frac{0.376}{0.362}(3.465 - 1.797) \\
&= 2.117 \log \mathrm{m}^3/\mathrm{s} \\
&= 131\ \mathrm{m}^3/\mathrm{s}
\end{aligned}
\tag{8.8.8}
$$

The computed figures for all AEPs are shown in columns 12, and the back-transformed values are shown in columns 13. It is of interest to calculate the AEPs of the concurrent tributary flows, and these may be calculated by first calculating the standard normal variate using:

$$z = \frac{(x - m)}{s} \tag{8.8.9}$$

For example, to calculate the AEP of the 74 m^3/s design flood estimate in the tributary:

$$
\begin{aligned}
z &= \frac{(\log(74) - 1.251)}{0.376} \\
&= 1.644
\end{aligned}
\tag{8.8.10}
$$

The corresponding standard normal cumulative distribution for this value of *z* is 0.95, which corresponds to1 in 20 AEP. Values for the other estimates are shown in column 14.

8.6. References

Abramowitz, M. and Stegun, I.A. (1964), Handbook of Mathematical Functions with Formulas, Graphs, and Mathematical Tables, Applied mathematics series, Dover Publications.

Laurenson, E.M. (1974), Modelling of stochastic-deterministic hydrologic systems. Water Resources Research, 10(5), 955-961.

BOOK 9

Runoff in Urban Areas

Runoff in Urban Areas

Table of Contents

1. Introduction 1
1.1. Introduction 1
1.1.1. Urban Stormwater Runoff 1
1.1.2. Stormwater Management Infrastructure 2
1.1.3. Modelling 2
1.1.4. Structure and Purpose of this Book 3
1.1.5. The Future 3
1.2. References 4
2. Aspects of Urban Hydrology 5
2.1. The Urban Hydrologic Cycle 5
2.2. Human Impact on the Hydrologic Cycle 6
2.2.1. Urban Water Balance 6
2.2.2. Lessons from a Detailed Water Balance Study at Curtin, ACT 12
2.2.3. Implications of the Urban Water Balance: Stormwater as a Resource 13
2.2.4. Comparison of Rural and Urban Water Balances 16
2.3. Aspects of Urban Stormwater Management Systems 16
2.3.1. Impervious Areas 16
2.3.2. Conveyance 19
2.3.3. Receiving Environments 20
2.3.4. More Complex than Rural 20
2.3.5. Combined and Separate Systems 21
2.4. References 21
3. Philosophy of Urban Stormwater Management 26
3.1. Introduction 26
3.2. The Journey from 1987 to 2016 26
3.3. Evolving Opportunities and Challenges 28
3.4. Urban Flooding 34
3.4.1. Overland Flooding 35
3.4.2. Fluvial Flooding (River and Creek Flooding) 37
3.4.3. The Overland and Fluvial Interface 37
3.5. Conveyance Systems 38
3.5.1. Capacity to Manage Flooding 39
3.5.2. Alignment and Configuration 40
3.5.3. Analysis 42
3.6. Stormwater Volume Management 42
3.6.1. Key Considerations 42
3.6.2. Selecting a Strategy 45
3.7. Stormwater Offsets 46
3.8. Acknowledgements 48
3.9. References 48
4. Stormwater Volume Management 52
4.1. Introduction 52
4.2. Volume Management Objectives 55
4.3. Components of a Volume Management Facility 58
4.3.1. Overview 58
4.3.2. Common Configurations in Australian Practice 60
4.4. Concept Design 62
4.4.1. Overview 62
4.4.2. Choose a Location 63
4.4.3. Choosing a Design Solution 64

4.4.4. Preliminary Sizing and Configuration 66
4.4.5. Collaboration and Integration 66
4.4.6. Emergence of Volume Management Research 67
4.4.7. Use of Computer Models 68
4.5. Detailed Design Considerations 69
4.5.1. Detention Basins 69
4.5.2. On-site Detention 73
4.5.3. Rain Water Harvesting 79
4.5.4. Bioretention Basins 82
4.5.5. Constructed Wetlands 85
4.5.6. Managed Aquifer Recharge 87
4.5.7. Infiltration Systems 90
4.5.8. Stormwater Harvest Ponds 95
4.6. References 96
5. Stormwater Conveyance 101
5.1. Introduction 101
5.2. Design Philosophy and Objectives 102
5.3. Conveyance Networks 106
5.4. Design of Conveyance Networks with Computer Models 112
5.5. Inlet Structures 114
5.5.1. Types of Inlet Structures 115
5.5.2. Inlet Capacities 116
5.5.3. Energy Losses 120
5.6. Conveyance Infrastructure 130
5.6.1. Hydraulic Models to Define Flow Characteristics 130
5.6.2. Overland Flow 133
5.6.3. The Hydraulic Grade Line (HGL) and Energy Grade Line (EGL) 138
5.6.4. Flows Through Conveyance Networks 140
5.6.5. Culverts 143
5.7. References 149
6. Modelling Approaches 153
6.1. Introduction 153
6.2. Urban Modelling Tasks 154
6.2.1. Impervious Surface Estimation 155
6.2.2. Conveyance Infrastructure 157
6.2.3. Waterway Crossings 158
6.2.4. Volume Management Infrastructure 159
6.2.5. Water Quality Treatment Performance 160
6.2.6. Data Collection and Collation 160
6.3. Model Selection 161
6.3.1. Overall Trends in Urban Modelling 161
6.3.2. Types of Urban Models 164
6.3.3. Model Scale 168
6.3.4. Flood Magnitude 172
6.3.5. Choosing a Model 173
6.3.6. Model Simplification 174
6.3.7. Model Resolution 175
6.4. Application to Urban Modelling 175
6.4.1. Urban Modelling Frameworks 176
6.4.2. Choice of Rainfall 179
6.4.3. Runoff From Properties 183
6.4.4. Sub-Catchment Runoff to Inlet Structures 188

6.4.5. From the Inlet to the Outlet 191
6.4.6. Downstream 203
6.5. References 215

List of Figures

9.2.1. Simple Model of Water Inputs, Storage and Flows in an Urban Catchment 6

9.2.2. Spatial Distribution of Average Annual Rainfall Depths for the Greater Melbourne Region (Coombes, 2012) 9

9.2.3. Spatial Distribution of Average Annual Frequency of Rainfall for the Greater Melbourne Region (Coombes, 2012) 10

9.2.4. Spatial Distribution of Average Annual Maximum Temperatures for the Greater Melbourne Region (Coombes, 2012) 11

9.2.5. Total Water Input to Curtin in ACT 13

9.2.6. Total Water Output from Curtin in ACT 13

9.2.7. Average Annual Water Balances from Households in Adelaide, Ballarat, Brisbane, Canberra, Melbourne, Perth and Sydney 14

9.2.8. Water Cycle Processes in the Ballarat Water District from 1999 to 2012 15

9.3.1. Schematic of Traditional Urban Catchments and Cumulative Stormwater Runoff Processes 29

9.3.2. The Full Spectrum of Flood Events (Adapted from Weinmann (2007)) 30

9.3.3. Cumulative Impacts of Distributed Management 32

9.3.4. Schematic of the Connectivity of Urban Water Networks 33

9.3.5. Example Overland Flow Path Map Generated Using a Two Dimensional Model. 35

9.3.6. Example of Fluvial Flow Path with Interface with Overland Flow Path 38

9.3.7. Typical Configurations of Major Minor Conveyance Systems Deployed in Australian Practice 41

9.4.1. Typical Hydrograph Change Generated by a Temporary Storage (Without Harvesting) 53

9.4.2. Rural and Developed Catchment Hydrographs 54

9.4.3. Developed Catchment with Retention as Compared to Detention and Slow Drainage Strategies 54

9.4.4. Potential Overlapping Volume Management Design Objectives 58

9.4.5. Detention Basin Typical Section 69

9.4.6. Typical Detention Basin Primary Outlets 72

9.4.7. Typical Section Through a Below Ground On-Site Detention 73

9.4.8. Frequency Staged Below Ground On-Site Detention System (adapted from Upper Parramatta River Trust (2005)) 75

9.4.9. Rain Water Tank with Dedicated Air Space (adapted from Coombes et al. (2001)) 82

9.4.10. Components of a Bioretention Basin (Healthy Waterways by Design, 2014) 83

9.4.11. Schematic Layout of a Typical Constructed Wetland 85

9.4.12. Photo of a Typical Constructed Wetland (Source: Steve Roso) 86

9.4.13. Example of a Managed Aquifer Recharge Scheme in an Unconfined Aquifer (Adapted from (Natural Resource Management Ministerial Council et al., 2009)) 88

9.4.14. Leaky Well Infiltration System (adapted from Argue (2017)) 92

9.4.15. Main Components of an Infiltration Swale (with Filter Strip) (adapted (Argue, 2017),(Argue, 2013)) 93

9.5.1. Stormwater Conveyance and Volume Management Within an Urban Stormwater Network 102

9.5.2. Minor and Major Concepts for Conveyance Networks 104

9.5.3. The Stormwater Design Process 106

9.5.4. Examples of Configurations of Conveyance Networks in New South Wales (Left) and Queensland (Right) 108

9.5.5. A Typical Configuration of a Conveyance Network 110

9.5.6. A Simple Example of a Stormwater Conveyance Network 110

9.5.7. Example of Alternative Configurations of Inlet Pits at a Road Intersection 112

9.5.8. Idealised Hydraulic Issues Impacting on Inlet Structures 115

9.5.9. Basic Types of Inlet Structures 115

9.5.10. Kerb Inlet Gutter Depressions from HEC-22 (NHI, 2013) 118

9.5.11. Kerb Inlet Throat Angles from HEC-22 (NHI, 2013) 119

9.5.12. Inlet Capacities for On-grade Pits 119

9.5.13. Idealised Grade Lines at an Inlet Structure 122

9.5.14. Approximate Pressure Change Coefficients, k_u, for Inlet Structures 125

9.5.15. Pressure Change Coefficient Chart (QUDM, 2013) 127

9.5.16. Pressure Change Coefficient Chart (QUDM, 2013) 128

9.5.17. Decrease in Pressure Change Coefficient as a Result of Benching (QUDM, 2013) 130

9.5.18. Schematic of Three Hydraulic Models of Conveyance Networks 131

9.5.19. Flow Capacity Chart for One Side of an 8 m Carriageway with 3% Cross-fall 135

9.5.20. Gutter Flow Characteristics. 137

9.5.21. Flow Behaviour in a Surcharged Pipe System Showing Energy Grade Lines and Hydraulic Grade Lines 140

9.5.22. Example of Full Flows in a Pipe 141

9.5.23. Vena Contracta or Contraction at a Culvert Entrance 144

9.5.24. Orifice Coefficients (Vennard and Street, 1982) 145

9.5.25. Example of Inlet Control (U.S. Department of Transport, 2005) 146

9.5.26. Inlet Control versus Elevation of Headwaters (U.S. Department of Transport, 2005) 147

9.5.27. Example of Outlet Control Situations (U.S. Department of Transport, 2005) 148

9.5.28. Shapes of Weir Crests (Laurenson et al., 2010) 149

9.6.1. Types of Urban Hydrologic Models and their Likely Application Range 173

9.6.2. Stepwise Flowchart for Selecting an Urban Stormwater Model 174

9.6.3. Example of a Box and Whisker Plot of Peak Stormwater Runoff Utilised to Select the Critical Storm Burst Ensemble and Other Design Information 180

9.6.4. Rainfall Runoff Processes in Urban Catchments 181

9.6.5. Stormwater Runoff from Roofs and Properties – Lot Scale Effects 184

9.6.6. Stormwater Management System for Larger Properties with Complex Land Uses 186

9.6.7. Example Distribution of Available Storage Prior to Storm Events versus Annual Exceedance Probability (AEP) of Storm Events 188

9.6.8. Example of a Simple Urban Stormwater Sub-Catchment 189

9.6.9. Changes in Design Modelling Techniques for Urban Areas 192

9.6.10. Design Process that Utilises Rainfall Ensembles in Hydrology to Select the Rainfall Pattern Closest to Mean Peak Flows for use in Hydraulic Analysis 193

9.6.11. Design Process that Utilises Rainfall Ensembles in Hydrology and Hydraulic Simulations to Select the Mean Pattern for Analysis of Flooding 193

9.6.12. The Woolloomooloo Catchment in Sydney 195

9.6.13. Use of Ensembles of Storm Bursts (1% AEP) in the Hydraulic Model to Select Critical Duration 198

9.6.14. Example of Runoff from ARR 1987 Single Storm Burst and Ensembles of Storm Bursts from this guideline (1% AEP) 199

9.6.15. Upper Catchment Volume Check for Direct Rainfall Model (Prior to Corrections) 200

9.6.16. Comparison of Treatment of Initial Conditions in Overland Flows Generated by Coupled Direct Rainfall Models Near the Top of the Catchment (Top Pane: Riley Street) and Near the Bottom of the Catchment 202

9.6.17. Outflow Hydrographs Catchment Showing the Significance of Surface Flows 203

9.6.18. Catchment prior to development 204

9.6.19. Developed Catchment 205

9.6.20. Estimated Rural Peak Flows using the RFFE 206

9.6.21. Regional Flow Gauges used in the RFFE Estimate of Rural Peak Flows 206

9.6.22. Calibration of Rural Flows to RFFE Flow Estimates 207

9.6.23. Pre-Development Peaks Flows for 50% AEP Events 207

9.6.24. Pre-Development Peak Flows for 10% AEP Events 208

9.6.25. Pre-Development Peak flows for 1% AEP Events 208

9.6.26. Selection of the 10% AEP Design Storm for Preliminary Infrastructure Design 209

9.6.27. Development Peak Flows for 50% AEP Events 209

9.6.28. Development Peaks Flows for 10% AEP Events 210

9.6.29. Development Peak Flows for 1% AEP Events 210

9.6.30. Overview of the Planned Conveyance Network in the Urban Development 211

9.6.31. Overview of the Planned Storage Basin Below the Urban Development 212

9.6.32. Peak flows from the Basin for 50% AEP Events 212

9.6.33. Peak Flows from the Basin for 10% AEP Events 213

9.6.34. Peak Flows from the Basin for 1% AEP Events 213

9.6.35. Peak Water Levels the Basin for 1% AEP Events 214

9.6.36. Peak Water Levels in the Basin and on Roads for 1% AEP Events Subject to Climate Change 215

List of Tables

9.2.1. Annual Water Balance Data from Suburbs of Australian Cities. 7

9.2.2. Water Balances for Selected Regions 11

9.2.3. Water Balance for Curtin Catchment in Canberra for the Period 1979 – 1995. (Adapted from (Mitchell et al., 2003)) 12

9.3.1. Factors Influencing the Balance between Capacities of Major and Minor Systems in Design 39

9.4.1. Summary of Volume Management Design Objectives 55

9.4.2. Volume Management Facility Components 58

9.4.3. Common Volume Management Facility Configurations in Australian Practice 60

9.4.4. Indicative Suitability of Common Volume Management Design Solutions 65

9.4.5. Detention Basin Freeboard Requirements (Adapted from Queensland Department of Energy and Water Supply (2013)) 71

9.4.6. Above Ground On-Site Detention Storage Design Considerations 76

9.4.7. Below Ground On-Site Detention Storage Design Considerations (Department of Irrigation and Drainage, 2000), (Department of Irrigation and Drainage, 2012) 77

9.4.8. Interim Relationship between Annual Exceedance Proability and 'Emptying Time (Argue, 2017) 95

9.5.1. Suggested Design and Severe Blockage Conditions for Inlet Pits Book 6, Chapter 6 120

9.5.2. Correction Factors for k_u and k_w for Submergence Ratios (S/D_o) not Equal to 2.5 (QUDM, 2013) 129

9.6.1. Common Types of Urban Models 165

9.6.2. Generic Classification of Model Estimation Capability 167

9.6.3. Typical Urban Model Scales 169

9.6.4. Example Flood Generation Processes at Different Model Spatial Scales 170

9.6.5. Summary of Updated Design Rainfall Processes 181

9.6.6. ARR 1987 Rainfall Loss Parameters 196

9.6.7. ARR 2016 Rainfall Loss Parameters for Urban Areas 197

9.6.8. Adopted ARR 2016 Rainfall Loss Parameters 197

9.6.9. Assumed Capacity of Inlet Pits for 1% AEP Rain Events 198

9.6.10. Characteristics of the Upper Catchment used in the Volume Check 200

Chapter 1. Introduction

Peter Coombes, Steve Roso

Chapter Status	Final
Date last updated	14/5/2019

1.1. Introduction

There have been profound changes to the science and practice of urban hydrology and stormwater management since the last edition of Australian Rainfall and Runoff (ARR) published in 1987 (Pilgrim, 1987). During this period analysis methods have evolved from use of the slide rule to the computer age and beyond. The revision of ARR has aimed for an evidence based approach that incorporates 30 years of additional data, science and knowledge. This includes a move away from simple design rainfall burst event methods towards Ensemble and Monte Carlo approaches to better capture variability. There is less reliance on the rational method, more data available, new Intensity Frequency Duration (IFD) data and better flow estimates for ungauged catchments (refer to Book 3, Chapter 3).

There are new challenges and gaps in knowledge about urban hydrology that is part of an increasingly complex urban water cycle and town planning processes. The designer now aims to retain stormwater within urban landscapes, manage stormwater quality, maximize the potential of the stormwater resource and to slow flows into receiving waterways. Australian Rainfall and Runoff now employs Australian data which ensures that urban designers can better represent real local systems and address these new challenges.

Wherever possible this version of ARR provides information about the uncertainty of methods and inputs. This will better equip urban designers to understand risks in the urban environment. The Urban Book (Book 9 – Runoff in Urban Areas) has been constructed to utilise and complement the broader set of tools in ARR used to manage the water cycle. The over-arching objective of this book is to provide revised and up-to-date guidance for analysis and management of urban stormwater runoff.

1.1.1. Urban Stormwater Runoff

Urban stormwater runoff and associated stormwater management responses are part of a linked urban water cycle which includes stormwater quantity and quality, water supply, sewerage, urban form and waterways. Urban runoff has hydrologic characteristics such as flow rate and volume which differ considerably from natural and rural systems. As a result there is significant potential for impacts on natural processes and on society. These include nuisance flooding, disruption of traffic and business functions, flood disasters and damage, stream erosion, and destruction of natural waterway form and function. These water balance and linked systems issues are discussed in Book 9, Chapter 2 and Book 9, Chapter 3.

Whilst urban runoff can be a problem to be managed, it is also a potential opportunity to be exploited if viewed as an environmental resource. There are urban runoff design and investigation techniques that can be used to achieve better economic, social and environmental outcomes. The discussion of managing urban stormwater runoff in this Book also intersects with managing stormwater quality which is addressed in a number of guidelines throughout Australia such as Australian Runoff Quality. Many of these practices are introduced in this book.

1.1.2. Stormwater Management Infrastructure

Urban runoff was traditionally managed using networks of pipes and channels to convey stormwater rapidly away from urban areas. The definition of drainage has now broadened to incorporate both conveyance and management of stormwater volumes via a wider range of measures, including natural and man-made infrastructure to restore natural flood behavior where possible.

Two classes of stormwater management infrastructure are described in this book; volume management (Book 9, Chapter 4) and conveyance systems (Book 9, Chapter 5).

Volume management includes measures that can store runoff for a period of time, promote infiltration and store harvested stormwater for beneficial uses. Modern best practice aims to achieve a range of hydrologic and water quality objectives within these facilities. Volume management is a key element of stormwater management and flood control which has increased in importance and will continue to evolve into the future. Stormwater volume controls have been subject to substantial and increased research effort since 1987.

Conveyance systems allow runoff to pass through urban areas and provide connections through the catchment. Conveyance systems can be classified in different ways, for example underground versus surface and trunk versus non-trunk. The traditional description of urban stormwater management involves a minor and major event management philosophy where the minor concept involves pipe drainage networks and the major concept addresses flood events that are conveyed as surface flows. A minor versus major design concept is also still relevant in order to efficiently convey urban runoff while mitigating nuisance, damage and disaster. Regardless, the focus for conveyance should be careful management of surface flows and restoration of natural flow behaviour wherever possible.

Volume management facilities and conveyance systems are interlinked to form a network with volume management most often at discrete locations connected by more linear conveyance systems. Both conveyance and volume management can exist at multiple scales from lot scale (source control) to regional scale (end of pipe). In the context of Book 9, natural and semi-natural urban waterways are considered part of the network of conveyance and storage infrastructure.

1.1.3. Modelling

The unique characteristics of urban modelling include measurement and assessment of the hydrologic and hydraulic effect of impervious surfaces, conveyance systems and hydraulic structures including volume management facilities. Analysis of urban areas involves data intensive and complex processes. There is a need for complex computing tasks aided by software to assist with modern investigation. A wide range of computer software is available to the designer. Hand calculations are generally unsuitable for most urban applications other than basic checks and approximations.

Choice of computer software such as urban hydrology and hydraulic models depends on a number of factors including the spatial scale of the investigation area and the magnitude of the floods of interest. Book 9, Chapter 6 provides guidance on how to pick a short list of suitable models based on these factors. The aim should be to best match the selected model with the type of investigation being undertaken.

Once a suitable model has been selected, the challenge is to ensure the model is applied correctly. Book 9, Chapter 6 does not provide guidance on how to use specific modelling software and instead describes the urban modelling process in a software independent manner. Some models can be simplified and the physical resolution reduced, depending on the spatial scale of the investigation and experience of the modelling team. Urban modelling frameworks are described providing guidance for key segments of urban catchments from the behaviour of land uses within sub-catchments that flow to inlet structures, through urban stormwater networks, and into the receiving waterway.

1.1.4. Structure and Purpose of this Book

This Urban Book is a guideline rather than a standard or recipe as Australia is too diverse and the urban practice involves increasing complex combinations of solutions. A primary audience of this book includes readers from multiple disciplines and early career urban designers.

This book focusses on the entire spectrum of runoff events and potential flooding outcomes. Book 9, Chapter 2 provides an overview of the characteristics of urban hydrology. Book 9, Chapter 3 introduces some of the key concepts in urban stormwater management as part of an urban water cycle and urban systems. It is built around Book 9, Chapter 4 and Book 9, Chapter 5 which describe the key stormwater design elements of volume management and conveyance. Book 9, Chapter 6 provides guidance on urban modelling including model selection and application. Two case studies are also provided in Book 9, Chapter 6.

1.1.5. The Future

There is a need to allow changes in interpretation of the stormwater components of this book to accommodate contemporary and integrated approaches to water cycle management in urban areas, which starts with the integration of land and water planning across time horizons and spatial scales. This guidance encourages advances in urban water cycle management, and expects advances in science and professional practice over the next 30 years. There is an enabling framework of guidance in all ARR Books, which encourages and permits advanced analysis techniques and innovative designs. The guidance in ARR does not intend to hold back advances in analysis of integrated solutions.

In some jurisdictions, there has been disproportionate focus on mitigating nuisance in the minor system at the expense of a proper analysis of the major system. Replacement of the minor or major drainage approach with the relativity of mitigating nuisance or disaster may be a future innovation of stormwater management. Allowing space for a major system can help manage large events and provides flexibility for adapting stormwater management to incorporate integrated systems and better management of nuisance.

It is expected that policy frameworks will evolve to further integrate land and water management with design processes at all spatial scales from local to regional and which also applies to urban renewal and asset renewal or replacement choices. Future design methods fomakr integrated solutions are likely to include most of the variability of real rainfall events by using continuous simulation, Monte Carlo frameworks and techniques that consider complete storms, frequency of rainfall volumes and the spatial variability of events.

Good urban runoff management will only be achieved when it is integrated with the complete management of the urban water cycle and includes proper consideration of runoff quality. The guidance in the Urban Book must be linked with Australian Runoff Quality (ARQ) (Engineers Australia, 2006) and other water quality guidelines so that urban stormwater management is an integrated part of the urban water cycle and avoids duplication of infrastructure. An integrated approach to stormwater management should avoid installation of infrastructure to meet separate objectives that, in combination, create unexpected diminished performance. There is a need to consider integrated approaches for future urban water management. Integrated systems have the capacity to produce solutions that respond to multiple objectives including economic, social and environmental criteria.

This Book on Runoff in Urban Areas is part of the evolving story of stormwater management and aims to encourage innovation into the future.

1.2. References

Engineers Australia (2006), Australian Runoff Quality - a guide to Water Sensitive Urban Design, Wong, T.H.F. (Editor-in-Chief), Barton.

Pilgrim, DH (ed) (1987) Australian Rainfall and Runoff - A Guide to Flood Estimation, Institution of Engineers, Australia, Barton, ACT, 1987.

Chapter 2. Aspects of Urban Hydrology

Anthony Ladson

With contributions from the Book 9 editors (Peter Coombes and Steve Roso)

Chapter Status	Final
Date last updated	14/5/2019

2.1. The Urban Hydrologic Cycle

Hydrologic analysis for both urban and non-urban situations begins with the water cycle. In rural areas, hydrologists are concerned with catchment inputs, especially rainfall, outputs such as evaporation and runoff, and water storage. The fundamental processes are the same for urban catchments, however, development profoundly changes water storages and flows (Figure 9.2.1):

- Inputs increase as mains water is supplied to urban catchments along with rainfall.
- Less water is stored within urban catchments. Paved surfaces replace much of the landscape to diminish infiltration of rainfall into soil profiles. Hydraulically efficient conveyance networks rapidly remove surface water from urban areas.
- There are dramatic changes in quantity, quality and timing of water leaving the catchment. Runoff volumes are often substantially increased. Wastewater networks provide an alternative flow path that interacts with stormwater and groundwater. There may be less opportunities for water to evaporate if it can quickly drain from a catchment.

The change in the rate and volume of inputs, outputs and storage explains the hydrologic behaviour in urban areas: the rapid response to rainfall and increased flood magnitude and frequency that correlates with development. This chapter explores aspects of urban hydrology, the impact of development and urban stormwater conveyance networks, focussing on areas where the effect of urbanisation needs to be considered for estimation of floods.

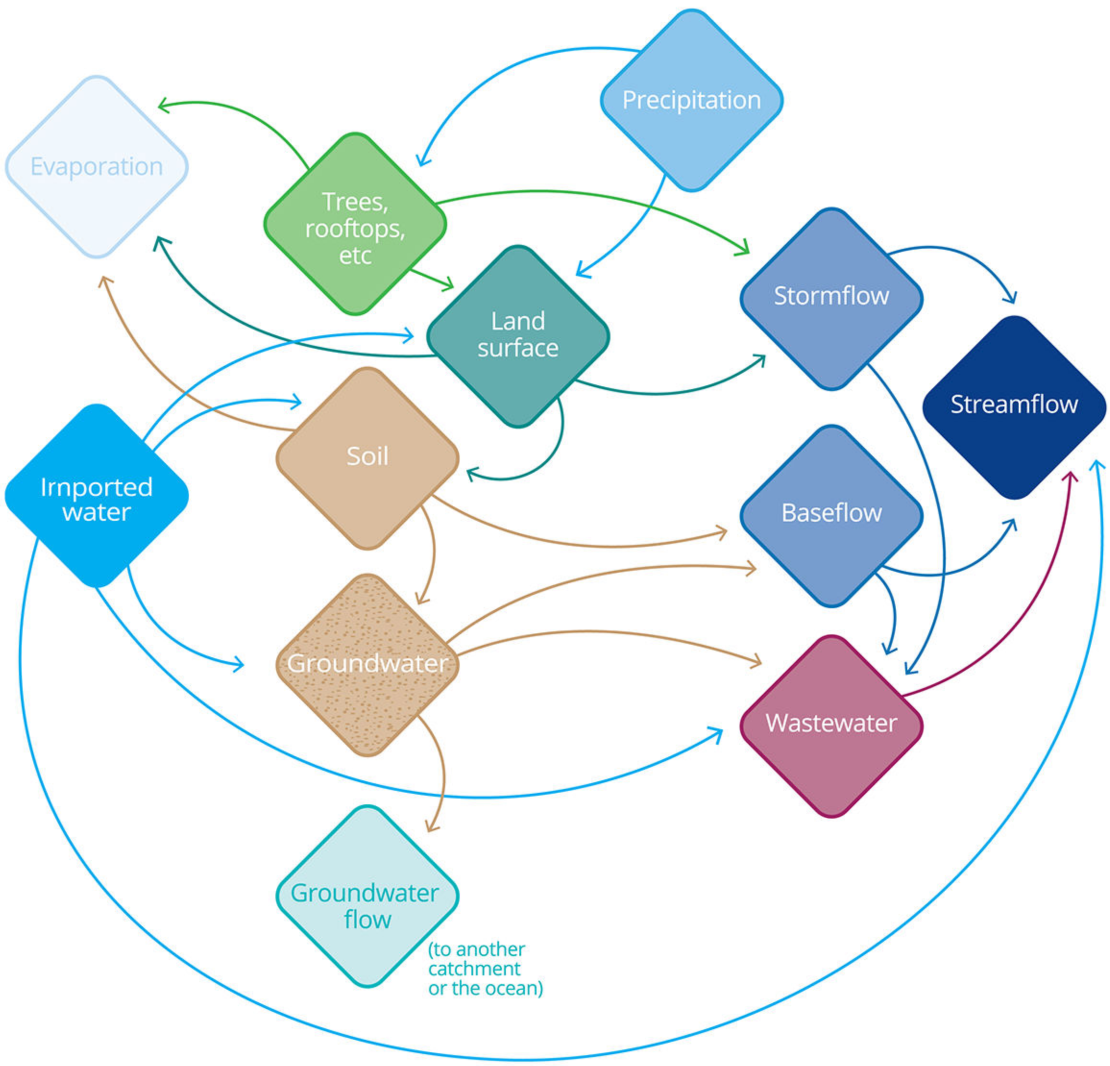


Figure 9.2.1. Simple Model of Water Inputs, Storage and Flows in an Urban Catchment

2.2. Human Impact on the Hydrologic Cycle

2.2.1. Urban Water Balance

The hydrological cycle must be considered at different temporal and spatial scales to gain an insight into urban hydrology. A water balance can identify the influence of imported water on catchment hydrology at the spatial scale of a suburb or city.

The water balance for an urban catchment, during a selected time period, can be expressed by equating the change in the amount of for water stored to the sum of catchment inputs minus the sum of catchment outputs (Mitchell et al., 2003).

$$\Delta S = (P + I) - (E_a + R_s + R_w) \qquad (9.2.1)$$

Where:

ΔS is the change in catchment storage

P is precipitation

I is imported water

E_a is actual evapotranspiration

R_s is stormwater runoff

R_w is wastewater discharge

There have been several studies of water balances in the urban areas of Australia including Canberra, Melbourne, Perth, Sydney and South-East Queensland (Table 9.2.1). Although there are substantial differences in climate of these study areas, and the number of selected examples is small, the data provides some insights.

- Wastewater leaving a catchment should be less than 59% to 86% of the amount of water imported, since imported water contributes to stormwater and evapotranspiration. This means that imported water contributes to stormwater and/or evapotranspiration. As a result, stormwater plus evaporation exceeds precipitation, according to all case studies.
- Imported water is about 30% to 39% of precipitation. This means imported water substantially increases catchment inflows.
- The volume of imported water is about the same as, or less than, wastewater plus stormwater. This suggests the potential for augmentation of water supply by some combination of rainwater harvest, stormwater harvest and wastewater reuse.

Table 9.2.1. Annual Water Balance Data from Suburbs of Australian Cities.[a]

Location	Input			Output				Wastewater /Imported Water (%)
	Rainfall (mm)	Imported Water (mm)	Imported Water as a Percentage of Rainfall (%)	Actual Evapo-transpiration (mm)	Storm Water Runoff (mm)	Waste Water Runoff (mm)	Change In Store (Misclose) (mm)[b]	
Curtin, ACT (Mitchell, et al. 2003) (1979-1996)	630	200	32%	508	203	118	1	59%
Sydney (Bell, 1972) (1962-1971)	1150	349[c]	30%	736	501	262	0	75%
Sydney (Kenway et al., 2011) (2004-2005)	952	370	39%	766	281	319	-44	86%

Location	Input			Output				Wastewater /Imported Water (%)
	Rainfall (mm)	Imported Water (mm)	Imported Water as a Percentage of Rainfall (%)	Actual Evapo-transpiration (mm)	Storm Water Runoff (mm)	Waste Water Runoff (mm)	Change In Store (Misclose) (mm)[b]	
Subiaco-Shenton Park Perth (McFarlane, 1985)	788	285 + 96[d]	36%	766	104	154	117[e]	54%
Melbourne (Kenway et al., 2011) (2004-2005)	763	237	31%	688	165	190	-43	80%
South-East Queensland (Kenway et al., 2011) (2004-2005)	1021	374	37%	814	390	179	12	49%

[a]The National Water Accounts reported by the Bureau of Meteorology (Bureau of Meteorology, 2015) contain information on water use in regions that include the urban areas of Adelaide, Canberra, Melbourne, Perth, South East Queensland and Sydney.

[b]See original studies for details

[c]Includes imported water and use of groundwater

[d]Inflow of stormwater from upstream area

[e]Adjusted for change in groundwater storage

Assessment of water balances for cities or urban regions also need to account for the spatial and temporal variation of paramaters throughout an area. For example, the spatial distribution of rainfall depth, frequency (rain days per year) and maximum temperatures are shown for the Greater Melbourne region in Figure 9.2.2, Figure 9.2.3 and Figure 9.2.4.

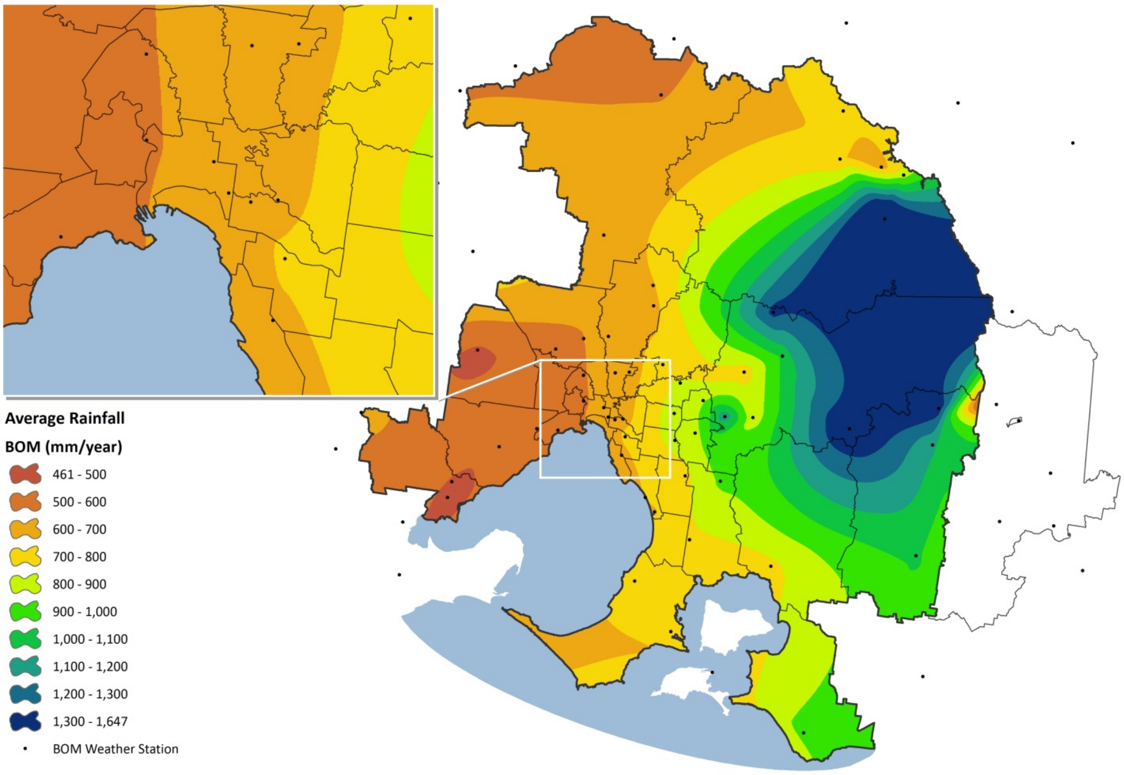


Figure 9.2.2. Spatial Distribution of Average Annual Rainfall Depths for the Greater Melbourne Region (Coombes, 2012)

Figure 9.2.2 demonstrates that average annual rainfall depths range from less than 470 mm to greater than 1640 mm across the Greater Melbourne region. The spatial distribution of rainfall will impact on the assessment of the water balance for the region and also impact on selection of stormwater management strategies. The spatial distribution of the frequency of rainfall will also impact on the determination of a water balance (Figure 9.2.3) (Walsh et al., 2012).

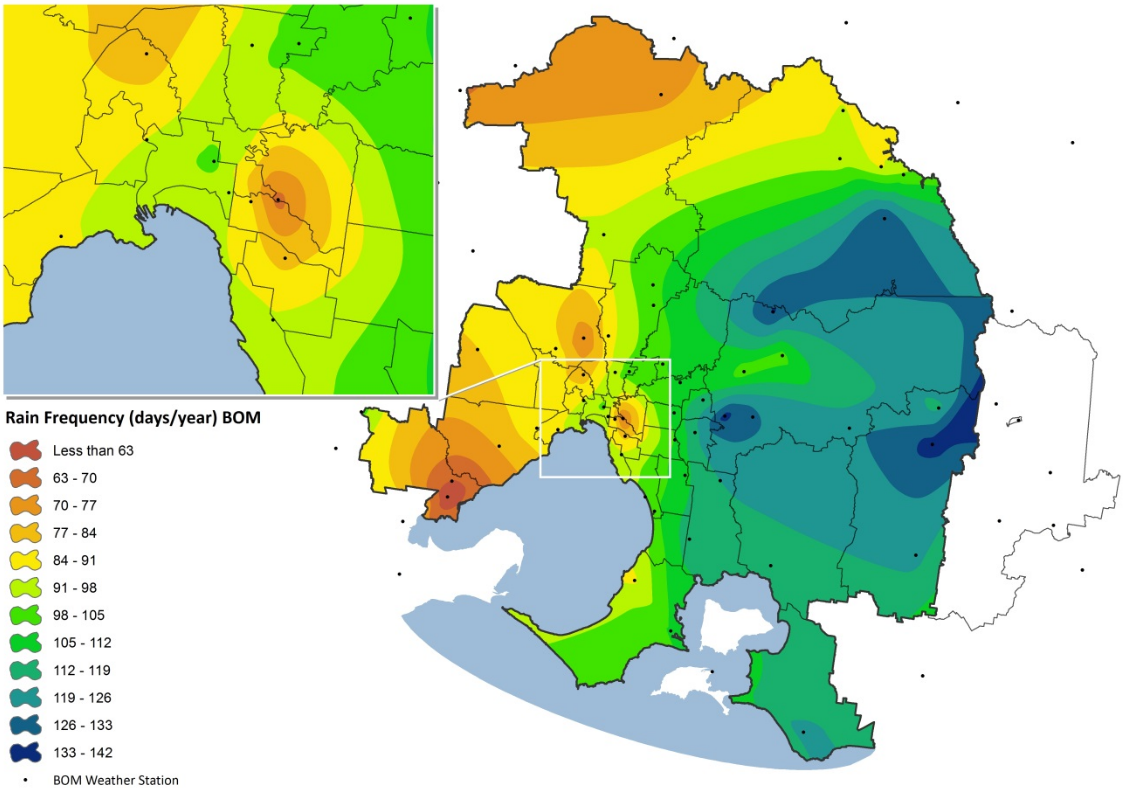


Figure 9.2.3. Spatial Distribution of Average Annual Frequency of Rainfall for the Greater Melbourne Region (Coombes, 2012)

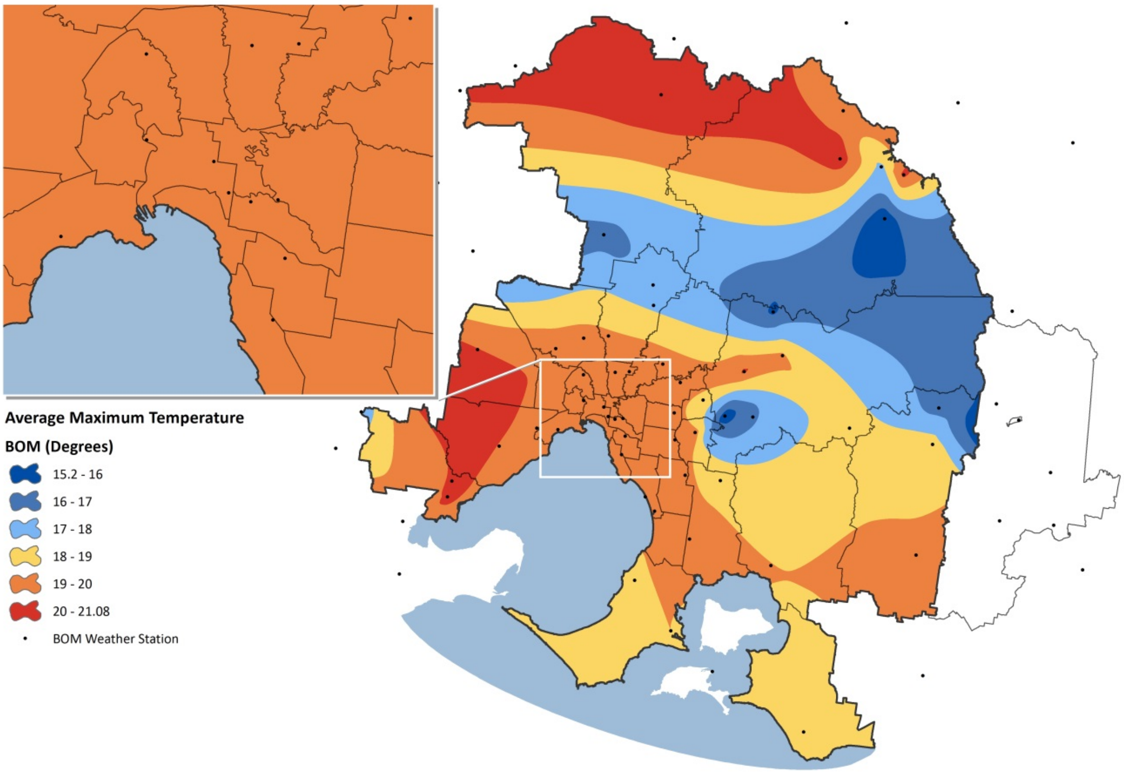


Figure 9.2.4. Spatial Distribution of Average Annual Maximum Temperatures for the Greater Melbourne Region (Coombes, 2012)

A range of recent detailed investigations that also considered the spatial and temporal variation of parameters was used to define water balances for Greater Melbourne, Greater Sydney, Greater Perth, and South-East Queensland regions. Water balances for 2013 were extracted from these studies to provide the examples presented in Table 9.2.2.

Table 9.2.2. Water Balances for Selected Regions

Region	Study	Average Annual Volume (GL)		
		Water	Wastewater	Stormwater
Greater Melbourne	Coombes & Bonacci (2012)	394	381	440[a]
Greater Sydney	Coombes & Barry (2012)	524	497	564
Greater Perth	Coombes & Lucas (2005)	249	131[b]	525
South East QLD	Coombes (2012)	278	265	470

[a]only includes stormwater runoff from urban surfaces. The total runoff volume of 650 GL/annum included open space and parks. These results are similar to the research by Walsh (2018) that estimated a total annual volume of 608 GL.

[b]there are less properties connected to centralised wastewater networks than connections to mains water supply.

Table 9.2.2 demonstrates that each region is subject to substantially greater volumes of stormwater runoff than demands for mains water. In addition, the volumes of wastewater discharges are similar to water demands. However, this result may be misleading as there are less wastewater connections (especially for Perth) than water supply connections in each region. Households in some areas are reliant on local wastewater management measures (such as septic tanks) and receive mains water supplies.

2.2.2. Lessons from a Detailed Water Balance Study at Curtin, ACT

Detailed information about an urban water balance is available for Curtin in ACT where Mitchell et al. (2003) obtained sufficient information to construct an annual water balance between January 1978 and June 1996. This study provides information on the variability in the urban water balance over time and the influence of climate (Table 9.2.3).

Table 9.2.3. Water Balance for Curtin Catchment in Canberra for the Period 1979 – 1995. (Adapted from (Mitchell et al., 2003))

Year	Rainfall (mm)	Imported Water (mm)	Actual Evapotranspiration (mm)	Stormwater Runoff (mm)	Wastewater Discharge (mm)	Change in Storage
Driest	247	269	347	74	107	-12
Average	630	200	508	203	118	1
Wettest	914	141	605	290	126	34

The average annual input and output of the catchment was about 830 mm. Approximately 24% (200 mm) of water was imported to the catchment via the supply system. Precipitation (rainfall) contributed the remaining 630 mm. Outputs included actual evapotranspiration (61%, 508 mm), stormwater runoff (24%, 203 mm) and wastewater discharge (14%, 118 mm).

The volume of imported water exceeded the volume of wastewater in all years and thus contributed to stormwater runoff, and at least in the driest years, to evapotranspiration. More water left the catchment as evapotranspiration and as stormwater runoff than was input via precipitation. In addition, in all but the driest years, wastewater and stormwater were greater than imported water, indicating the potential for harvest of suburban discharges to meet water demands. This highlighted the requirements for water imports under drought conditions.

Climate had a substantial influence on several of the water fluxes. Annual precipitation was highly variable ranging between 214 mm to 914 mm. On average, there was three times as much rainfall as water imports but in the driest year, more water was imported to the catchment than fell as rainfall. In the wettest year, imported water made up only 13% of water input. Figure 9.2.5 shows the relative amounts of precipitation and imported water for the driest, average and wettest years. The area of pie charts are proportional to total input. The proportion of imported water increases in drier years.. The proportion of imported water increases in drier years.

Considering outputs, the largest term is evapotranspiration, which represents 59% or more for each year. Although the total evapotranspiration varies between 347 mm and 605 mm for dry and wet years, the proportion of water lost as evapotranspiration is reasonably constant (59% to 66%). Figure 9.2.6 shows that relative amounts of actual evapotranspiration, stormwater and wastewater for the driest, average and wettest years (area of pie chart is proportional to total output). The proportion of stormwater increases in wetter years.. The proportion of stormwater increases during wetter years. The total volume and percentage of wastewater output does not seem to be greatly influenced by climate, as it is consistent between wet, average and dry years.

Stormwater runoff is highly reliant on climate, changing by a factor of about 4 mm from 74 mm during the driest year to 290 mm during the wettest year. Woolmington and Burgess (1983) demonstrated the direct link between garden watering and augmentation of low flows in Canberra urban streams, although this is moderated by water restrictions.

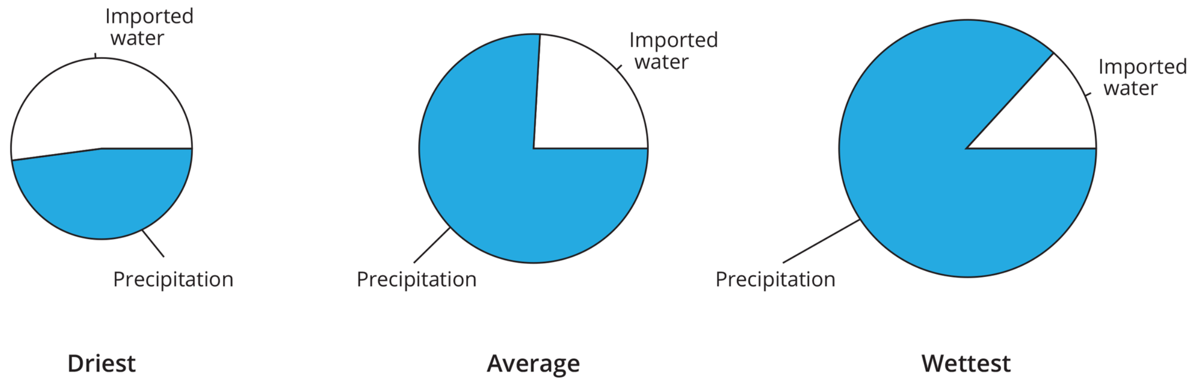


Figure 9.2.5. Total Water Input to Curtin in ACT

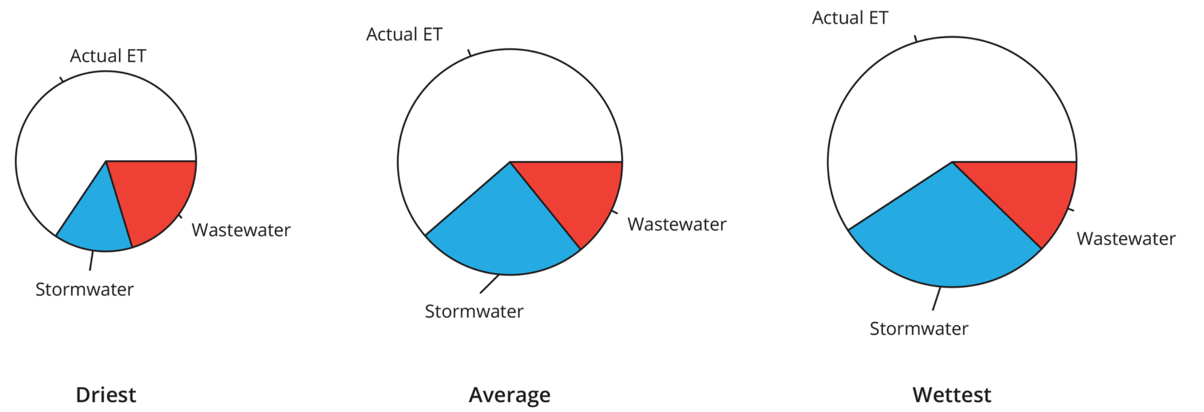


Figure 9.2.6. Total Water Output from Curtin in ACT

In summary, at the annual scale the urban water balance indicates the human impact on the hydrologic cycle. Water is imported into urban catchments and this exceeds the amount of wastewater exported, therefore there must be a net increase in outputs. Data from Curtin in the ACT shows that in dry years more than half of water inputs are via the mains supply system.

2.2.3. Implications of the Urban Water Balance: Stormwater as a Resource

Stormwater management throughout Australia was the subject of a recent Senate inquiry that recognised urban stormwater runoff as an under-utilised resource that creates significant environmental and flooding challenges (Commonwealth of Australia, 2015). Urban areas generate substantially greater stormwater runoff and pollutant loads compared to natural landscapes and are degrading our urban waterways and receiving waters. These additional flows substantially increase the discharges and overflows from sewer networks.

The volumes of stormwater runoff from urban areas exceed the water demand in many cities.

The water balance in cities include stormwater runoff, wastewater discharges and imported reticulated mains water. To illustrate this, Figure 9.2.7 presents the average annual water balance from the perspective of households in a range of Australian cities (Coombes, 2015).

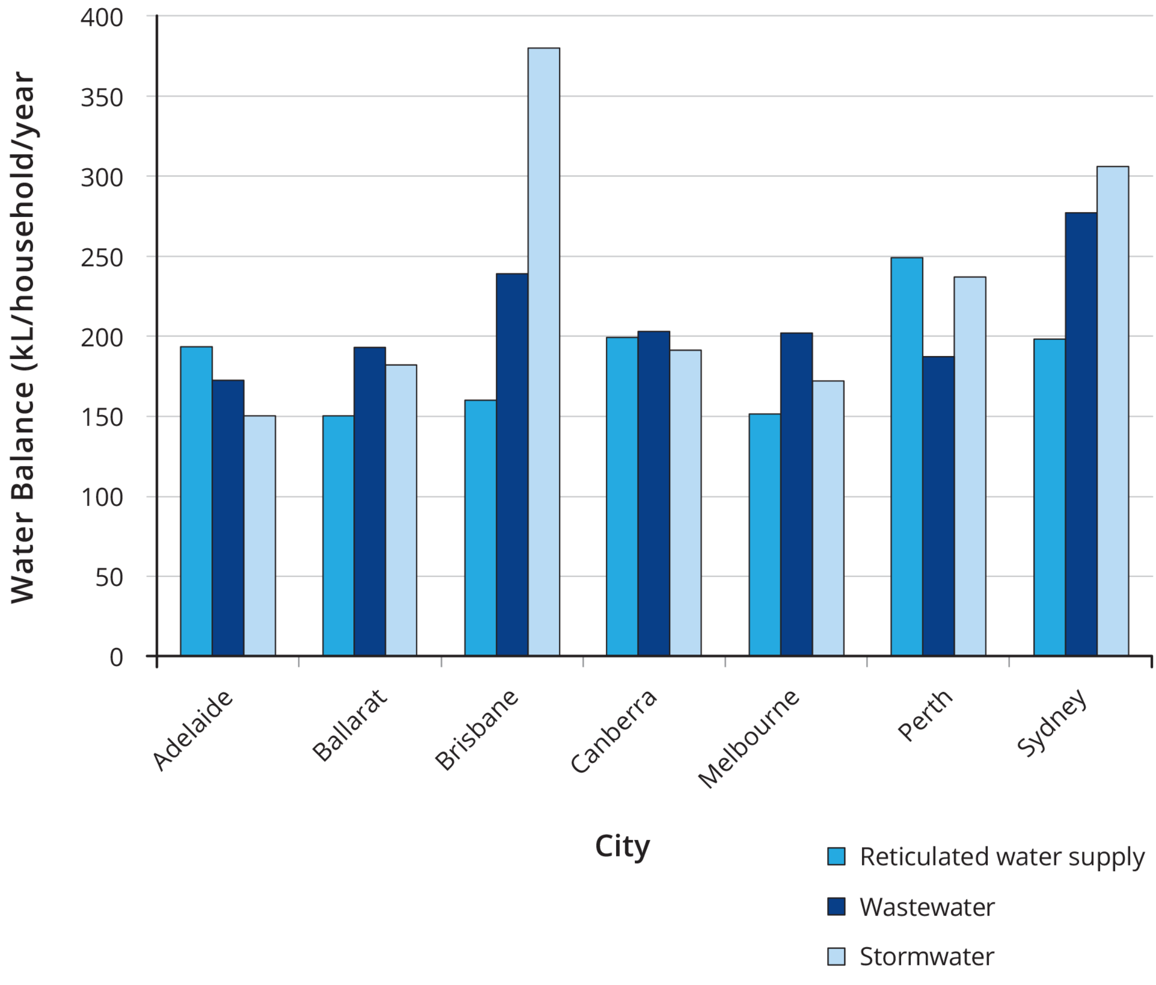


Figure 9.2.7. Average Annual Water Balances from Households in Adelaide, Ballarat, Brisbane, Canberra, Melbourne, Perth and Sydney

Figure 9.2.7 reveals how the combined volumes of stormwater runoff and wastewater discharging from households (and their properties) in each of the cities are greater than the volume of imported reticulated water supply at each location. Indeed, the average annual volumes of stormwater runoff from residential properties is similar to or greater than the average reticulated water demand from most of the properties. Improving stormwater management provides an opportunity to supplement urban water supplies as well as enhancing the amenity of urban areas and protecting the health of waterways in most cities.

The timing of water balances (rainfall, local and imported surface water supplies, groundwater, metered water use, sewage collected and stormwater runoff from urban surface) in the Ballarat Water District during recent drought is provided in Figure 9.2.8 as an example of water cycle processes (Coombes, 2015).

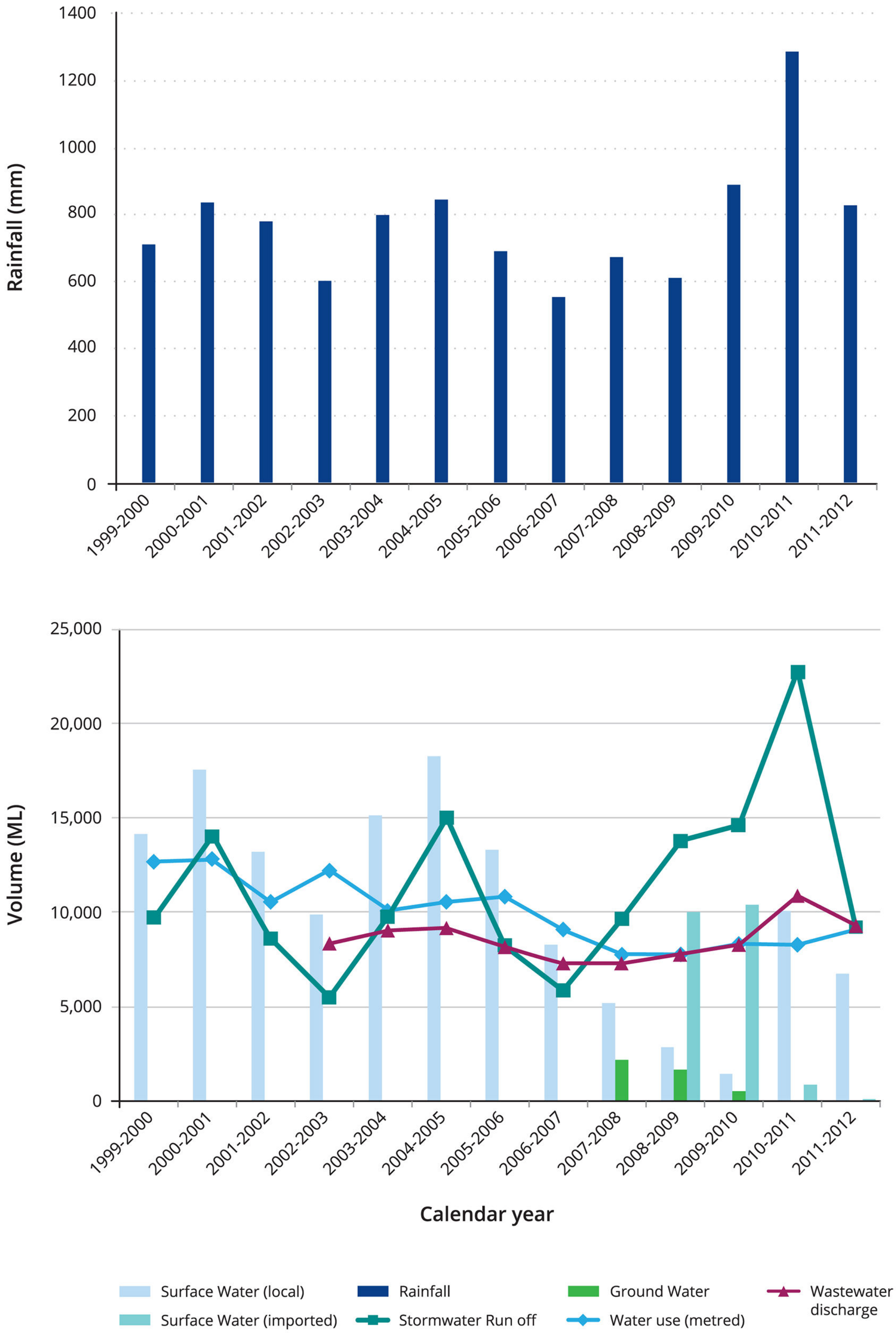


Figure 9.2.8. Water Cycle Processes in the Ballarat Water District from 1999 to 2012

Figure 9.2.8 indicates how the Ballarat Water District was dependent on surface water from nearby dams on local waterways [surface water (local)], until the worst of the drought in 2006. The reduced flows into local dams were supplemented using local ground water and the surface water imported from the Goulburn River (Murray-Darling Basin). Citizen's actions to reduce water use in response to water restrictions, installation of water efficient appliances and rainwater harvesting also halved the demands for utility water supply [water use (metred)] of the Ballarat Water District. The Council and the Water Authority also implemented stormwater harvesting and wastewater reuse solutions. In combination with the availability of ground water and imported surface water from the Goulburn River, these actions ensured that the City of Ballarat did not exhaust water supplies during drought. Despite rainwater and stormwater harvesting, there were still substantial stormwater runoff events suggesting that additional water was available albeit at additional cost.

The integrated action across the water cycle by the entire Ballarat community was a success from a water supply perspective that demonstrates the value of integrated solutions and understanding urban water balances. Nevertheless, this example also highlights the variable and temporal nature of urban water balances and connectivity with surrounding systems. Some of the key insights highlighted in Figure 9.2.8 are that substantial stormwater runoff events occur during drought, annual volumes of wastewater discharges were similar to water demands during water restrictions. Increases in stormwater runoff drive increases in wastewater discharges to be greater than water demands. The integrated solution for Ballarat was able to overcome the jurisdictional and institutional boundary conditions that limit opportunities for catchment based solutions in many cases.

2.2.4. Comparison of Rural and Urban Water Balances

A few studies that contrast water balances for urban and neighbouring natural catchments (Grimmond and Oke, 1986; Stephenson, 1994; Bhaskar and Welty, 2012). As expected, there is an increase in runoff, which we explore in the next section. The impact on evapotranspiration is less clear and depends on specific conditions as was apparent in the data for Curtain (Mitchell et al., 2003).

The partitioning of outflow between evaporation and stormwater runoff depends on water availability, conveyance infrastructure, storage in the catchment and the extent of irrigated parkland and gardens. There are a few examples, other than for Curtain, where this has been looked at in detail in an Australian context. In Melbourne, during a time of highly restricted water use for irrigation, Coutts et al. (2009) found that rapid stormwater runoff resulted in much reduced water availability and decreased evapotranspiration in urban areas compared to neighbouring rural sites. The result was a very dry urban landscape with energy partitioned into heating the atmosphere (which drove hot dry conditions) or into heat storage (which increased overnight temperature).

Bell (1972) suggests a similar decrease in evapotranspiration in Sydney (and consequent increase in runoff) as urbanisation increased. Recent investigations by Parker (2013) for Melbourne and by Argueso et al. (2014) for Sydney discuss a significant urban heat island effect, that is driven by increased heat storage capacity of urban structures and reduced evaporation from cities.

2.3. Aspects of Urban Stormwater Management Systems

2.3.1. Impervious Areas

An annual water balance illustrates the long term hydrologic changes caused by urbanisation. There are also substantial changes to flow events that are caused by:

- The expansion of impervious areas; and
- Efficient conveyance networks (Hollis, 1988; Schueler, 1994; Jacobson, 2011).

Urbanisation results in impervious surfaces replacing vegetated landscapes and this:

- Decreases the storage of water within soil profiles and on the ground surface and so increases the proportion of rain that runs off;
- Increases the velocity of overland flow; and
- Reduces the amount of rainfall that recharges groundwater.

Additionally, the natural stream network is augmented by conveyance networks (pipes and channels) that directly collects water from roofs and roads throughout the urban catchment. The expanded conveyance (drainage) network:

- Reduces the overland flow distance before water reaches a stream;
- Increases flow velocity because constructed drains are smoother and straighter than natural channels or overland flow paths;
- Reduces the storage of water in the channel system and on the catchment;
- Decreases the amount of water lost to evaporation because the water is quickly removed by the drainage network; and
- Means that almost all areas will contribute flow to a stream because the piped drainage network often extends to the furthest reaches the catchment.

As a result, although the exact effect of urbanisation on stream hydrology depends on the specific circumstances, there are some general comments that apply to many urban waterways in Australia. Urbanisation results in:

- Increased volumes of stormwater runoff;
- Increased frequency of high flow events;
- Increased magnitude of high flow events;
- Increased rates of change (both rising and falling limb of hydrographs);
- Increased catchment responsiveness to rainfall – more runoff events;
- Increased speed of catchment response;
- Reduced seasonality of high flows – high flow events occur year round rather than being mainly concentrated in a wet season;
- Greater variation in daily flows;
- Increased frequency of surface runoff to streams; and
- Reduced infiltration of rainfall.

Hydrologic changes caused by urbanisation occur at the same time as, and partly cause, changes to sediment loads, stream ecology and water quality (Walsh et al, 2005). Key hydrologic changes are considered in more detail in the following sections.

Increased Flow Volumes

More rainfall is converted to runoff in urban catchments from impervious surfaces and from pervious areas that are commonly compacted or irrigated by imported water (Harris and Rantz, 1964; Cordery, 1976; Hollis and Ovenden, 1988a; Hollis and Ovenden, 1988b; Hollis, 1988; Ferguson and Suckling, 1990; Boyd et al., 1994; Walsh et al., 2012; Askarizadeh et al., 2015).

Increased Flood Frequency and Magnitude

The increase in magnitude of flooding because of urbanisation has been recognised for many decades (Leopold, 1968). Urbanisation causes up to a 10-fold increase in peak flood flows in the range 4 EY to 1 EY with diminishing impacts on larger floods (Tholin and Keifer, 1959; ASCE, 1975; Espey and Winslow, 1974; Hollis, 1975; Cordery, 1976; Packman, 1981; Mein and Goyen, 1988; Ferguson and Suckling, 1990; Wong et al, 2000; Beighley and Moglen, 2002; Brath et al., 2006; Prosdocimi et al., 2015).

Increased flood magnitudes have been confirmed by analysis of paired catchment data in Australia as demonstrated by the comparison of urban Giralang and rural Gungahlin catchments in Canberra (Codner et al., 1988) as well as numerous modelling studies (Carroll, 1995). The impact of this increased flooding is substantial and makes up a large proportion of overall average annual flood damage estimates (Ronan, 2009).

Faster Flood Peaks – Flashiness

Runoff in urban streams responds more rapidly to rainfall in comparison to rural catchments and recedes more quickly. The quick response means there are more flow peaks in urban streams (Mein and Goyen, 1988; McMahon et al., 2003; Baker et al., 2004; Heejun, 2007; Walsh et al., 2012). Urbanisation was found to reduce the volume of channel storage by a factor of 30 in Canberra (Codner et al., 1988). This contributes to the rapid response of urban streams and increased flood flows.

The lag time – the time between the centre of mass of effective rainfall and the centre of mass of a flood hydrograph – decreases by 1.5 to 10 times in response to urbanisation (Packman, 1981; Bufill and Boyd, 1989).

Increased Runoff Frequency

Increased frequency of stormwater runoff is correlated with increased area of impervious surfaces. Small rainfall events of 1 to 2 mm will cause runoff from impervious surfaces (ASCE, 1975; Codner et al., 1988; Boyd et al., 1993; Walsh et al., 2012) but much more rainfall is usually required to produce runoff from grassland or forest (Hill et al., 1998; Hill et al., 2014). The frequency of stormwater runoff can increase by a factor of ten or more.

The increased responsiveness of urban landscapes to rainfall means that seasonality of flows in urban streams is different to rural streams. In many areas, rural catchments will only produce runoff after saturation of soil profiles following long periods where rainfall exceeds evapotranspiration. This result produces seasonal stream flows in many rural catchments with little runoff, when catchments are dry even when there is heavy rainfall (Western and Grayson, 2000). In urban streams, flows occur anytime there is rainfall. In temperate urban catchments, the largest urban runoff often occurs following intense thunderstorm rain during summer when, in equivalent rural catchment, there is little flow (Codner et al., 1988; Smith et al., 2013).

Changed Base Flows

The influence of urbanisation has complex impacts on groundwater and base flow in streams. Various features of urbanisation have confounding effects and their relative magnitude will determine the overall influence on base flow in streams. These features include:

- Reduced vegetation cover;
- Increases in impervious surfaces that limits infiltration and reduces evaporation of shallow groundwater;
- Infiltration from irrigation of gardens;
- Water leaking from pipes which contributes to ground water; and
- Drainage of groundwater into pipes or the gravel-filled trenches that surround pipes.

The most common response to urbanisation is that base flow in urban streams is decreased. More impervious areas means less opportunity for water to infiltrate so groundwater storage, for storage in soil profiles and discharges are reduced (Simmons and Reynolds, 1982; Lerner, 2002; Brandes et al., 2005). Less commonly, there may be increased base flow, particularly where stormwater is deliberately infiltrated (Ku et al., 1992; Al-Rashed and Sherif, 2001; Barron et al., 2013).

2.3.2. Conveyance

Urbanisation changes the processes of conveying water. The network of urban stormwater conveyance infrastructure is denser and more extensive than the natural stream system it replaces. This means that water is conveyed rapidly from both pervious and impervious surfaces throughout an urban catchment. Resistance to flows is lower in straight and smooth drainage paths of urban waterways, as compared to their natural counterparts.

The way water is conveyed from impervious areas can enhance or mitigate the influence of impervious areas. Modelling by Wong et al (2000) suggests that condition of the waterways also influences peak discharges that follow urbanisation. The largest impacts occur when urban streams are lined and made hydraulically efficient.

The importance of stormwater conveyance was confirmed in catchments with similar imperviousness but with and without conventional drainage infrastructure. This alteration of hydraulic behaviour was substantially reduced in suburbs with less efficient informal stormwater infrastructure that included roofs drained to gardens or rainwater tanks, and sealed roads which lacked curbs and drained to surrounding forest or earthen or vegetated swales (Hardy et al., 2004; Walsh et al, 2005).

Conveyance of Flood Flows

Understanding the conveyance of water in urban areas during times of overland flooding is a critical part of the analysis and design of urban stormwater management strategies. The major/minor principle requires that overland flow paths must be considered once the capacity of conveyance conduits is exceeded. This behaviour can be complex. Modelling of overland flow paths is used in many areas to guide zoning of land to control development and so reduce flood risk (Baker et al., 2005).

The catchment boundary for overland flows will often differ from boundaries of flows in conduits. This means that the behaviour of large floods may be substantially different from smaller events and has the potential to produce unexpected behaviours. An example is a suburb protected from riverine flooding by a levee. Stormwater is usually discharged under the levee into the river. If overland stormwater flooding cannot reach the river because of the levee it may, instead, back up and cause flooding. This type of unexpected and rapid flooding can be dangerous, as people are unlikely to be prepared for these types of events.

2.3.3. Receiving Environments

Many urban areas are adjacent to estuaries or bays that are the downstream boundary for water levels in streams. Coincident stormwater and estuarine flooding needs to be considered and is addressed in detail in Book 6, Chapter 5. Water authorities will often have mandated sea levels that must be used as part of the analysis flooding scenarios for planning (e.g; Melbourne Water (2012)).

Major rivers flowing through urban centres are also receivers of urban stormwater. These rivers will determine the base level to be used for modelling and additional analysis of the river system may be required to ensure flood risks are adequately considered.

The impact of urbanisation on major rivers can be contrasted with the effect on urban stormwater conveyance systems. Much of the water that is used in cities is harvested from the rivers that flow through them, for example, the Yarra River in Melbourne, the Hawkesbury-Nepean in Sydney and the Brisbane River. This results in lower flows and reduces flooding in main streams. There is a paradox here. The main rivers in urban areas have much reduced flow while in urban waterways flows are increased. For example, in Melbourne, there is about 125 km of streams and estuaries where flow has been substantially decreased by harvesting for urban water supply, and 1700 km of urban streams with substantially increased flow from urban catchments. From a citywide perspective, stormwater management needs to consider both of these impacts.

2.3.4. More Complex than Rural

Many aspects of urban flooding are more complex than similar issues in rural areas and require careful and thorough analysis. Key differences include:

- Very rapid response to rainfall;
- A greater proportion of rainfall converted to flood flow;
- Large numbers of people potentially affected by flooding;
- Development in one area adversely affecting flood risk in distant areas;
- Catchment areas than can change with the magnitude of flooding;
- Increased influence of the spatial pattern of rainfall because catchments respond to short rainfall events which are more spatial variable;
- Flooding from both riverine (fluvial) and stormwater (pluvial) overflows; and

- Floods can occur at any time of the year and may be most severe when triggered by summer thunderstorms - there is often no requirement for antecedent rainfall to wet the catchment to generate flooding.

In reviewing the components of average annual flood damage, Ronan (2009) suggested that, in general, risks from riverine flooding were reasonably well addressed but that stormwater flooding was a major issue that was yet to be adequately considered.

2.3.5. Combined and Separate Systems

The discussion in this section has generally assumed that suburbs have separate sanitary sewers and stormwater management systems. This is mostly true for Australian towns and cities. However, two areas have combined sewers – a single pipe that carries both wastewater and stormwater. These are the central area of Launceston, Tasmania and a small area in the CBD of Sydney. When the first sewers were built in Sydney, around 1857, there were five combined sewer systems: Woolloomooloo, Blackwattle Bay, Hay Street, Tank Stream and Bennelong. These discharged to Sydney Harbour. Most of these original sewers were converted to carry stormwater only following the construction of the Bondi Ocean Outfall Sewer in 1889 and wastewater was discharged in the ocean. Later developments in Sydney and elsewhere adopted separate stormwater drainage.

For an analysis of decision-making between separate and combined systems of sewerage, see Tarr (1979). For a history of urban drainage approaches, see Delleur (2003).

2.4. References

ASCE (American Society of Civil Engineers) (1975), Aspects of hydrological effects of urbanization. ASCE Task Committee on the effects of urbanization on low flow, total runoff, infiltration, and groundwater recharge of the Committee on surface water hydrology of the Hydraulics Division. Journal of the Hydraulics Division, (5), 449-468.

Al-Rashed, M.F. and Sherif, M.M. (2001), Hydrogeological aspects of groundwater drainage of the urban areas in Kuwait City. Hydrological Processes15: pp: 777-795.

Argueso D., Evans, J.P., Fita, L. and Bormann, K.J. (2014), Temperature response to future urbanisation and climate change. Climate Dynamics. 42(7), 2183-2199.

Askarizadeh, A., Rippy, M.A., Fletcher, T.D., Feldman, D.L., Peng, J., Bowler, P., Mehring, A.S., Winfrey, B.K., Vrugt, J.A., AghaKouchak, A., Jiang, S.C., Sanders, B.F., Levin, L.A., Taylor, S. and Grant, S.B. (2015), From rain tanks to catchments: use of low-impact development to address hydrologic symptoms of the urban stream syndrome. Environmental Science and Technology, 49(19), 11264-11280.

Baker, D.B., Richards, R.P., Loftus, T.T. and Kramer, J.W. (2004), A new flashiness index: characteristics and applications to Midwestern rivers and streams. Journal of the American Water Resources Association, 40: 503-522.

Baker, A., Rasmussen, P., Parkyn, K., Catchlove, R. and Kazazic, E. (2005), A Case Study of the December 2003 Melbourne Storm: the Meteorology, Rainfall Intensity, and Impacts of Flash Flooding. 29th Hydrology and Water Resources Symposium. Canberra. Engineers Australia.

Barron, O.V., Barr, A.D. and Donn, M.J. (2013), Effect of urbanization on the water balance of a catchment with shallow groundwater. Journal of Hydrology, 485: 162-176.

Beighley, R.E. and Moglen, G.E (2002), Trend assessment in rainfall-runoff behaviour in urbanising watersheds. Journal of Hydrologic Engineering, 7(1), 27-34

Bell, F.C. (1972), The acquisition, consumption and elimination of water by the Sydney Urban System. Proceedings of the Ecology Society of Australia, 7: 161-176.

Bhaskar, A. and Welty, C. (2012), Water Balances along an Urban-to-Rural Gradient of Metropolitan Baltimore, 2001-2009. Environmental & Engineering Geoscience, 28(1), 37-50.

Boyd, M.J., Bufill, M.C. and Knee, R.M. (1994), Predicting pervious and impervious storm runoff from urban drainage basins. Hydrological Sciences Journal, 39(4), 321-332.

Boyd, M.J., Bufill, M.C. and Knee, R. (1993), Pervious and impervious runoff in urban catchments. Hydrological Sciences Journal, 38(6), 463-478.

Brandes, D., Cavallo, G.J. and Nilson, M.L. (2005), Base flow trends in urbanizing watersheds of the Delaware river basin. Journal of the American Water Resources Association, 41(6), 1377-1391.

Brath, A., Montanari, A., and Moretti, G. (2006), Assessing the effect on flood frequency of land use change via hydrological simulation (with uncertainty), Journal of Hydrology, 324(1-4), 141-153.

Bufill, M.C. and Boyd, M.J. (1989), Effect of urbanization on catchment lag parameters. Hydrology and Water Resources Symposium. Christchurch, New Zealand, 23-30 November. pp: 171-178.

Carroll, D.G. (1995), Assessment of the effects or urbanisation on peak flow rates. Second International Symposium on Urban Stormwater Management 1995: Integrated Management of Urban Environments. Engineers Australia, Canberra. pp: 201-208.

Codner, G.P., Laurenson, E.M. and Mein, R.G. (1988), Hydrologic Effects of Urbanisation: A case study. Hydrology and Water Resources Symposium, ANU Canberra, 1-3 Feb 1988, Institution of Engineers Australia, pp: 201-205.

Commonwealth of Australia (2015), Stormwater management in Australia. The Senate. Environment and Communications Reference Committee. Australian Government, Canberra.

Coombes P.J. (2012), Effectiveness of rainwater harvesting for management of the urban water cycle in South East Queensland. Report by Urban Water Cycle Solutions for the Rainwater Harvesting Association of Australia and the Association of Rotomoulders Australasia, Carrington, NSW. Available at http://urbanwatercyclesolutions.com.

Coombes (2015), Submission to the Senate Standing Committees on Environment and Communications inquiry into the quantum of stormwater resource in Australia and impact and potential of optimal management practices in areas of flooding, environmental impacts, waterway management and water resource planning. The Senate. Environment and Communications Reference Committee. Australian Government, Canberra.

Cordery, I. (1976), Some effects of urbanisation on streams. Civil Engineering Transactions, The institution of Engineers, Australia, CE18(1), 7-11.

Coutts, A.M., Beringer, J., Jimi, S. and Tapper, N.J. (2009), The urban heat island in Melbourne: drivers, spatial and temporal variability, and the vital role of stormwater. Stormwater 2009. Stormwater Industry Association.

Delleur, J.W. (2003), The evolution of urban hydrology: past, present and future. Journal of Hydraulic Engineering, 129: 563-573.

Espey, W.H. and Winslow, D.E. (1974), Urban flood frequency characteristics. Journal of the Hydraulics Division. American Society of Civil Engineers, 100(HY2), 279-293.

Ferguson, B.K. and Suckling, P.W. (1990), Changing rainfall-runoff relationships in the urbanizing Peachtree Creek Watershed, Atlanta, Georgia. Water Resources Bulletin, 26(2), 313-322.

Grimmond, C.S.B. and Oke, T.R. (1986), Urban Water Balance: 2. Results From a Suburb of Vancouver, British Columbia. Water Resources Research, 22(10), 1404-1412.

Hardy, M.J., Coombes, P.J. and Kuczera, G.A. (2004), An investigation of estate level impacts of spatially distributed rainwater tanks, 2004 International Conference on Water Sensitive Urban Design, Engineers Australia, Adelaide, Australia.

Harris, E.E. and Rantz, S.E. (1964) Effect of urban growth on streamflow regimen of Permanente Creek Santa Clara County, California: Hydrological effects of urban growth. Geological survey water-supply paper 1591-B. US Department of the Interior.

Heejun, C. (2007), Comparative streamflow characteristics in urbanizing basins in the Portland Metropolitan Area, Oregon, USA. Hydrological Processes, 21(2), 211-222.

Hill, P.I., Mein, R.G. and Siriwardena, L. (1998), How much rainfall becomes runoff: loss modelling for flood estimation. Cooperative Research Centre for Catchment Hydrology, Melbourne.

Hill, P.I., Graszkiewicz, Z., Taylor, M. and Nathan, R. (2014), Loss models for catchment simulation: Phase 4. Analysis of Rural Catchments. Australian Rainfall and Runoff Report No. P6/S3/016B, Engineers Australia.

Hollis, G.E. (1988), Rain, Roads, Roofs and Runoff: Hydrology in Cities. Geography 73(1), 9-18.

Hollis, G.E. (1975), The effect of urbanization on floods of different recurrence interval Water Resources Research, 11: 431-435.

Hollis, G.E. and Ovenden, J.C. (1988a), One year irrigation experiment to assess losses and runoff volume relationships for a residential road in Hertfordshire, England. Hydrological Processes, 2: 61-74.

Hollis, G.E. and Ovenden, J.C. (1988b), The quantity of stormwater runoff from ten stretches of road, a car park and eight roofs in Hertfordshire, England during 1983. Hydrological Processes, 2: 227-243.

Jacobson, C.R. (2011), Identification and quantification of the hydrological impacts of imperviousness in urban catchments: a review. Journal of Environmental Management, 92: 1438-1448.

Ku, H.F.H., Hagelin, N.W. and Buxton, H.T. (1992), Effects of urban storm-runoff control on Ground-water recharge in Nassau County, New York, Ground Water, 30(4), 507-514.

Leopold, L.B. (1968), Hydrology for urban land planning - A guidebook on the hydrologic effect of urban land use. U.S Geological Survey, Circular. 554: 18, available at http://pubs.usgs.gov/circ/1968/0554/report.pdf

Lerner, D.N. (2002), Identifying and quantifying urban recharge: a review. Hydrogeology Journal, 10: 143-152.

McMahon, G., Bales, J.D., Coles, J.F., Giddings, E.M.P. and Zappia, H. (2003), Use of stage data to Characterize hydrologic conditions in an urbanizing environment. Journal of the American Water Resources Association, 39(6), 1529-1546.

Mein, R.G. and Goyen, A.G. (1988), Urban runoff. Civil Engineering Transactions, CE30(4), 225-238.

Melbourne Water (2012), Planning for sea level rise. 5 June 2012. Available at http://www.melbournewater.com.au/Planning-and-building/Forms-guidelines-and-standard-drawings/Documents/Planning-for-sea-level-rise-guidelines.pdf

Mitchell, V.G., McMahon, T.A and Mein, R.G. (2003), Components of the Total Water Balance of an Urban Catchment. Environmental Management, 32(6), 735-746.

Packman, J.C. (1981), Effects of catchment urbanization on flood flows. ICE, Flood studies report - Five Years On. Proceedings of a conference organized by the Institution of Civil Engineers, held in Manchester, 22-24 July. Thomas Telford Ltd, London, pp: 121-129.

Parker, A. (2013), Melbourne heat island contamination of temperature trend maps in Victoria, Australia. Journal of Non-linear Engineering, 2(1-2), 39-62.

Prosdocimi, I., Kjeldsen, T.R. and Miller, J.D. (2015), Detection and attribution or urbanization effect on flood extremes using nonstationary flood-frequency models. Water Resources Research, 51(6), 4244-4262.

Ronan, N.M. (2009), Future flood resilience - Victoria's next strategy. Albury, Floodplain Management Association.

Schueler, T. (1994), The importance of imperviousness. Watershed Protection Techniques, 1(3).

Simmons, D.L. and Reynolds, R.J. (1982), Effects of urbanisation on base flow of selected south-shore streams, Long Island, New York. Water Resources Bulletin, 18(5), 797-805.

Smith, B. K., Smith, J.A., Baeck, M.L., Villarini, G. and Wright, D.B. (2013), Spectrum of storm event hydrologic response in urban watersheds. Water Resources Research, 49: 2649-2663.

Stephenson, D. (1994), Comparison of the water balance for an undeveloped and a surburban catchment. Hydrological Sciences Journal, 39(4), 295-307.

Tarr, J.A. (1979), The separate vs. combined sewer problem: a case study in urban technology design choice. Journal of Urban History, 5(3), 308-339.

Tholin, A.L. and Keifer, C.J. (1959), The hydrology of urban runoff. Journal of the Sanitary Engineering Division, 85(SA2), 47-106.

Walsh C.J., (2018), The water we should be using first - stormwater runoff from roofs and roads, https://urbanstreams.net/tools/melbrunoff/.

Walsh, C.J., Roy, H., Feminella, J.W., Cottingham, P., Groffman, P.M. and Morgan, R. (2005), The urban stream syndrome: current knowledge and the search for a cure. Journal North American Benthological Society, 24(3), 706-723.

Walsh, C.J., Fletcher, T.D. and Burns, M.J. (2012), Urban stormwater runoff: a new class of environmental flow problem. PLOSone DOI: 10.1371/journal.pone.0045814.

Western, A.W. and Grayson, R.B. (2000), Soil moisture and runoff processes at Tarrawarra. In: Grayson, R. G. and Blöschl, G. (eds) Spatial Patterns in Catchment Hydrology: Observations and Modelling. Cambridge University Press, pp: 209-46.

Wong, T., Breen, P.F. and Lloyd, S. (2000), Water sensitive road design - design options for improving stormwater quality of road surfaces. Cooperative Research Centre for Catchment Hydrology, Melbourne.

Woolmington, E. and Burgess, J.S. (1983), Hedonistic water use and low-flow runoff in Australia's national capital, Urban Ecology, 7(3), 215-227.

Chapter 3. Philosophy of Urban Stormwater Management

Peter Coombes, Steve Roso

Chapter Status	Final
Date last updated	14/5/2019

3.1. Introduction

Urban stormwater management is historically described as the hydraulic design of urban drainage networks that safely conveys stormwater runoff to receiving environments. The industry's approach to urban water management in Australia has changed significantly since the establishment of centralised and separate water supply, stormwater and wastewater paradigm in the 1800s.

Urban water management evolved over time to include waterways protection, mitigation of stormwater quality, use of Water Sensitive Urban Design (WSUD), Integrated Water Cycle Management (IWCM), Water Sensitive Cities (WSC), Integrated Water Management (IWM), and many other approaches. Although these approaches are relatively new, they have wide adoption and support in legislation and policies for water management throughout Australia. Similar changes in approach to urban stormwater management in other countries include Sustainable Urban Drainage Systems (SuDS) (Bozovic et al., 2017) and Low Impact Design (LID) (USEPA, 2008). Consequently, the approach to urban stormwater management includes water supply and is based on retention and conveyance of stormwater runoff to meet multi-purpose design objectives that enhance livability of urban areas, mitigate nuisance, and avoid damage to property and loss of life.

3.2. The Journey from 1987 to 2016

Australia has experienced considerable improvement in urban water management since the 1800s, supported and underpinned by publications such as ARR (PMSEIC, 2007). Stormwater drainage in Australia evolved from combined sewers that rapidly discharged the accumulated rubbish, sewage, sullage and stormwater from streets to waterways (Armstrong, 1967; Lloyd et al., 1992). The impact on waterways and amenity of urban settlements drove the separation of sewage and stormwater infrastructure. Filling of swamps and development of contributing catchments to accommodate population growth resulted in frequent flooding of early settlements. Drainage solutions emerged to avoid stagnant water, local flooding and health impacts in urban areas. Nation building works programs during economic depressions (for example in 1890 and 1920) and following wars provided large scale drainage infrastructure throughout Australia.

The ARR 1987 guideline focused on collection and conveyance of peak stormwater flows in drainage networks. The guideline's advice on hydrologic and hydraulic analysis was consistent with the emerging computer age and hand calculation while programmable calculator and computer methods were discussed. The increasing complexity of the different methods and an associated requirement for use of computers was highlighted.

Use of statistical design rainfall bursts was recommended to calculate inflows to drainage networks and the Rational Method was described as the best known method for estimation of urban stormwater runoff. The main objective of urban drainage was to convey stormwater from streets and adjoining properties without nuisance from minor rain events, and to avoid property flooding and associated damage from major rain events (the minor/major design approach).

In contrast to the introductory comments, urban drainage was presented as a prescriptive approach using pipes to convey minor flows, with streets, open space and trunk drains used to transport major flows. Trunk drainage was described to include designs for open channels, detention and retention basins to control peak discharge, and bridges. While urban stormwater management was presented and interpreted as a drainage approach, Chapter 14 in ARR 1987 highlighted how urban drainage solutions should also:

- limit pollutants entering receiving waters;
- consider water conservation;
- integrate overall planning schemes;
- be based on measured or observed real system behaviour;
- be viewed in relation to the total urban system; and
- maximise benefits to society.

Drainage solutions solely focused on developed catchment and were mostly designed by engineers. The simplicity of methods for estimating stormwater runoff implied accuracy and certainty of design performance for many users. Urban water management further evolved in the mid-1990s to cover protection of waterways, mitigation of urban stormwater quality, WSUD (Whelans and Maunsell, 1994), IWM and IWCM (Coombes and Kuczera, 2002) approaches. Nevertheless urban stormwater runoff creates complex impacts on urban stream ecosystems and receiving waterways (Walsh et al., 2005; Paul and Meyer, 2001). Increases in runoff volumes and rates from urban areas (flow regimes) contribute to degradation of riparian ecosystems and promotes geomorphic changes within urban streams (Walsh et al., 2012). Although these approaches are relatively new, they have subsequently gained widespread adoption and support throughout Australia. To support this evolution, Engineers Australia published 'Australian Runoff Quality – A Guide To Water Sensitive Urban Design' in 2006 (Engineers Australia, 2006).

The acceptance of WSUD, IWCM and related approaches is manifested in three significant ways:

- the development of benchmark projects (e.g; Lynbrook estate (Lloyd et al, 2002), Fig Tree Place (Coombes et al., 2000) and Little Stringy Bark Creek (Walsh et al., 2015)) that provided evidence that these new approaches were successful;
- the creation of local policies and plans for integrated water management; and
- the adoption of policies for sustainable water management by state and federal governments.

Recent droughts, such as the 'millennium drought' also triggered many other changes in the urban water sector, largely associated with water conservation, harvesting, recycling and reuse (Aishett and Steinhauser, 2011).

Urban areas are complex systems that are subject to dynamic interaction of economic, social, physical and environmental processes across time and space (Forrester, 1969;

Coombes and Kuczera, 2003; Beven and Alcock, 2012). Continuous intervention is required to renew urban economic, technical and social structures to maintain human welfare and protect ecosystem services (Forrester, 1969; Meadows, 1999). Understanding these processes into the future also encounters the uncertainty created by non-stationary data that describes past processes. Design and analysis processes should include distributed approaches to account for the time based dynamics of essential data. The integrated nature of contemporary water management approaches is different to the objectives and design solutions envisaged in 1987. Urban water management is now required to consider multiple objectives (e.g. resilience, livability, sustainability and affordability) and the perspective of many disciplines. Advances in computing power, more available data and associated research also allows the analysis of increasingly complex systems to understand the trade-offs between multiple objectives (Coombes and Barry, 2014). Design of urban water management seeks to integrate land and water planning. Use of more comprehensive datasets revealed a greater range of potential outcomes that needs understanding to develop integrated solutions.

According to Argue (2017), the urban designer aims at managing the impact of urban stormwater runoff 'at source' and at multiple scales by retaining stormwater in landscapes and soil profiles, rainwater harvesting and disconnecting impervious surfaces from drainage networks (Poelsma et al., 2013). Consistent with the philosophy of source control and systems analysis, stormwater runoff is now seen as an opportunity and is valued as a resource (Clarke, 1990; Mitchell et al., 2003; McAlister et al., 2004). Modern design criteria may include analysis of the volumes, timing and frequency of stormwater runoff to determine peak flow rates, water quality and requirements to mimic natural flow regimes to protect waterway health (Walsh, 2004).

3.3. Evolving Opportunities and Challenges

Urbanisation generates dramatic changes within the natural water cycle. Impervious surfaces and directly connected drainage infrastructure decreases evapotranspiration and infiltration to soil profiles. This increases the volume and frequency of stormwater runoff and reduces baseflows; which can create flooding and affect waterway health. Drainage strategies that are reliant on conveyance can transfer additional stormwater runoff and pollutant loads generated by urban areas to other locations. The different regional scale responses within a river basin and a linked urban catchment are presented in Figure 9.3.1.

The impervious surfaces and hydraulically efficient infrastructure associated with urban catchments increases the magnitude and frequency of stormwater runoff whilse reducing the infiltration to soil profiles and subsequent baseflows in waterways. The accumulation of stormwater flows within urban catchments is highlighted. The first response at A is the (undisturbed) ecosystem upstream from urban impacts, the second response at B includes the impact of water extraction to supply the urban area (changed flow regime in rivers created by water supply) and the third response at C includes water discharges from the urban catchment (changed flow and water quality regime from both stormwater runoff and wastewater discharges) into the river basin.

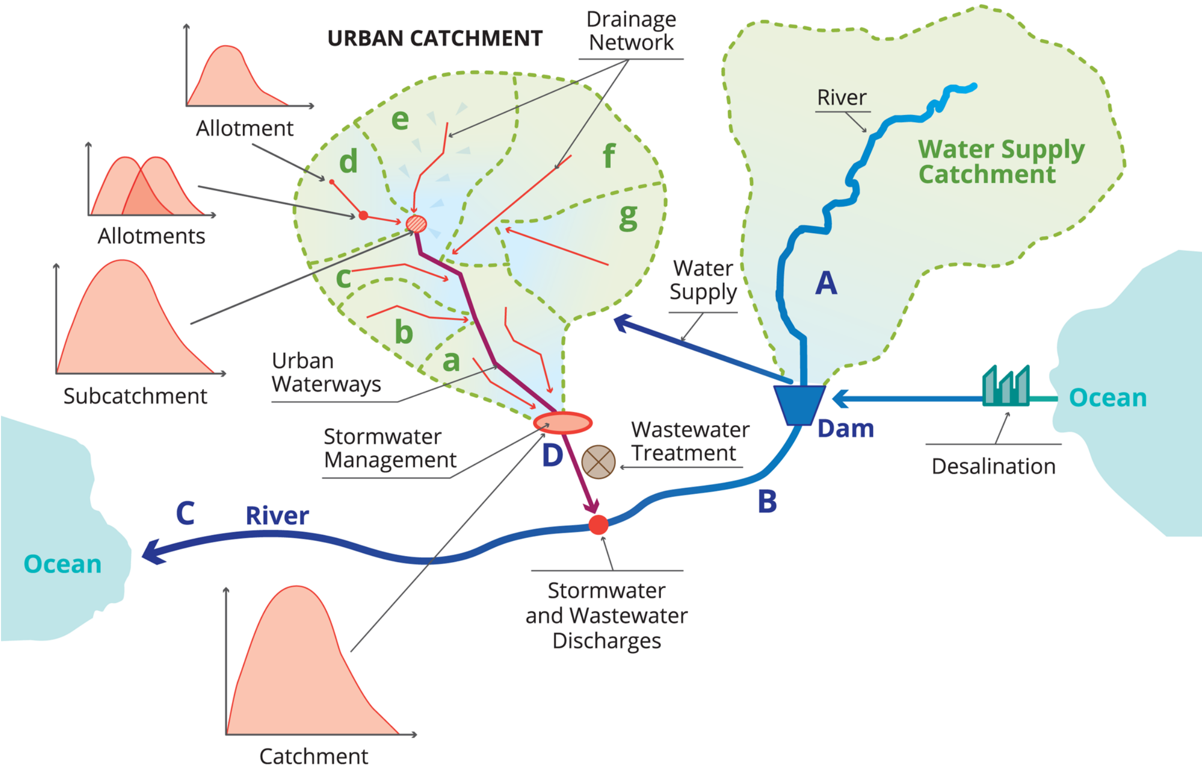


Figure 9.3.1. Schematic of Traditional Urban Catchments and Cumulative Stormwater Runoff Processes

Figure 9.3.1 demonstrates analysis and solutions at point D at the bottom of urban catchments; it can exclude understanding of impacts within the urban catchment (sub-catchments a-h) and external impacts to the river basin at B and C. Traditional analysis of urban catchments is from the perspective of rapid discharge and accumulation of stormwater via drainage networks (in sub-catchments a-h) with flow and water quality management at the bottom of the urban catchment (D) using retarding basins, constructed wetlands, and stormwater harvesting. However, the benefits for flood protection, improved stormwater quality, and protection of the health of waterways from this approach do not occur within the urban catchment upstream of point D.

Figure 9.3.1 also highlights how distributed land uses (allotments or properties) produce hydrographs of stormwater runoff into the street drainage system. This system accumulates stormwater runoff from multiple inputs, creating progressively larger volumes of stormwater runoff, which ultimately flows into urban waterways or adjoining catchments (Pezzaniti et al., 2002). This process results in significant changes in volume and timing of stormwater discharge to downstream environments.

There has been an emerging understanding that this issue can be solved by viewing urban stormwater as an opportunity to supplement urban water supplies and enhance the amenity of urban areas (Mitchell et al., 2003; Barry and Coombes, 2006; Wong, 2006). This includes development of green infrastructure and microclimates that reduce urban heat island effects. Urban catchments with impervious surfaces are substantially more efficient than conventional water supply catchments in translating rainfall into surface runoff. Rainwater and stormwater harvesting can extend supplies from regional reservoirs and the restoration of environmental flows in rivers subject to extractions for water supply (Coombes, 2007). These insights are consistent with earlier applied research by Goyen (1981) that both volumes and peak flows of stormwater runoff are required to design stormwater infrastructure, and the local property scale is the building block of cumulative rainfall runoff processes (Goyen, 2000). Reducing urban stormwater runoff volumes via harvesting and retention in upstream catchments can also decrease stormwater driven peak discharges and surcharges in wastewater infrastructure (Coombes and Barry, 2014).There has been an emerging understanding that this issue can be solved

Changes in land use, climate, increased density of urban areas and decline in hydraulic capacity of aging drainage networks can result in local flooding and damage to property. Climate change is expected to reduce annual rainfall and generate more intense rainfall events in a warming climate (PMSEIC, 2007; Wasko and Sharma, 2015). This will intensify the challenges of providing secure water supplies and mitigating urban stormwater runoff. There may also be the need to replace stormwater conveyance networks installed during post-war urban redevelopment that are nearing the end of useful life. In this situation, the capacity of an aging network or increased runoff from increasing development density can be supplemented by source control measures and integrated solutions (Barton et al., 2007). Integrated solutions and flexible approaches to design can avoid costly replacement of existing infrastructure.

Flood management issues for many urban areas are driven by runoff discharged towards waterways (overland flooding) rather than from flood flows originating at waterways (fluvial flooding). There is a need to consider more extensive range of stormwater runoff events, from frequent to rare or extreme and the associated impacts on urban environments (Weinmann, 2007). Management of these flood related impacts require integrated management of the full spectrum of flood events (Figure 9.3.2).

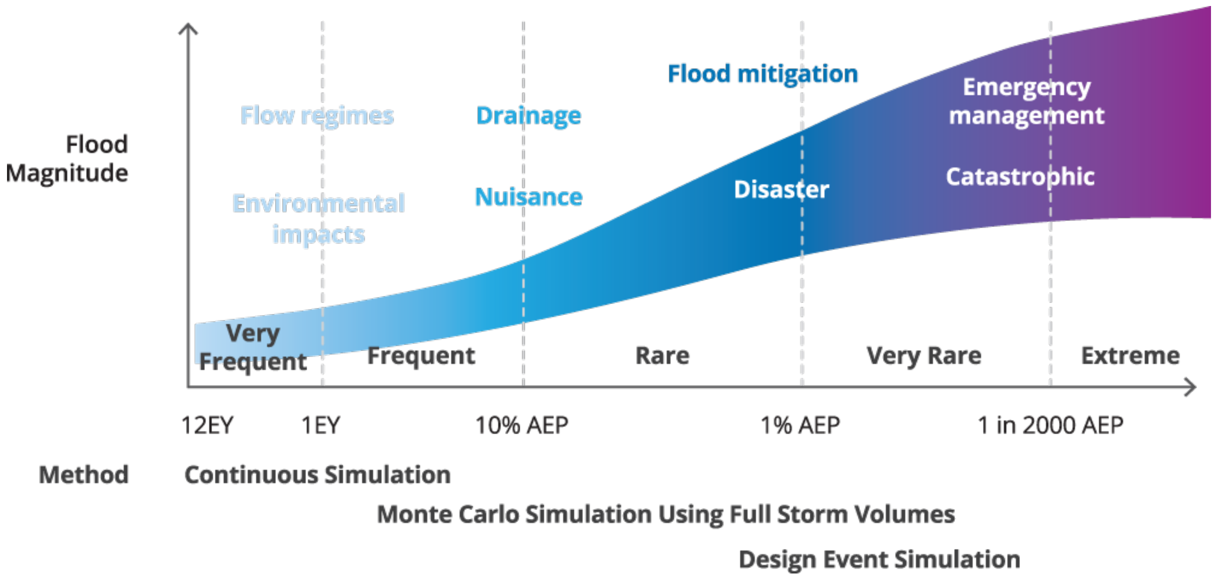


Figure 9.3.2. The Full Spectrum of Flood Events (Adapted from Weinmann (2007))

Figure 9.3.2 highlights the evolving methods of analysis, including continuous simulation and Monte Carlo simulation of full storm volumes that are likely to be required to account for the full spectrum of rainfall events as defined by Exceedance per Year (EY) or Annual Exceedance Probability (AEP). The definition of rain events is currently a mix of assumptions regarding frequency and magnitude that is clarified in this version of ARR to allow effective advice on design of stormwater management schemes. This includes development of green infrastructure and microclimates with reduction of urban heat island effects.

Strategic use of water efficiency, rainwater, stormwater and wastewater at multiple scales can supplement the performance of centralised water supply systems to provide more sustainable and affordable outcomes (Victorian Government, 2013). These integrated strategies diminish the requirement to transport water, stormwater and wastewater across regions with associated reductions in costs of extension, renewal and operation of infrastructure (Coombes and Barry, 2014). This leads to decreased requirement to augment regional water supplies and long run economic benefits. These strategies also focus on restoring more natural flow regimes in waterways and they will be beneficial in reducing remedial works in waterways and will provide reduction in size or footprint of quality treatment measures (Poelsma et al., 2013).

Current approaches to stormwater management include separate design processes and infrastructure for flooding, drainage and water quality. Jurisdictional and institutional boundary conditions are often imposed on analysis (Brown and Farrelly, 2007; Daniell et al., 2014). Integrated design includes solutions that meet multiple objectives, the catchment boundaries of each element and aims to avoid redundant infrastructure. Realisation of these benefits is dependent on integrated design approaches that account for changes in the timing and volumes of stormwater runoff, and respond to multiple objectives. Analysis of the economic benefits of integrated designs and drainage networks should be evaluated across an entire system from the perspective of whole of society. The methods and objectives for estimating urban stormwater runoff and the design of pipe drainage networks from 1987 do not include these additional considerations.

A challenge to integrated solutions is presented by engineering and economic methods of estimating performance that are reliant on average assumptions and judgements as inputs to empirical methods of estimating performance. Consequently, optimum design based on average assumptions and model approximations may not represent the actual integrated response of a project.

Educated empirical input assumptions and estimation processes can be reasonably approximated as generic processes for known historical and static problems (Kuczera et al., 2006; Weinmann, 2007). However, these processes may not replicate performance of multiple solutions within a system. For example, with respect to intersection of local water cycle solutions with town planning processes and regional infrastructure and, therefore, cannot understand or value a system that changes runoff behaviour from the smallest distributed scales (from the ‘bottom up’) (Argue, 2017; Coombes and Barry, 2014; Goyen, 2000). For example, cumulative actions at the smallest scale, such as retaining stormwater in the soil profile on each property can produce significant changes in responses throughout urban systems as shown in Figure 9.3.3.

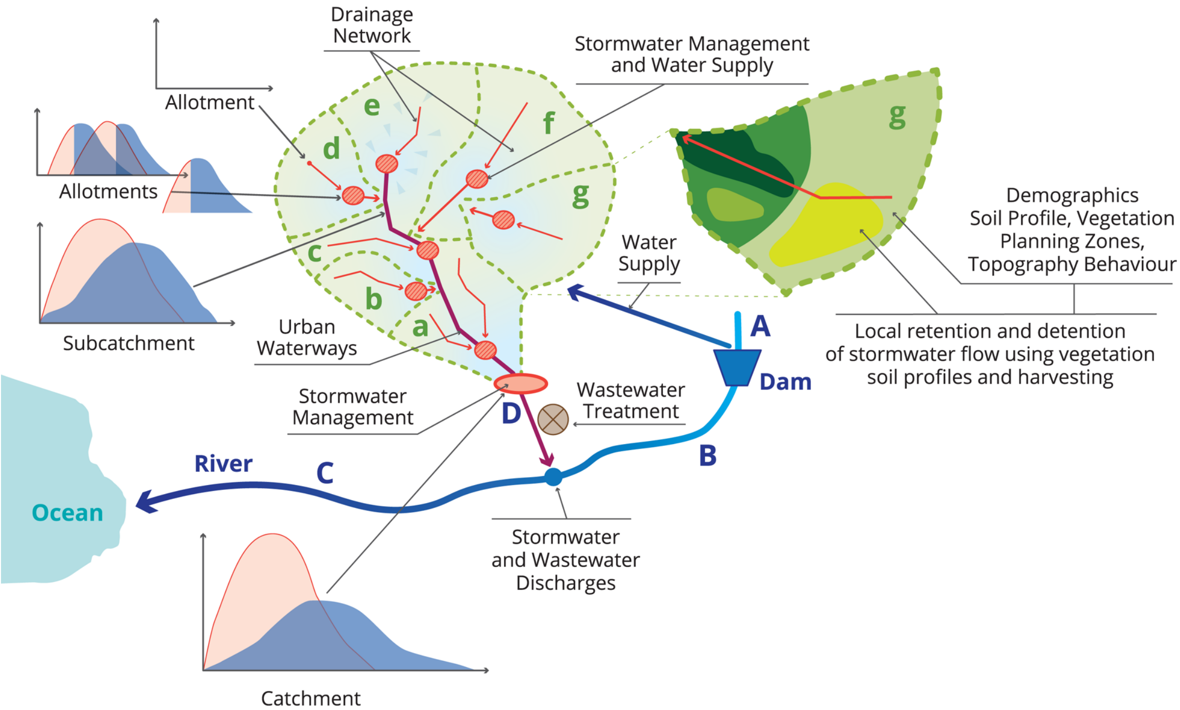


Figure 9.3.3. Cumulative Impacts of Distributed Management

It also follows that historical ‘top down’ design processes may not evaluate distributed processes because a small proportion of the available data may be simplified as whole of system average or fixed inputs (such as a runoff coefficient and average rainfall intensity). Thus, the signals of linked distributed performance (such as local volume management measures) in a system are smoothed or completely lost by partial use of data as averages and by the scale of analysis. Therefore, there is no direct mechanism to capture cascading changes in behaviours throughout a system. This can lead to competing objectives (For example: local versus regional), inappropriate solutions and information disparity such as provision of a wetland and retarding or detention basin downstream of an urban area when management is required within the urban area to protect urban amenity, stream health and avoid local flooding. This paradox can only be resolved through a broader analysis framework which recognises location based principles of proportionality and efficient intervention.

For example, consider the connectivity of contemporary water cycle networks presented in Figure 9.3.4.

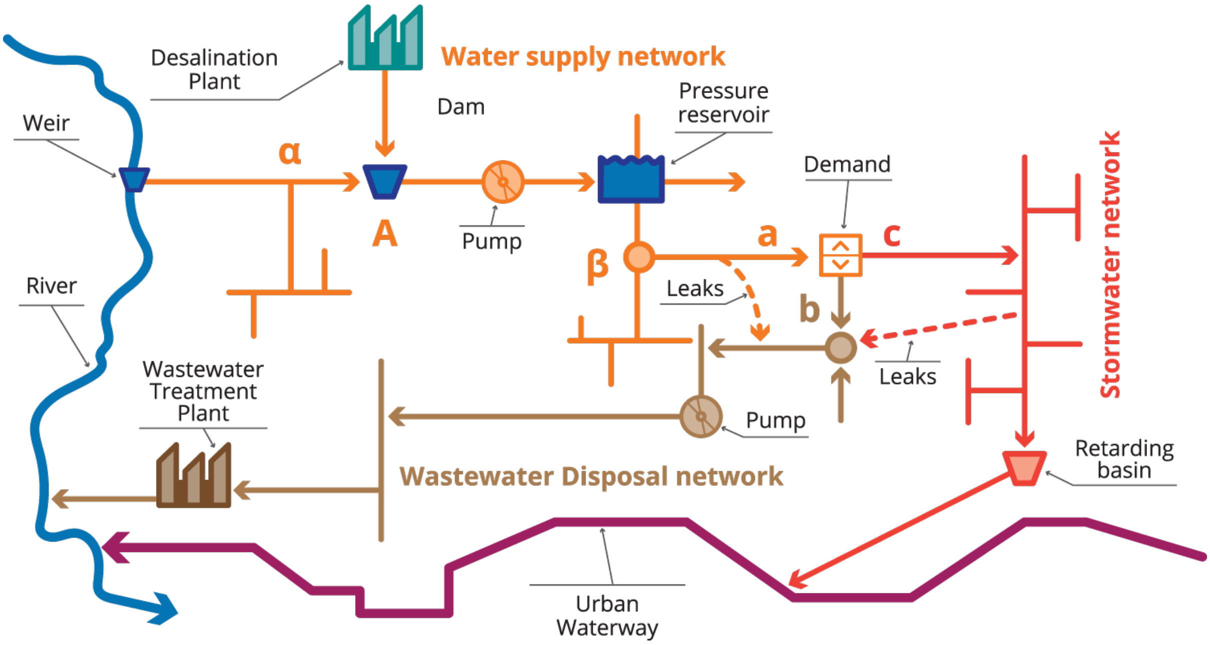


Figure 9.3.4. Schematic of the Connectivity of Urban Water Networks

Figure 9.3.4 shows that an input, or extraction at any point α or β, or an increase in water storage in a reservoir, at location A, will have some influence on flows and capacities at many other points in the system. These in turn, will translate into changes in performance and costs across the linked networks of infrastructure. Similarly, changes in behaviour (demand) at any point in the system will generate different linked impacts a, b and c on water, wastewater and stormwater networks respectively. Analysis and design of integrated solutions needs to account for the linked dynamic nature of the urban water cycle and demography. The inclusion of rainwater and stormwater harvesting, and wastewater reuse further increases the level of connectivity of urban water networks.

The historical practice for estimation of stormwater runoff rates and the design of drainage (conveyance) infrastructure is based on a methodology where all inputs, other than rainfall, are fixed variables. The fixed values of the input variables are selected to ensure that the exceedance probability of stormwater runoff is similar to that of regional rainfall statistics. However, catchments that contain cascading integrated solutions involving retention, slow drainage, harvesting of stormwater and disconnection of impervious surfaces require enhanced design methods (Kuczera et al., 2006; Wong et al, 2008; Coombes and Barry, 2008). These emerging methods for analysis and design of integrated solutions include the following considerations:

- Long sequences of rainfall that include full volumes of storm events are required to generate probabilistic designs of integrated solutions;
- Peak rainfall events may not generate peak stormwater runoff from projects with integrated solutions;
- The frequency of peak rainfall may not be equal to the frequency of peak stormwater runoff from integrated solutions;
- Stormwater runoff from urban catchments is influenced by land use planning, and the connectivity and sequencing of integrated solutions across scales;

- The probability distribution of the parameters that influence the performance of the integrated solutions (for example human behaviour, rainfall and soil processes) and the ultimate stormwater runoff behaviour are unknown for each project;

- Integrated solutions often meet multiple objectives (for example water supply, stormwater drainage, management of stormwater quality, provision of amenity and protection of waterways) and are dependent on linked interactions with surrounding infrastructure; and

- We should be mindful that the limitations of design processes are not always apparent and diligence is required to ensure that substantial problems are avoided.

In this situation, continuous simulation using historical or synthetic sequences of rainfall in a Monte Carlo framework may be required to understand the probability of stormwater runoff and the design of infrastructure (Kuczera et al., 2006; Weinmann, 2007). There are approximately 20,000 daily rainfall records with sufficient continuous rainfall records (more than 3,500) to allow continuous simulation using real or synthetic continuous rainfall records. Similarly, the designer can use ensembles of full volumes design storm event to test an integrated design solution. Assumptions and methods of analysis imposed by approval authorities in accordance with ARR 1987 can constrain the use of more appropriate analysis techniques required for better understanding the behaviour of integrated solutions. Similarly, a default requirement by approval authorities for drainage (conveyance) networks that are designed using peak storm bursts alone can limit the adoption of innovative and integrated solutions.

A combination of event based estimation techniques, directly or indirectly, may not reliably produce probabilistic design of drainage, water quality, and water or wastewater infrastructure within integrated strategies. While use of best available event based design approximations are an accepted default or deemed to comply approach for design of infrastructure, there is a need for more advanced methods for design of integrated solutions.

The absence of an integrated approach to design and planning in stormwater catchments may lead to missed opportunities and poor investment decisions, which ultimately results in higher costs with diminished social and environmental benefits (Coombes, 2005). Estimation of stormwater runoff and design of drainage (conveyance) networks for mitigation of urban flooding needs to be enhanced to provide integration with water cycle management within a systems framework.

The definition and purpose of minor or major drainage system is unclear in the context of modern approaches to water cycle management. Replacement of minor or major drainage descriptions with a definition of managing nuisance or disaster respectively, would provide a clearer focus on the relative importance of both concepts. To avoid nuisance, one may be too focused on a prescriptive drainage approach to the minor system. A well-designed major system to avoid disaster is likely to allow more opportunity for integrated solutions that will also mitigate nuisance. We also need to be cognisant that water supply and stormwater quality options can also assist in avoiding disaster and mitigating nuisance.

3.4. Urban Flooding

Urban flooding may include overland (pluvial) flooding and fluvial flooding (river and creek flows). This distinction can be important as the two types of flooding have different behaviours that may require particular analysis and management approaches.

3.4.1. Overland Flooding

Overland flooding is typically generated by short durations (minutes to hours) of intense rainfall on small catchments up to approximately 1 km^2 in area. This rainfall causes significant concentrations of surface runoff at low points and depressions throughout the urban topography. These concentrations of flows continue downslope and discharge into larger natural waterways with defined banks such as creeks, rivers or lakes where flows become fluvial in character.

Overland flooding can be responsible for significant damage. Adequate major flow paths must be provided or retained to manage these events. A stormwater management strategy is required that includes systematic identification of overland flow paths and design practices that recognise and respond to overland flood risks. Simple design practices such as slightly elevating property and floor levels above the surrounding terrain can effectively eliminate most overland flood risks.

Approaches to analysis have been developed in recent years to assist identification of overland flow paths that involve use of two dimensional hydrodynamic models where real and design rainfall events are applied throughout sub-catchments. These methods use digital terrain models of land profiles that are usually derived from LiDAR and aerial photogrammetry information. Hydrodynamic models can predict the accumulation of runoff across these surfaces and the generation of concentrated flows. Depth and velocity depth thresholds can be applied to model outputs and mapped spatially to allow identification of the most significant accumulation of flow. A map of a fluvial flow path prepared using a two dimensional hydraulic model is presented in Figure 9.3.5.

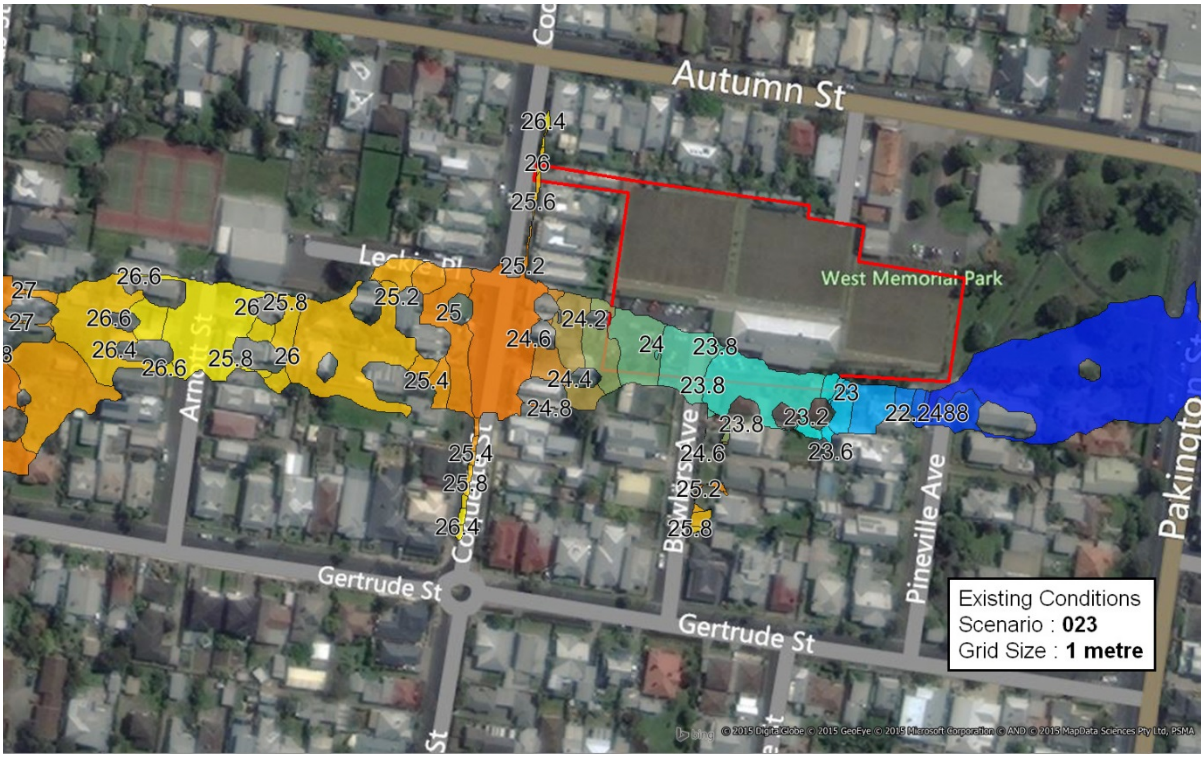


Figure 9.3.5. Example Overland Flow Path Map Generated Using a Two Dimensional Model.

This modelling approach is complex and is undertaken by a designer with suitable experience to ensure reliable outcomes. However, successful application of this method can be efficient and reveal a range of important stormwater management issues, including overland flow paths.

Approaches to analysis with less complexity may be more practical for smaller areas or simpler stormwater management strategies. This may involve the capture of detailed ground survey and inspection of the data by a suitably experienced designer to manually estimate the location of low points and likely flow paths. Simple hydrologic and hydraulic calculations (refer to Book 9, Chapter 5) could then be applied to estimate the depth and width of stormwater at regular intervals throughout overland flow paths.

Caution should always be employed when interpreting the mapping of results for stormwater flows and inundation as there may be significant uncertainties about the results caused by:

- obstructions to flow paths such as buildings and fences;
- rapidly changing flow conditions throughout a flow path;
- limitations in the accuracy of survey information; and
- limited opportunity for calibration.

The application of two dimensional modelling approach produces results that reveal hydrologic uncertainty due to use of the hydraulic model to simulate the natural physical processes of stormwater flows. These results may be in contrast to empirical or statistical relationships between rainfall and runoff that are used to estimate stormwater runoff in some traditional hydrologic modelling software.

Identification of overland flow paths allows development of stormwater management strategies. These may include:

- mapping of flooding to promote public awareness of flood risks;
- education about flood risks;
- investigation of potential upgrades to stormwater management networks; and
- building and development controls.

Flood warning emergency systems are usually inappropriate for overland flooding, as the potential warning times are too short. However, incorporation of overland flooding information with radar rainfall forecasts may assist in providing emergency management warnings.

Building and development controls should include provisions that prevent the erection of new buildings within overland flow paths or set minimum floor levels that are deemed safe. Other building controls may also require measures that minimise potential blockage and obstruction to flows within effected building envelopes. Application of these controls to particular sites may require detailed site-based flood investigations to more accurately estimate flood levels and behaviours.

A freeboard allowance above a calculated flood level is applied to determine the minimum level of infrastructure such as a habitable dwelling. Freeboard is required to account for the uncertainties that are inherent in the calculation of flooding. A typical minimum value of 0.3 m above a flood surface is suggested. However, this value can be varied to account for local factors such as the sensitivity of specific infrastructure to flood damage and expected uncertainty in estimates of flood level estimates for a site. Uncertainty about flood levels are variable and dependent on many factors including the nature of the catchment and the cross-sectional profile across the flow path.

Freeboard should not be used to protect against measurable uncertainties for example risk of blockage and climate change. If these risks are a concern for the site then they should be explicitly incorporated into the basic flood level estimates before freeboard is applied.

3.4.2. Fluvial Flooding (River and Creek Flooding)

Fluvial flooding is often referred to as river and creek flooding, and is generally caused by long durations (hours to days) of intense rainfall across large catchments. These catchments range in area from 1 km^2 to many thousands of km^2. Excess runoff from these catchments accumulates and is concentrated as flows in creeks, rivers and lakes that have natural features such as a main channel and defined banks. Stormwater escapes the main channel at locations where hydraulic capacity exceeded and caused inundation of surrounding land. This flooding can occur across vast areas of flat or low-lying terrain. The extent of flooding can be quite narrow and well defined at locations where the natural topography is incised. Fluvial flooding is generally easier to analyse than overland flooding because the channels are more readily identified and represented using computer models.

This type of flooding is natural. However, careful urban planning is required to avoid substantial damage to infrastructure and property. Fluvial flooding is recognised as one of the most significant natural hazards in Australia that is responsible for a significant proportion of economic losses and damage to property. Therefore, fluvial flooding has been the target of significant government programs for mapping of flood hazards and implementation of measures that mitigate potential economic losses and damage to property.

Fluvial flooding is a constraint to urban stormwater management that needs to be understood as it may heavily influence the type's solutions that are proposed. Numerical methods for the estimation of flood behaviour and identification of fluvial flood hazard are well established and tested. These methods are described in Book 6, Book 7 and Book 8.

The management of hazards created by fluvial flooding differs from overland flooding as the quantity of floodwaters can be much greater and therefore more difficult to control and contain using physical changes to the floodplain. It is often preferable and more cost effective to avoid these hazards using a process of careful urban planning. This is best achieved by the use of strategic plans and a suite of flood related building and development controls.

Public flood awareness mapping, flood education, flood mitigation and flood warning emergency systems become more important where development has already occurred within parts of the floodplain subject to fluvial flooding. Catchments that generate fluvial flooding are often large and the lag between rainfall and runoff can be sufficient which increases the feasibility of flood warning and emergency management strategies.

3.4.3. The Overland and Fluvial Interface

There is often an interface zone within catchments where both fluvial and overland flow paths may exist and differentiation between the two types of flowpaths becomes subjective. For example, a small gully drains through a town directly into a major creek as presented in Figure 9.3.6.

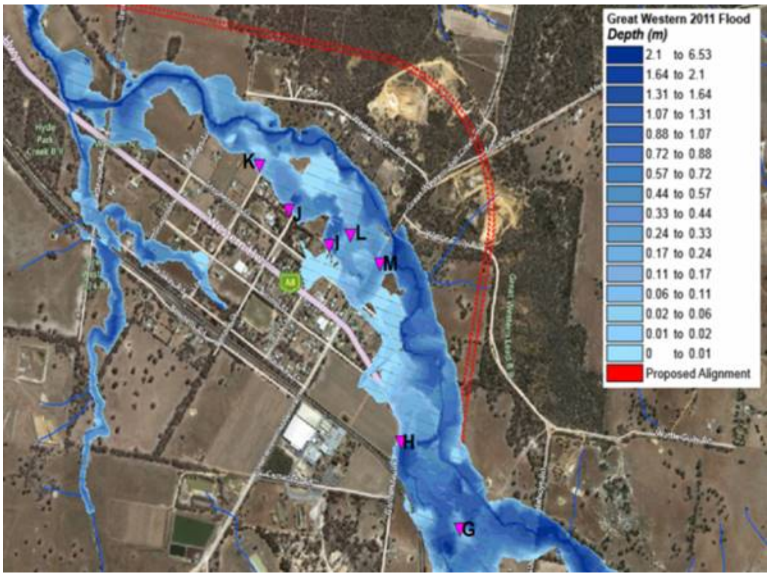


Figure 9.3.6. Example of Fluvial Flow Path with Interface with Overland Flow Path

Analysis of stormwater management strategies at the interface zone requires first principles assessment of management techniques from the perspective of both overland and fluvial flooding.

Both types of flooding can occur simultaneously. However, this is unlikely since the rainfall mechanisms that typically cause each type of flood are different. It is more likely that overland and fluvial flooding will occur at different times and possibly not during a single rainfall event. This complex behaviour can confuse attempts to communicate flood risks and implement management strategies. Confusion also arises when insurance claims are made for loss and damage because the decision to pay a claim sometimes relies upon whether the flooding was overland or fluvial in nature. In addition, the insurance industry has begun to offer fluvial flood insurance cover, which may reduce this problem in future. Nevertheless, it is important for practitioners to recognize the potential for both forms of flooding and carefully assess flood behaviour at each site and for each flood event from first principles.

3.5. Conveyance Systems

A typical stormwater (drainage) conveyance system must convey a wide range of flows within a confined corridor of land (refer to Book 9, Chapter 5). At the same time the system must meet appropriate standards of flood safety and be delivered for low life-cycle cost. This challenge is best addressed through application of a design approach referred to as a 'major and minor stormwater management system'.

A Major and Minor Stormwater Management System Has Two Parts:

- The minor system manages nuisance. This runoff is conveyed in a manner that maintains safety, minimises nuisance and damage to property. The infrastructure is also provided to avoid potential maintenance problems for example ponding and saturation of designated areas. Importantly, the minor system also includes volume management measures that aim to hold water within urban landscapes and sub-catchments (refer to Book 9, Chapter 4) – these solutions may include ponding of stormwater within a defined area. The minor system must withstand the effects of regular stormwater inundation.

- A major system primarily intended to mitigate disaster. The major system typically includes overland flow paths on roads and through open space, and trunk conveyance infrastructure. This system conveys additional stormwater runoff produced during larger less probable and rarer storm events with the intent of managing the potential for flood disaster. Overland conveyance of stormwater from large events is potentially hazardous due to the velocity and depth of flows, and must be safely contained within a defined corridor of major system flows.

3.5.1. Capacity to Manage Flooding

The overall combined capacity of the major and minor drainage system to manage flooding or inundation needs to be established for each design. This capacity is normally expressed in terms of the exceedance probability of design rainfall, creating a flood that must be contained within the conveyance or drainage system. It is common practice to set the capacity of the major system at a similar exceedance probability as the flood event used for regional flood planning (e.g. 1% AEP discharge).

However, there may be justification to deviate from this practice where a suitable risk assessment identifies the need. For example where the consequences of flooding at a particular location are high, it may be necessary to expand the overall system capacity to cater for more extreme events. This is not commonly required and this type of decision must have regard to the overall life-cycle cost and benefits that a larger capacity system may deliver.

The threshold at which the capacity of the minor system is exceeded and the major system begins to convey runoff is also a matter for consideration at the design stage or for policy makers at the time when preparing local design standards for stormwater management. The capacity of minor system is typically established to manage stormwater events ranging from 50% AEP to 5% AEP. Documentation of these standards can be found in drainage design guidelines prepared by local government and relevant state authorities. No single universally appropriate capacities of minor systems can be applied in practice.

Some factors that may influence the balance between the capacity of major and minor systems are described in Table 9.3.1. These factors may generate a number of different capacity standards for minor systems that account for different locations and jurisdictions.

Table 9.3.1. Factors Influencing the Balance between Capacities of Major and Minor Systems in Design

Factor	Description
Land availability	Sufficient land may be available for major systems to safely convey additional surface flows and reduce the proportion of flows conveyed by minor systems. The use of volume management and WSUD approaches can also change the proportion of flows assigned to minor and major systems.
Local rainfall patterns	In some areas, such as tropical northern Australia, runoff generated by frequent storms may be too large to cost effectively convey using minor systems. Major flow paths will need to be expanded accordingly to manage a proportion of these flows.
Likely level of exposure to the major flow path hazard	Major systems that are highly frequented by people or vehicles, for example in city streets or major motorways, involve greater exposure to floodwaters and corresponding risks. In these cases, it may be appropriate for a greater proportion of runoff to be conveyed in minor systems.

Factor	Description
Physical and downstream constraints	When new stormwater management systems are required for an existing urban area, it may be impractical or cost prohibitive to achieve an ideal capacity and compromise may be required.
Erosion	Natural or otherwise unlined minor systems may be subject to erosion when flow durations and or velocities are too high. If volume management options (as discussed in Book 9, Chapter 4) are not available, then lowering the capacity of the minor systems and forcing a greater proportion of flow into the major system may be one way to manage these effects.
Blockage potential	Where the capacity of minor systems is reduced by a likelihood of blockage with debris, resources should be directed towards safer and more durable surface flow paths within major systems.
Climate change	The expected future increases in short duration rainfall intensities may require appropriate design responses to increase the capacity of minor systems or change the relationship with major systems to maintain current levels of service.

3.5.2. Alignment and Configuration

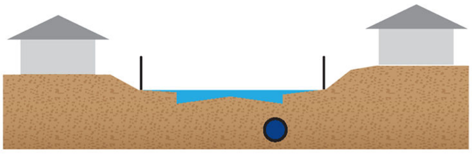
The characteristics of urban form including the layout of roads, location of urban parkland and topography will influence the alignment and configuration of stormwater management networks. It is difficult to modify the stormwater management network after installation. A design process should aim for a long service life. Concept planning for major and minor stormwater management systems should therefore be undertaken carefully as an early task in the design of new urban developments.

The depth and velocity of flows along any proposed surface flow paths are considered when calculating the dimensions of stormwater conveyance corridors and must meet relevant standards for design, safety and maintenance. A design should also ensure that operation of a conveyance network during severe storms does not cause unexpected or catastrophic consequences (for example, an unintended diversion of flows into an adjoining catchment because of blockage or extreme events).

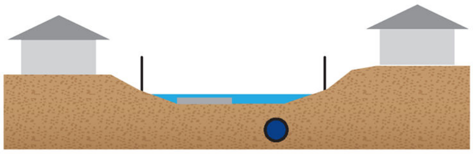
Wherever possible the width of the land corridor set aside for stormwater management should be generous to improve the constructability of the system and reduce the costs of any future renewal and maintenance activities. Opportunities for co-location of stormwater management within urban parklands should be considered. The alignments of stormwater conveyance networks typically follow natural low points to minimise earthworks. However, some re-alignment away from the natural low points may occur to account for urban form and limit conflicts with other urban infrastructure. However, the design of conveyance networks should also consider minimising damage to existing ephemeral waterways.

Alignments of major systems are often parallel to minor systems and should be continuous until intersection with a natural watercourse or receiving waters. The design should include adequate management to avoid nuisance or risks at crossings, for example roadways or footpaths.

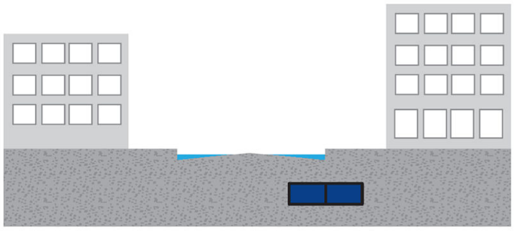
Configuration of stormwater management strategies (including conveyance networks) will depend on the land use within and alongside the selected overland flow paths (refer to Book 9, Chapter 5). This configuration may also vary throughout a stormwater management solution. Some of the typical configurations deployed in Australian design practice are presented in Figure 9.3.7. The most common configuration (shown in Figure 9.3.7) comprises an underground conveyance (inlet structures and pipes) network (minor system) within surface flow paths on roads (major system).



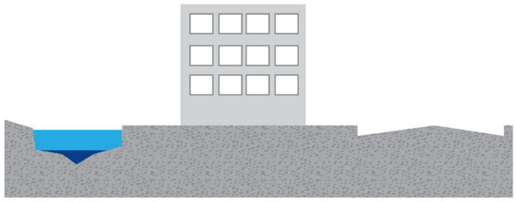
Residential Precinct
- underground pipe minor system (dark blue) with road as major system (light blue)



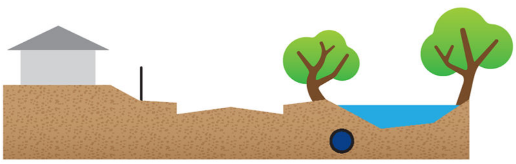
Residential Precinct
- underground pipe minor system (dark blue) with pathway as major system (light blue). Only suitable for small catchments.



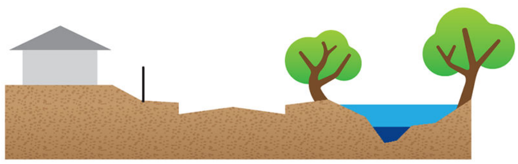
Commercial Precinct
- underground box culvert minor system (dark blue) with road as major system. Most of design capacity contained underground for safety reasons



Commercial Precinct
- open channel minor and major system combined in a single lined channel



Parkland Precinct
- underground pipe minor system (dark blue) with parkland as major system (light blue)



Parkland Precinct
- naturalised or lined open channel minor system (dark blue) with parkland as major system (light blue)

Figure 9.3.7. Typical Configurations of Major Minor Conveyance Systems Deployed in Australian Practice

The design of the major and minor systems should integrate smoothly with other urban infrastructure and manage impacts on natural environments. In particular, innovative design of urban parks can be used to achieve drainage objectives while also enhancing aesthetic and environmental outcomes.

Innovative approaches to stormwater management strategies can reduce construction costs and requirement for land area. This opportunity should be given early consideration in the concept design phase from perspective of multi-disciplinary teams.

3.5.3. Analysis

Suitable hydrologic and hydraulic calculation methods, described in Book 9, Chapter 4, Book 9, Chapter 5 and Book 9, Chapter 6, are used to estimate depths and velocities of stormwater flows with associated extents of flooding throughout major and minor systems, which facilitates the design of various components. The methods selected for analysis or design must be able to simulate the complexity of the stormwater management strategy. A design problem may include complex flow behaviours, for example parallel underground and surface flow paths, multiple inflows and the effects of storage and tail water conditions.

These methods must have the capacity to predict the hydraulic performance of the overall system and of each different component within the system for example inlet structures, pipes and channels. Hydraulic performance must be assessed using a range of storm events and configurations. Ideally, a design should be challenged by ensembles of full volume storm events to determine the critical storm duration and shape for each AEP.

The available software modelling tools can facilitate most of these complex calculations. However, emerging engineering practice and software tools aim to seamlessly handle the full range of linked hydrologic and hydraulic calculations required to account for surface flow behaviours throughout complex conveyance networks. These complex scenarios may require combinations of hydrologic models linked to hydraulic models with one dimensional conveyance network and two dimensional surface flows.

3.6. Stormwater Volume Management

3.6.1. Key Considerations

The historical practice of designing urban stormwater management has traditionally focused on peak flows and conveyance. Design standards have evolved to require comprehensive management of hydrologic changes created by urbanisation. It is now recognized that volume and regimes of stormwater runoff need to be managed (Beven and Alcock, 2012; Poelsma et al., 2013).

Typically, this is achieved through the design and installation of volume management facilities. Detailed aspects of these facilities are described in Book 9, Chapter 4, however at a philosophical level the questions that need consideration when developing a catchment-wide volume management strategy are:

What are the Volume Management Objectives for the Catchment?

Volume management objectives can include control of peak discharge, harvesting or infiltration of water and water quality treatment (refer to Book 9, Chapter 4, Section 2). These objectives are achieved using a volume management facility (either a single facility or a number of them) which can store and release runoff at different times, or even store runoff for later use.

The impact on downstream floodplains and receiving waters must be determined by assessing the catchment–wide consequences of compounding peak flow and volume discharges (increases in runoff volume and peak discharges) from different sub-catchments, as well as increased duration of flows in ephemeral aquatic ecosystems. This impact assessment will then help inform a decision about the volume management objectives to be pursued.

Phillips and Yu (2015) suggest whilst undertaking these assessments, catchment managers should also consider whether to use an ensemble of complete storms with a storm burst of around the critical duration or a storm burst only to determine the benchmark condition(s). The decision of what design to adopt can be informed through identifying the level of risk the community is willing to accept within the catchment.

Should the Objectives be Achieved in Combined Facilities?

It is preferable to provide infrastructure that meets multiple objectives. Where multiple volume objectives are sought for the catchment, it is possible to design separate volume management facilities that each target only a single objective.

For example, a facility might only manage peak discharge from a site for a single probability design flood event used for regional flood planning (e.g. 1% AEP). This might be achieved by storing a proportion of the hydrograph volume and releasing it later during the storm event through a constricted outlet. This is commonly called a detention basin or retarding basin.

Separate facilities might be required to also meet other stormwater volume objectives for example a rainwater tank for harvesting and a bio-retention basin for water quality improvement.

A more comprehensive facility might aim to achieve a peak discharge control objective alongside other volume objectives, by storing a proportion of the hydrograph volume and releasing it well after the storm event has finished, or even store it for later use (i.e. not released into the stormwater system at all). For example a constructed wetland (water quality) with an extended detention storage compartment above (peak discharge control), providing pre-treatment for a stormwater harvest facility (retention).

What is the Performance Level Sought?

For each facility and objective it is necessary to determine whether the facility must achieve a low or high level of performance.

For example, it may be sufficient to retain the hydrologic conditions equivalent to a pre-developed condition, which might be considered a low level of performance.

In some circumstances a higher level of performance might be required, for example, a return of hydrologic conditions back to a natural state.

The performance level sought will be related to the sensitivity of the downstream receiving waterway and whether the local community aspires to achieve a high performance solution.

Where should Volume Management be Achieved in the Catchment?

In some circumstances, there is opportunity to make broad strategic decisions about the distribution of these facilities across a catchment. Some typical volume management strategies that can be followed include:

- An ‘at source’ management strategy: this employs small facilities, widely distributed across the catchment, many of which will only service a small catchment or single property. Strategies of this type are most commonly part of a more comprehensive and integrated urban water strategy.
- A ‘neighbourhood scale’ management strategy: this strategy employs larger facilities that are less widely distributed than lot scale facilities but servicing larger catchments. These facilities are normally publicly managed and co-located alongside a watercourse or drainage reserve at the interface between underground and surface conveyance paths.

- A ‘regional scale’ management strategy: this strategy uses very large facilities that are located at the catchment outlet and service all properties in the watershed. These are normally publicly owned and co-located with major parkland. This is also referred to as an ‘end of pipe’ strategy.

How does Existing Urban Development Influence the Volume Strategy?

Some typical types of urbanising catchments and their associated volume strategy considerations are:

- **Future growth areas** where there is currently limited urban development (also commonly referred to as ‘Greenfields’ development). For these catchments the over-riding strategic objective commonly applied is to preserve the nature and amenity of their waterways in terms of hydrology (flow and channel geometry) and aquatic communities. This can be achieved using ‘source control’ measures applied throughout their contributing catchments. These measures include rainwater tanks, bio-retention facilities, ‘rain gardens’, infiltration trenches, ‘soakaways’ and access to aquifers where soil and geological conditions are favourable.

  Since there is often opportunity to forward plan in ‘greenfields’ catchments there may also be opportunities for comprehensive ‘neighbourhood scale’ and ‘regional scale’ placement strategies.

  Every effort should be made in these catchments to encourage 'informal' drainage, green spaces and to disconnect as much impervious surface as possible. The criterion for successful design of these systems is keeping the volume discharged from each site the same after development as before, for design flood events. Use of these practices, is referred to by Argue (2017) as a ‘regime-in-balance’ strategy. It is suggested that adoption of such a strategy can keep urban waterways operating as natural systems for many years before increased urbanisation might then require the introduction of rectification strategies such as increased channel lining.

- **Highly urbanised catchments** where the strategic objective is often to minimise the need for further modification or upgrades to conveyance networks as development and re-development continues. For these catchments land availability may constrain opportunities for wide adoption of ‘neighbourhood scale’ and ‘regional scale’ placement strategies. However volume management objectives can be achieved in a similar manner to a ‘greenfields’ catchment using ‘source control’ practices as re-development takes place. An additional opportunity, ‘roof gardens’, is provided by the presence of multi-storey and high-rise elements of this class of development.

  The objective for successful design of these systems is keeping the volume discharged from each site the same after development as before, for a design flood event. This objective is more difficult to achieve than in ‘greenfields’ catchments giving rise to the more common use of temporary on-site storages holding stormwater after flood peaks have passed. This problem can be solved by ‘slow release’, infiltration or harvesting to ensure storages are empty ahead of closely-spaced storm events. With such provisions in place, the supporting infrastructure can continue to operate successfully without enlargement.

  Prediction of Australia’s urban growth to mid-21st Century suggests that development within catchments of this type will provide the majority of new urbanisation.

- **Over-developed catchments** are a particular case of highly urbanised catchments described above, and apply to many of our older, inner-city suburbs. These catchments are characterised by frequent episodes of flash flooding and resulting community disruption.

The criterion for successful design of these systems is not just to match pre-development conditions but to go further and minimise the volume discharged from each site after re-development. This is referred to by Argue (2017) as the ‘yield-minimum’ strategy. The nature of re-development in an already over-developed urban catchment is frequently large-scale, for example urban renewal projects. These lend themselves to complete re-organisation of local drainage infrastructure and, hence, opportunities for less discharge during the ‘design’ runoff events. Every component of re-development incorporated under the ‘yield-minimum’ strategy moves the catchment in the direction of a balance between runoff being generated and infrastructure capacity.

Are There Other Constraints that may Influence the Strategy?

Catchment managers will also need to take into account the local landscape and soil conditions, which may limit the application of certain volume and quantity management solutions. For example, heavy clay soils may limit the application of infiltration based solutions, whereas sandy soils may promote such solutions.

Other examples of constraints that may have strategic influence are:

- sensitive riparian vegetation communities
- land ownership and development patterns, and
- different choices may be required depending on the nature of the catchment and the asset policies of the local stormwater authority.

3.6.2. Selecting a Strategy

Once the above questions have been considered it might be appropriate to establish and document a catchment-wide strategy for stormwater volume management. Such a strategy should be used to assist with the design and assessment of individual volume management proposals.

Typical catchment management strategies (as designed using bottom up or top down methods or other analyses) can include a number of different approaches which reflect the local authority’s commitment to WSUD principles, as well as commitment to restore overloaded systems to balance.

Three examples of management strategies for catchment-wide volume management are provided by Argue (2017). These are consistent with the risk management framework discussed in Book 1, Chapter 5 and are defined as follows:

Yield-maximum: maximise the quantity of storm runoff captured at the end of the catchment and ensure that the floodwaters are contained within a defined floodplain. This strategy is most suitable for local authorities with a desire to have large centrally controlled systems, rather than distributed local solutions.

Regime-in-balance: maintain the harmonious and synergistic relationship that exists between continuing urban development and ‘acceptable’ use of the floodplain for agricultural and amenity pursuits. This strategy is most suitable for catchment or sub-catchments where development has occurred or is likely to occur and will discharge to a nearly intact or sensitive receiving environment.

Yield-minimum: improve the performance of the urban flood control infrastructure through minimisation of stormwater discharge from each development site (including redevelopment sites). This strategy is most suitable for catchment or sub-catchments with already poorly controlled urban development with a history of flood damage and ecosystem deterioration.

Large catchments, where urbanisation is actively occurring, and over an extended period, may contain precincts where a mix of these strategies might be appropriate. Notably, all strategies will benefit from urban planning that promotes rainfall infiltration, harvesting and retains natural hydrologic function.

3.7. Stormwater Offsets

Tradeable permits or offset schemes are also known as market mechanisms and are established methods within the pollution control industry, in water markets and for management of nutrient or salinity loads in river basins. These processes commonly involve financial contributions paid by a landholder for provision of pollution control works at another location, construction of an alternative mitigation scheme instead of a conventional solution in the landholders development site, or the sale of a water licence from a landholder to another landholder at another location.

Tradeable permits for pollution control are attractive as they provide opportunities for economic efficiency, flexibility and incentives for innovation (Kraemer et al., 2004; Haensch et al., 2016). The international experience with water pollution emission trading is not extensive but does include some successful examples (Shortle, 2013). Trading of pollution abatement responsibilities can cause water quality to deteriorate at different times and rates in some parts of a catchment. Therefore designing a tradeable permit or offset scheme needs to take spatial, temporal and environmental equivalence effects into account.

At the time of writing, Melbourne Water (MWC, 2018) and Queensland Healthy Waterways (Water by Design, 2014) (for example) operate stormwater quality offset schemes. These schemes involve a financial contribution paid by developers for stormwater management works to be undertaken in another location to meet catchment wide objectives for managing stormwater and protecting waterway health. These schemes respond to the assumption that regional stormwater management is more cost and time effective than distributed smaller scale solutions. These off-set schemes can be useful for urban areas subject to infill development that may have limited space for infrastructure.

There are limited examples of trading or offset schemes for management of stormwater runoff volumes or peak flows. The District of Columbia Water and Sewer Authority (for example: DCWater (2018)) provide an impervious area charge incentive program for customers to reduce effective impervious surfaces and, therefore, stormwater runoff on their properties which avoids regional works. Properties that use best management practices such as rain gardens, rainwater harvesting, green spaces and pervious paving are considered to reduce effective impervious surfaces and results in a reduced stormwater charge. Similarly, the historical on-site detention (OSD) strategies by the Upper Parramatta River Catchment Trust (for example: UPRCT (2005)) offset the need for regional stormwater basins by use of detention storages (OSD) on properties.

Use of formal stormwater off-set schemes to transfer local management of stormwater volumes and peak flows to regional facilities is not common, but these types of approaches are embodied in most developer contribution schemes for regional infrastructure. Stormwater off-set schemes for management of runoff volumes and peak flows should include the following key principles:

- Transfer of stormwater management to another location should not negatively impact on surrounding local properties at any (legal) point of discharge

- The spatial, temporal and cumulative allocation of required treatment capacity must be defined using a catchment management strategy. It is unlikely that transfer of local stormwater management requirement to a downstream regional location will be a linear or average process

- A scheme must result in the desired and measurable changes in flow (and water quality) resulting from the infrastructure and stormwater strategies within the same catchment

- The funds obtained from stormwater off-sets must be tied to measurable deliverables in the catchment

- The scheme must provide for regional infrastructure in a reasonable time period that is consistent with the timing of upstream development.

- The relative financial contributions from upstream developers must be proportional to their flow and pollutant loads that will be managed by the regional scheme

- The scheme must have the same life cycle or equivalent life cycle as the life cycle of the upstream development (e.g. short-term mitigation strategy, such as flow and erosion management, cannot be used for a long-term offset to a developed area stormwater management)

- If water quality is part of the scheme, consideration should be given to the bio-availability of pollutants removed through the different upstream and catchment wide management methods

- Clear ownership and rules about the off-set scheme should be established and risk should be mitigated through the adoption of appropriate ratios, and

- The ongoing maintenance and renewal costs associated with the regional infrastructure must be allocated to ensure the performance of the scheme does not deteriorate over time.

Stormwater off-set schemes that transfer management of stormwater volumes and peak flow to other locations have the potential for ecological impacts in local waterways or downstream receiving waters. The ultimate objectives of an off-set scheme should include performance targets that also consider secondary effects (such as impacts on local waterways) and monitoring strategies should be implemented to measure effects of strategies.

Chee (2015) highlights that there is limited evidence of success of stormwater off-set schemes and formal monitoring strategies would provide an opportunity to more critically consider the evidence of how well schemes that have been implemented and their operation. It is also emphasized that achieving equivalence in stream biodiversity and ecological function is extremely difficult.

Coker et al. (2018) argue that stormwater off-sets should not result in avoided management of stormwater runoff. They emphasize the substantial challenge of adequately considering spatial, temporal and environmental influences of off-sets, and the importance of quantifying the spatial extent of stormwater impacts from the development in question. It is highlighted that unmitigated stormwater runoff from relatively small proportions of urban areas may propagate severe impacts a long way downstream which can render the practice of offsetting within a single catchment a difficult undertaking.

3.8. Acknowledgements

This chapter benefited from the support and guidance provided by Mark Babister and Engineers Australia. The substantial contributions of the editorial panel of Andrew Allan, Andrew King, Stephen Frost, Mike Mouritz, Tony McAlister and Tony Webber in shaping this discussion are gratefully acknowledged. Additional review comments from James Ball, Bill Weeks and Christopher Walsh also assisted in clarifying the content of this paper.

3.9. References

Aishett E., and Steinhauser, E. (2011), Does anybody give a dam. The importance of public awareness for urban water conservation during drought. Submission by the Australian National University to the Productivity Commission.

Argue, J.R. (2017), Water Sensitive Urban Design: basic procedures for 'source control' of storm water - a handbook for Australian practice. Urban Water Resources Centre, University of South Australia, Adelaide, South Australia, in collaboration with Stormwater Industry Association and Australian Water Association. Adelaide.

Armstrong, J.W. (1967), Pipelines and People, Halstead Press, Sydney.

Barry, M.E. and Coombes, P.J. (2006), 'Optimisation of Mains Trickle Topup Supply to Rainwater Tanks in an Urban Setting', Australian Journal of Water Resources, 10(3), 269-276.

Barton A.F., Coombes, P.J., Sharma, A. (2007), Impacts of innovative WSUD intervention strategies on infrastructure deterioration and evolving urban form. Rainwater and Urban Design Conference 2007, Engineers Australia, Sydney.

Beven K.J. and Alcock R.E. (2012), Modelling everything everywhere: a new approach to decision making for water management under uncertainty, Freshwater Biology, 57(1), 124-132.

Bozovic, R., Maksimovic, C., Mijic, A., Smith, K. M., Suter, I., Van Reeuwijk, M., (2017), Blue Green Solutions. A Systems Approach to Sustainable and Cost-Effective Urban Development, Imperial College, London, UK.

Brown, R.R. and Farrelly, M. (2007), Institutional Impediments to Advancing Sustainable Urban Water Management: a Typology. International Rainwater Catchment Systems Conference and the 5th International Water Sensitive Urban Design Conference, Sydney, Australia.

Chee Y.E. (2015), Principles underpinning biodiversity offsets and guidance on their rise. In van der Ree R. Smith DJ and Grilo C (eds.) Handbook of Road Ecology. John Wiley and Sons, Ltd, Chichester, West Sussex, pp: 51-59.

Clarke, R.D.S., (1990), Asset replacement: can we get it right? Water. pp.22-24

Coker, M.E., Bond, N.R., Chee, Y.E. and Walsh C.J. (2018), Alternatives to biodiversity offsets for mitigating the effects of urbanization on stream ecosystems, Conservation Biology.

Coombes, P.J. (2007), Energy and economic impacts of rainwater tanks on the operation of regional water systems. Australian Journal of Water Resources, 11(2), 177-191.

Coombes, P.J. (2005), Integrated Water Cycle Management: Analysis of Resource Security. Water, 32: 21-26.

Coombes, P.J. and Barry, M.E. (2014), A systems framework of big data driving policy making - Melbourne's water future. OzWater14 Conference. Australian Water Association, Brisbane.

Coombes, P.J. and Barry, M.E. (2008), The relative efficiency of water supply catchments and rainwater tanks in cities subject to variable climate and the potential for climate change, Australian Journal of Water Resources, 12: 85-100.

Coombes P.J. and Kuczera, G. (2002), Integrated Water Cycle Management - moving towards systems understanding. WSUD02 Conference, Engineers Australia, Brisbane, Australia.

Coombes, P. and Kuczera, G. (2003), Analysis of the performance of rainwater tanks in Australian capital cities, Proceedings of the 28th International Hydrology and Water Resources Symposium, Engineers Australia, volume 2, Wollongong.

Coombes, P.J., Argue, J.R. and Kuczera, G.A. (2000), Figtree Place: a case study in water sensitive urban development (WSUD), Urban Water, 1(4), 335-343.

DCWater (2018), Impervious Area Charge, https://www.dcwater.com/impervious-area-charge [accessed on 16/04/2018], District of Columbia Water and Sewer Authority, USA.

Daniell, K.A., Coombes, P.J. and White, I. (2014), Politics of innovation in multi-level water governance systems. Journal of Hydrology, 519(C), 2415-2435.

EA., (2006). Australian Runoff Quality: a guide to Water Sensitive Urban Design. Editor in Chief: T.H.F. Wong. Engineers Australia. Canberra.

Forrester J.W. (1969), Urban Dynamics. Massachusetts Institute of Technology Press, Cambridge, Massachusetts, USA.

Goyen A.G. (1981), Determination of rainfall runoff model parameters. Masters Thesis, NSW Institute of Technology, Sydney.

Goyen, A.G. (2000), Spatial and Temporal Effects on Urban Rainfall / Runoff Modelling. PhD Thesis. University of Technology, Sydney. Faculty of Engineering.

Haensch, J., Wheeler, S.A., Zuo, A. and Bjornlund, H. (2016), The impact of water and soil salinity on water market trading in the southern Murray-Darling Basin, Water Economics and Policy, 2(1), article No.1650004, pp: 1-26.

Kraemer, R.A., Kampa, E., and Interwies, E., (2004), The role of tradable permits in water pollution control, Inter-American Development Bank.

Kuczera G.A., Lambert, M., Heneker, T., Jennings, S., Frost, A. and Coombes, P.J. (2006), Joint probability and design storms at the crossroads. Australian Journal of Water Resources, 10: 63-79.

Lloyd, S.D., Wong, T.H.F. and Porter, B. (2002), The planning and construction of an urban stormwater management scheme. Water Science and Technology. International Water Association, 45(7), 1-10.

Lloyd, C.J., Troy, P. and Schreiner, S. (1992), For the Public Health, Longman Cheshire, Melbourne.

Melbourrne Water Corporation (MWC) (2018), Stormwater offsets explained, https://www.melbournewater.com.au/planning-and-building/developer-guides-and-resources/drainage-schemes-and-contribution-rates [accessed on 16/04/2018], Melbourne Water Corporation.

McAlister, T., Coombes, P.J. and Barry, M.E. (2004), Recent South East Queensland Developments in Integrated Water Cycle Management - Going Beyond WSUD. Cities as Catchments:WSUD2004, Adelaide, SA, Australia.

Meadows D.H., (1999), Leverage points: places to intervene in the system, The Sustainability Institute, Hartland, USA.

Mitchell, V.G., McMahon, T.A. and Mein, R.G. (2003), Components of the total water balance of an urban catchment. Environmental Management, 32: 735-746.

PMSEIC. (2007), Water for Our Cities: building resilience in a climate of uncertainty. A report of the Prime Minister's Science, Engineering and Innovation Council working group, Australian Government, Canberra.

Paul, M.J. and Meyer, J.L., (2001), Streams in the urban landscape,. Annual Review of Ecology and Systematics, 32: 333-365.

Pezzaniti, D., Argue, J.R. and Johnston, L. (2002). Detention/retention storages for peak flow reduction in urban catchments: effects of spatial deployment of storages. Australian Journal of Water Resources, 7(2), 1-7.

Phillips, B.C. and Yu, S. (2015), How robust are OSD and OSR Systems?. Proceedings, 3rd International Erosion Control Conference and 9th International Water Sensitive Urban Design Conference, 20-22 October 2015, Darling Harbour.

Poelsma, P.J., Fletcher, T.D. and Burns, M.J. (2013). Restoring natural flow regimes: the importance of multiple scales. Proceedings of Novatech Conference, Lyon, France, pp: 23-27.

Shortle, J., (2013), Economics and environmental markets: Lessons from water-quality trading. Agricultural and Resource Economics Review, 42(1), 57-74.

UPRCT (2005), Onsite stormwater detention handbook, Upper Parramatta River Catchment Trust.

United States Environmental Protection Authority (USEPA) (2008), Urban stormwater management in the United States, National Research Council. The National Academies Press, DC.

Victorian Government (2013), Melbourne's Water Future.

Walsh, C.J. (2004), Protection of in-stream biota from urban impacts: minimise catchment imperviousness or improve drainage design?. Marine and Freshwater Research, 55: 317-326.

Walsh, C.J., Roy, A.H., Feminella, J.W., Cottingham, P.D., Groffman, P.M. and Morgan, R.P. (2005), The urban stream syndrome: current knowledge and the search for a cure. Journal of the North American Benthological Society, 24: 706-723.

Walsh, C.J., Fletcher, T.D. and Burns, M.J. (2012), Urban stormwater runoff: a new class of environmental flow problem, PLoS ONE 7(9), e45814.

Walsh, C.W., Fletcher, T.D., Bos, D.G. and Imburger, S.J., (2015), Restoring a stream through retention of urban stormwater runoff: a catchment-scale experiment in a social-ecological system. Freshwater Science, 34(3).

Wasko, C. and Sharma, A. (2015), Steeper temporal distribution of rain intensity at higher temperatures within Australian storms, Nature Geoscience, 8(7), 527-529.

Water by Design (2014), Off-site stormwater quality offsets discussion paper, Healthy Waterways, Brisbane.

Weinmann, E. (2007), Hydrology and Water Resources: the challenge of finding the water balance. Australian Journal of Water Resources, 11(2), 121-132.

Whelans and Halpern Glick Maunsell (1994), Planning and Management Guidelines for Water Sensitive Urban (Residential) Design, report prepared for the Department of Planning and Urban Development the Water Authority of Western Australia and the Environmental Protection Authority.

Wong, T.H.F. (2006), Water Sensitive Urban Design: the journey thus far. Australian Journal of Water Resources, 10(3), 213-221.

Wong, T.H.F., Knights, D. and Lloyd, S.D. (2008), Hydrologic, water quality and geomorphic indicators of catchment Effective Imperviousness, Australian Journal of Water Resources, 12(2), Engineers Australia, pp: 111-119.

Chapter 4. Stormwater Volume Management

Steve Roso, Marlene van der Sterren

With contributions from John Argue, Brett Phillips and Urban Book Editor Peter Coombes

Chapter Status	Final
Date last updated	14/5/2019

4.1. Introduction

Progressing from the urban stormwater philosophy discussed in Book 9, Chapter 3, this chapter provides introductory guidance on the design of 'volume management facilities'. These are discrete infrastructure measures in various forms and configurations, each of which are designed to store and release runoff volumes to manage the changes caused by urbanisation. They are linked by conveyance infrastructure (refer to Book 9, Chapter 5) to form an urban stormwater network.

This chapter focusses on the concept design phase of a volume management facility and outlines the detailed design process. Before applying the content in this chapter it is assumed that the general position of the facility within the catchment is already largely understood, and preferably informed by a catchment strategy, as discussed in Book 9, Chapter 3 and Book 9, Chapter 5.

Stormwater storages receive runoff volumes from the catchment via upstream conveyance infrastructure. The manner in which these runoff volumes are managed depends on the practice that is adopted. The storage and release of runoff changes the characteristics of the runoff hydrograph and is a fundamentally important feature of all volume management facilities.

There is considerable legacy terminology used to describe these facilities including detention (or retarding), retention, extended detention or slow release. These terms are a derivative of outlet structures and different operational strategies that change the behaviour of stormwater storages.

Stormwater storages designed in accordance with 'detention' practices include those where runoff is temporarily stored and simultaneously released via an outlet structure (Figure 9.4.1). This process typically lowers peak discharge and attenuates the hydrograph so that the average time of release is delayed. The storage volume and capacity of the outlet must be determined by catchment wide modelling to achieve target outflow peak discharges at the catchment outlet.

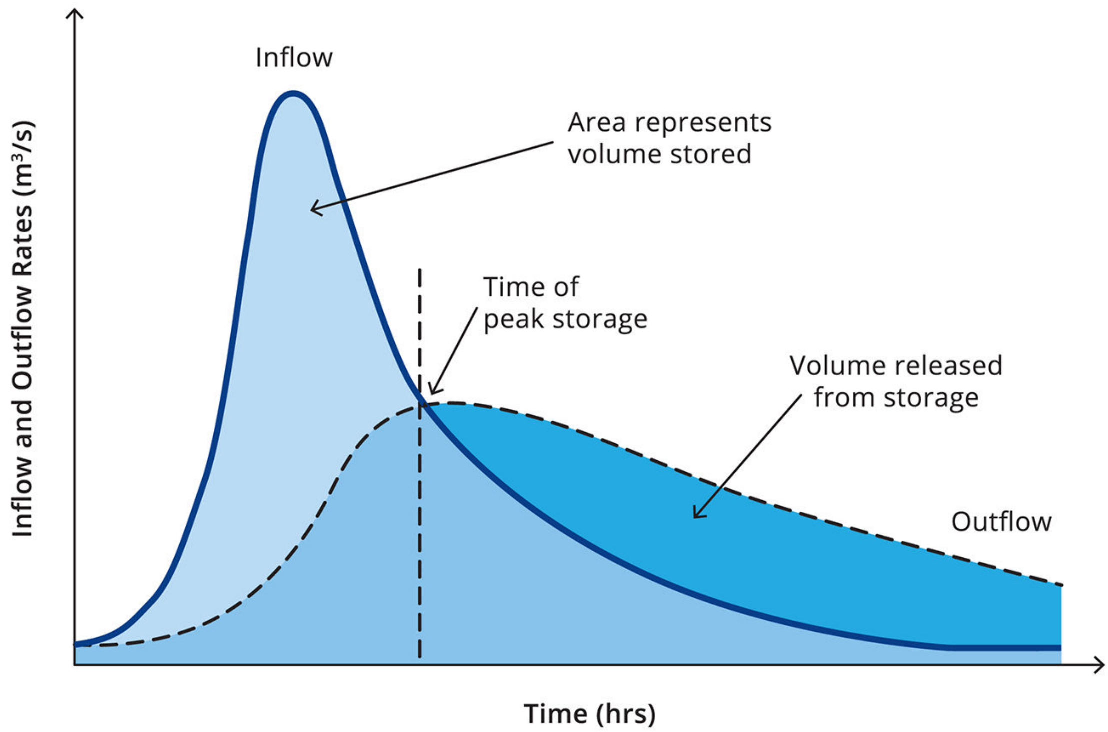


Figure 9.4.1. Typical Hydrograph Change Generated by a Temporary Storage (Without Harvesting)

Assuming the stormwater storage is empty at the beginning of a storm, the potential hydrograph change that can occur depends on:

- the outlet's discharge capacity relative to the peak discharge of the storm;
- the size of the storage basin volume relative to the total runoff volume from the storm; and
- the volume of water harvested from the storage.

As a general rule, if the storage volume is large relative to the total runoff volume, the greater the potential hydrograph attenuation that can occur. This performance also depends on the outlet capacity. A small outlet capacity relative to peak inflows will tend to favour attenuation of small storms and large storms it will overflow early, whereas a large outlet capacity will tend to favour attenuation of large storms and small storms will pass through the facility without attenuation in storage. While the storage and outlet structure are separate physical components of a volume management facility, they must be designed in an integrated manner since the capacity of the storage will effect the performance and sizing of the outlet structure and vice versa. This is a critical aspect of the design of a volume management facility with detention characteristics that requires an iterative approach to sizing.

Stormwater storages designed in accordance with 'retention' practices provide sufficient storage in the volume management facility to contain additional runoff from urban development. The volume of stored stormwater is then drawn down by infiltration, harvesting or slow release. Typical hydrographs of flows from a rural catchment and subsequent urban development of the catchment are presented in Figure 9.4.2. Inflow and outflow hydrographs which apply to a volume management facility used in a typical retention strategy, are shown in Figure 9.4.3.

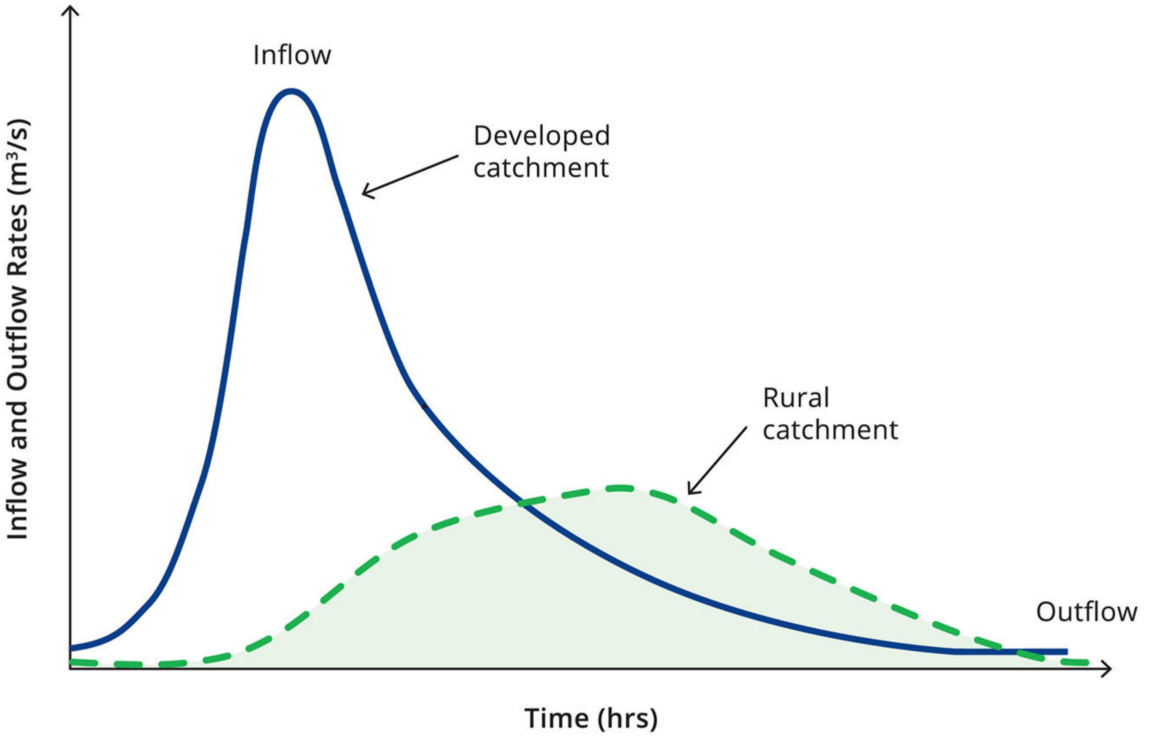


Figure 9.4.2. Rural and Developed Catchment Hydrographs

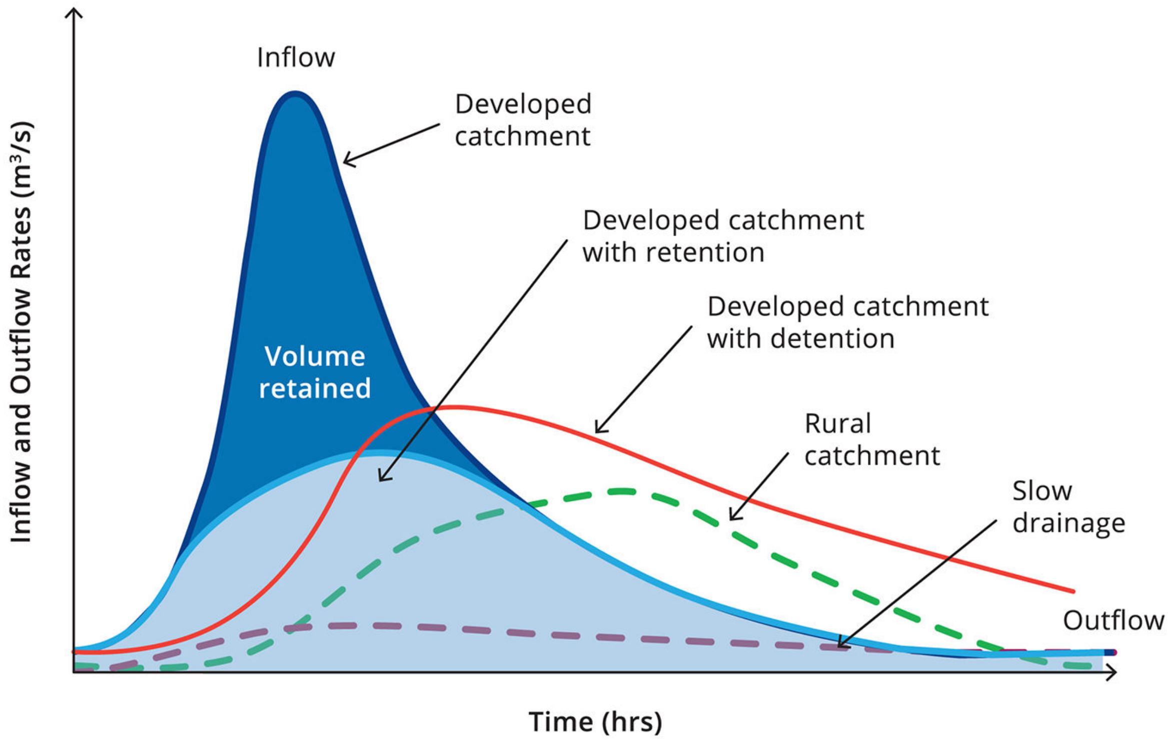


Figure 9.4.3. Developed Catchment with Retention as Compared to Detention and Slow Drainage Strategies

The hydrographs in Figure 9.4.2 represent runoff from a rural catchment and from the urban landscape developed on it. Ideal retention performance of the storage is reproduction of the rural hydrograph followed by outflow of the remaining stored runoff via slow release over a longer duration (typically greater than 24 hours). Argue (2017) outlines that it is difficult to achieve this outcome and recommends a storage volume equal to the total additional runoff expected from the development and the emptying time of volume management facility is a function of outlet infrastructure.

The outflow hydrograph resulting from this approach should be similar to that shown in Figure 9.4.3 (developed catchment with retention). A first approximation solution is likely to produce a different outflow hydrograph from the required result. Continuous simulation of the volume management facility is recommended with the aim of adjusting the design i.e. storage and outflow configuration, to produce the desired outflow hydrograph.

The concept design phase of volume management facilities commences with a thorough understanding of the volume management objectives intended for the facility (refer to Book 9, Chapter 4, Section 2). Once these objectives are defined, consideration can be given to the configuration of the facility and how its components might be sized and positioned to best meet the objectives and local site conditions (refer Book 9, Chapter 4, Section 3 and Book 9, Chapter 4, Section 4). Detailed design then follows to comprehensively define the facility to permit construction (refer to Book 9, Chapter 4, Section 5).

4.2. Volume Management Objectives

The design of a volume management facility must include objectives which are relevant to the site, the surrounding catchment and receiving waterways. A summary of the most commonly encountered volume management design objectives in Australian practice is provided in Table 9.4.1. Each objective has ‘associated benefits’ that are also listed to help distinguish the relevance of each objective to a particular site and design.

An adequate number of facilities are required within catchments to ensure that the controls will significantly affect peak discharges, volume targets and water quality targets at catchment outlets. A key aspect of the design of storage based measures is to ensure that the storages are empty or nearly empty at the commencement of a flood producing rain event. It is essential to determine the spectrum of design flood events that these facilities will manage (refer to Book 9, Chapter 3).

Table 9.4.1. Summary of Volume Management Design Objectives

Objective	Potential Associated Benefits
Control Peak Discharges This objective seeks to limit the peak flood flows and volumes discharging from a catchment to a pre-determined and acceptable level. Commonly the acceptable level is set at the natural or ‘pre-development’ condition. In some cases the acceptable level may be set below the natural condition in order to achieve a net benefit or offset an impact elsewhere. In highly developed catchments (infill development), the acceptable level may correspond to flows from the original development. These objectives may seek to change the total volume of stormwater leaving a site (retention), or delay the volume for a	• Reduced property flood damage • Reduced personal safety risks due to flooding • Reduced infrastructure damage • Reduced conveyance infrastructure

Objective	Potential Associated Benefits
short period of time (hours) (detention or retarding) which may reduce the peak of the flood hydrograph discharging from a catchment. Careful consideration of the spectrum of design flood events needs to be given and its impact on downstream receiving systems (for example stream forming flows and flood flows), which can result in 'slow release' systems. Emerging stormwater management practices seek to reduce the volume and timing of stormwater discharges from catchments. This combined approach is particularly relevant for managing stormwater runoff from increasing urban density (refer Book 9, Chapter 3). This objective is a very commonly sought outcome. It is of most relevance to urban catchments where there is a constrained floodplain downstream or sensitive ecosystem that cannot accommodate increase in peak flood discharges or volumes.	requirements (downstream)
Harvest or Infiltrate Rainwater or Stormwater This objective seeks to extract a proportion of the runoff volume from a catchment and either use this water for a consumptive purpose (i.e. consistent use to ensure draw down of storages), or infiltrate the runoff directly into local soils or subterranean aquifers (possibly for later extraction). These integrated design approaches can require interaction with soil properties, capacity of aquifers, urban form and demands of water (refer Book 9, Chapter 3). The designer should account for the elements in the design of a catchment wide strategy to ensure that adequate storage space is available in storages to achieve the objectives of the strategy. Analysis of these measures must include continuous simulation and the use of full volume storms to understand the required storage capacity for a given set of rainfall events.	• Maintain waterway stability and reduce scour • Maintain groundwater behaviour • Maintain hydrologic behaviour including natural runoff regimes • Increase volume of water stored in an aquifer • Increased availability of water for harvesting and use
Improve Water Quality This objective seeks to reduce concentrations and loads of contaminants within urban runoff to pre-determined and acceptable levels. This is achieved by: delaying some of the runoff volume for a period of time (hours to days) (detention), or storing part of the stormwater on-site (retention) and passing the retained water through treatment processes where physical, chemical and biological processes reduce contaminants in the water column. Storage of stormwater can also provide some limited water quality treatment through settlement, even where this objective is not necessarily sought.	• Maintain aquatic health • Maintain visual amenity • Improved water quality prior to discharge or prior to harvesting activities

An early design task should examine the relevance of the objectives from Table 9.4.1 for a design in the context of prior studies, investigations, catchment strategies and receiving waterbody conditions. This process allows the designer to establish a preliminary understanding of the behaviour of the site, the catchment and receiving system. Another important task is to check local stormwater authority and state government policy requirements and standards. In the absence of background studies and local authority guidance, the designer should critically assess the relevance of the above-listed objectives from first principles. The ‘associated benefits’ listed in Table 9.4.1 may assist.

Volume management initially emerged as a design consideration to control of peak discharges in catchments. This was driven by a need to manage flood impacts associated with development and an emerging understanding that the stormwater runoff behaviour of urban catchment is volume dependent. Nevertheless, the design process was driven by peak rainfall bursts rather than the full volumes of storm events. Progressively, as our understanding of urban impacts on waterways has broadened, standards have changed to the point where it is now quite common for the other volume management objectives listed in Table 9.4.1 to also be considered. Facilities that target these multiple objectives have a stronger business case and are therefore more commonly sought after in modern practice and use the full spectrum of storms to protect the downstream receiving systems.

If there are indeed multiple objectives sought for a design, it may be advantageous to design a single facility that will meet all the desired objectives. However, current stormwater management practice incorporates multiple solutions across scales to better manage risk profiles (refer Book 9, Chapter 3). Figure 9.4.4 shows how more than one design objective can be relevant to a site or a catchment, or an entire stormwater management strategy. For example, design objectives for a facility or a strategy may include:

- control peak discharges and harvest (or infiltrate) stormwater;
- control peak discharge and improve water quality;
- improve water quality and harvest (or infiltrate) stormwater; and
- control peak discharge, improve water quality and harvest (or infiltrate) stormwater.

Where possible the design process should pursue performance characteristics that target all the desired objectives. This goal is most likely to be achieved when a particular management strategy is selected as the primary objective, for example peak discharge reduction or water quality improvement, and the subsidiary objectives are incorporated by exploiting opportunities made available by the primary objective.

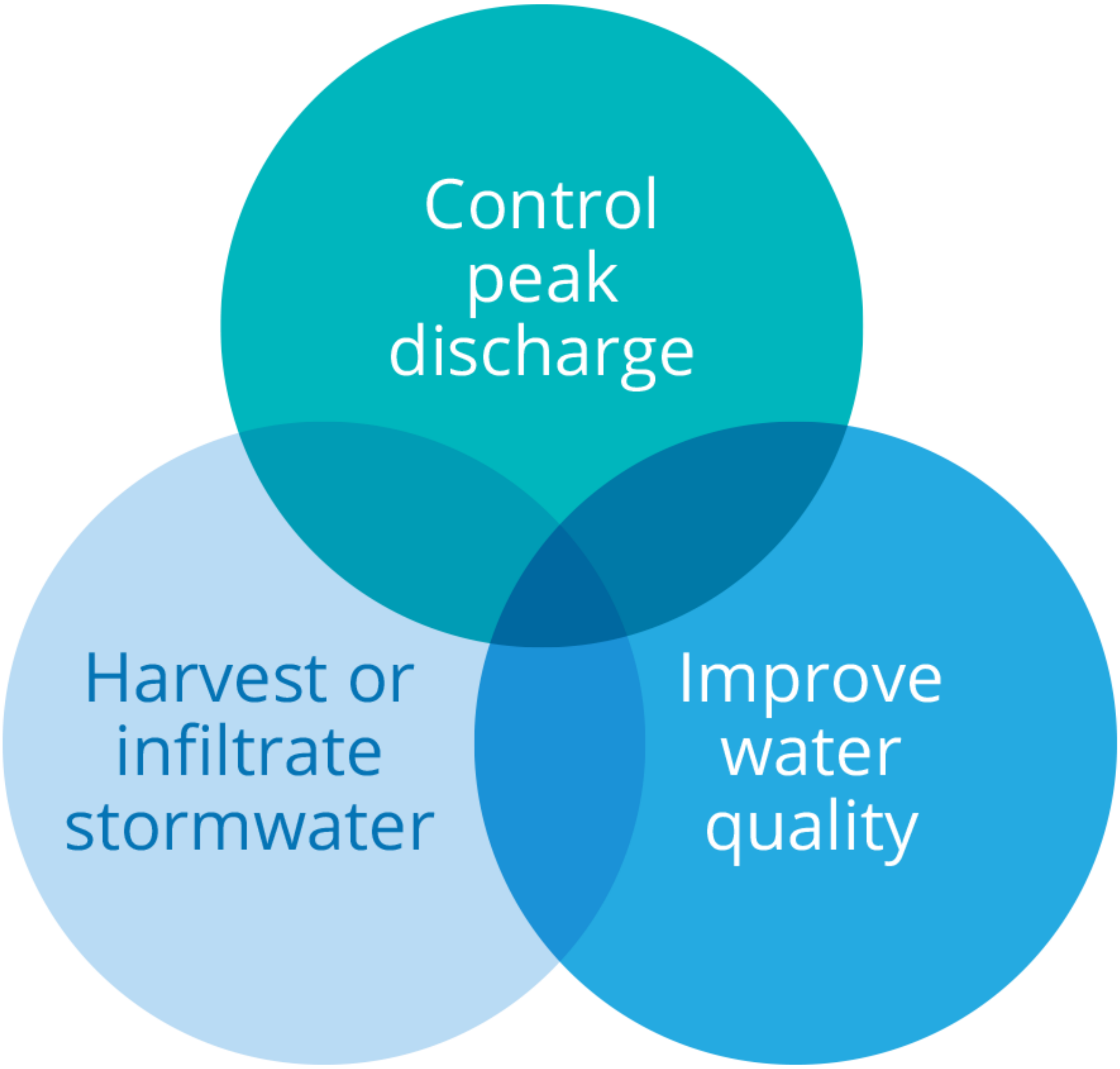


Figure 9.4.4. Potential Overlapping Volume Management Design Objectives

4.3. Components of a Volume Management Facility

4.3.1. Overview

There are up to four generic infrastructure components that are common to majority of volume management facilities; an inlet structure, storage, an outlet structure, and treatment media. These are described in Table 9.4.2.

Table 9.4.2. Volume Management Facility Components

Component	Purpose	Examples
Inlet Structure A conduit or flow path that controls the inflow into the facility and connects the	To transition flows from the upstream conveyance system into the storage device in a controlled manner (refer Book 9, Chapter 5 for	• Headwall outlet structure with riprap • Level spreader

Component	Purpose	Examples
upstream conveyance network to the storage.	more details on conveyance system outlets).	• Energy dissipator
Storage An area of land or a storage structure that contains water after rainfall occurs.	To receive and store a pre-determined volume of water for a pre-determined period of time. Partial discharge from the site and partial retention in on-site storage facilities.	• Small storages such as a On-Site Detention (OSD) tank • Large storages such as basins • 'Nested' basins • Roof gardens, rainwater tanks, bio-retention facilities, raingardens, infiltration trenches, soakaways, access to aquifers
Outlet structure A conduit or flowpath that connects the storage basin to downstream conveyance infrastructure	To control water release from the storage at a pre-determined rate and direct it to the appropriate location downstream. To control water release from the storage to a pre-determined slow release rate. Control outflow from the storage to satisfy a required emptying time criterion.	• Pipe or box culvert through an embankment (with headwall or pit entry) • Discharge control pit • Rainwater distribution system • Spillway across the top of an embankment • High overflow discharge pipe • Aquifer infiltration zone • Combinations of the above
Treatment processes A physical installation located, in-line or off-line, usually within a storage, upstream of a site, neighbourhood or regional discharge point. A material or process that removes water-borne contaminants from runoff as it passes through the storage basin.	To reduce or remove concentrations of contaminants from runoff as it passes through the device towards the outlet.	• Sediment forebay • Gross pollutant trap • Aquatic plants • Vegetated soil media • Sand, gravel or other filtration media • Storage processes including settlement, bio-reaction and natural flocculation

Each of these components can be configured and combined with the other components in different ways to meet different design objectives. The size, shape and material of each component can also be selected to respond to performance criteria and site constraints. Some components can be omitted depending on the design objectives. For example treatment processes are only required where the design seeks to improve water quality or the impacts of the storage on improving water quality need to be enhanced.

4.3.2. Common Configurations in Australian Practice

There are a large number of potential sizing and configuration options available to the designer. Changes to the relative sizes of each component (from Table 9.4.2), along with combinations of different materials and different hydraulic designs can adjust the way in which an overall facility or strategy will perform against the volume management objectives and respond to different site constraints.

The volume management facility configurations that are in common use in Australia are listed and described in Table 9.4.3. Further guidance on selecting a specific design configuration is also provided in Book 9, Chapter 4, Section 4 and Book 9, Chapter 4, Section 5

Table 9.4.3. Common Volume Management Facility Configurations in Australian Practice

Common Description	Storage Basin	Outlet Structure	Treatment Processes[a]	Typical Catchment Scale[b]
Detention Basin (Retarding Basin)	A storage basin excavated into the ground surface and partially formed by embankment on downslope side. The size of storage to be determined from catchment-wide analysis focused on the target peak flow at the catchment outlet. Normally dry.	A concrete pipe or box culvert passing through the embankment at the base level of the storage. A spillway at the top level of the storage to pass flow in excess of the culvert capacity.	Nil	Neighbourhood Precinct
On-Site Detention (OSD)	A small underground tank or surface depression. Normally dry.	A small pipe at the base level of the storage with an orifice to reduce outlet flow rates. A small weir at the top of the storage to pass flow in excess of orifice capacity (Figure 9.4.7 and Figure 9.4.8).	Nil	Lot Site
Rainwater Harvesting	Surface or underground storages capturing runoff from roof surfaces and consumed for indoor and outdoor purposes. The storage has a permanent storage volume and may have an air space above	Constant water usage (for example indoor demands) draws down storage volumes prior to rainfall events. A small pipe may link to the downstream stormwater network at the	Volume reduction processes reduce erosion of streams and reduces transport of urban pollutants.	Lot Site Neighbourhood Precinct

Common Description	Storage Basin	Outlet Structure	Treatment Processes[a]	Typical Catchment Scale[b]
	the permanent storage for stormwater detention.	top level of the permanent storage. A second pipe at the top level of the air space caters for high level overflows (Figure 9.4.9).		
Bioretention Basin	A storage basin excavated into the ground surface and partially formed by embankment on downslope side. Shallow storage over filter media. Experiences a cycle of wetting and drying.	A network of sub-soil drainage at the bottom of the filter media. Outlet pit and pipe culvert for flows that exceed the permeability of the filter. A spillway at the top level of the storage to pass flow in excess of the culvert capacity (Figure 9.4.10).	Sandy loam filter media with high permeability and suitable vegetation.	Site Neighbourhood
Constructed Wetland	A storage basin excavated into the ground surface and partially formed by embankment on downslope side. Normally wet with bathymetry designed to support healthy range of aquatic plants. Ephemeral wetlands are subject to a cycle of wetting and drying that replicate natural processes.	Outlet pit and pipe culvert. High flow bypass (directs high flows away from wetland area). A spillway at the top level of the storage to pass flow in excess of the culvert capacity (Figure 9.4.12).	Aquatic plants growing in a suitable soil substrate.	Precinct
Managed Aquifer Recharge	An infiltration zone, in the floor of a basin, with good permeable connectivity to the groundwater system or a gravel filled soakaway with aquifer access via a bore pipe.	A permeable soil layer in the floor of the basin with connectivity to an aquifer A spillway at the top level of the basin to pass flow in excess of the permeable layer (Figure 9.4.13).	Removal of stormwater volumes decreases erosion of streams and reduces transport of urban pollutants. Normally requires pre-treatment.	Neighbourhood Precinct
Infiltration System	An infiltration zone, in the floor of a drainage pit, swale, basin, trench or	A porous floor in the base of the structure with	Removed contaminants and volumes of	Lot Site

Common Description	Storage Basin	Outlet Structure	Treatment Processes[a]	Typical Catchment Scale[b]
	pavement with good permeable connectivity to the groundwater system. Overflow from a rain water tank passed into bio-retention, raingarden, gravel filled trench or soakaway, normally dry, or directly to a local aquifer	connectivity to deeper sub-soils. A spillway, pipe or channel at the top level of the structure to pass flow in excess of the permeable layer (Figure 9.4.14 and Figure 9.4.15).	stormwater from flows. This further reduces transport of pollutants.	Neighbourhood
Stormwater Harvest Pond	A large storage pond formed by excavation into the ground surface and possibly formed by embankment on downslope side.	A pump system to extract water for use. A spillway at the top level of the pond to pass flow in excess of demand.	Reduce runoff volumes diminishes erosion of streams and reduces transport of urban pollutants. Normally requires pre-treatment.	Neighbourhood Precinct

[a]Note those devices without treatment processes may still provide water treatment benefits due to the effects of temporary storage and/or harvesting of runoff.

[b]Scale definitions taken from Book 9, Chapter 6

4.4. Concept Design

4.4.1. Overview

Concept design is an important phase in the overall infrastructure delivery process. It provides early insight into the likely physical characteristics of a facility, and allows design integration with other nearby infrastructure including, for example, stormwater conveyance infrastructure, open space, roads and buildings. If an approval is required, then the concept design will form part of the evidence needed for a submission. A concept design will also be needed to establish a financial budget.

The following sub-sections outlines the concept design phase of a typical volume management facility. Four concept design tasks are described:

- Choosing the best location for the facility (Book 9, Chapter 4, Section 4)
- Choosing the best design solution, having regard to the design objectives and site variables (Book 9, Chapter 4, Section 4)
- Preliminary sizing and configuration (Book 9, Chapter 4, Section 4)
- Collaboration and integration with other relevant professional disciplines (Book 9, Chapter 4, Section 4)

While these tasks are presented in this sequence, the tasks should not necessarily be completed in this sequence nor in a linear fashion. There is often a need for iteration and concurrent completion of design tasks. For example, collaboration and a preliminary sizing may be required to inform the selection of a preferred location. Once the preferred location is determined, the preliminary sizing must be updated.

Concept design can only commence once an overall catchment strategy has been established (refer to Book 9, Chapter 3) and design objectives determined (refer to Book 9, Chapter 4, Section 2). These foundational design aspects are assumed to have been resolved prior to implementing the following guidance. In particular a decision must be made as to the general position of the facility or strategy within the catchment. For example, it should be decided prior to commencing concept design whether the facility will be constructed to service a catchment comprising a single lot, a neighbourhood, or an urban precinct that is large in scale. With this overall constraint in mind, the following concept design tasks should be considered.

4.4.2. Choose a Location

The site chosen for a volume management facility is important to the success of the design. The site will have associated site variables, such as topography, soil types, catchment characteristics and groundwater characteristics. In some circumstances, the design may need to trade off some capabilities or require special features to completely respond to these site variables, and avoid constructability and long-term performance issues.

Where there is flexibility, it is best to choose a site that presents the smallest design challenge and meets the objectives for the project. The following discussion is intended to assist in this regard.

Topography

Volume management facilities may be located on or adjacent to the lowest point in the catchment to be serviced. This maximises the catchment area to be managed. Similarly the location may also need to capture flows from upstream conveyance infrastructure. If the site cannot easily service the relevant upstream sub-catchment then performance against the design objectives may be compromised.

While catchment hydrology (refer to Book 9, Chapter 6) is an integral part of the design process, even before such calculations are undertaken, the concept design should be informed by a general appreciation of the catchment draining to the proposed facility. As a minimum, the size of the catchment area draining to the facility needs to be determined so that preliminary sizing can be undertaken.

The location chosen may need to be adequately elevated (or able to be raised using an embankment), so that hydraulic performance of the outlet structure is not adversely influenced by backwater. This is a particular consideration for facilities that have treatment processes and vegetation or where the storage is intended to be well drained.

Areas in low-lying coastal districts must also consider the effects of high-tide and possible future changes to the tide level due to sea-level rise (refer to Book 4, Chapter 4 and Book 6, Chapter 5). Frequent backwater flooding from regional flood events should also be avoided, unless its impact can be assessed and proven acceptable.

The average ground slope in the location chosen should ideally be no steeper than 5%. Steeper sites are not precluded, however they will require more careful consideration of the type and shape of storage to avoid excessive earthworks. It may also introduce the need for vertical retaining wall elements which may be undesirable if they hamper access, introduce safety risks, increase maintenance and increase longer-term facility replacement and renewal costs.

Soils

Ideally the soils in the chosen location will be suitable for construction and sufficiently deep to avoid excavation into rock.

Where an embankment is to be formed, the soil properties should allow tight compaction in layers to form a cohesive matrix and stable slope within the range of 1 in 2 to 1 in 10.

The soils used to construct any embankments or spillways should also have a very low permeability, particularly where significant volumes of water are to be stored or where long-term water storage is intended. If the soil type is not suitable then other soil materials will need to be imported for blending or replacement, or other materials considered such as clay liners.

Sites with dispersive and acid sulphate soils will require a careful selection of storage solution. If unavoidable, then the design must include appropriate management measures.

Groundwater Characteristics

Where stormwater infiltration is one of the overall design objectives, the site selected must be underlain by geologic strata that allow this infiltration to occur. Long-term groundwater behaviour in the vicinity should also be profiled, and a site selected where the elevation of the infiltration zone is not substantially below normal groundwater levels.

If infiltration is not required or desired, then a site should be chosen where the groundwater profile is unlikely to intersect the storage profile. This will simplify construction and ensure the storage can be more easily drained.

The stream baseflow, flow regimes and runoff water quality characteristics will also be relevant where water quality improvement or stormwater harvesting or infiltration objectives are targeted.

The quality of the groundwater store should also be investigated and water quality criteria for infiltration will need to be observed in accordance with local guidelines and Australian and New Zealand Environment and Conservation Council (ANZECC, 2000).

Vegetation

The selected site should not require the damage or removal of valuable trees or large stands of native vegetation. If it is determined that this cannot be avoided then special approvals may be required and a flora and fauna specialist should be engaged to assist to provide advise the design team. An environmental offset planting may be necessary.

If the facility is intended to be vegetated then an appropriate depth and quality of surface soil is required to support healthy plant growth.

4.4.3. Choosing a Design Solution

A design solution should be selected that best targets the established objectives and provides an optimum response to the constraints and variables of the site. A listing of common design solutions is provided in Table 9.4.4.

This is a basic guide aimed to provide an indicative starting point for the inexperienced urban stormwater designer and should not be interpreted as a barrier to innovative strategies or a replacement for first principles analysis. Those with experience will recognise opportunities for hybrid solutions that have broader application. For example, a hybrid facility involving a detention (retarding) basin with managed aquifer recharge (retention) and stormwater harvesting (retention) may provide a more comprehensive design solution to a volume management problem and for protection of urban waterways.

It is noted that in Table 9.4.4 there are several solution and objective combinations that are flagged as “suitable with limitations”. This means that the solution may not always perform well with respect to the relevant objective, however it can in some circumstances. For example, a particular managed aquifer recharge facility may not normally provide control of peak discharge in large floods when the water levels in the aquifer are high. However, it may still afford some benefits in small floods and greater benefits if aquifer levels are low. Some further information about these possible limitations is provided in Book 9, Chapter 4, Section 5.

Table 9.4.4. Indicative Suitability of Common Volume Management Design Solutions

Solution	Control Peak Discharge	Improve Water Quality	Harvest or Infiltrate Stormwater
Detention (Retarding) Basin (refer Book 9, Chapter 4, Section 5)	Suitable	Not suitable	Not suitable
On-Site Detention (OSD) (refer Book 9, Chapter 4, Section 5)	Suitable	Not suitable	Not suitable
Rainwater Harvesting (refer Book 9, Chapter 4, Section 5)	Suitable with limitations	Suitable	Suitable
Bioretention Basin (refer Book 9, Chapter 4, Section 5)	Suitable with limitations	Suitable	Suitable with limitations
Constructed Wetland (refer Book 9, Chapter 4, Section 5)	Suitable with limitations	Suitable	Suitable with limitations
Managed Aquifer Recharge (refer Book 9, Chapter 4, Section 5)	Suitable with limitations	Suitable with limitations	Suitable
Infiltration System (refer Book 9, Chapter 4, Section 5)	Suitable with limitations	Suitable with limitations	Suitable
Stormwater Harvest Pond (refer Book 9, Chapter 4, Section 5)	Suitable with limitations	Suitable with limitations	Suitable

4.4.4. Preliminary Sizing and Configuration

The approximate physical footprint of the structure must be understood to confirm the availability of sufficient space at the site. Where the surrounding infrastructure has yet to be planned, space requirements can be communicated early to other members of the design team.

The size of the structure is the first aspect to investigate. Ultimately the size of the structure is determined by detailed calculation and modelling, however in the very early stages of planning it may be possible to use simple hand calculations and ‘rules of thumb’.

Preliminary sizing will depend on local rainfall conditions, climate patterns and performance criteria. A value is often selected based on prior experience with the design of other nearby facilities. For example, in the case of an infiltration measure, the estimated surface area can then be combined with length and width limitations to estimate the total requirement for land area at a preliminary level of accuracy.

The shape of the facility must then be considered. The shape of the facility will be largely governed by a combination of factors including:

- Minimising and balancing earthworks – to suit the site topography and drainage and minimise the volume of earthworks relative to the volume of runoff stored. At the same time have regard to the design of adjoining infrastructure such as stormwater conveyance, roads and buildings.

- Visual and landscape objectives – there may be visual and landscape objectives sought for the facility that might influence overall shape of the facility.

- Maintenance and safety objectives – Suitable allowance should be made for maintenance access and safe batter slopes.

- Achieving suitable length to width ratios – where the facility targets water quality improvement the length to width ratio must sit within a suitable range, typically between 3:1 and 10:1.

While determining the preliminary shape of the structure, consideration should also be given to the need for any vertical wall elements, the location of outlet structures and the position and alignment of any embankments.

4.4.5. Collaboration and Integration

The best integrated outcomes for an urban design project involving stormwater are only achieved when stormwater professionals are consulted at the very beginning.

The design of a volume management facility is a task best undertaken in close collaboration with the client representative, relevant stakeholders and the overall urban design team including:

- Urban Designers;

- Local authorities including Councils and government departments;

- Civil Engineers;

- Landscape Architects;
- Environmental Engineers, Geomorphologists and Ecologists; and
- Geotechnical Engineers.

This collaboration should occur early in the design process to minimise re-work and maximise the potential for integrated outcomes. For example good opportunities exist for co-location of volume management facilities within areas that also perform recreation, landscape and environmental functions.

Since the position of volume management facilities is often tightly controlled by site topography and hydraulic constraints, it is also important that the design is undertaken in conjunction with the overall bulk earthworks and stormwater conveyance solution to yield an overall efficient and low cost design.

4.4.6. Emergence of Volume Management Research

The use of volume management measures distributed throughout urban areas to assist in the management of peak discharges at the outlets of catchments has been the topic of emerging research and practical investigations since the 1990s by an increasing number of authors and practitioners (for example Joliffe (1997), Argue and Pezzaniti (2007), Argue and Pezzaniti (2009), Argue and Pezzaniti (2010), Argue and Pezzaniti (2012), Andoh and Declerck (1999), Coombes et al. (2000), Coombes et al. (2001), Coombes et al. (2002a), Coombes et al. (2015), van der Sterren et al. (2013), van der Sterren et al. (2014)).

More recently, investigations have also focused on understanding the performance of entire linked systems of water cycle management within urban catchments that can reveal the cumulative impacts of integrated or combined strategies that better represent real systems (Coombes et al., 2002b; Coombes, 2005; Walsh et al., 2012; Coombes and Barry, 2015). These issues are discussed in Book 9, Chapter 3. This body of research and practice has evolved since the previous version of ARR 1987 (Pilgrim, 1987) and represents significant new thinking in the stormwater industry.

Many authors have established that the use of volume management at a distributed scale may not be required to provide significant reductions in peak discharges at the property scale because reducing runoff volumes at the top of catchment provide substantial reductions in peak flows throughout catchments (for example: Herrmann and Schmida (1999), Andoh and Declerck (1999), Argue and Scott (2000), Vaes and Berlamont (2001)). Argue and Scott (2000) used a large catchment scale model to conclude that distributed peak discharge control (on-site detention) and volume management (rainwater harvesting) systems produce similar hydrographs at the catchment outlet. It was acknowledged that the peak discharges on a lot scale may be larger for volume management than for flow management. However, it was found for medium to large catchments that the cumulative effect of volume reductions obliterates the effect of peak discharges at individual sites. This indicates that the cumulative effects of distributed reductions in stormwater runoff volumes can be significant at a catchment scale due to the reduction in overall volume discharged to the catchment outlet (refer to Book 9, Chapter 3). These results are consistent with the basic elements of peak flows which are volume and time. Reducing either element must reduce peak flows within the catchment.

Coombes et al. (2001), Coombes et al. (2003) also found that at the lot scale the flow management (detention) systems reduced the peak discharge at the lot scale and volume management (rainwater harvesting) provided smaller changes in peak discharges at lot scale but significantly reduced the volumes of stormwater runoff which reduced peak discharges at the street and catchment scale. It was argued that flooding is a volume driven process and peak discharges at the lot scale had little or no bearing on the floods at a catchment scale. Use of first principles processes such as continuous simulation and detailed systems analysis rather than empirical assumptions (for example antecedent conditions associated with event based analysis) has also revealed that the shape of catchment hydrographs may be significantly altered by distributed and integrated solutions within catchments (for example; Coombes and Barry (2009), Coombes (2015)). van der Sterren (2012), Burns et al. (2013) and Coombes (2015) highlight the benefits of replacing the common design requirements with treatment trains on properties and throughout urban areas to manage peak discharges and flow regimes throughout and at the outlet of urban catchments.

4.4.7. Use of Computer Models

A coupled analysis of storage basin volume and outlet capacity is necessary in order to determine the most appropriate configuration for a facility. This analysis is usually iterative. Firstly, dimensions of the storage basin and outlet are estimated and tested by numerical calculation and then progressively adjusted to achieve the design objectives. This is normally undertaken using computer models that have been developed to assist with these calculations.

The design and analysis of these facilities must include the interactions with other stormwater management facilities and urban form in the catchment and catchment behaviours. The adopted modelling approach should also use rainfall time series and resolve full hydrographs of a total duration that is relevant to the objective being analysed. For peak discharge control, this may only be minutes or hours. For water quality improvement and stormwater harvesting applications, this may be years or decades. The model must have sufficient catchment resolution and detail to adequately represent the linked hydrologic processes in the catchment. Lumped models that simplify catchment representation and behaviours should be used with caution.

The modelling approach should allow different storm scenarios to be tested since the performance of a volume management facility may be highly sensitive to the selected storm characteristics and volumes. For example, volume management facilities will have a greater impact on peak discharges under conditions where the storm burst occurs in front of a storm, rather than under conditions when the storm burst occurs towards the back of a storm, when the detention storage is already partially full.

A designer may therefore need to consider using an ensemble of complete storms with a storm burst of around the critical duration or a storm burst only to determine the benchmark condition(s) (Phillips and Yu (2015); Book 9, Chapter 6). If a design approach adopts a storm burst only approach, then for a given Annual Exceedance Probability (AEP) the peak flows are assessed for a range of storm burst durations and the storm burst duration that gives the highest peak flow is adopted as the critical storm.

If a design approach adopts an ensemble of complete storms of a given AEP, then the designer will need to determine if the benchmark condition is to be based on the 50th percentile peak flow or on a different percentile of peak flow. Preliminary testing indicates that adopting the 50th percentile is a very good indicator of the results from more complex Monte-Carlo approaches in most circumstances. Ultimately, the decision of what percentile of peak flow to adopt can be informed through identifying the level of risk the community is willing to accept within the catchment.

Once a base model is established, which includes the proposed facility, the model should be capable of iterative changes to the dimensions of the storage and the outlet structure. Using a judgement driven and iterative approach, the model is used to determine an optimised configuration that results in the required hydrologic performance for the selected range of storms.

For more detailed guidance regarding the use of computer modelling in urban stormwater design refer to Book 9, Chapter 5 and Book 9, Chapter 6.

4.5. Detailed Design Considerations

This section provides introductory level detailed design guidance for each of the most common volume management facility types, as listed in Table 9.4.3. Furthermore comprehensive design guidance reflecting local design standards should be sought from the relevant local stormwater authority. References to some useful guidelines are provided in each of the following sections.

4.5.1. Detention Basins

Detention basins, also sometimes called retarding basins, are measures which temporarily store stormwater to reduce peak discharge. Outflows are typically controlled by a low-level pipe or culvert and a high-level overflow spillway as shown in Figure 9.4.5.

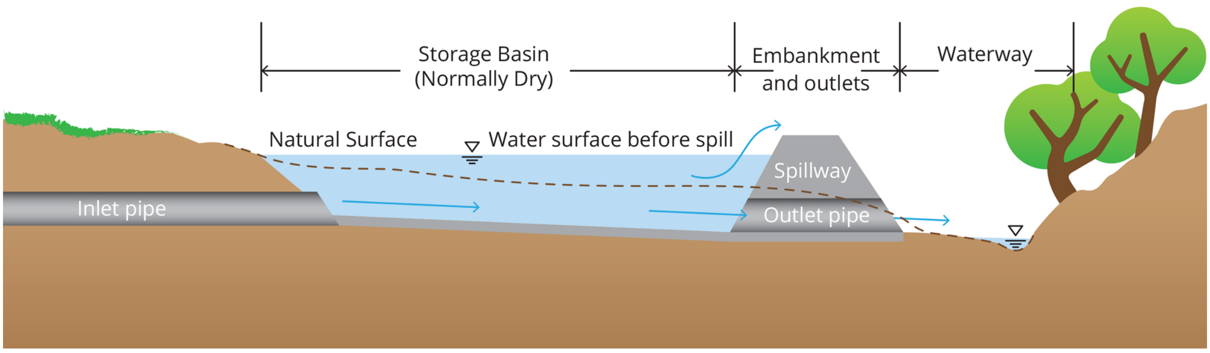


Figure 9.4.5. Detention Basin Typical Section

Detention basins can be designed to suit a range of catchment sizes. Community and regional scale basins may have considerable community benefits as areas for recreation and may be built around specific sizes and shapes of fields for sports such as football, netball and cricket. The sides of basins are usually sloping earth embankments, suitable for occasional spectator use. Basins used for passive recreation may include stands of trees (within the basin but not on any fill embankment), lawns and other vegetation.

Basins may be placed directly across a watercourse, or located off-stream, with flows in excess of a certain flow rate being diverted into them. They can be arranged in a widened section of drainage easement zoned both for recreation and drainage purposes.

Detention basins themselves are not suited to the improvement of water quality or harvesting and infiltration of stormwater. However other types of volume management facilities can be nested inside. For example a constructed wetland can be located in the floor of a large detention basin storage to also target water quality improvements.

Available Guidelines

There are many guidelines on community and regional detention including ACT Department of Urban Services (1998), Hobart City Council (2006), Department of Water, Western Australia (2007), Melbourne Water (2010), Queensland Department of Energy and Water Supply (2013). These guidelines can be readily used for designing and modelling detention systems, using the modelling and storm patterns as described in Book 9, Chapter 6.

Detailed Design Considerations

Flood Capacity

The final sizing of any basin should be completed with the aid of a computer model. The selected model must accurately simulate the hydraulic behaviour of the basin outlet, especially when a partially full pipe flow or tailwater submergence occurs (Queensland Department of Energy and Water Supply, 2013). When located in-stream, the hydraulic modelling should also represent the stream conditions and the stream flows discharging through the basin in addition to the urban areas directed to it.

Large community and regional basins can be considered dams, as they can store significant volumes of stormwater, and therefore may pose a potential threat to communities residing downstream of a basin. As a result, the design must have regard to the ANCOLD (Australian National Committee on Large Dams, 2000) guidelines. A detailed risk assessment of a storm exceeding the Dam Crest Flood should be considered in the design of a detention basin within an urban area due to the potential severe consequences of the sudden failure of a basin on any urban development located on the floodplain downstream.

Detention basins should be designed with a flood capacity to convey appropriate extreme storms safely through the basin in accordance with the Hazard Category of the basin as defined by ANCOLD, as is the case for conventional dams.

An ‘Initial Assessment’, as defined by ANCOLD’s guidelines within the ANCOLD (2000a) should be undertaken for any proposed detention basin to determine the hazard category of the structure.

Depending on the findings of the ‘Initial Assessment’ a more detailed assessment (ANCOLD, 2000b) including a Dam Break analysis for both ‘flood failure’ and ‘sunny day’ scenarios may be required.

With increasing urbanisation there are now many catchments which contain a series of detention basins. Each basin within a catchment should be investigated not only individually but also collectively within the catchment, including all basins modelled as a whole (Melbourne Water, 2010).

In addition, two further issues should be considered:

- The consequences of one basin failure cascading downstream into lower basins should be evaluated; and
- The effect of long period releases from upper basins superimposing on flows through lower basins may require a revision of the basins’ operation throughout the catchment.

Embankment Design

The embankments of detention basins should be designed using appropriate stability analysis and geotechnical design practices. Particularly, appropriate foundation treatment should be specified. For earthen embankments suitable compaction levels, vegetation cover and stabilisation should be specified and protection provided to cater for cracking or dispersive soils. Impervious zones of an earthen embankment should take the form of a centrally located 'core' rather than an upstream face zone to reduce the effects of drying which may lead to cracking.

If the earth fill for any embankment is taken from borrow areas, these areas should be kept as far away from the embankment(s) as practicable. Should the borrow area penetrate any alluvial sand layers or lenses, the embankment's cut-offs should be taken to at least one metre below the estimated depth of such sand layers/lenses at the detention basin floor.

Chimney intercept filters and filter/drainage blankets should be used for all high and extreme hazard category detention basins. Such filters may also be required for lower hazard category detention basins. All earthen embankments constructed from dispersion soils must have a chimney filter and downstream filter/drain (Melbourne Water, 2010).

Suggested basin freeboard requirements for a variety of basins are provided in Table 9.4.5.

Table 9.4.5. Detention Basin Freeboard Requirements (Adapted from Queensland Department of Energy and Water Supply (2013))

Situation	AEP	Maximum Depth or Level
Basin Formed by Road Embankment		
(a)	5%	Bottom of pavement box
(b)	2%	0.3 m below edge of shoulder
Basin Formed by Railway Embankment	2%	Underside of ballast
Large Basins with Separate High Level Spillway	1%	Embankment crest with freeboard ≥1% AEP storage depth and with minimum freeboard = 0.3 m [1]

External earthen embankment slopes and their protection should take into account long term maintenance of the structure. The side slopes of a grassed earthen embankment and basin storage area should not be steeper than 1(V):4(H) to prevent bank erosion and to facilitate maintenance and mowing.

The surfaces of an earthen embankment and overflow spillway must be protected against damage by scour. The degree of protection required is subject to the calculated flow velocity.

The following treatments are recommended as a guide (NSW Government, 2004):

- V ≤ 2 m/s a dense well-knit turf cover using for example kikuyu;
- 2 m/s < V < 7 m/s a dense well-knit turf cover incorporating a turf reinforcement system; and
- V ≥ 7 m/s hard surfacing with concrete, riprap or similar.

Practical maintenance access should be provided to the full length of the embankment and any hydraulic structures passing through it.

Basin Floor

The floor of basin shall be designed with a suitable grade that provides positive drainage to the basin outlet and to prevent water logging. Detention basins may require underdrains to positively drain the bottom of the detention facility for ease of maintenance. If there are frequent trickle flows entering the basin then a low flow channel or pipe passing through the basin should be considered.

Primary Outlets

The key function of primary outlets is to release flows from a detention basin at the designed discharge rate. Some typical primary outlets are shown in Figure 9.4.6. Book 6 details how these outlets can be hydraulically designed.

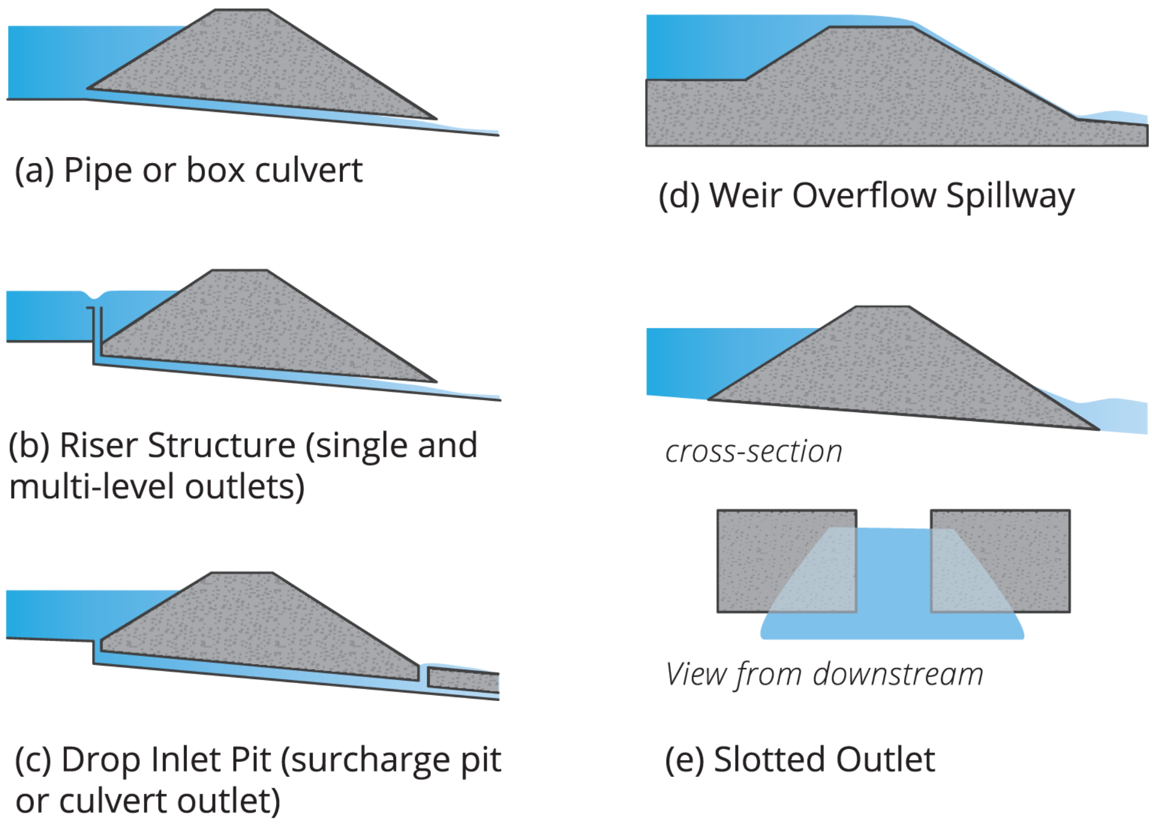


Figure 9.4.6. Typical Detention Basin Primary Outlets

Pipe or box culverts are often used as outlet structures for detention basin facilities. The design of these outlets can be for either single or multi-stage discharges. A single stage discharge system typically consists of a single culvert entrance system, which is not designed to carry emergency overflows (for example, when pipes are blocked). A multi-stage inlet typically involves the placement of a control structure at the inlet to the culvert. In particular, details on the hydraulics of rectangular weirs are given in Book 6, Chapter 3 and Book 9, Chapter 5.

Secondary Outlets

In general, the capacity of secondary outlets (typically spillways) should be based on the hazard rating of the structure as defined by the ANCOLD seven level rating system. The hazard rating defines the required 'Fall back' Design Flood. In some cases where the required 'Fall back' Design Flood is considered to be impractical, a full risk assessment of the basin may allow a lesser capacity spillway in line with ALARP (As Low As Reasonably Practicable) principles(Melbourne Water, 2010).

The design capacity of spillways should account for the possible reduced capacity of primary outlets which have the potential to become blocked during a major storm. The assessment of the possible blockage should be undertaken in accordance with the guidance provided in Book 6.

Recommendations for the design of outlet structures are provided by (ASCE, 1985) while the Design of Small Dams US Bureau of Reclamation (1987) provides procedures for the sizing and design of free overfall, ogee crest, side channel, labyrinth, chute, conduit, drop inlet (morning glory), baffled chute and culvert spillways.

Details on the hydraulics of rectangular weirs, sharp-crested rectangular weirs, broad-crested rectangular weirs, trapezoidal weirs, circular-crested weirs and compound weirs are provided in Book 6, Chapter 3.

4.5.2. On-site Detention

In many urban areas detention has been implemented, and in particular since 1975 the use of detention basins has been widespread in NSW (Institution of Engineers, Australia, 1985). However in urbanised areas the available sites for large detention basins (as described previously in Book 9, Chapter 4, Section 5) are limited or are fully utilised over time.

To avoid exacerbating what can be already substantial flooding problems in an urbanised catchment, planning and development controls are sometimes implemented at the lot scale to mitigate the impact of increased impervious surfaces. These are commonly described as On-Site Detention (OSD) as shown in Figure 9.4.7.

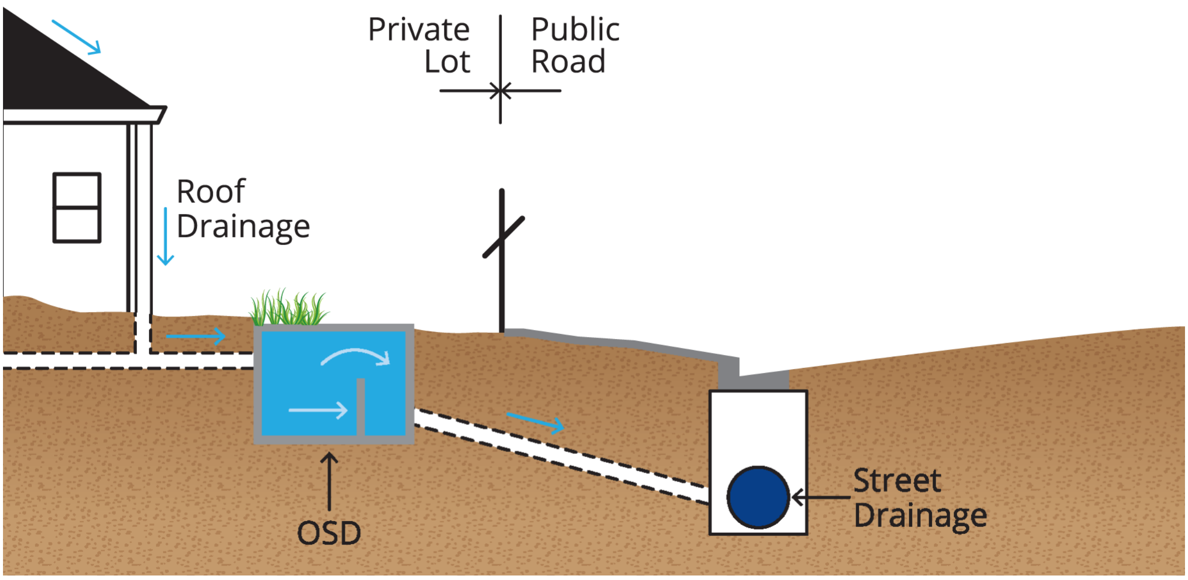


Figure 9.4.7. Typical Section Through a Below Ground On-Site Detention

In New South Wales, OSD was developed and first implemented by Ku-ring-gai Council, closely followed by Wollongong City Council (O'Loughlin et al., 1995). Since then many councils in Greater Sydney and elsewhere have implemented OSD systems. Other Councils outside of NSW have also adopted On-Site Detention, such as Hobart City Council (TAS), City of Casey (VIC), Manningham City Council (VIC), Melton Shire Council (VIC) and the City of Tea Tree (SA).

It is important to note that the imposition of OSD requirements at the lot scale is often done on the assumption that there are broader flood benefits at a catchment scale. However, in some cases there may be little or no catchment wide benefit from OSD, as the overall volume of runoff is not reduced, merely detained for a period of time. This effect is not always sufficient to influence catchment scale floods. OSD performance is also sensitive to the temporal pattern of rainfall.

Establishment of OSD policy therefore needs careful assessment at the outset using a catchment wide strategy to ensure the overall catchments to which the policy is intended to be applied are indeed suitable.

Available Guidelines

There are many guidelines on the sizing or design of OSD, for example Department of Irrigation and Drainage (2000), Upper Parramatta River Trust (2005), Hobart City Council (2006) and Derwent Estuary Program (2012). These guidelines can be readily used for designing OSD systems, using the modelling approaches outlined in Book 9, Chapter 6.

These documents can assist in the design of OSD systems, however, designers are encouraged to determine if the method identified in the guidelines are consistent and make suitable for using the contemporary flood estimation techniques identified in Book 9, Chapter 6 and the issues identified in Book 9, Chapter 3.

Detailed Design Considerations

Flood Capacity

Historically, the primary objective of OSD controls was to manage flooding in a 1% AEP event only. Further implementation and development on OSD has resulted in many authorities now requiring OSD systems to reduce the post-development flows to adopted benchmark peak discharges over a range of AEPs up to and including the 1% AEP event.

OSD discharge control requirements should be based on a catchment wide assessment. A catchment wide assessment has been typically downscaled to site control requirements, such as:

- Permissible Site Discharge (PSD) or Site Reference Discharge (SRD), which are defined as the maximum allowable discharge leaving the site (determined using catchment-based assessment of lot-based measures) with PSD giving a single discharge rates and SRD giving multiple discharge rates for different rainfall frequencies; and
- Site Storage Requirement (SSR), which is defined as the volume required for overall storage.

It should be noted that if the objective of OSD control is to manage flooding in a 1% AEP event only then typically only a single set of PSD and SSR values are defined. However, where authorities require OSD systems to perform over a range of AEPs a nest of frequency staged storages and outlets is required with multiple PSD and SSR values. An example of an OSD design is provided in Figure 9.4.8.

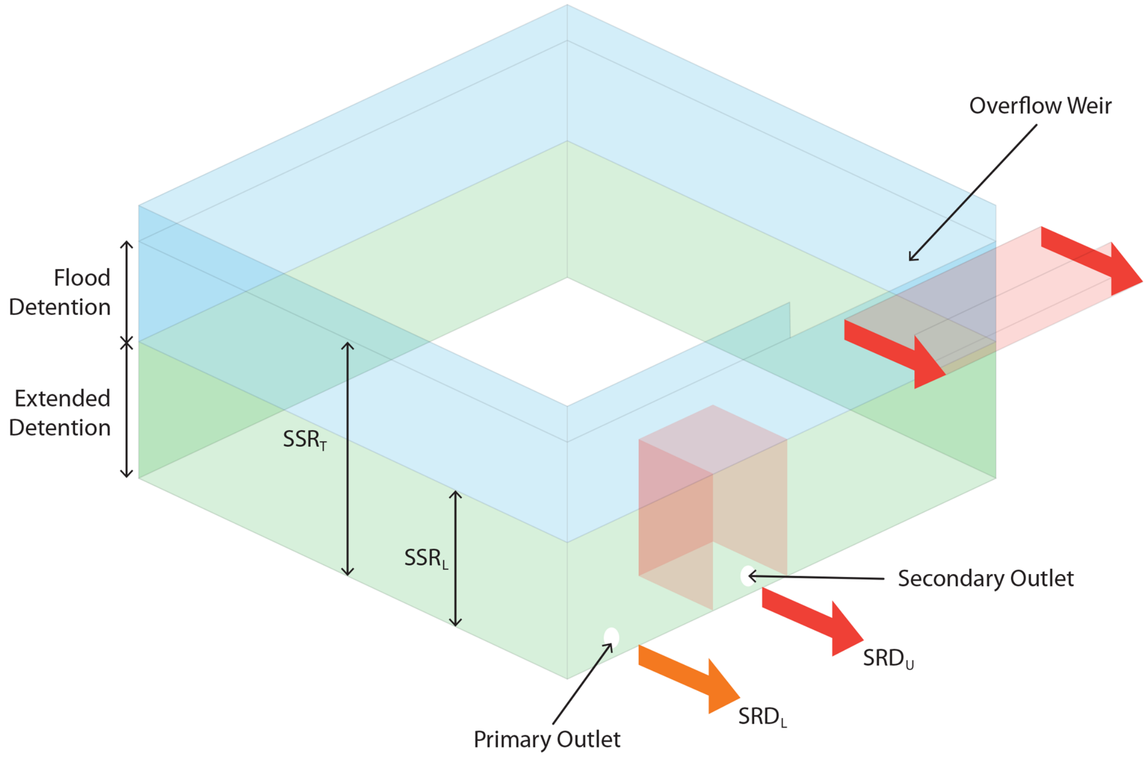


Figure 9.4.8. Frequency Staged Below Ground On-Site Detention System (adapted from Upper Parramatta River Trust (2005))

In the event that catchment wide assessments have not been conducted, one of the following site controls can be applied to enable the design of OSD systems:

1. The post-development flows of the subject site should be controlled to meet the pre-development flows for the site for a range of complete storms; or

2. Determine the capacity of the drainage system and divide by the area of lots that drain to the system. This gives an indicative estimate of the amount of the unit runoff (i.e. the PSD).

Either of these approaches are not as effective as designs based on a holistic catchment assessment, but may assist in the short term in managing nuisance flows in existing systems immediately downstream from sites.

On-Site Detention Types

OSD systems may comprise above-ground storage or underground storage or a combination of both. Above ground storage has advantages in terms of flexible configuration of site levels to achieve the required storage volume, capacity to incorporate retention through infiltration and pollutant removal landscaping features, reduced construction cost and easier maintenance. The advantages of underground storage are typically a reduced footprint in comparison to above ground storages and limitation of ponding on runoff on the surface. It is critical to select an appropriate storage type by considering the site layout, costs and effectiveness of OSD.

Above Ground On-Site Detention

OSD systems may comprise above ground storage or underground storage or a combination of both. It is critical to select an appropriate storage type by considering the site layout, costs and effectiveness of OSD.

Above ground storage has advantages in terms of flexible configuration of site levels to achieve the required storage volume, capacity to incorporate water quality treatment through infiltration and treatment media, low construction cost and potentially low maintenance.

The main types of above ground storages include landscaped storages, parking and paved storages, and rain water tanks with dedicated airspace for detention.

Where storage is not provided by a rain water tank the typical requirements listed in Table 9.4.6 should be considered.

Table 9.4.6. Above Ground On-Site Detention Storage Design Considerations

Design Aspect	Typical Considerations
Structural Adequacy	Design of surrounding embankments or retaining walls should consider structural and geotechnical aspects such as the need for reinforcement, compaction requirements and stable slopes. This includes when the storage is both full and empty.
Storage Configuration	Ponding depths shall not exceed the maximum storage depth requirements required by local standards. As an initial guide a maximum of 0.6 m is suggested for landscape areas with low pedestrian use (Department of Irrigation and Drainage, 2012). A Council may approve deeper ponding in individual cases where it is demonstrated that safety issues have been adequately addressed. For example, warning signs and or fencing should be installed where the depth exceeds0.6 m or adjacent to pedestrian traffic areas. Where ponding occurs in areas for recreational purposes (e.g. a playground) suitable velocity and depth should be selected to ensure the safety of children and the elderly. The storage volume should be increased by 20% to compensate for the potential loss of storage due to construction inaccuracies and the build-up of vegetation growth over time.
Floor Slope	The minimum ground surface slope should be 1.0%, while the desirable minimum surface slope is 1.5%.
Vegetation and Soils	Subsoil drainage around the outlet should be designed to prevent the ground becoming saturated during prolonged wet weather. Appropriate plant species for the vegetated areas should be selected that can withstand prolonged inundation and frequent wetting and drying. Any direct inflow point into a vegetated system (e.g. roof drainage or driveway runoff) should include a small energy dissipation device to reduce velocity and prevent erosion of the basin floor. Mulch utilised in the above ground storages should not be able to float and plants should be capable of withstanding frequent inundation as per the design depth and frequency.

Design Aspect	Typical Considerations
Overflow	An overflow should direct the flows to the legal point of discharge in a controlled and safe manner.
Freeboard	There should be freeboard above the stored flood level and adjacent habitable floor levels in accordance with local standards.
Safety and Access	Balustrades (fences) must comply with the Building Code of Australia (refer to Section D2.16 of the Code), while safety fences should comply with any legislated requirements for swimming pool fencing. Surface storages should be constructed so as to be easily accessible, with gentle side slopes permitting walking in or out. A maximum gradient of 1(V):4(H) (i.e. 1 vertical to 4 horizontal) should be required on at least one side to permit safe egress in an emergency. Where steep or vertical sides are unavoidable, due consideration should be given to safety aspects, such as the need for fencing or steps or a ladder, both when the storage is full and empty.
Frequency of Inundation	Frequent ponding can create maintenance problems or personal inconvenience to property owners. The initial 10%-20% of the storage should be provided in an area able to tolerate frequent inundation, e.g. a paved outdoor entertainment area, a permanent water feature, or a rock garden. Alternatively, a frequency staged storage approach should be adopted.

Below Ground Storages

Below ground storage tanks may be considered under the following conditions:

- Infeasible to construct above ground storages due to site constraints or topography; and
- Frequent inundation areas causing maintenance problems and inconvenience to the property owners or community members.

Below ground OSD storage tanks are usually made of reinforced concrete and can be pre-cast or cast in-situ to meet individual site requirements. When designing below ground tanks then typical design considerations include those listed below in Table 9.4.7.

Table 9.4.7. Below Ground On-Site Detention Storage Design Considerations (Department of Irrigation and Drainage, 2000), (Department of Irrigation and Drainage, 2012)

Design Aspect	Typical Considerations
Structural Adequacy	Storages must be structurally sound and be constructed from durable materials that are not subject to deterioration by corrosion or aggressive soil conditions. Tanks must be designed to withstand the expected live and dead loads on the structure, including external and internal hydrostatic loadings. Buoyancy should also be checked, especially for lightweight tanks, to ensure that the tank will not lift under high groundwater conditions. The soils and their impacts on concrete structure should be assessed to ensure that the correct structural specification is made.
Storage Configuration	Site geometry will dictate how the OSD system configured in plan. While a rectangular planform is typical and offers certain cost and maintenance

Design Aspect	Typical Considerations
	advantages site constraints will sometimes dictate a variation from a rectangular planform.
Floor Slope	To permit easy access to all parts of the storage for maintenance, the floor slope of the tank should be in the range 1% to 10%.
Ventilation	An important consideration for below ground storage systems is ventilation to minimise odour problems. Ventilation may be provided through the storage access opening(s) or by separate ventilation pipe risers and should be designed to prevent air from being trapped between the roof of the storage and the water surface.
Overflow	An overflow system must be provided to allow the storage to surcharge in a controlled manner if the capacity of the tank is exceeded due to a blockage of the outlet pipe or in the event of a storm with a magnitude greater than the design storm.
Freeboard	There should be freeboard above the stored flood level and adjacent habitable floor levels in accordance with local standards.
Safety and Access	A suitable amount of access hatches should be provided to enable contractors to readily adopt working in confined spaces techniques and equipment. Below-ground storage tanks should be provided with openings to allow access for maintenance. An access opening should be located directly above the outlet for cleaning when the storage tank is full and the outlet is clogged. A permanently installed ladder or step iron arrangement should be provided below each access opening if the storage is deeper than 1200 mm.
Frequency of Inundation	There should be no constraints on the frequency of inundation of the storage basin.

Below ground storage could be provided by modular system which could include one or more parallel rows of pipes connected by a common inlet and outlet chamber. The size of a modular unit is determined by the storage volume requirements, site constraints and the number of conduits or modular units which can be installed. When designing modular storage systems typical design considerations are similar to the design considerations for below ground storages as outlined above. Further guidance on conduit storage systems is provided by Department of Irrigation and Drainage (2000), Department of Irrigation and Drainage (2012).

Combined Above and Below Ground Storage

The designer of an OSD system faces a challenging task to achieve a balance between creating sufficient storages that are attractive and complementary to the architectural design, minimising personal inconvenience for property owners/residents and limiting costs.

These demands can be balanced by providing storage with a frequency staged storage approach. Under this approach, the design of OSD adopts combined storages multiple outlet approach, which can consist of an above ground storage and below ground storage. Underground storage is designed to store runoff for more frequent storm events, whilst the remainder of the required storage, up to the design storm event, is provided as above-ground storage.

This approach is likely to limit the depth of inundation and extent of area inundated in the above ground storage so that the greatest inconvenience to property owners or occupiers occurs very infrequently. It recognises that people are generally prepared to accept flooding which causes inconvenience as long as it does not cause a significant damage or does not happen too often. Conversely, the less the personal inconvenience the more frequently the inundation can be tolerated.

Outlet Structures

The outflows from OSD systems are typically controlled by orifices. Details on the hydraulics of orifices are discussed in Steward (1908); Medaugh and Johnson (1940); Lea (1942); Brater et al0. (1996); Bryant et al. (2008) and USBR (2001).

The orifice outlets should have a minimum internal diameter of at least 25 mm and need to be protected by a mesh screen to reduce the likelihood of the primary or secondary outlets being blocked by debris.

Upstream Drainage

The stormwater drainage system (including surface gradings, gutters, pipes, surface drains and overland flowpaths) for the property must:

- be able to collectively convey all runoff to the OSD system in a 1% AEP event with a duration equal to the time of concentration of the site; and
- ensure that the OSD storage is by-passed by all runoff from neighbouring properties and any part of the site not being directed to the OSD storage, for events up to and including the 1% AEP event.

Maintenance

While Councils are ultimately responsible for ensuring these systems are maintained through field inspections and enforcing the terms of any positive covenant covering OSD systems, the designer's task is to minimise the frequency of maintenance and make the job as simple as possible (Upper Parramatta River Trust, 2005).

4.5.3. Rain Water Harvesting

Rain water harvesting at the property or lot scale has been historically used for water supply throughout Australia and for the management of stormwater runoff in cities since the 1990s. Rain water harvesting systems that provide water supply for more frequent indoor uses can reduce catchment runoff (peak flow and volumes), improve urban stormwater quality and provide a supplementary water source. The effectiveness of distributed rain water harvesting solutions for management of stormwater within and at the outlet of catchments is dependent on the number of facilities, integration with other strategies in the catchment, density of development, climate regimes, and magnitude and frequency of demand for rain water supply.

Available Guidelines

Further guidance on rain water harvesting can be found in the following documents:

- Guidance on Use of Rain water Tanks (enHealth, 2012)
- Rain water Tank Design and Installation Handbook HB 230 refer to http://www.rainwaterharvesting.org.au

- Interim Rain water Harvesting System Guidelines (NSW Department of Planning and Environment, 2015)
- Design and Operation of Rain Water Harvesting Systems refer to http://urbanwatercyclesolutions.com

Detailed Design Considerations

Modelling

Rain water harvesting systems were historically designed and considered as a stand-alone facility. This process results in assumptions that rain water harvesting systems do not contribute to the control of quantity or quality of stormwater discharges from a site or throughout a catchment. It was often argued that rain water harvesting does not provide these benefits due to the uncertainty associated with antecedent conditions of storm events (how full is the storage prior to the design storm?). Methods to determine the antecedent conditions in rain water storages prior to storm events and for design rain water harvesting systems were developed and demonstrated by Coombes et al. (2001), Coombes et al. (2002b), Hardy et al. (2004), Coombes (2005), Coombes and Barry (2007), Coombes and Barry (2009) and Coombes (2009). This applied research and monitoring has provided a design process for rain water harvesting systems that requires continuous simulation at sub-daily intervals – preferably six minute time steps to determine the dynamic airspace (drawn down of storages by water demands). This process can also determine any detention airspace requirements of rain water storages prior to given storm events for allow integration with surrounding stormwater management strategies and use in catchment models reliant on design storms. These concepts have been applied by Phillips et al. (2005) for example and can be used to address concerns about antecedent conditions in linked stormwater designs. The design process for rain water harvesting systems has been enhanced by many authors including Burns et al. (2013) and van der Sterren (2012) (for example) to also account for flow regimes to protect urban waterways.

The rain water or stormwater harvesting system should be designed using continuous simulation (as identified in Coombes and Barry (2007)) and should consider the following:

- Rainfall at the site;
- Potential magnitude and frequency of rain water use and any rate of leakage from a leaky tank;
- Roof or catchment area draining to the tank;
- Size of inlet configuration, overflow and use (e.g. can the rate of flow be discharge into the tank and out of the storage without surcharging); and
- When underground – the backflow potential from downstream systems.

Upstream Drainage

The design of rain water harvesting systems can include gutter guards, leaf diverters, first flush devices and filter socks can limit the transfer of sediment and debris into rain water storages. Mesh screens on inlets, outlets and overflow devices will exclude animals and mosquitoes and other insects from entering storages therefore minimising the risk of harmful microorganisms and disease-carrying mosquitoes entering the tanks.

Runoff that is not collected in the storage and overflows from the storage should be diverted away from storage foundations, buildings or other structures (enHealth, 2012).

Storage Location

The location of the storage infrastructure will be dependent on aesthetic and space requirements for the chosen device. The tank must also be located where sufficient roof area can be drained by gravity to the top of the tank.

If the storage system is below-ground, site soil characteristics and surface flows will need to be considered. Surface flows should be prevented from entering the tank and soil conditions are particularly important if there are salinity or acid sulphate soil concerns which would affect the integrity of the structure (Department of Water, Western Australia, 2007).

Pumps and Connections

The tanks should be connected to internal domestic demands, typically toilet flushing. Appropriate flow rates need to be maintained for the occupant and therefore the majority of rain water supply systems will require a pump to distribute water to internal and external plumbing fixtures. A pump should be sized to balance the required flow and pressure for the intended uses of the rain water from the storage while minimising energy use. Generally flows of less than 30 L/min are suitable for most residential applications (NSW Department of Planning and Environment, 2015).

Local government or State Government policy requirements may exist in regards to pumps and connections.

Outlet

Runoff that is not collected in the storage and overflows should be diverted away from storage foundations, buildings or other structures (enHealth, 2012). This water should be directed into gardens, infiltration systems or the public stormwater management network. The overflow water should not be allowed to cause nuisance to neighbouring properties or to areas of public access.

Tanks with Dedicated Airspace

The increased uptake of rain water harvesting also creates an opportunity to adopt an integrated approach to lot scale stormwater management by designing the facility to control of peak discharge and harvest runoff volume. This approach may result in rain water tanks with three outlets, one for use of rain water (e.g. connected to selected indoor plumbing or garden irrigation) down the bottom of the tank, one for orifice discharge (i.e. the OSD outlet) half way up the tank, and the third outlet is an overflow at the top of the tank (as per Figure 9.4.9) as originally proposed by Coombes et al. (2001). The dedicated airspace above the minimum level outlet provides for additional attenuation of peak discharges.

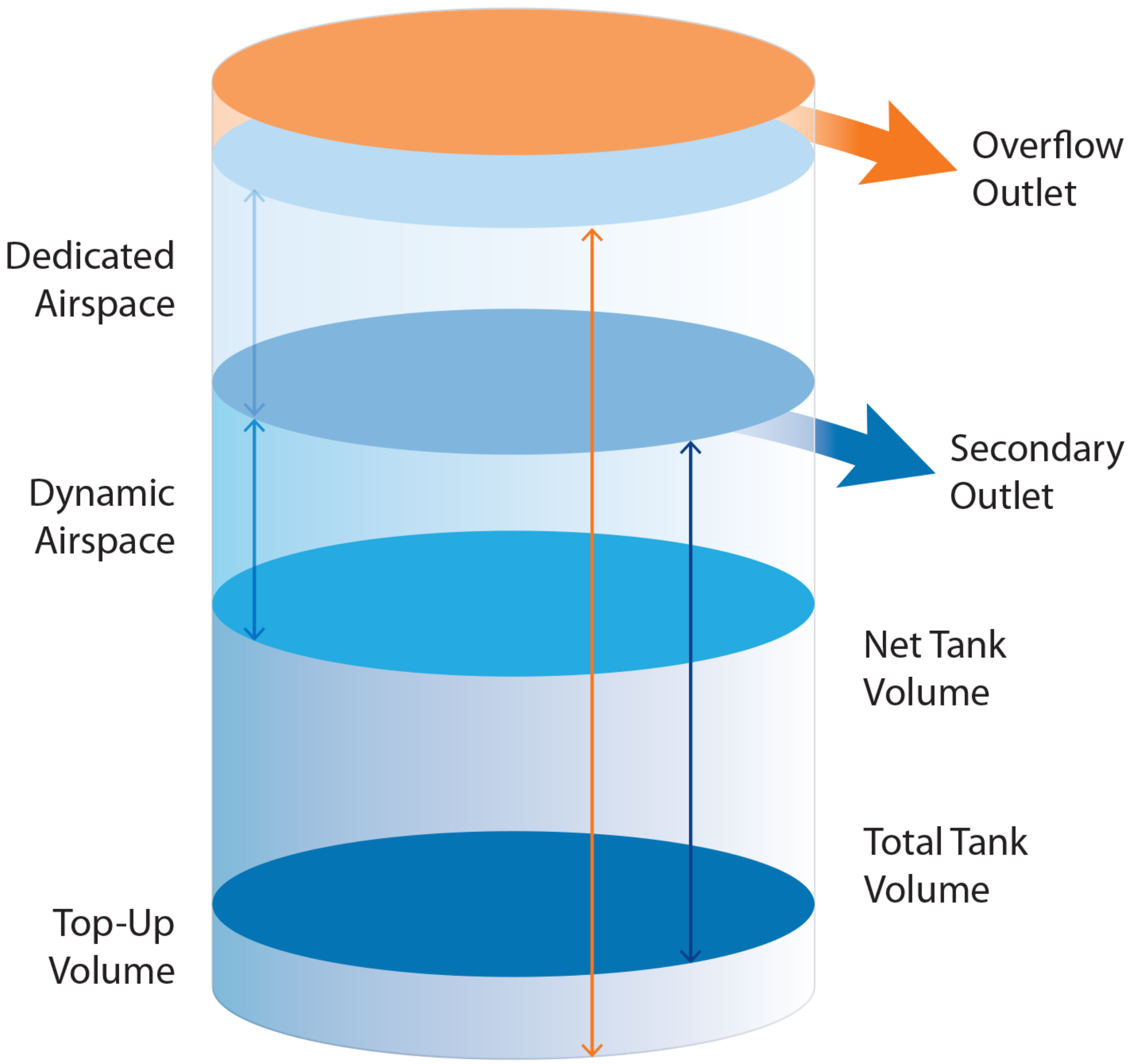


Figure 9.4.9. Rain Water Tank with Dedicated Air Space (adapted from Coombes et al. (2001))

4.5.4. Bioretention Basins

A bioretention basin is a shallow depression with a network of under-drainage and a soil-based filter media (refer to Figure 9.4.10). The filter media is vegetated with plants that tolerate periodic inundation. Stormwater is directed into the basin and percolates vertically through the soil and plant root zone providing water treatment. These facilities are sometimes also referred to as ‘rain gardens’.

Bioretention basins primarily target water quality treatment objectives for small to medium catchments. In some circumstances it may also contribute to peak discharge control. Certain design types can also be used to promote the infiltration of stormwater into the groundwater system.

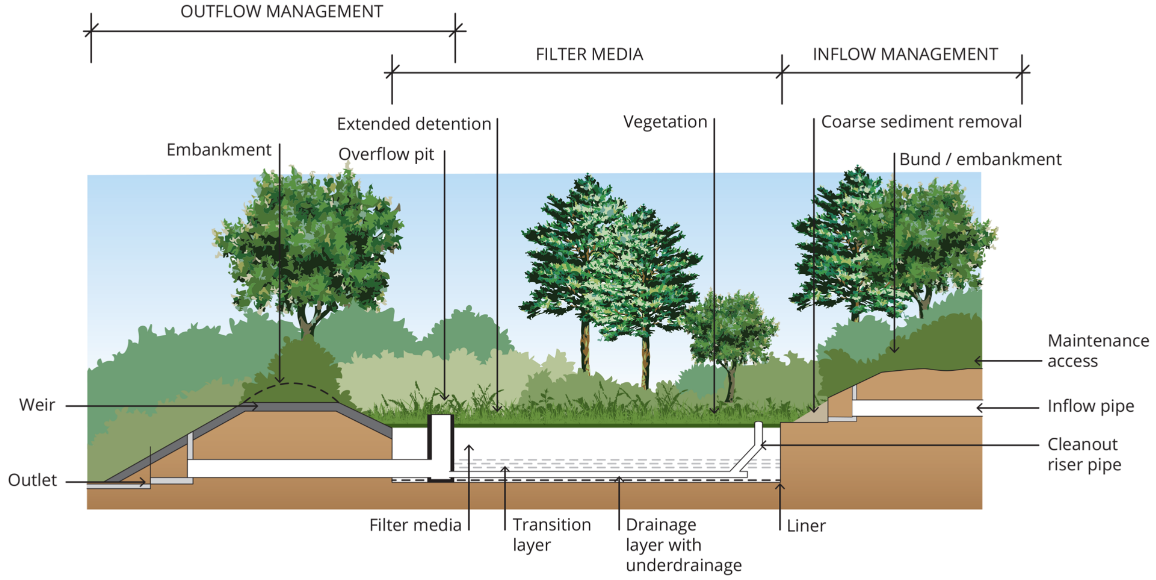


Figure 9.4.10. Components of a Bioretention Basin (Healthy Waterways by Design, 2014)

Available Guidelines

- Healthy Waterways by Design (2014) “Bioretention Technical Design Guidelines”
- Department of Water, Western Australia (2007) “Stormwater management manual for Western Australia”. Department of Water, WA, Perth, August.
- Facility for Advanced Water Biofiltration (2009) “Guidelines for Filter Media in Biofiltration Systems”

Design Considerations

Basin Layout and Sizing

The core elements of a bioretention facility including a basin with filter media, an inlet structure and an outlet structure.

In practice a typical basin filter area requirement is between 1% and 2% of the catchment it serves. However the overall size of the basin will vary depending on its catchment and the treatment performance sought.

The shape of the basin is flexible but needs to facilitate even distribution of inflows across the filter media’s surface. The shape factor should therefore ideally approach a length to width ratio of 1 (i.e. square), though rectangular layouts are acceptable and common.

An indicative maximum catchment area constraint of about 10 hectares applies since areas greater than this normally produce trickle flows which can compromise the performance of the vegetation and filter media. It also becomes more difficult to evenly distribute inflows across a large filter area and manage scour velocities. This catchment area constraint will vary depending on local climate and soils.

Designs can be scaled down to lot scale and street scale sub-catchments. These facilities are sometimes referred to as bio-pods, rain gardens and tree pits.

The basin is designed to be frequently inundated for a short period of time, however the volume temporarily stored and the release rate are not normally effective at controlling peak discharge in large floods. Hybrid design opportunities exist where the bioretention basin is nested inside a larger detention basin facility to target peak flood discharge as well as water quality.

Filter Media and Layers

The floor of the basin comprises of a carefully blended soil filter media, minimum 400 mm depth, with a prescribed hydraulic conductivity of between 100 mm and 300 mm/hr. Over time the conductivity changes as the media settles and plants establish. The plant root zone enhances the water quality treatment performance of the filter and also helps to maintain an equilibrium level of hydraulic conductivity in the media.

Beneath the filter media are a sand transition layer and then a gravel drainage layer. The sand transition layer limits progressive migration of the filter media into the drainage layer. The drainage layer includes a network of slotted pipes that collect treated stormwater for discharge. This drainage layer can be designed as a saturated sump to sustain plant growth during extended dry seasons.

Bioretention basins are normally lined with low permeability clay or a plastic membrane. It is possible to design the system without a liner to encourage infiltration into the local groundwater table, however success with this approach will heavily depend on plant choice and climate.

Inlet Structures

The inlet structure receives flow from the upstream conveyance network. Typically the inlet comprises a small headwall pipe outlet, roadside kerb and gutter or an open channel swale. For large catchments a high-flow bypass is required to limit velocities within the basin and avoid scour of plants and filter media. For large catchments a coarse sediment capture zone (sometimes referred to as a 'sediment forebay') is also required to capture sediment and prevent smothering of the filter media. Regular clean-out of the coarse sediment capture zone is required. Maintenance access is therefore important.

Outlet Structures

The primary outlet is the filter media underdrainage system described previously. This is collected into an outlet pit before discharge into the downstream conveyance system. The secondary outlet normally comprises of an overflow pit or weir that is engaged once the hydraulic conductivity of the filter media is exceeded. The level of the weir is normally between 0.1 m and 0.3 m above the filter surface level. For larger systems a small armoured spillway or weir may also be provided to augment outlet capacity during a large storm.

The outlet discharge level should be sufficiently elevated above local backwater and tide levels to ensure the overall facility is free-draining. Emptying time for these systems can be critical and should be checked.

Vegetation and Landscape Integration

Bioretention basins should be thickly vegetated to encourage water treatment, enhance the long-term performance of the filter media and suppress weed growth. A wide range of plant species may be suitable, but those that tolerate dry conditions, can be periodically inundated and have fibrous root systems are preferred. Native sedges, rushes, grasses, tea tree, paper bark and swamp oak have all been found to perform well. The planting scheme that is chosen should blend with the surrounding landscape and habitat.

4.5.5. Constructed Wetlands

A constructed wetland is a system of water bodies that store water and sustain a range of aquatic macrophytes and semi-aquatic plants (Figure 9.4.11 and Figure 9.4.12). Stormwater is directed into the wetland and detained for a period of approximately 48 hours. During this time, physical, chemical and biological processes result in removal of water-borne pollutants.

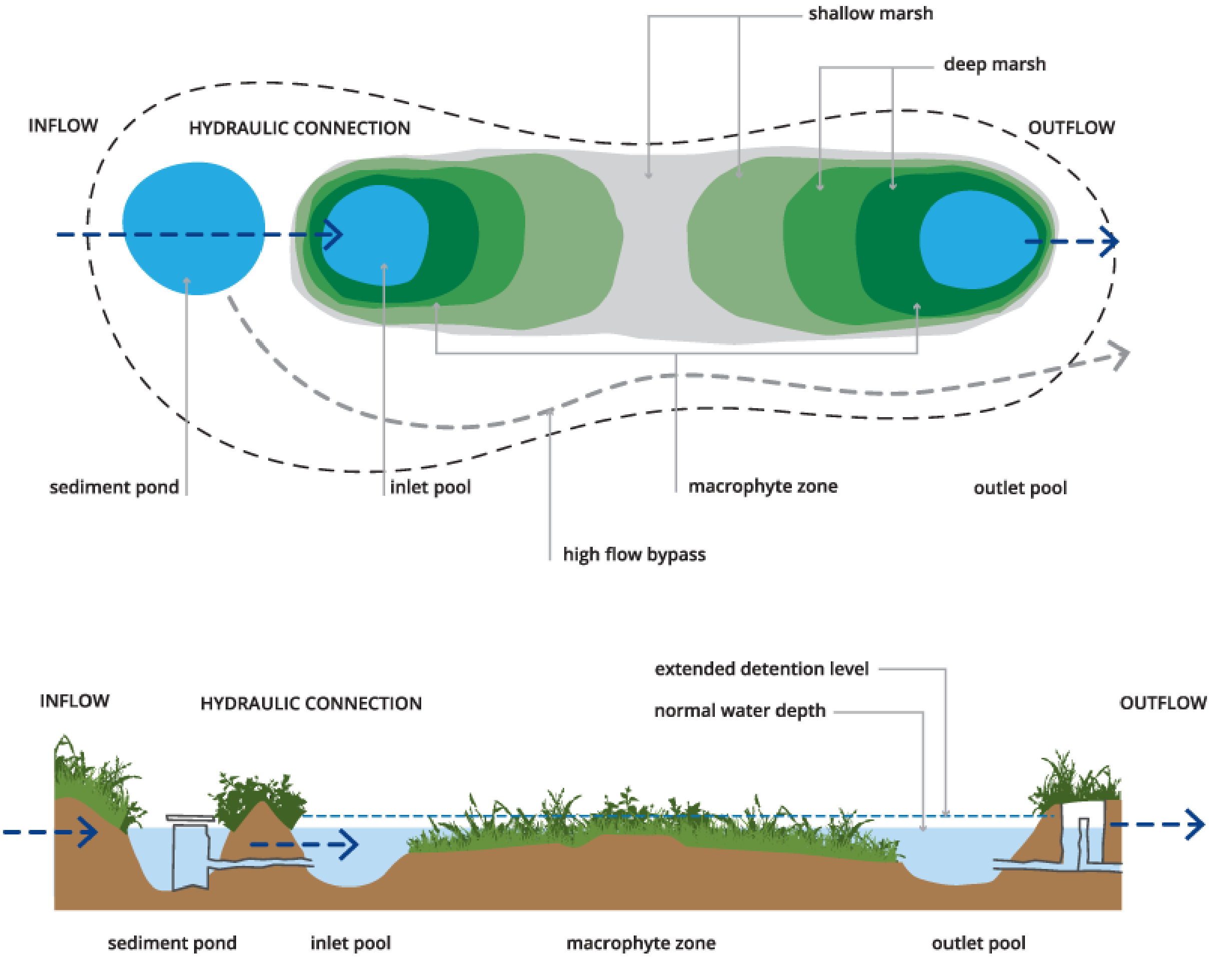


Figure 9.4.11. Schematic Layout of a Typical Constructed Wetland

Figure 9.4.12. Photo of a Typical Constructed Wetland (Source: Steve Roso)

A constructed wetland is most suitable for water quality improvement on catchments larger than approximately 10 hectares (indicative only and subject to local climate and design features). Subject to design and location it may also provide some peak discharge control. It is not directly suitable for harvesting or infiltration of stormwater as this can compromise the sustainability of vegetation. If this objective is sought a separate downstream pond facility should be provided.

Available Guidelines

- Water by Design (2017) “Wetland Technical Design Guidelines”, Brisbane Queensland
- Melbourne Water (2016) Design, construction and establishment of constructed wetlands: design manual (Final Draft), Melbourne, Victoria
- Laurenson and Kuczera (1998) “The Constructed wetlands manual” Sydney, New South Wales

Design Considerations

Inlet Pond and High Flow Bypass

The inlet pond receives stormwater inflow from the upstream conveyance system. The depth and size of the pond should be sufficient to lower flow velocities and promote settling of course sediment particles. Regular clean-out of sediment from this area is required. Reliable maintenance access for machinery should therefore be considered in the design.

The inlet zone contains drainage structures that direct low flows out of the inlet pond and into a downstream wetland area.

During a storm the wetland area fills to a depth of about 0.5 m above the normal operating level. Once this threshold is reached, high flows are directed around the wetland area via a high flow bypass. This flow split is necessary to avoid re-suspension of sediment and plant damage in the wetland area.

Wetland Area

The wetland area is designed with a range of different ponding depths up to 1.5 m, perpendicular to the flowpath. These different depth zones promote a diversity of macrophytes and semi-aquatic plants and enhance the wetlands treatment capacity. The majority of the wetland area should comprise emergent macrophytes however deeper zones are important for diversity and to sustain the ecosystem during drier periods. The overall shape of the wetland should rest within a length to width ratio of between 3 and 10. Typically the total wetland area represents about 5% of the catchment area treated however this varies depending on the climate and treatment performance that is sought.

Outlet Structure

The stormwater that is temporarily held in the wetland after rain is progressively released via a restricted outlet. A typical residence time of 48 hours is sought, however this can vary depending on the site constraints and plant selection.

A secondary outlet is also required to limit the depth of submergence over the wetland.

Vegetation and Landscape Integration

Plant selection requires specialist input to design a planting scheme suited to the hydrologic regime and climate and therefore likely to establish and maintain a thick vegetation cover. The majority of the wetland footprint should be designed to support emergent macrophytes, however deeper zones are important for diversity and to sustain the ecosystem during drier periods. Regional biodiversity guidelines should be consulted for selection of appropriate plant species.

Opportunities should also be sought to integrate the wetland into passive open space recreation and/or local natural habitat.

The wetland should be well sealed with low permeability material to ensure water retention during dry periods. The bed of the wetland should also be lined with topsoil as a growth medium for the selected plants.

The initial establishment period is critical, careful maintenance is required including weeding and replacement of losses. Progressive flooding of the wetland is also needed to avoid drowning of small plants. Predation by birds is also sometimes a challenge that needs to be managed, particularly during the establishment phase.

4.5.6. Managed Aquifer Recharge

Managed Aquifer Recharge (MAR), also known as artificial recharge, is the infiltration or injection of water into an aquifer (Environmental Protection Authority, 2005) (refer to Figure 9.4.13). The water can be withdrawn at a later date, left in the aquifer for environmental benefits, such as maintaining water levels in wetlands, or used as a barrier to prevent saltwater or other contaminants from entering the aquifer (Department of Water, Western Australia, 2007).

MAR may be used as a means of managing water from a number of sources including stormwater. The MAR schemes can range in complexity and scale from the precinct scale, through local authority infiltration systems for road runoff and public open space irrigation bores, through to the regional scale, which involves infiltration or well injection of stormwater and provision of third pipe non-potable water supply for domestic use.

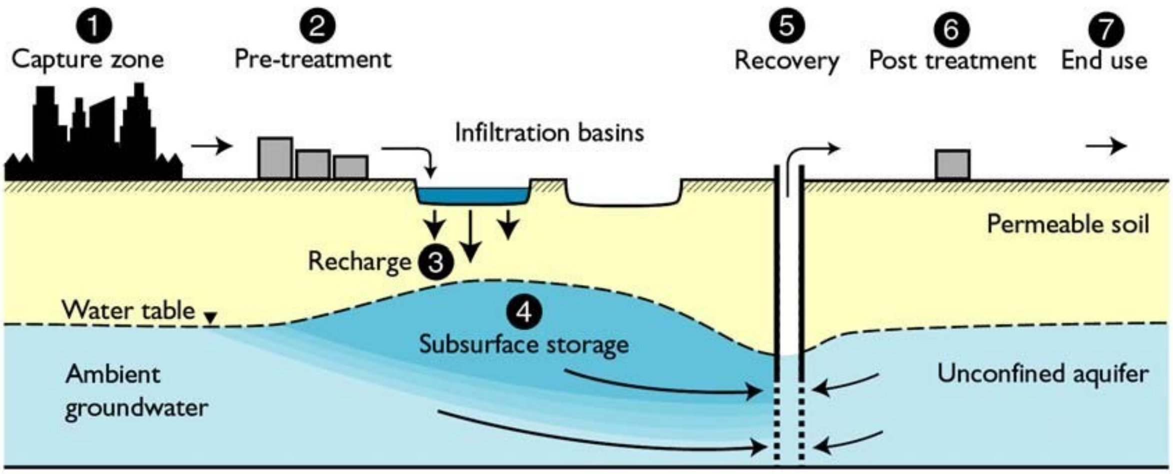


Figure 9.4.13. Example of a Managed Aquifer Recharge Scheme in an Unconfined Aquifer (Adapted from (Natural Resource Management Ministerial Council et al., 2009))

Available Guidelines

Further guidance on MAR can be found in the following documents:

- Melbourne Water (2005): WSUD: Engineering Procedures – Stormwater. Victorian Stormwater Committee, published CSIRO, Melbourne.
- Department of Water, Western Australia (2007) “Stormwater management manual for Western Australia”. Department of Water, WA, Perth, August.
- Natural Resource Management Ministerial Council et al. (2009) “Australian Guidelines for Water Recycling - Managed Aquifer Recharge - National Water Quality Management Strategy - Document No 24, Canberra.

Design Considerations

System Components

As an example, a MAR scheme for infiltration of treated stormwater into a shallow aquifer could contain the following structural elements (Melbourne Water, 2005; Department of Water, Western Australia, 2007):

- soakwells, swales or infiltration basins used to detain runoff and preferentially recharge the superficial aquifer with harvested stormwater;
- an abstraction bore to recover water from the superficial aquifer for reuse;

- a reticulation system (in the case of irrigation reuse) (will require physical separation from potable water supply);
- a water quality treatment system for recovered water depending on its intended use (e.g. removal of iron staining minerals);
- systems to monitor groundwater levels and abstraction volumes; and
- systems to monitor the quality of groundwater and recovered water.

An MAR system may also incorporate the following additional elements (Melbourne Water, 2005; Department of Water, Western Australia, 2007):

- a diversion structure from a drain;
- a control unit to stop diversions when flows are outside an acceptable range of flows or quality;
- some form of treatment for stormwater prior to injection;
- a constructed wetland, detention pond, dam or tank, part or all of which acts as a temporary storage measure (and which may also be used as a buffer storage during recovery and reuse);
- a spill or overflow structure incorporated in the constructed wetland or detention storage;
- well(s) into which the water is injected (may require extraction equipment for periodic purging);
- an equipped well to recover water from the aquifer (injection and recovery may occur in the same well);
- a treatment system for recovered water (depending on its intended use);
- sampling ports on injection and recovery lines; and
- a control system to shut down recharge in the event of unfavourable conditions.

Site Suitability

Factors to consider in evaluating the suitability of an aquifer for a MAR scheme include (Melbourne Water, 2005; Department of Water, Western Australia, 2007):

- environmental values of the aquifer including ecosystem maintenance of caves, wetlands, phyreatophytic vegetation, surface water systems and human uses (irrigation, drinking water supply);
- adverse impacts on the environment and other aquifer users (e.g. reduced pumping pressure for nearby irrigators);
- an existing and/or future drinking water source area;
- sufficient permeability and storage within the receiving aquifer;
- depth of abstraction from the aquifer;

- existing allocation of the aquifer and groundwater resource;
- existing ambient groundwater quality and contaminant concentrations;
- loss of aquifer permeability and/or infiltration due to precipitation of minerals or clogging;
- possible damage to confining layers due to pressure increases;
- higher recovery efficiencies of porous media aquifers;
- aquifer mineral dissolution, if any; and
- potential for local aquitard collapse or distortion.

4.5.7. Infiltration Systems

Infiltration systems can come in a number of different forms, each having different size and geometry but all with a common purpose to promote infiltration of stormwater. They comprise of two main components; a storage basin and an infiltration zone. They are best suited to locations where natural soils have high permeability.

These facilities assist to manage stormwater volume through infiltration of stormwater that enters the groundwater system. They may also contribute to peak discharge control where rainfall intensities are low relative to the permeability of the infiltration zone. They are not intended to provide standalone water quality treatment and should ideally be accompanied by a treatment facility to prevent groundwater contamination.

Available Guidelines

Further guidance on infiltration systems can be found in the following documents:

- Healthy Waterways by Design (2006) “Water Sensitive Urban Design Technical Design Guidelines for South-east Queensland”
- Department of Water, Western Australia (2007) “Stormwater management manual for Western Australia”. Department of Water, WA, Perth, August.
- Argue and Pezzaniti (2012) “WSUD: basic procedures for ‘source control of stormwater – a Handbook for Australian practice”

Design Considerations

System Types

There are several different types of infiltration systems that are available to the designer, each of which suit different sites and applications. These are:

- Infiltration Trenches;
- Infiltration Basins;
- Soakage Well;
- Permeable Pavement; and
- Infiltration Swales.

Each of these is described further below.

Infiltration Trenches

An infiltration trench is a trench filled with gravel or other aggregate (e.g. blue metal), lined with geotextile and covered with topsoil. Often a perforated pipe runs across the media to ensure effective distribution of the stormwater along the system. Recharge storages can also be formed using modular plastic open crates or cells which can be laid in a trench or in rectangular formation. Such systems are typically 0.5 m to 1.5 m deep, surrounded by geotextile and covered with topsoil. Stormwater discharged into these systems is often pre-treated to reduce ongoing maintenance of such systems. Systems usually have an overflow pipe for larger storm events. There are a range of products which have various weight-bearing capacities so that the surface of the system can be used for parkland or vehicle parking areas. These systems can be combined to treat a large area (Department of Water, Western Australia, 2007).

Infiltration Basins (also Known as Retention Basins)

Community and regional infiltration basins are typically installed within public open space parklands. They can consist of a natural or constructed depression designed to capture and store the stormwater runoff on the surface prior to infiltrating into the soils. Basins are best suited to sandy soils and can be planted out with a range of vegetation to blend into the local landscape. The vegetation provides some water quality treatment and the root network assists in preventing the basin floor from clogging. Pre-treatment of inflows may be required in catchments with high sediment flows (Department of Water, Western Australia, 2007).

Soakage Wells

One method for infiltration of urban runoff into suitable soils is using soakage wells (for soils with hydraulic conductivity values > 1×10^{-6} m/s). These systems are used widely in Western Australia as an at-source stormwater management control, typically in small scale residential and commercial applications, or as road side entry pits at the beginning of a stormwater system. Soakage wells can be applied in retrofitting scenarios and existing road side entry pits/gullies can be retrofitted to perform an infiltration function (Department of Water, Western Australia, 2007).

Soakage wells consist of a vertical perforated liner with stormwater entering the system via an inlet pipe at the top of the device (refer Figure 9.4.14). The base of the soakwell is open or perforated and usually covered with a geotextile. Alternatively, pervious material, such as gravel or porous pavement, can be used to form the base of the soakwell. Where source water may have a high sediment load, there should be pre-treatment, such as filtering, as soakage wells are susceptible to clogging.

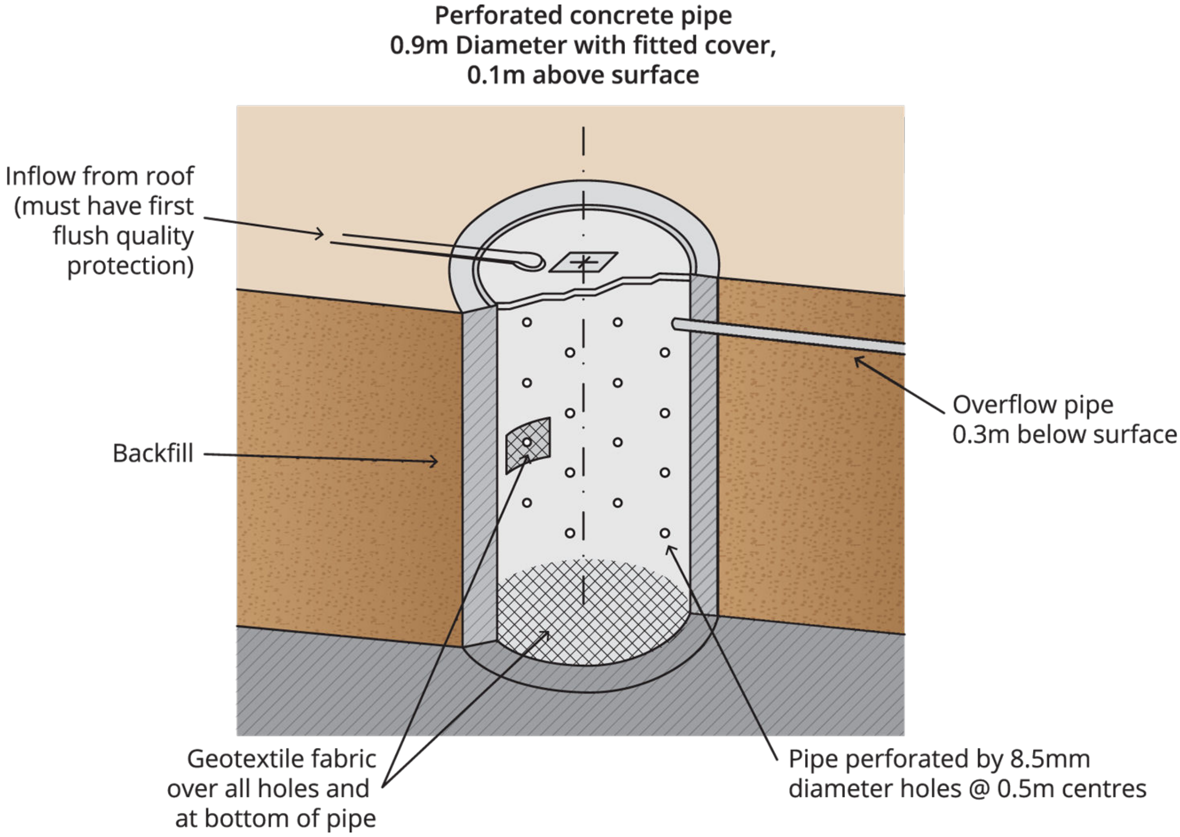


Figure 9.4.14. Leaky Well Infiltration System (adapted from Argue (2017))

Permeable Pavement

There are two types of pervious pavements that are effective in intercepting and diverting surface runoff into the host soil body:

- Permeable paving: concrete blocks incorporating slots or gravel-filled tubes providing (vertical) paths for surface flow to access gravel-filled ("leaky") storages; and
- Porous paving: grassed surface integrated with a sandy-loam and plastic ring-matrix layer laid above a substructure of sand/gravel mix placed under optimum moisture content conditions.

The abstraction capabilities of permeable paving system slots and gravel-filled tubes can be as high as 4,000 mm/h when new – a performance which can show little deterioration over time where surface sediment loads are "light" or where the supply is pre-treated. Pre-treatment in a typical urban street context would require the insertion of a simple sediment trap (2.0 m^2 capacity) immediately upstream of the paving (requiring annual clean-out). The alternative to pre-treatment is regular (five-year intervals) cleaning of the paved surface.

Grassed surface paving shows infiltration capacity of, typically, at least 100 mm/h when new and, like permeable paving, shows little deterioration over time where supply sediment loads are relatively "light". Porous paving is unsuited to the urban street context where permeable paving is used but can be relied upon for many decades of low maintenance service receiving runoff from, for example, a (conventional) paved carpark surface. "Low maintenance" in this context involves little more than regular mowing. The continued impressive performance of a porous paved surface is accounted for by the dynamic nature of the interaction – maintaining infiltration capacity - which takes place between the grass roots and the host soil.

Infiltration Swales

Infiltration swales are shallow grassed channels – typically 0.3 m to 0.5 m (maximum) deep, 5 m to 6 m wide in residential streets – with longitudinal slopes, preferably, less than 3%. They have wide application in stormwater retention systems for three main reasons:

- They can retain runoff through bed infiltration;
- They can be effective in retaining pollutants conveyed in stormwater (Breen et al., 1997; Lee et al., 2008) and
- They can fulfil a role in stormwater harvesting through soil moisture enhancement and, possibly, aquifer recharge and recovery.

The configuration of a typical infiltration swale in relation to a residential street carriageway is shown on Figure 9.4.15. This configuration includes a filter strip between the carriageway and the swale invert to provide pre-treatment and additional infiltration.

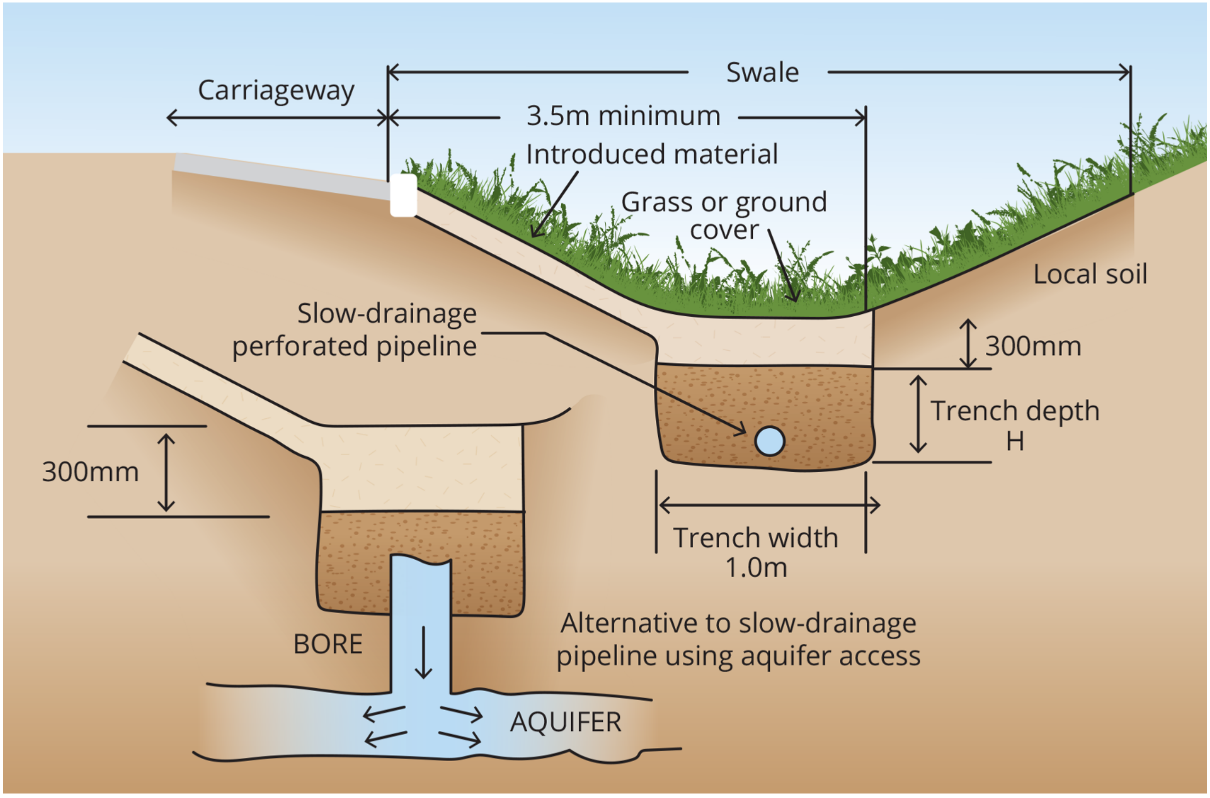


Figure 9.4.15. Main Components of an Infiltration Swale (with Filter Strip) (adapted (Argue, 2017),(Argue, 2013))

Swales only abstract flows up to a limit set by the infiltration capacity of the near-carriageway "filter strip" and channel bed. All exceedances above this capacity pass as open channel flow conveyed downstream within the boundaries of the swale. Another practice is to terminate such a swale in a "dry pond" perhaps in the vicinity of a major road intersection.

The process of abstraction is achieved through infiltration alone or by infiltration combined with sub-structure retention (gravel-filled trench or similar illustrated in Figure 9.4.7) with hydraulic disposal to aquifers (if available) or local waterways (slow-drainage) if necessary.

Site Selection

Due to their flexibility in shape, infiltration systems can be located in a relatively unusable portion of the site. However, design will need to consider clearance distances from adjacent building footings or boundaries to protect against cracking of walls and footings.

Identification of suitable sites for infiltration systems should also avoid steep terrain and areas of shallow soils overlying largely impervious rock (non-sedimentary rock and some sedimentary rock such as shale). An understanding of the seasonal and inter-annual variation of the groundwater table is also an essential element in the design of infiltration systems.

Soils

Soil types, surface geological conditions and groundwater levels determine the suitability of infiltration systems. Infiltration techniques can be implemented in a range of soil types, and are typically used in soils ranging from sands to clayey sands. While well-compacted sands are suitable these measures should not be installed in loose aeolian wind-blown sands.

Soils with lower hydraulic conductivities do not necessarily preclude the use of infiltration systems, but the size of the required system may typically become prohibitively large, or a more complex design approach may be required, such as including a slow drainage outlet system. Care should also be taken at sites with shallow soil overlying impervious bedrock, as the water stored on the bedrock will provide a stream of flow along the soil/rock interface (Department of Water, Western Australia, 2007).

Groundwater

The presence of a high groundwater table limits the potential use of infiltration systems in some areas, but does not preclude them. There are many instances of the successful application of infiltration basins on the Swan Coastal Plain where the basin base is located within 0.5 m of the average annual maximum groundwater level. The seasonal nature of local rainfall and variability in groundwater level should also be considered. Infiltration in areas with rising groundwater tables should be avoided where infiltration may accelerate the development of problems such as waterlogging and rising salinity (Department of Water, Western Australia, 2007).

Pre-treatment

In general, stormwater runoff should not be conveyed directly into an infiltration system, but the requirement for pre-treatment will depend on the catchment e.g. residential or industrial. Pre-treatment measures include the provision of leaf and roof litter guards along roof gutters, vegetated strips or swales, litter and sediment traps, sand filters and bioretention systems. To prevent infiltration systems from being clogged with sediment/litter during road and housing/building construction, temporary bunding or sediment controls need to be installed. It may also be necessary to achieve a prescribed water quality standard before stormwater can be discharged into groundwater (Department of Water, Western Australia, 2007).

Emptying Time

Emptying time is defined as the time taken to completely empty a storage associated with an infiltration system following the cessation of rainfall. This is an important design consideration as the computation procedures typically assume that the storage is empty prior to the commencement of the design storm event.

Ideally emptying time criteria should be ascertained by undertaking ‘continuous simulation’ modelling of a catchment (Argue, 2017) and should be conducted in accordance with Book 7 and combined with partial series analysis to determine the volume, frequency and rate of discharge from the site. In the absence of such assessments the emptying times for infiltration systems given in Table 9.4.8 are recommended in the interim.

Table 9.4.8. Interim Relationship between Annual Exceedance Proability and ‘Emptying Time (Argue, 2017)

	EY			AEP (%)			
	1	0.5	0.2	10	5	2	1
Emptying Time (days)	0.5	1.0	1.5	2.0	2.5	3.0	3.5

4.5.8. Stormwater Harvest Ponds

A stormwater harvest pond comprises of a storage area to collect surface runoff for later extraction and use, often for irrigation. Ancillary infrastructure is also required for the pre and post treatment and distribution of this water.

A stormwater harvest pond is best suited to applications that target the harvesting and re-use of larger quantities of stormwater for non-potable use. Below a certain size threshold a pond may not be an economic way of storing water, in which case an alternative may be an underground tank.

A stormwater harvest pond does not directly target the improvement of water quality, however it can provide a minor contribution to this outcome in some circumstances where there is suitable irrigation demand. Similarly a stormwater harvest pond does not directly target the control of peak discharge but it may contribute to minor reductions in peak flow downstream for smaller storms.

Available Guidelines

- Healthy Waterways by Design (2009) “Stormwater Harvesting Guidelines (Draft)”, Brisbane, Queensland

Design Considerations

Embankment Design

The guidance on embankment design given in Book 9, Chapter 4, Section 5 is also applicable to any dry ponds which are formed by embankments.

Configuration and Sizing

The configuration of a stormwater harvest pond is mostly influenced by physical site constraints and geotechnical limitations on batter slope. The shape of the pond does not directly affect its performance as a storage, but may affect the cost of civil construction. The most efficient shape in this regard approaches a circle or square.

The size of the pond relative to the estimated catchment yield is a balance between capital cost of construction and the reliability of supply. This sizing must be undertaken using a water balance of the site with realistic estimates of rainfall, runoff and demand.

Liners

A stormwater harvest pond requires a low permeability liner with freeboard above the normal maximum operating level. This can comprise of a non-dispersive compacted clay liner, or a synthetic membrane.

A well utilised stormwater harvest pond is not normally full and will experience significant fluctuations in water level. The liner may therefore need underdrainage to prevent excessive groundwater pressures developing on the outer wall of the membrane.

Treatment

Depending on the anticipated end-use of the stored water, the water extracted from the facility may require treatment to improve water quality to the required standard. This may involve filtration using a graded sand filter or similar.

Drainage Structures

A stormwater harvest pond requires a suitable inlet structure armoured against erosion and designed to accommodate potential inflows when the pond is fully drawn down.

The outlet structure typically comprises of an enclosed conduit or spillway, with invert set at the maximum operating level. The capacity of this spillway should be designed with the same level of consideration given to a detention basin spillway, with a capacity and freeboard matched to the level of accepted risk.

4.6. References

ACT Department of Urban Services (1998), Design standards for urban infrastructure, section 1 : stormwater, 1.9 Retarding Basins, p: 4.

ANCOLD (Australian National Committee on Large Dams) (2000a), Guidelines on assessment of the consequences of dam failure, May, p: 15.

ANCOLD (Australian National Committee on Large Dams) (2000b), Guidelines on selection of acceptable flood capacity of dams, March, p: 36.

Australian and New Zealand Environment and Conservation Council (ANZECC) (2000), Australian guidelines for urban stormwater management, National Water Quality Management Strategy Report No.10, Canberra, p: 72.

American Society of Civil Engineers (1985), Stormwater detention outlet control structures, New York: ASCE.

Andoh, R.Y.G. and Declerck, C. (1999), Source control and distributed storage - a cost effective approach to urban drainage for the new millennium?, Proceedings of the 8th International Conference on Urban Storm Drainage, Engineers Australia, 30 August - 3 Septemeber, Sydney.

Argue, J.R. (2017), Water Sensitive Urban Design: basic procedures for 'source control' of storm water - a handbook for Australian practice. Urban Water Resources Centre, University of South Australia, Adelaide, South Australia, in collaboration with Stormwater Industry Association and Australian Water Association. Adelaide.

Argue, J.R. (2013), WSUD and green infrastructure: a cost-effective and sustainable strategy for urban re-development, Proceedings of the 8th International WSUD Conference, 25-29 November 2013, Gold Coast.

Argue, J.R. and Pezzaniti, D. (2007), WSUD (Stormwater) Practices - Learning from nature: A discussion, Proceedings of the Rainwater and Urban Design Conference, IEAust. Sydney.

Argue, J.R. and Pezzaniti, D. (2009), The need for quantity-related criteria in regional WSUD guidelines: A demonstration, Proceedings of the 2009 SIA NSW and VIC Annual Conference, SIA NSW & SIA VIC, 7-10 July, Albury.

Argue, J.R. and Pezzaniti, D. (2010), The quantity domain of WSUD: Flooding and environmental flows, Proceedings of Stormwater 2010: SIA National Conference, 9-12 November, Sydney.

Argue, J.R. and Pezzaniti, D. (2012), Use of WSUD 'source control' practices to manage floodwaters in urbanising landscapes: Developed and ultra-developed catchments, Proceedings of Stormwater 2012, National Conference of the Stormwater Industry Association, 15-19 October, Melbourne.

Argue, J.R. and Scott, P. (2000), On-site Stormwater Retention (OSR) in residential xatchments: A better option?, Proceedings of the 40th Floodplain Management Conference, NSW Floodplain Management Authorities, 9-12 May, Parramatta.

Brater, E.F., King, H.W. and Lindell, J.E. (1996), Handbook of hydraulics for the solution of hydraulic engineering problems, 7th ed. New York: Mc-Graw-Hill.

Breen, L., Dyer, F., Hairsine, P., Riddiford, J., Sirriwardhena, V. and Zierholz, C. (1997), Controlling sediment and nutrient movement within catchments, Industry report 97/9. Melbourne: Cooperative Research Centre for Catchment Hydrology.

Bryant, D.B., Wilden, K.A. and Socolofksy, S.A. (2008), Laboratory measurements of tidal jet vortex through inlet with developed boundary layer, Proceedings of the 2nd International Synopsium on Shallow Flows, p: 6.

Burns, M.J., Fletcher, T.D., Walsh, C.J., Ladson, A.R. and Hatt, B. (2013), Setting objectives for hydrologic restoration: from site-scale to catchment-scale, Proceedings of the Novatech 8th International Conference, Lyon, France.

Coombes, P.J. (2005), Integrated Water Cycle Management: Analysis of Resource Security. Water, 32: 21-26.

Coombes (2015), Submission to the Senate Standing Committees on Environment and Communications inquiry into the quantum of stormwater resource in Australia and impact and potential of optimal management practices in areas of flooding, environmental impacts, waterway management and water resource planning. The Senate. Environment and Communications Reference Committee. Australian Government, Canberra.

Coombes, P.J. (2009), The use of rainwater tanks as a supplement or replacement for onsite stormwater detention (OSD) in the Knox Area of Victoria, Proceedings of the 32nd Hydrology and Water Resources Symposium, 30 November to 3 December, pp: 616-627.

Coombes, P.J. and Barry, M.E. (2015), A systems framework of big data for analysis of policy and strategy, Proceedings of the 2015 WSUD and IECA Conference, Sydney.

Coombes, P.J. and Barry, M.E. (2009), The spatial variation of climate, household water use and the performance of rainwater tanks across Greater Melbourne, WSUD09 Conference, Engineers Australia, Perth

Coombes, P.J. and Barry, M.E. (2007), Optimisation of mains trickle top-up volumes and rates supplying rainwater tanks in the Australian urban setting, Proceedings of the 13th International Conference on Rain Water Catcment Systems 2007, 21-23 August, Sydney, Australia.

Coombes, P.J., Kuczera, G and Kalma, J.D. (2000), Economic benefits arising from use of Water Sensitive Urban Development source control measures, Proceedings of the 3rd International Hydrology and Water Resources Symposium, IEAust. 20-23 November, Perth.

Coombes, P.J., Frost, A. and Kuczera, G. (2001), Impact of rainwater tank and on site detention options on stormwater management in the Upper Parramatta River Catchment. Newcastle: Department of Civil, Surveying and Environmental Engineering, University of Newcastle.

Coombes, P.J., Babister, M. and McAlister, T. (2015), Is the science and data underpinning the rational method robust for use in evolving urban catchments, Proceedings of the 36th Hydrology and Water Resources Symposium, Engineers Australia, Hobart.

Coombes P.J., Boubli, D. and Argue, J.R. (2003), Integrated water cycle management at the Heritage Mews development in Western Sydney, Proceedings of the 28th International Hydrology and Water Resources Symposium, 10-13 November, Wollongong.

Coombes, P.J., Kuczera, G. Kalma, J.D. and Argue, J.R. (2002a), An evaluation of benefits of source control measures at the regional scale, Urban Water, 4(4), 307-320.

Coombes, P.J.,Frost, A., Kuczera, G., O'Loughlin, G. and Lees, S. (2002b), Rainwater tank options for stormwater management in the Upper Parramatta River Catchment, Proceedings of the 27th Hydrology and Water Resources Symposium, 20-23 May, Melbourne, pp: 474-482.

Department of Irrigation and Drainage (2000), Urban stormwater management manual for Malaysia (Manual Saliran Mesra Alam Malaysia), volume 7, Detention (Chapter 18, 19, 20), volume 8 Retention (Chapters 21, 22).

Department of Irrigation and Drainage (2012), Urban stormwater management manual for Malaysia (Maunual Saliran Mesra Alam Malaysia), ed. 2, Chapter 6: Rainwater Harvesting, p.18.

Department of Water, Western Australia (2007), Stormwater management manual for Western Australia, Perth: Department of Water, W.A.

Derwent Estuary Program (2012), Water sensitive urban design: Engineering procedures for stormwater management in Tasmania, Hobart.

Environmental Protection Authority (2005), Strategic advice on managed aquifer recharge using treated wastewater on the Swan coastal plain, Bulletin Report No. 1199, EPA, Perth, Western Australia, Available at: <http://www.epa.gov.wa.au>.

Facility for advancing water biofiltration (2009), Guidelines for filter media in biofiltration systems, Melbourne: FAWB.

Hardy, M.J., Coombes, P.J. and Kuczera, G.A. (2004), An investigation of estate level impacts of spatially distributed rainwater tanks, 2004 International Conference on Water Sensitive Urban Design, Engineers Australia, Adelaide, Australia.

Healthy Waterways Water By Design (2014), Bioretention technical design guidelines, Brisbane.

Healthy Waterways Water By Design (2006), Water sensitive urban design technical design guidelines for South-east Queensland, Brisbane.

Healthy Waterways Water By Design (2009), Stormwater harvesting guidelines (Draft), Brisbane.

Herrmann, T. and U. Schmida (1999), Rainwater utilisation in Germany: Efficiency, dimensioning, hydraulic and environmental aspects, Urban Water, volume 1(4), pp. 307-316.

Hobart City Council (2006), Water sensitive urban design site development guidelines and practice notes, Hobart: Hobart City Council.

Institution of Engineers, Australia (1985), Guidelines for the design of detention basins & grassed waterways for urban drainage systems, April , pp: 11.

Joliffe, I.B. (1997), Roof tanks - A role in stormwater management, Proceedings of Future Directions for Australian Soil and Water Management, 9-12 September, Brisbane.

Laurenson, E.M. and Kuczera, G.A. (1998), Annual exceedance probability of probable maximum precipitation - report on a review and recommendations for practice. Report prepared for the NSW Department of Land and Water Conservation and the Snowy Mountains Hydro-Electric Authority.

Lea, F. C. (1942), Hydraulics, 6th ed. New York: Longmans, Green and Co.

Lee, A., Hewa, G., Pezzaniti, D. and Argue, J.R. (2008), Improving stream low flow regimes in urbanised catchments using water sensitive urban design techniques, Australian Journal of Water Resources, 12(2), 121-132.

Medaugh, F.W. and Johnson, G.D. (1940), Investigation of the discharge coefficients of small circular orifices, Civil Engineer, 7(7), 422-424.

Melbourne Water (2010), Guidelines for the design and assessment of flood retarding basins, p: 9.

Melbourne Water (2016), Design, construction and establishment of constructed wetlands: design manual (Final Draft), Melbourne.

Melbourne Water (2005), WSUD: Engineering procedures - Stormwater. Victorian Stormwater Committee, Melbourne: CSIRO.

New South Wales Department of Planning and Environment (2015), Interim rainwater harvesting system guidelines (Building Sustainability Index: BASIX), Sydney: NSW Government.

New South Wales Government (2004), Managing urban stormwater: Soils and construction, 1(4), Sydney: NSW Government.

Natural Resource Management Ministerial Council, Environment Protection and Heritage Council, (2009), Australian guidelines for water recycling - Managed aquifer recharge - National water quality management strategy - Document No.24, Canberra: NRMMC.

O'Loughlin, G., Beecham, S., Lees, S., Rose, L. and Nicholas, D. (1995), On-site stormwater detention systems in Sydney, Water Science and Technology, 32: 169-175.

Phillips, B.C. and Yu, S. (2015), How robust are OSD and OSR Systems?, Proceedings of the 3rd International Erosion Control Conference and 9th International Water Sensitive Urban Design Conference, 20-22 October 2015, Darling Harbour.

Phillips, B.C., Goyen, A.G. and Lees, S.J. (2005), Improving the sustainability of on-site detention in urban catchments, Proceedings of the 10th International Conference on Urban Drainage, IAHR/IWA, 21-26 August, Copenhagen.

Pilgrim, DH (ed) (1987) Australian Rainfall and Runoff - A Guide to Flood Estimation, Institution of Engineers, Australia, Barton, ACT, 1987.

Queensland Department of Energy and Water Supply (2013), Queensland urban drainage manual, Provisional Third Edition, Chapter 5: Detention/retention systems, p: 17.

Steward, C.B. (1908), Investigation of flow through large submerged orifices and tubes, Bulletin, No.216. Madison: University of Wisconsin.

United States Department of Interior: Bureau of Reclamation (1987), Design of small dams, Washington: USBR.

United States Department of the Interior: Bureau of Reclamation (2001), Water measurement manual, revised edition. Denver, U.S.A.: USBR.

Upper Parramatta River Catchment Trust (2005), On-site stormwater detention handbook, 4th edition, Parramatta: UPRCT, p: 212.

Vaes, G. and J. Berlamont (2001), The effect of rainwater storage tanks on design storms. Urban Water, 3(4), 303-307.

Walsh, C.J., Fletcher, T.D. and Burns, M.J. (2012), Urban stormwater runoff: a new class of environmental flow problem. PLOSone DOI: 10.1371/journal.pone.0045814.

Water by Design (2017), Wetland Technical Design Guidelines, Brisbane Queensland.

enHealth. Environmental Health Risk Assessment-Guidelines for Assessing Human Health Risks from Environmental Hazards; Department of Health and Ageing and enHealth Council: Sydney, Australia, 2012.

van der Sterren, M. (2012), Assessment of the impact of rainwater tanks and on-site detention on urban run-off quantity and quality characteristics. PhD Thesis, University of Western Sydney.

van der Sterren, M., Rahman, A. and Dennis, G.R. (2013), Quality and quantity monitoring of five rainwater tanks in Western Sydney, Australia, Journal of Environmental Engineering, 139(3), 332-340.

van der Sterren, M., Rahman, A. and Ryan G. (2014), Modelling of a lot scale rainwater tank system in XP-SWMM: A case study in Western Sydney, Australia, Journal of Environmental Management, 141: 177-189.

Chapter 5. Stormwater Conveyance

Benjamin Kus

With contributions from the Book 9 editors (Peter Coombes and Steve Roso)

Chapter Status	Final
Date last updated	14/5/2019

5.1. Introduction

Stormwater conveyance combines hydrological and hydraulic methods to safely convey stormwater generated by rain falling on urban surfaces to an outlet. Analysis of conveyance infrastructure typically includes the hydrology of sub-catchments that transfer rainfall runoff to inlet structures feeding a network of other conveyance infrastructure including pipes, open channels, roadways and open space.

Conveyance infrastructure is one of the many tools available to the designer for urban stormwater management which is part of the process of managing the water cycle. For example a stormwater management strategy for an urban area will include a wide range of measures to manage stormwater runoff volumes and flow rates, for example on-site detention, bio-retention, rain water, stormwater harvesting and infiltration systems. These may alter the inflows to and the design of the stormwater conveyance network as shown in Figure 9.5.1. These volume management measures (as described in Book 9, Chapter 4) can operate at different scales such as source and neighbourhood controls that alter inputs to conveyance networks and regional controls that mitigate outflows from conveyance networks.

This chapter focuses on the design and analysis of stormwater conveyance networks.

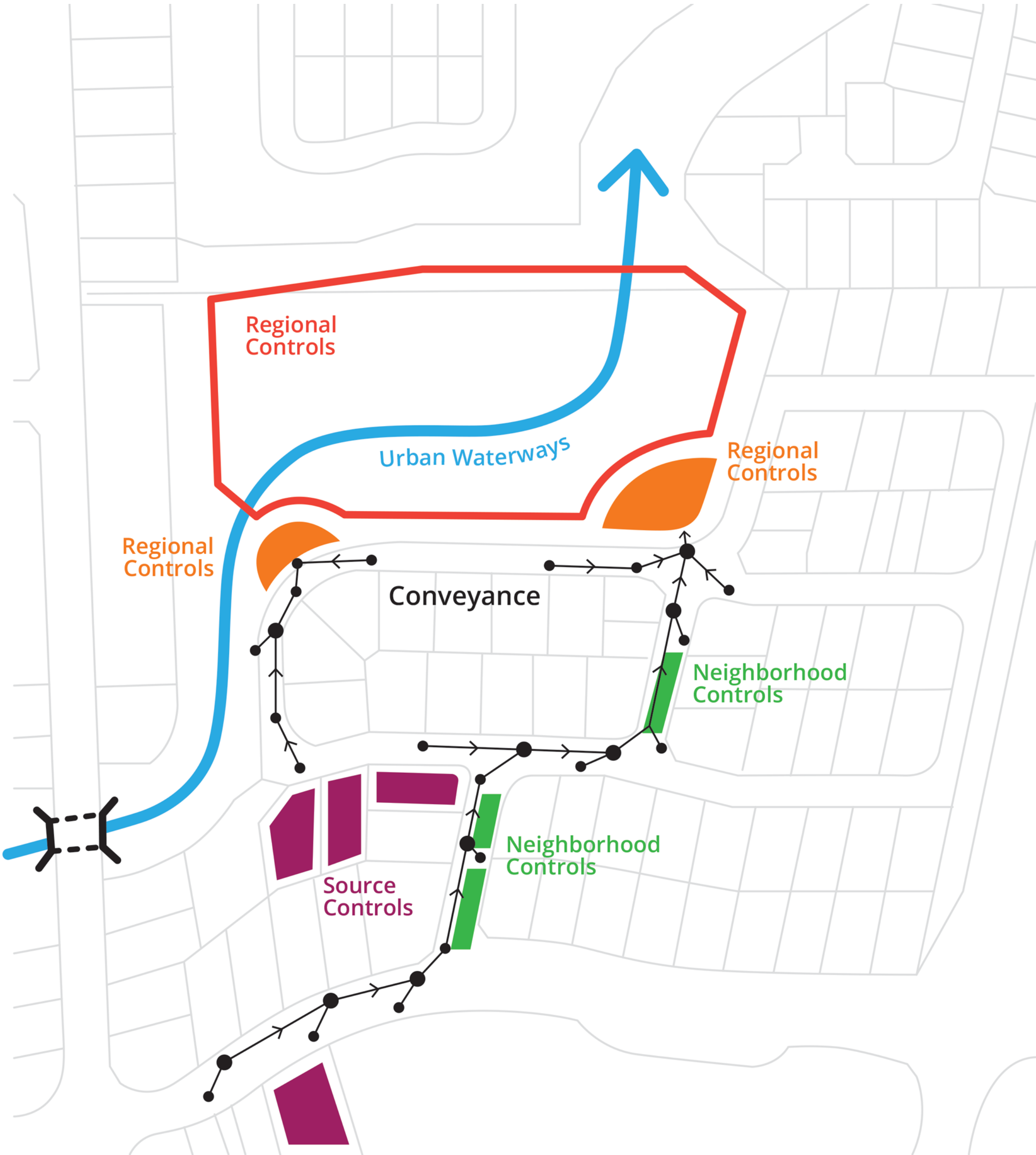


Figure 9.5.1. Stormwater Conveyance and Volume Management Within an Urban Stormwater Network

5.2. Design Philosophy and Objectives

Design of urban stormwater conveyance networks has been comprehensively addressed in guidelines within Australia and internationally. These guidelines include aspects of the design of stormwater infrastructure with different levels of detail that often concentrate on key areas of design focus (such as urban developments or main highways) or on problematic areas of concern that are specific to a locality or past events. It is generally the responsibility of the designer to select an adequate design procedure. However, the objectives and attributes of stormwater conveyance networks are often specified by the local approval authority. These authorities may base their guidance on other design specifications and guidelines such as Aus-Spec, Austroads, or the Victorian Infrastructure Design manual.

This section provides an overview of the philosophy and objectives for design of stormwater conveyance networks. The primary focus of this section is hydraulics and hydrology, and design safety requirements. Nevertheless, there are other important aspects that should be considered during the planning and design of conveyance networks. These include constructability, aesthetics, future maintenance, direct costs, long term economic factors, and the potential liability created by a conveyance network. The design should also account for the practicality of replacing conveyance infrastructure at the end of its design life.

A key hydraulic criterion is to define a conveyance network that restricts surface flows to safe limits. The primary design requirement is that stormwater depths should not be greater than a threshold value above the top of inlet pit or invert of a road gutter. This prevents inlet pits filling to the brim under design conditions which inhibits stormwater flows from entering conveyance networks. The threshold depth is typically set by the relevant approval authority and is in the order of 150 mm. Approval authorities also typically specify maximum velocities of surface flows and minimum velocities of flows in conveyance infrastructure.

In situations where surface flows are conveyed through public places, including footpaths, roads and public places, it is important to ensure that unacceptable hazards to people are not created (refer to Book 6, Chapter 7). Keeping the depth and velocity-depth attributes of surface flow within acceptable limits will minimise these hazards. When the primary purpose of a pathway is for conveyance of stormwater, it will usually be more efficient to convey flows in a dedicated watercourse that can accept higher velocity and depths of flows. These types of flow paths can be designed for dual uses (stormwater conveyance and public access) provided that the design ensures that people cannot be trapped by stormwater flows.

These limits are intended to ensure that stormwater conveyance networks operate at given levels of service without causing flooding of properties, nuisance or hazard to pedestrians and to traffic on streets. An approval authority typically specifies the design AEP of the minor and major storm events required for different land uses. Designs usually involve minor system capacity criteria for design of conveyance infrastructure and major system assumptions to ensure the urban area can safely cope with larger storm events as shown in Figure 9.5.2.

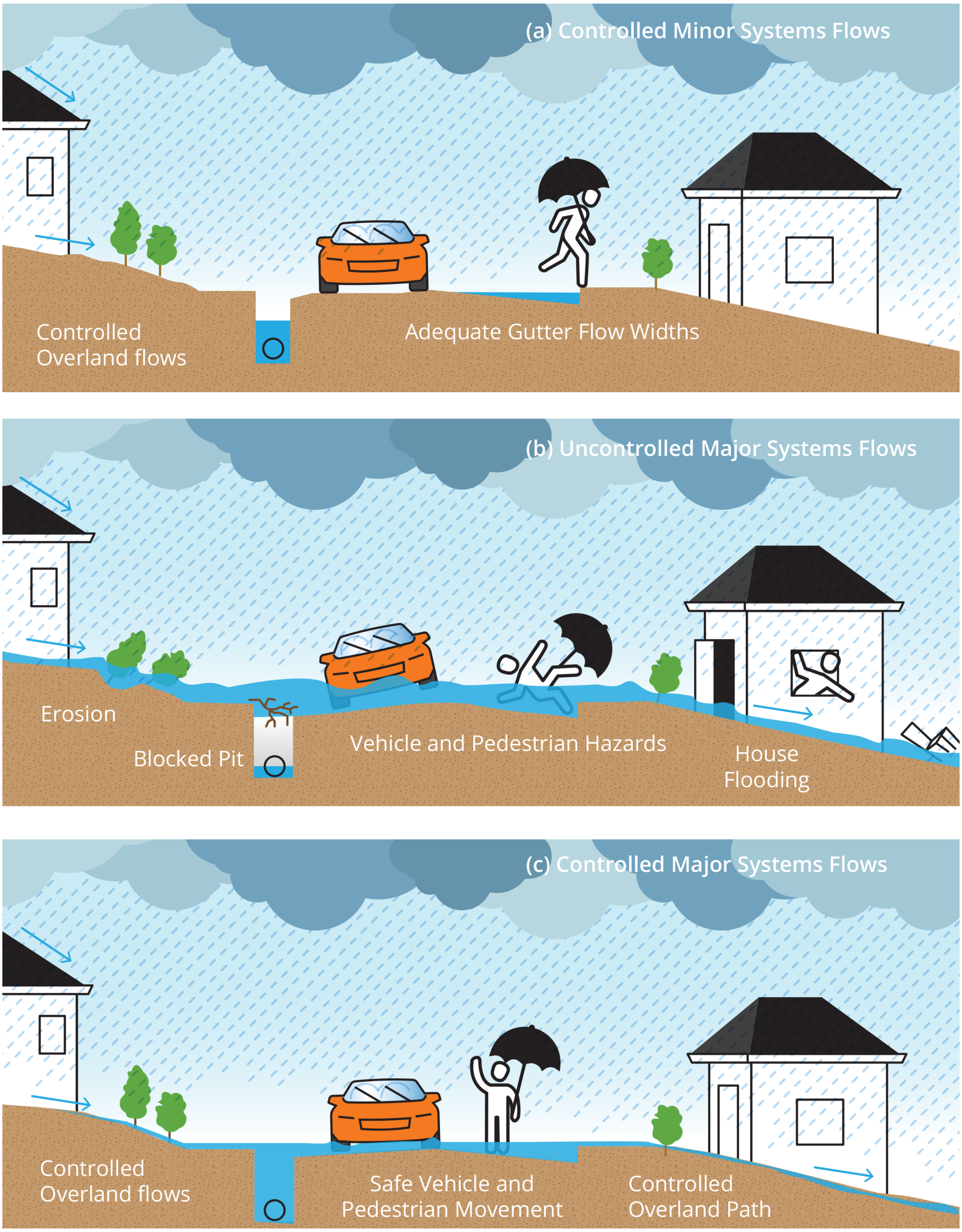


Figure 9.5.2. Minor and Major Concepts for Conveyance Networks

Figure 9.5.2 shows that the minor system is used to define the performance of the conveyance networks which include overland and bypass flows on roads, and performance of conveyance infrastructure (such as pipes and culverts). The major conveyance system includes the road profile and overland flow paths, and aims to ensure the safety of pedestrian and vehicle traffic whilst avoiding property damage and risk to life. In the absence of guidance from a consent authority, the design AEP storm events are selected to reflect the importance of a facility or urban area and the consequences of failure. Some examples are:

- Roof drainage systems: 5% AEP to 1% AEP;
- Conduit drainage systems through lots or sites: 0.5 EY to 1% AEP, depending on consequences of failure;
- Conveyance networks in streets: 0.5 EY to 5% AEP for minor flows, 2% AEP or 1% AEP for major flows (refer to Book 9, Chapter 3) (note that the street profile is part of the major conveyance network);
- Trunk conveyance networks: 1% AEP or higher, with checks on effects created by PMP storm events;
- Stormwater quality and sediment control devices: 4 EY to 1 EY but may address the full spectrum of rainfall frequencies (refer to Book 9, Chapter 3);
- On-site detention (refer to Book 9, Chapter 4): the requirements vary but should aim to improve the performance of stormwater management scheme at a sub-catchment scale; and
- Large detention basins (refer to Book 9, Chapter 4 and Book 9, Chapter 6) that may endanger lives if failure occurs: 1% AEP with checks using probable maximum precipitation (PMP) storm events.

Both design and analysis processes involve modelling the operation of a conveyance network that is subject to critical rare storm events that produce maximum flow rates for the selected AEP events. The selection of critical storm events will involve finding storm durations or particular storm patterns within ensembles or continuous sequences of rainfall that create maximum outputs for a particular location. Typically, the design of a conveyance network is shaped and sized to cater for critical storms for selected AEP events. This approach recognises that:

- It is not practical or economically feasible to design conveyance networks to be free of failure for all events. An attempt to do this would result in very large and expensive conveyance networks that would occupy a considerable land space. This would impact on the optimum provision other infrastructure services, such as water pipes and electricity conduits;
- Failures can occur in response to rare or extreme storm events or other factors such as blockages due to poor maintenance, and exacerbating circumstances such as high tide levels in coastal areas;
- A risk management approach should be adopted that accepts controlled failure;
- Ideally the acceptable level of risk should be set by community values and economic analysis, and;
- The effects of potentially rare failures should be limited by providing a ‘fail safe’ system that does not fail disastrously.

Analysis techniques should include sensitivity checks to ensure that damage and risks to lives due to failures are limited. Some failures of the network and overflows can be expected during major storm events, as shown in Figure 9.5.2, but the network should operate without causing safety hazards or large-scale property damage.

5.3. Conveyance Networks

The design or analysis process for conveyance networks requires observation of the real world situation; definition of the problem and objectives of the design; development of a conceptual model of real world behaviours; calibration using observed data; and predictions or design. A conceptual model of a conveyance network involves hydrological and hydraulic modelling including components such as inlet pits, pipes, open channels, roadways and storages. A general overview of the modelling and design process is presented in Figure 9.5.3.

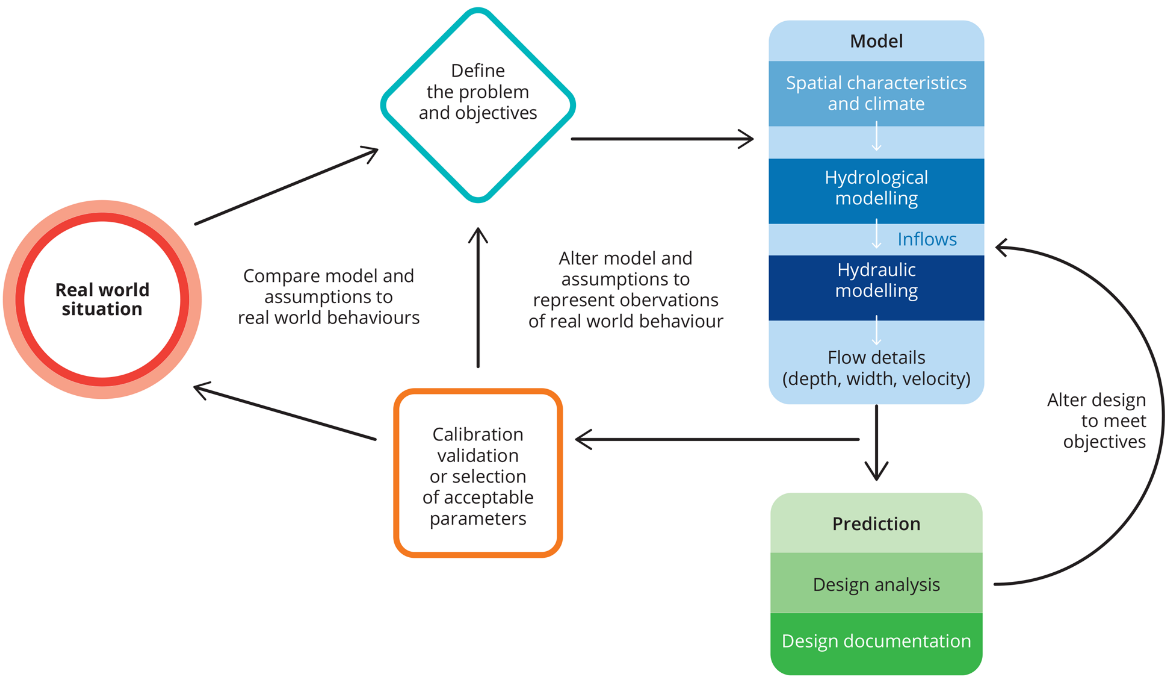


Figure 9.5.3. The Stormwater Design Process

Figure 9.5.3 shows that the stormwater design process includes hydrological modelling of rainfall runoff from urban surfaces to generate inputs to hydraulic modelling of the conveyance network. This process usually incorporates a hydrological model than translates design or real rainfall patterns into design flow rates and volumes of stormwater arriving at inlet structures within a conveyance network. A hydraulic model then converts these inflows into flow characteristics (depths, elevations, widths, velocities, and volumes) throughout the network. The design analysis then determines attributes of the conveyance infrastructure, including pipe diameters and invert levels. The steps in the conveyance design process include:

1. Define the real world situation to be modelled. This will include land use, demographics, topography, urban form, local climate, upstream and downstream conditions, and location within a river basin or waterway catchment.

2. Determine the objectives and design standards that should apply to the drainage network.

3. Locate any available rainfall runoff data that can be used to calibrate models used to design the drainage network or collate the most appropriate parameters for the catchment.

4. Choose the rainfall inputs, hydrological and hydraulic modelling methods for design or analysis:

   a. Rainfall inputs may be design storm temporal patterns of storm bursts or full volume storms, ensembles of peak burst or full storms, or long sequences of real rainfall (refer to Book 2 and ARR Data Hub).

   b. The hydrology and hydraulic models may be hand calculations but will typically be some form of computer model (refer to Book 5 and Book 7).

5. Analyse land uses, road and open space networks, and topography to develop the connectivity stormwater runoff processes throughout the catchment. This includes gathering information such as:

   a. survey and information defining topography;

   b. geotechnical and soil information;

   c. plans of the development or facility to be designed; and

   d. identifying constraints, such as easements and external drainage networks.

6. Define a model network of sub-catchments and drainage infrastructure that is an acceptable approximation of the real system.

7. Using topography, rainfall, land uses, the spatial location of other urban infrastructure and knowledge of the capacity of various drainage inlet structures, define the spacing of nodes in the conveyance network and the routing processes. The routing processes can include gutter flows, overland flows, bypass flows and pipe, culvert or channel flows. The routing processes can include gutter flows, overland flows, bypass flows and pipe or culvert or channel flows.

8. Calibrate or validate the hydrology and hydraulics of the existing catchment to any gauged data or nearby flood frequency information or accepted parameters for the area.

9. Use the model to design the capacity and spacing in inlet structures, and to size the conveyance infrastructure. This design process will be guided by the objectives and design standards that are applied to the project at Step 2. This process includes:

   a. definition of a trial layout of a drainage system made up of inlets, pipes, open channels, and storages; and

   b. using a model to define the sizes and locations of components.

10. Determine the adequacy and safety of the design for all relevant storm events.

11. Prepare plans, specifications and design reports and provide essential instructions on how to build the conveyance network.

1 Review the design, obtain approval from the required authorities and proceed with
2. construction or implementation.

Urban stormwater conveyance networks are usually a dendritic or tree-like structure that transports stormwater by gravity. Stormwater runoff is collected using inlet structures (pits) in different branches that converge at junctions along main lines and flow toward an outlet. Inlets structures located at the top of and along network branches:

- admit stormwater runoff into the conveyance infrastructure;
- provide locations where pipe diameters and directions can change;
- provide access for inspection and maintenance; and
- provide overflow points (if necessary).

Examples of underground pipe conveyance networks used in New South Wales and Queensland are provided in Figure 9.5.4.

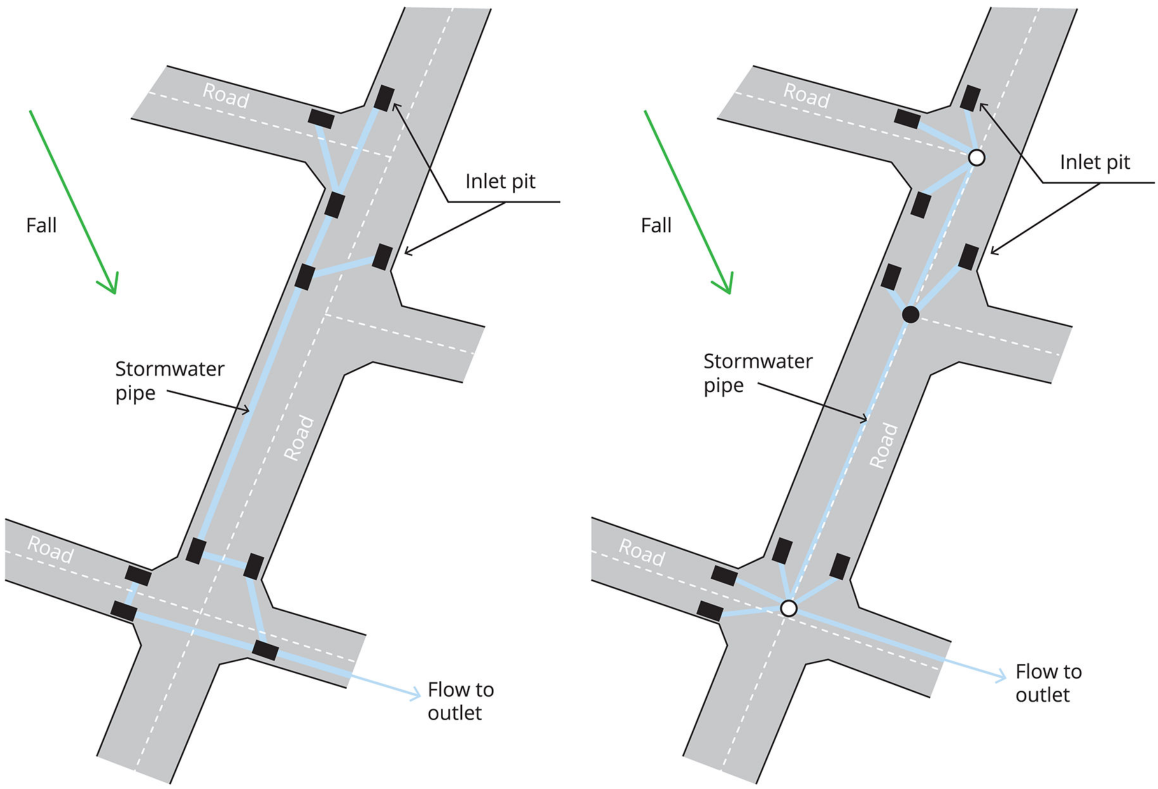


Figure 9.5.4. Examples of Configurations of Conveyance Networks in New South Wales (Left) and Queensland (Right)

Figure 9.5.4 shows different configurations of conveyance networks used in New South Wales and Queensland which highlights that a range of configurations are favoured across different jurisdictions. For example, in some Queensland jurisdictions, pipes are located under road centrelines and manholes at junctions in the conveyance network are used as collectors from inlet pits. Differences in terminology also occur across jurisdictions. For example, in New South Wales 'kerb and gutter' is used, while in Victoria and Queensland the term 'kerb and channel' is employed.

In some cases, maintenance holes, junctions or junction boxes (pits) are provided as nodes linking branches in the conveyance network. Other pits that are intended to overflow are called surcharge pits, overflow pits, or ‘bubble up’ pits. In established urban areas, looped networks may occur where additional pipes are added to provide additional conveyance capacity which can change the behaviour of the original conveyance network.

Conveyance infrastructure (for example: pipes, culverts, channels, and swales) are, mostly, constructed as straight sections with constant slope. Pipe and culvert conveyance infrastructure are available in standard dimensions supplied by the manufacturers. For example, the diameters of PVC pipes range from 90 mm to about 600 mm, and the diameters of reinforced concrete pipes start at 225 mm and increase to over 2 metres. Road authorities usually specify a minimum pipe diameter of 300 mm to 375 mm within road reserves to improve maintenance outcomes.

It is vital that conveyance networks include overland flow paths to control major stormwater runoff events. These overland flow paths should be within road profiles or through open space and pedestrian pathways. Flow paths through private property should be provided as a last resort and will require an easement (a legal instrument providing a right to drain stormwater through a property and permitting authorities to enter the site for maintenance). Overland flows directed through private property can create hazards and inhibit the development and value of the property as the required easement cannot be blocked or built upon.

Conveyance infrastructures (pipes) are designed to limit surface flows on roads to avoid nuisance to pedestrians and motorists. This process incorporates the design of roads including profiles of high locations (most often along road centrelines) and low locations (most often the inverts of gutters). The trapped low points in road networks require the provision of sag pits which will usually inform the required network of conveyance infrastructure (pipes) that can be realised by ‘joining up the dots’ between pits as shown in Figure 9.5.5.

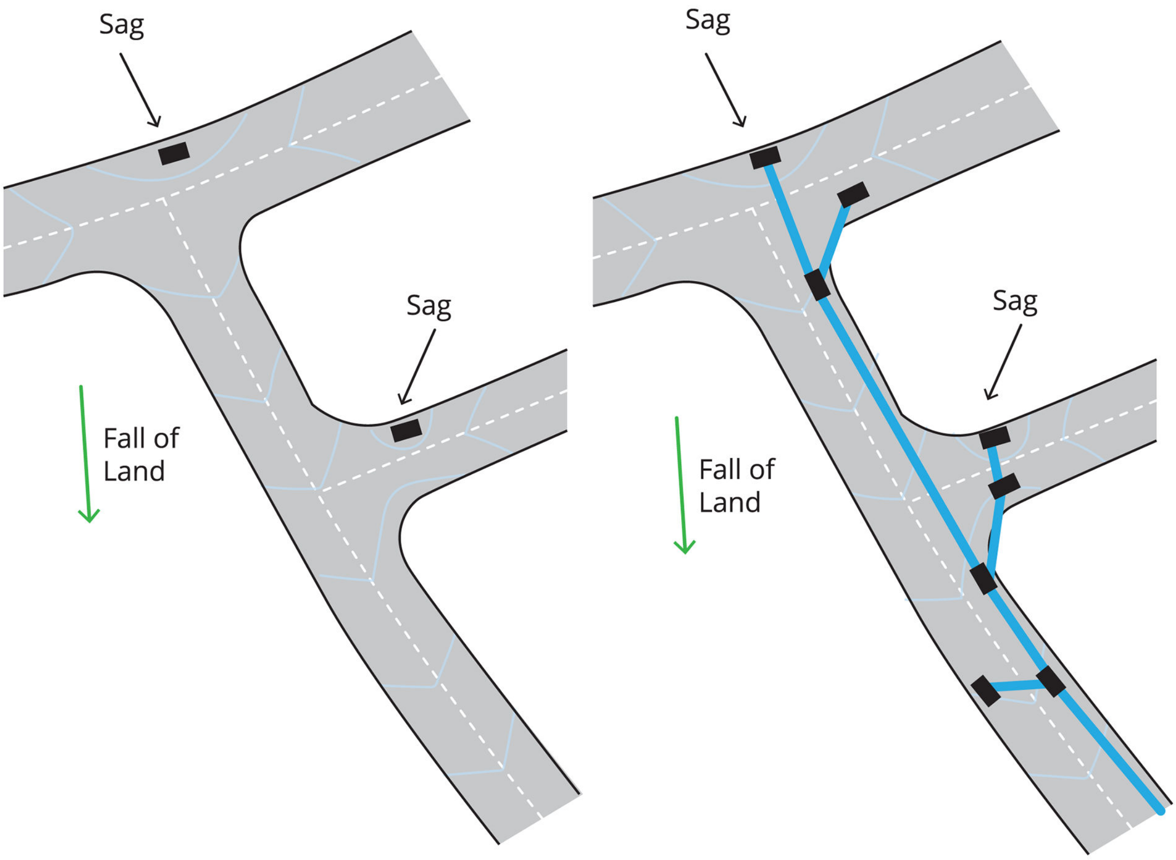


Figure 9.5.5. A Typical Configuration of a Conveyance Network

The configuration of an urban drainage network is demonstrated in a simple example of a single street in Figure 9.5.6.

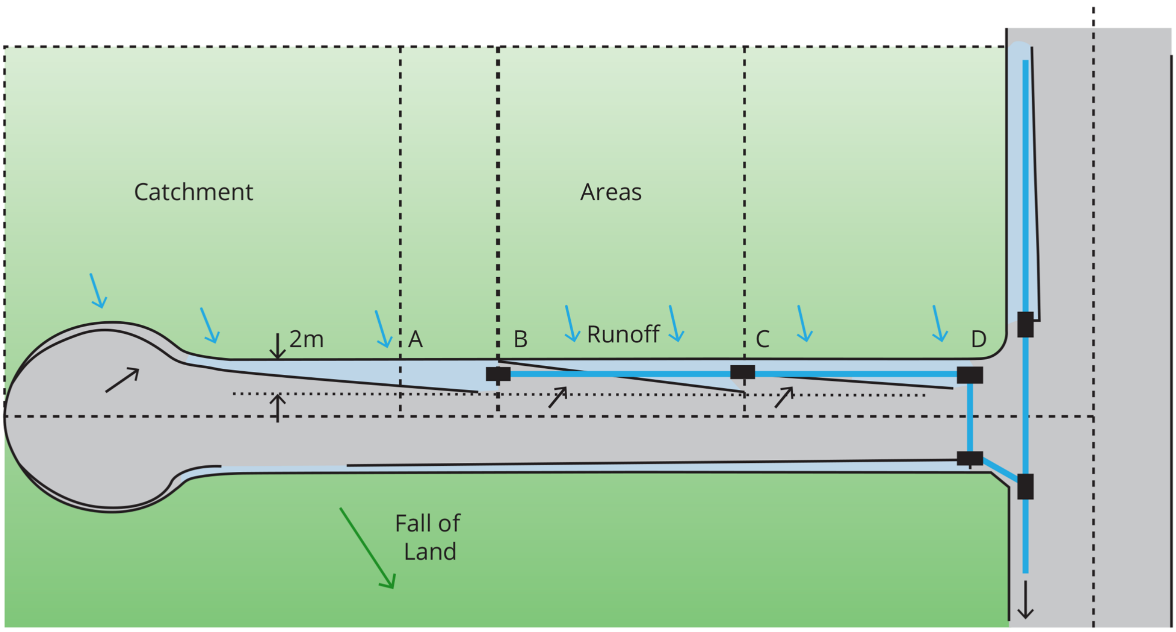


Figure 9.5.6. A Simple Example of a Stormwater Conveyance Network

Figure 9.5.6 indicates the location of inlet pits at the top of a conveyance network. The street gutters are part of the conveyance network and are utilised to transport stormwater towards inlet pits that are situated at intervals to ensure that acceptable flows are carried in road gutters. The width and depth of flows in gutters are limited to allow unimpeded access for pedestrians and vehicles. A maximum width of gutter flow of 2 m to 2.5 m with a maximum flow depth of 150 mm is generally acceptable. Local authorities typically provide guidance on these values. Locations of inlet pits to ensure adequate conveyance of stormwater may also be determined from percentages of stormwater runoff captured by each pit, and the depth of flow and the velocity-depth product of flows in the road gutter.

A designer typically prefers collecting all stormwater runoff from the upper side of the street as shown in Figure 9.5.6 with an inlet pit at Point D which avoids the need for a conveyance branch in the street. This possibility is evaluated by establishing a trial location (A) where stormwater runoff from the corresponding catchment is calculated and the corresponding width of gutter flow is estimated. The width of flows will increase along the gutter length as the areas of contributing catchments increase. A pit must be located whenever any of the criterion limits (such as flow width, depth, or velocity-depth product) are reached. Note that the design process is about limiting surface flows.

Capture of all stormwater runoff at inlet pit B reduces surface flow to zero just downstream of the pit, with surface flows increasing again along the gutter due to lateral inflows from the catchment. However, it is unlikely that on-grade pits will capture all stormwater runoff from catchment areas during minor storm events that create bypass flows downstream of the pit. This is shown at inlet pit C where the flow width increases and reduces due to the pit with a bypass, and some width of flow just downstream of the pit. The flow widths along the gutter will typically follow a saw-tooth pattern.

Figure 9.5.6 also highlights that an inlet pit must be located upstream of a tangent point at an intersection to prevent excessive surface flows at the kerb return. Bypass flows from this inlet pit are collected at inlet pit D. The other pits at the intersection are located along the path of surface overflows to collect both minor and major overflows. This configuration of inlet structures (pits) allows pedestrians to cross at street corners without being exposed to large widths of flows.

The location of inlet pits in the conveyance network may also be driven by a need to provide an inlet at a significant location, such as near a school with street crossings or at a change in road alignment. Aspects of good design practice include location of inlet pits upstream of driveways and avoidance of clashes with other services. A conveyance network also includes additional pipe connections from private property that should be incorporated in the design or analysis of the conveyance network for the street. This may include directly connected pipes from sources such as inter-allotment drainage, on-site volume management systems (such as onsite detention and rain water tanks), or major commercial developments. The first inlet pit in the conveyance network for the street may be receiving considerable pipe flow from upstream private property.

The designer needs to decide on the density of inlet pits in the conveyance network. This decision will typically be guided by the local authority. For example, the two arrangements of inlet pits at an intersection may be acceptable in two different scenarios as shown in Figure 9.5.7.

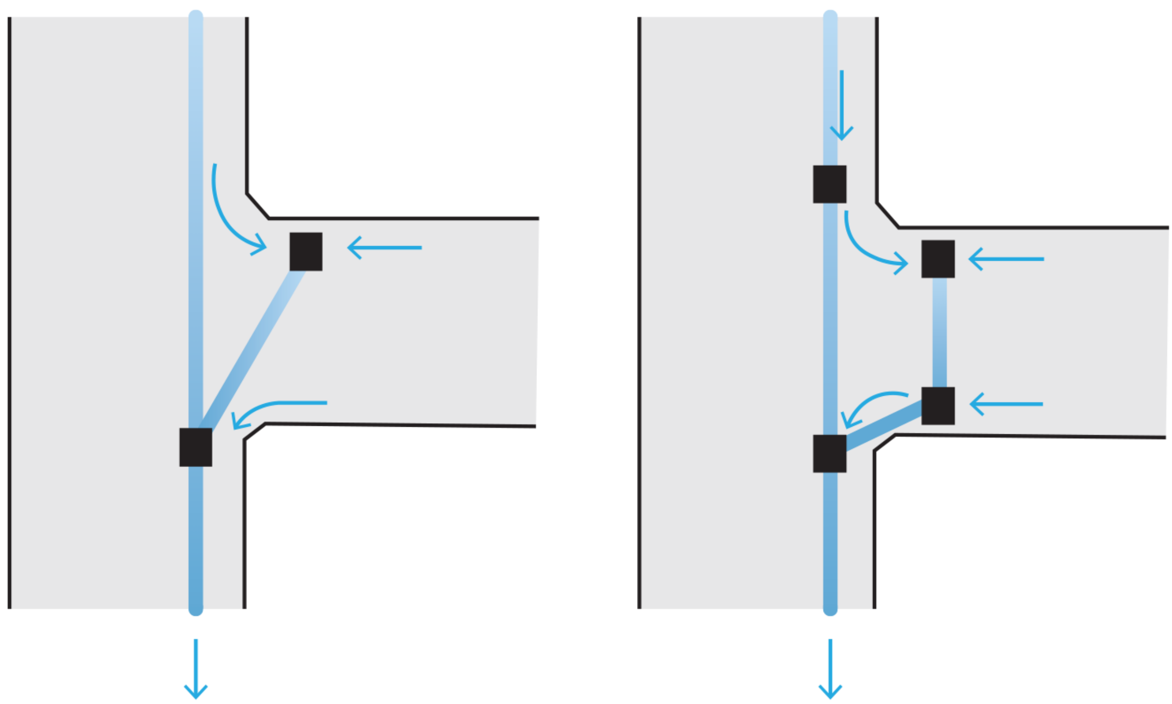


Figure 9.5.7. Example of Alternative Configurations of Inlet Pits at a Road Intersection

Figure 9.5.7 demonstrates different arrangements that may be required at an intersection that could use two or four inlet pits. The decision about the configuration of inlet pits is dependent on the magnitudes and consequences of the flows that may bypass the inlet pits. In a densely-developed area, where overflows or bypass flows may cause nuisance and damage, a greater number of inlet pits will be preferred. Fewer inlet pits can be used in a lower density development where surface flows are more easily managed and the consequences of overflows or bypass flows are small.

5.4. Design of Conveyance Networks with Computer Models

Design of urban stormwater conveyance networks has a long history in Australia. Hand calculations using the urban Rational Method was discussed in the 1958, 1977 and 1987 editions of ARR as the most utilised method for estimation of stormwater inflows to conveyance or drainage networks. There is significant ongoing concern about the reliable characterisation of the parameters (such as runoff co-efficient and time of concentration) underpinning the Rational Method due to insufficient rainfall runoff observations in urban areas (Coombes et al., 2015).

A transition into the computer age heralded the design and analysis of urban conveyance networks using computers to operate drainage software or spreadsheet manipulation that often implemented the Rational Method. The sizing of conveyance infrastructure was based on estimates of peak flows. Increases in computing power allowed greater access to software that integrated hydrology and hydraulics to more accurately analyse or design conveyance networks using hydrograph methods. Additional details about urban modelling approaches are provided in Book 9, Chapter 6.

The characteristics of contemporary urban stormwater management have evolved to be different to the objectives and design solutions for urban stormwater drainage or conveyance networks as envisaged in 1987 (Coombes, 2015). Since the 19th century, the Rational Method and hand calculations has evolved into modern rainfall runoff models (refer to Book 9, Chapter 3). The catchment area has been subdivided into sub-catchments. Average rainfall intensity derived from storm bursts has been modernised to include temporal patterns, spatial variation, relationships between different burst rainfall depths and durations, and the capture of partial areas effects. The runoff coefficient for estimation of stormwater runoff has been replaced with processes that account for the degree of urbanisation and spatial distribution of different land uses, addition of loss models to determine rainfall excess, accounting for pervious and directly or indirectly connected impervious surfaces, and inclusion of depression storages.

Rainfall runoff models have also incorporated connective components including:

- the shape of drainage networks;
- addition of drainage network conveyance, travel times and system storages;
- a separation of minor and major systems;
- response times of different components (such as roads and gutters); and
- bypasses of drainage pits and storages in sag pits.

These evolving models account for modern urban features including distributed storages such as rain water tanks, bio-retention and on-site detention; detention basins and the spatial distribution of urban features (refer to Book 9, Chapter 4). Modern design criteria include analysis of the volume, timing and frequency of stormwater runoff to determine peak flow rates, water quality and requirements. This is done to mimic natural regimes of volumetric flows to protect waterway health (Walsh, 2004; Walsh et al., 2016). Management of the volume of stormwater runoff and the frequency of runoff events from urban catchments is now seen as a key design objective to mitigate downstream flooding and protect the health of urban waterways.

Predictions of peak stormwater flows using the Rational Method may not adequately represent the fundamental processes occurring within contemporary urban catchments (Coombes et al., 2015). This concern is particularly relevant to modern stormwater management methods, such as Water Sensitive Urban Design (WSUD), that include cascading integrated solutions involving retention, slow drainage via vegetation, harvesting and reuse of stormwater and the disconnection of impervious surfaces. These distributed solutions within catchments alter runoff volume and timing in a variable manner throughout a catchment (refer to Book 9, Chapter 3). These dynamics are more likely to be revealed by advanced analysis methods. Importantly, provision of optimum designs for urban stormwater management is dependent on testing solutions across the full range of urban dynamics. The limited urban data available for characterising the parameters underpinning the urban Rational Method for average urban conditions remains a challenge.

The design procedure using computer models is typically implemented more easily and accurately than the simpler design methods. A main advantage is the ability of a computer model to rapidly perform design procedures once a system is set up and the necessary data is entered. In addition, use of computer software allows simultaneous analysis of both minor and major storm events to adequately size inlet structures and conveyance infrastructure, ensuring safe overland flow outcomes.

The procedures for design of conveyance networks have evolved from simplifying assumptions required for hand calculations, such as assuming that pipes are flowing full but not under pressure. Modern methods include more calculations and checks, and can apply unsteady flow hydraulic simulations throughout conveyance networks. These complex calculations are implemented using computers. The amount of calculations is now so large that simple numerical checks using hand calculations are not possible. However, 'sanity checks' can (and should) be made to compare results from models using simplified procedures such as estimating flowrates per unit area. These simple checks will provide estimates that are different to the results produced by computer models, however, this process should assist in avoiding gross errors.

Peak flowrates and hydrographs calculated by rainfall-runoff models are inputs to hydraulic models that determine the characteristics (elevations, depths, widths and velocities) of stormwater flows throughout catchments. Hydraulic modelling is based on physics and requires that the geometry of components of a conveyance network should be carefully defined. Key hydraulic concepts such as Continuity, Conservation of Mass, Energy, and the Bernoulli's Equation, are covered in Book 6, Chapter 2. The Friction Equations including Darcy-Weisbach, the Manning and the Colebrook-White Equations (Book 6, Chapter 2) are all important considerations for the hydraulic design of conveyance networks.

A range of hydraulic models can be used to design conveyance networks. The performance of the models can be illustrated by Hydraulic Grade Lines (HGL) and Eenergy Grade Lines (EGL, also called Total Energy Line, TEL). These grade lines are described in books on fluid mechanics and hydraulics and are useful for understanding flow phenomena.

The design of conveyance infrastructure is highly dependent on the capacity of inlet structures. Inlet structures (pits) are chosen, and then surcharges from each structure are calculated, which determines the possible cumulative surface flows throughout sub-catchments. A check is made to see whether the hydraulic characteristics of surcharges exceed performance objectives for the network (such as safety and access criteria). If the performance objectives are exceeded, the size of inlet structures needs to be increased using the next largest inlet structures and calculations proceed. During the design of inflows and associated dimensions of inlet structures, conveyance infrastructure (pipes) are also sized to ensure that the performance objectives are met. This process continues until a satisfactory level of surface flow is reached.

5.5. Inlet Structures

The performance of urban conveyance networks is dependent on the effectiveness of the inlet structures (pits) that capture stormwater runoff at regular intervals throughout the network. Relationships for the capacity of inlet structures determine the magnitudes of bypass flows and are an essential part of the design of conveyance networks. Designers should be concerned that flow widths and depths are within appropriate limits, both upstream and downstream of an inlet structure.

Designers need to consider that the effectiveness of inflow structures is impacted by the inflow of stormwater through grates or kerb inlets, and by the energy losses or pressure changes that are created by inlet structures (refer to Figure 9.5.8). Historically, pit losses were simplified as a single simple coefficient that approximates the reality of entry losses to the pit, losses within the pit and exit losses from the pit. A simple single coefficient is generally used for many different types of inlet structures.

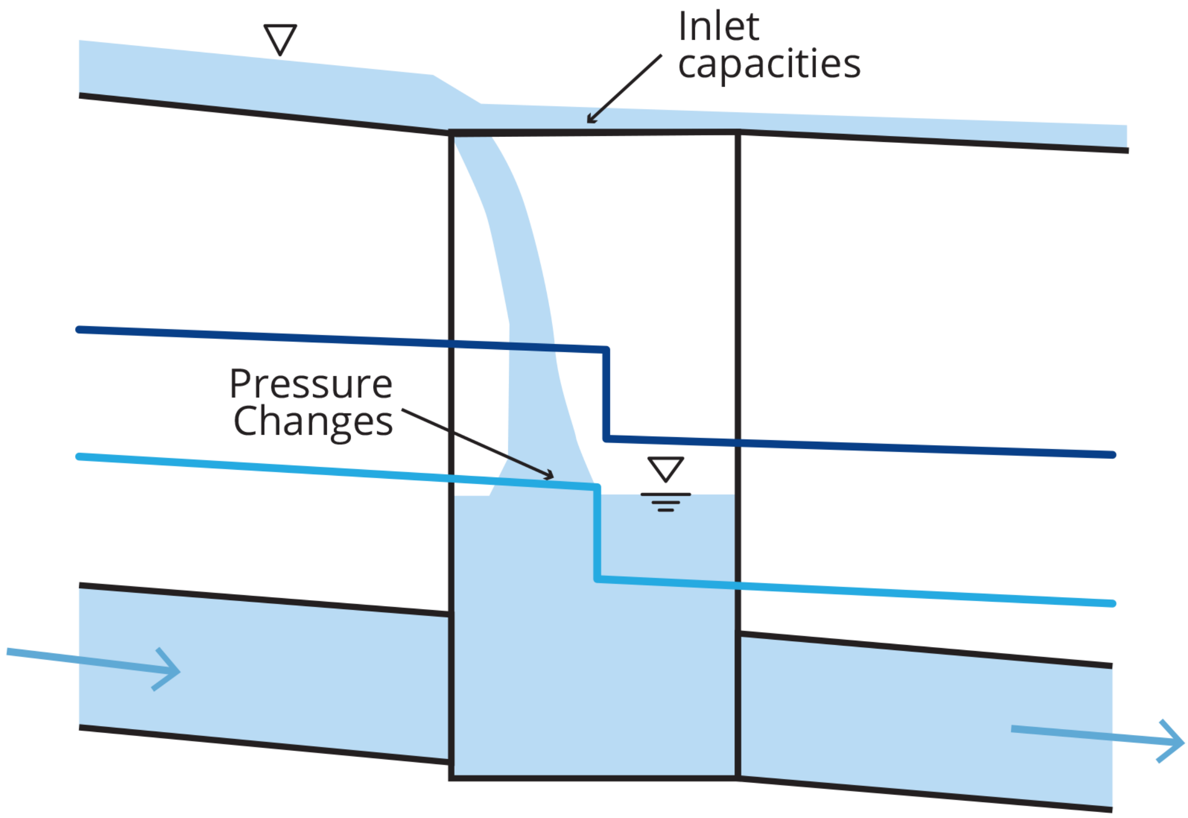


Figure 9.5.8. Idealised Hydraulic Issues Impacting on Inlet Structures

5.5.1. Types of Inlet Structures

A majority of urban stormwater runoff enters conveyance networks via inlet structures located in gutters and medians of roads. These inlet structures or drainage pits are classified by shape or configuration, and are also defined by location on a slope (on-grade pits) or in a depression (sag pits), as shown in Figure 9.5.9. On-grade and sag inlet structures are subject to different hydraulic processes. The behaviour of on-grade pits links inlet capacities to approaching flowrates and resulting bypass flows. Performance of sag pits is dependent on stormwater inflows, pipe outflows and depths of ponded water over pits which cannot escape without passing over footpaths or crowns of roads.

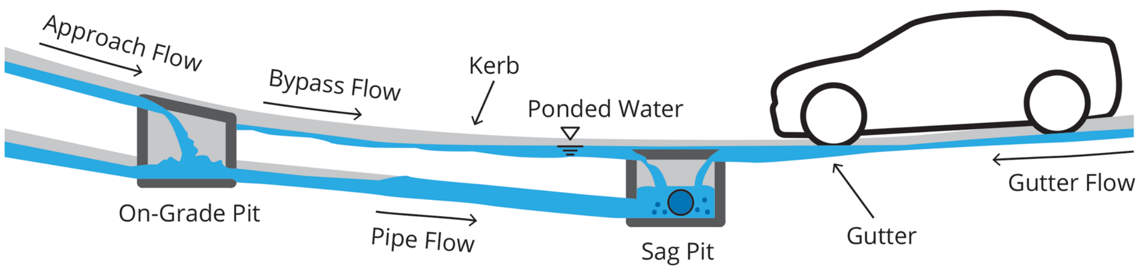


Figure 9.5.9. Basic Types of Inlet Structures

It is desirable for an inlet structure to maximise collection of stormwater runoff. However this objective must also include the safety and convenience of pedestrians, cyclists and motorists, and costs of infrastructure. Open pit structures that may provide the greatest inlet capacity are unacceptable in most environments. The design of inlet structures must not permit children to enter the pit or the conveyance network.

Grates and kerb inlet pits (also referred to as side entries or lintels) are typical inlet structures that are deployed either separately or in combination. Capacities of inlet structures can be improved by providing extensions to kerb inlets, deflectors (ribs or grooves that direct water into an inlet), depressed grates and gutters, or clusters of inlet structures that include adjacent installation of two or three standard pits. Grates and depressions of inlet structures should not be hazardous to road users, including cyclists, and their use should be avoided on busy narrow roads. Aspects of inlet structures for bicycle safety are discussed by the U.S. Federal Highway Administration (Burgi and Gober, 1977).

There is limited information on simple relationships available for the capacity of many types of inlet structures. Many investigations of pit entry capacities have utilised hydraulic models. A range of significant historical studies were published by Burgi and Gober (1977), the Australian Road Research Board (1979), NSW Department of Main Roads (1979) and Marsalek (1982). More general information about capacities of inlet structure are provided by Searcy (1969), Jens (1979), Marsalek (1982), Mills and O'Loughlin (1986), and Argue (1986).

More recent laboratory experiments have examined capacities of different inlet structures at the Manly Hydraulics Laboratory in NSW and at the University of South Australia. The relationships obtained from laboratory tests do not extend to flow rates that may occur in extreme flood events such as 1% AEP or probable maximum floods. However, these relationships are still useful for most design problems as inlet structures in urban areas are predominantly used to admit inflows from minor or more frequent events into conveyance networks.

The US Federal Highway Administration (NHI, 2013) has published the general procedure for determining inflow capacities of on-grade pits in their Hydraulic Engineering Circular No. 22 (HEC-22). The efficiency of various grate types and impacts on inlet capacities for a range of approach grades and velocities are important considerations for urban conveyance networks. In addition to grate and kerb inlets, the capacities of slotted drain inlet structures are also relevant for locations where interception of wide sheet flow is desirable and low sediment and debris is expected. The HEC-22 pit inlet procedures are a useful source of information to aid design of inlet structures.

5.5.2. Inlet Capacities

The hydraulic behaviour of on-grade and sag pits is quite different. These differences are discussed below.

Sag Inlet Pits

The capacities of sag pits are generally independent of upstream gutter slopes and are governed by weir and orifice equations which are dependent on the depth of ponding. The weir equations apply to flows that enter the pit at its edges or at the edges of bars in a grate. Alternatively orifice equations are applied when water ponds above the inlet structure at depths typically exceeding about 0.2 m. The depth of ponding increases to a threshold level and stormwater will overflow as bypass flow by passing over a ‘weir‘ such as a road crown or driveway hump or wall.

The approach and cross-fall grades of roads can affect the availability of storage volumes surrounding sag pits which can indirectly affect the overall behaviour of sag pits. These issues can be considered using hydrodynamic analysis of sag pits as small detention structures. Sag pits must have sufficient inflow capacity to accept the total inflows of stormwater runoff to avoid undesirable ponding of stormwater in intersections to limit obstruction to turning traffic, onto footpaths, into adjacent private properties or basement car parks, or over the crown of a road during a minor storm.

Basic calculations for determining approximate inlet relationships for grated sag pits were derived by Searcy (1969). However, it is preferable to utilise the HEC 22 procedures rather than the sag pit Equation (9.5.1) and Equation (9.5.2) when side entry inlet relationships are required.

For a grate,

$$Q_i = BF \times 1.66\,Pd^{1.5} \qquad \text{up to about 0.12 m of ponding } (d < 0.12) \tag{9.5.1}$$

or

$$Q_i = BF \times 0.67A(2gd)^{0.5} \qquad \text{over 0.43 m of ponding } (d > 0.43) \tag{9.5.2}$$

where

Q_i is the inlet flow rate (m^3/s),

BF is the Blockage Factor

d is the average depth of ponding (m),

P is the perimeter length of the pit excluding the section against the kerb (m) (bars can be disregarded),

A is the clear opening of the grate (m^2), i.e. total area minus area of bars, and

g is acceleration due to gravity (approximately 9.81 m/s^2).

The relationship for inlet capacity between depths of 0.12 and 0.43 is described by ARR 1987 as indefinite and Equation (9.5.1) was recommended in that situation. For an inlet structure that is not located in a depression, the following relationships are recommended:

For ponding up to 1.4 times the height of the inlet:

$$Q_i = BF \text{ x } 1.66Pd^{1.5}h \; (d \le 1.4h) \tag{9.5.3}$$

or

For ponding greater than 1.4h (d > 1.4h):

$$Q_i = BF \text{ x } 0.67A\left(2g\left(d - \frac{h}{2}\right)\right)^{0.5} (d > 1.4h) \tag{9.5.4}$$

where Q_i is the inlet flow rate (m^3/s),

h is the height of the inlet

BF is the Blockage Factor

d is the average depth of ponding (m)

L is the inlet width (m),

A is the clear opening of the grate (m^2), i.e. total area minus area of bars, and

g is acceleration due to gravity (approximately 9.81 m/s^2).

Charts of the inlet capacity of depressed kerb inlets at sag points are provided by Searcy (1969) and in (NHI, 2013).

On-Grade Inlet Pits

Calculation of relationships for inlet capacity of on-grade pits is more complex than for sag pits as several factors can change the capacity of inlets. These factors include:

- grade of the approach gutter (or channel) which will vary flow velocity;
- road cross-fall which impacts the flow width and consequently the maximum allowable flow depth at the inlet;
- roughness of the gutter and road pavement (or channel);
- efficiency of the grate; and
- entry conditions leading into the pit chamber such as gutter depressions (Figure 9.5.10) and the angle of the throat (inlet to the pit) (Figure 9.5.11).

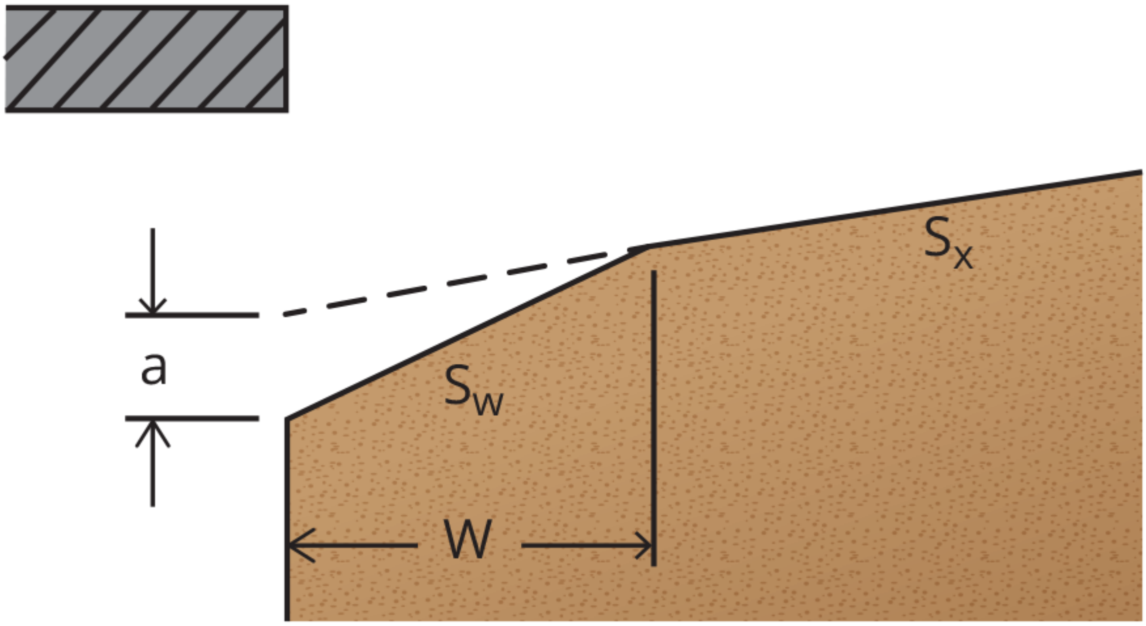


Figure 9.5.10. Kerb Inlet Gutter Depressions from HEC-22 (NHI, 2013)

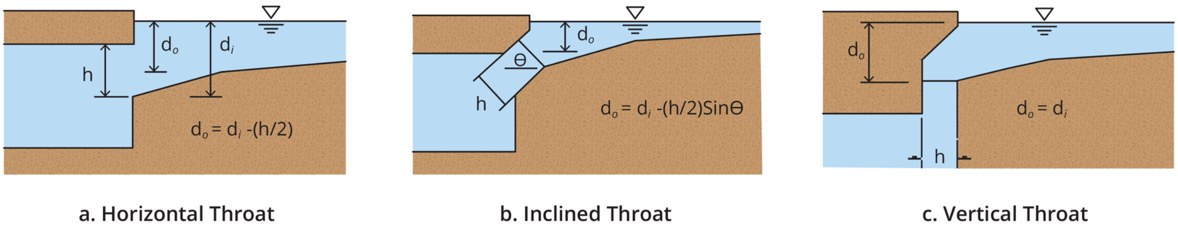


Figure 9.5.11. Kerb Inlet Throat Angles from HEC-22 (NHI, 2013)

The basic calculations to determine approximate relationships for inlet capacities of grate, side entry and combination inlets are provided by Searcy (1969). However, Equation (9.5.3) and Equation (9.5.4) should not be used in preference to HEC 22 procedures which have been hydraulically tested, and where the efficiency of various grate types is provided along with calculations for throat entry conditions. As an illustration, typical relationships for 1 m and 2 m on-grade kerb inlets are shown in Figure 9.5.12 that were derived using the HEC 22 procedures.

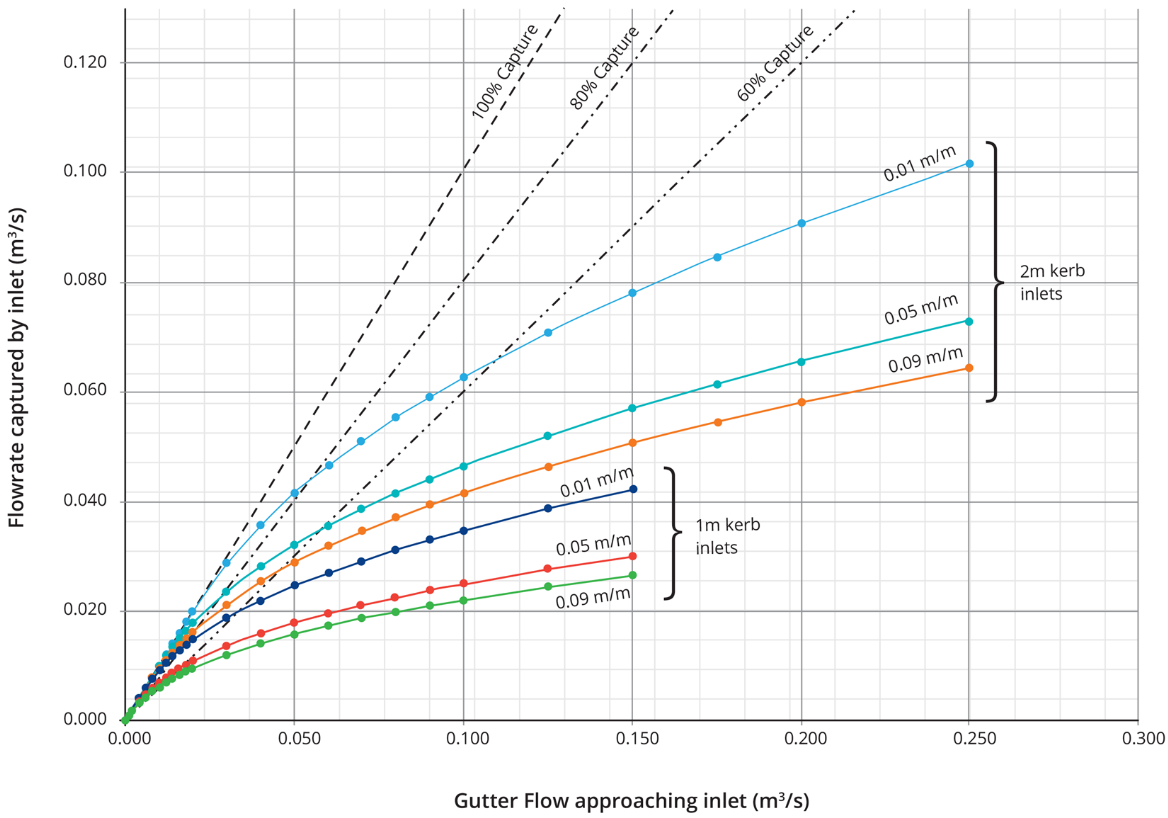


Figure 9.5.12. Inlet Capacities for On-grade Pits

Additional Information

Many different types of inlet structures are used across Australia and this chapter has only discussed some of the configurations. It is recommended that local capacity relationships, knowledge and experience, and types of inlet structures should be employed in designs for urban stormwater conveyance networks. In the absence of mandated design procedures that may be provided by a local authority, preferences should be given to local knowledge and experience, and to laboratory based methods. The designer and local authority should also accept first principles hydraulic analysis and evolving science in the selection of inlet capacities.

Additional resources available to the designer include those provided local and state authorities such as Vic Roads, the (QUDM, 2013) and from older resources including the National Capital Development Commission (1981), the Victoria Country Roads Board (1982) and the New South Wales, Department of Housing (1987).

The usual pit entry capacity relationships may not be adequate for analysis of conveyance networks subject to major rainfall events. In these situations, larger depths of surface flows, velocities and loads of debris may occur, and the inlet capacities of pits will be make for additional discussion). A blockage factor of 50% is generally applied for sag pits for minor and major systems in situations when experimental results or observations are not available. The blockage factor for on-grade pits can vary from 0% and 20% in response to local conditions. Additional advice on blockage factors is provided by Weeks et al. (2013) as shown in Table 9.5.1. Higher blockage factors are often applied for events rarer than the 1% AEP.

Table 9.5.1. Suggested Design and Severe Blockage Conditions for Inlet Pits Book 6, Chapter 6

Type of structure		Blockage conditions	
		Design blockage	Severe blockage
Sag kerb inlets	Kerb inlet only	0-20%	100% (all cases)
	Grated inlet only	0-50%	
	Combined inlets	Capacity of kerb opening with 100% blockage of grate	
On grade kerb inlets	Kerb inlet only	0-20%	100% (all cases)
	Grated inlet only (longitudinal bars)	0-40%	
	Grated inlet only (transverse bars)	0-50%	
	Combined inlets	10% blockage of combined inlet capacity on continuous grade	

Ultimately relationships for the capacity of inlet structures determine the magnitudes of bypass flows and are an essential consideration in the design of conveyance networks. Designers must be concerned that flow widths, depths and product of depths and velocities are within appropriate limits at locations upstream and downstream of an inlet structure. These factors can be controlled by the careful location of inlet structures and by limiting bypass flows using infrastructure with sufficient inlet capacities.

5.5.3. Energy Losses

Significant pressure losses may be created by inlet structures and junctions in conveyance networks. Hydraulic losses are generally reduced when open channel flows occur in conveyance infrastructure (pipes) and benching or smooth transitioning is provided within inlet structures. Higher losses occur at inlet structures when conveyance infrastructures (pipes) are full and surcharging in response to pressure flows. These losses at pits are offset by the increased capacity of pressurised pipes and the entire pressurised conveyance network may cope with greater flow rates. Energy losses at inlet structures are expressed as a function of the velocity V_0 in the outlet or downstream pipe:

$$h_L = k \, . V_0{}^2 / 2g \tag{9.5.5}$$

Where:

h_L is the loss in metres,

k is a dimensionless energy loss coefficient, and

g is acceleration due to gravity (m/s^2).

This energy loss at the inlet structure creates a change in the total energy line (TEL) as shown in Figure 9.5.13. The associated change in the hydraulic grade line (HGL) is likely to be different in response to different pipe diameters and flow rates upstream and downstream of the structure. The position of the HGL is important to designers as it determines the location of the water surface and the degree of surcharge or overflow which may occur at that location in the conveyance network.

The change in pressure head is estimated as:

$$\Delta P / \gamma = k_u . V_0{}^2 / 2g \tag{9.5.6}$$

Where:

$\Delta P/\gamma$ is the pressure head change (m) relating to a change of pressure of ΔP kN / m^2 and the specific weight of water kN/m^3, and

k_u is a dimensionless coefficient of change in pressure.

A similar relationship can be applied to water levels within inlet structures which may be slightly higher than the HGL level due to the conversion of some kinetic energy to pressure energy when stormwater flows through a pit:

$$WSE = k_w . V_0{}^2 / 2g \tag{9.5.7}$$

Where:

WSE is the elevation of the pit water surface (m) relative to the downstream HGL elevation, and k_w is a dimensionless coefficient.

These effects are illustrated in Figure 9.5.13.

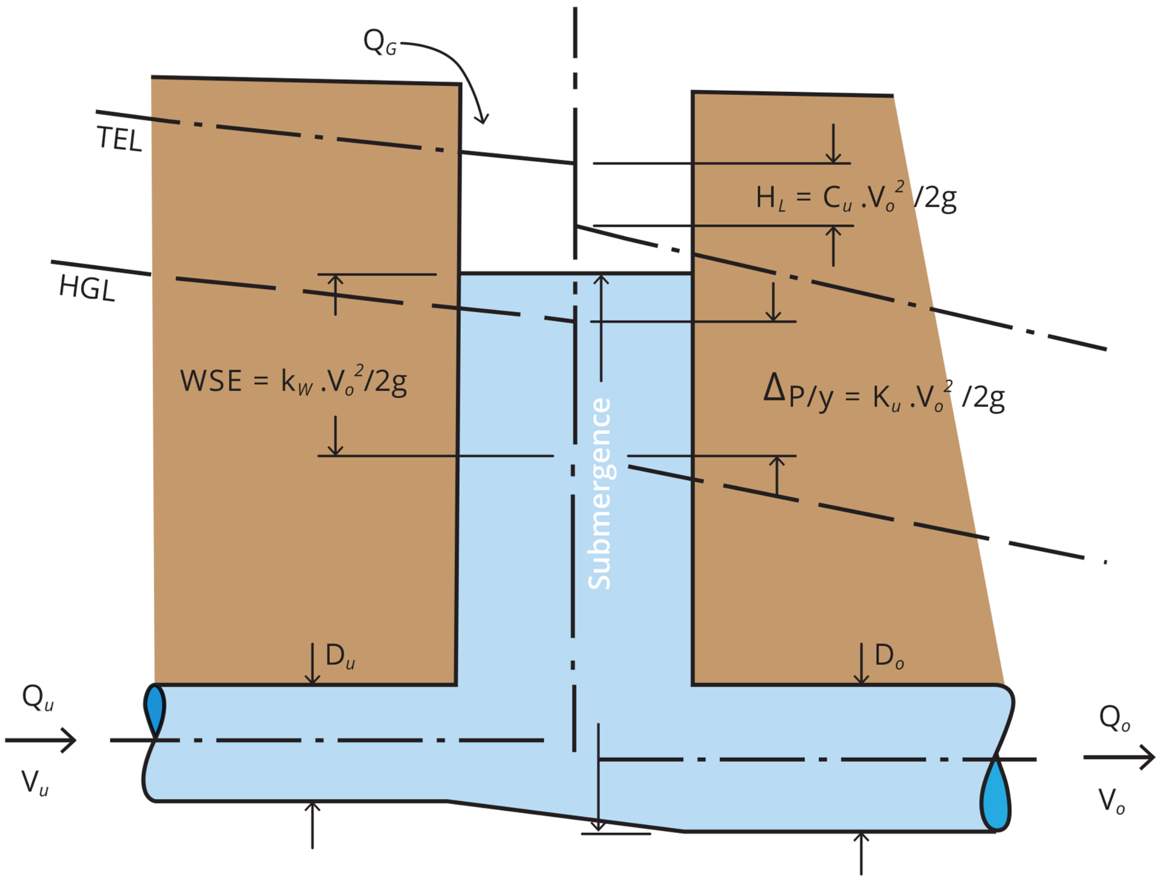


Figure 9.5.13. Idealised Grade Lines at an Inlet Structure

Where Q_o is the downstream discharge, V_o is the downstream flow velocity, D_o is the downstream pipe diameter, Q_g is the surface inflow to the pit, D_u is the upstream pipe diameter, Q_u is the upstream flow rate and Vu is the upstream flow velocity. The parameters k_u and k_W are similar for most configurations of inlet structures and the water level in a pit can be assumed to coincide with the HGL level. The arrangement of grade lines in Figure 9.5.13 is an idealised situation that assumes all changes occur at the centreline of the inlet structure. Losses actually occur across the structure and immediately downstream in the conveyance infrastructure. The convention in measurement of the performance of inlet structures is to project grade lines (measured by manometers in the upstream and downstream pipes) forwards or backwards to the pit centreline and to accept the difference as the overall loss or pressure change.

Available Methods of Determining Pressure Loss Coefficient k_u

Studies using hydraulic models can be used to derive reliable values of energy losses and pressure changes for different types of pits and junctions. A significant study by Sangster et al. (1958) dealt with pipes flowing full and produced a set of design aids for a selected configuration of inlet structures which are now called “Missouri Charts”. Hare (1980); Hare (1983) produced information on other configurations. The charts are complex and provide many possible geometric configurations of inlet structures. Careful judgement is required to select the appropriate chart for a particular configuration of a structure, and in practice, iterative calculations are required to converge to a suitable value of the pressure loss coefficient.

This iterative process can be quite time consuming for large conveyance networks. Attempts have been made to replace dependence on charts with semi-analytical methods. These range from relatively simple methods suggested by Argue (1986), Hare et al (1990) and Mills and O'Loughlin (1998) to more in-depth methods suggested by Parsell (1992) and the US FHWA HEC-22 procedure from which the algorithm described by GKY and Associates Inc (1999) and Stein et al. (1999) has been developed. The FHWA HEC-22 procedure was developed using research and laboratory efforts improving the methodologies of the 'Corrective Coefficient Energy-Loss Method' (Chang and Kilgore, 1989) and the 'Composite Energy Loss Method" Chang et al. (1994). It is also the only method which considers part-full and full pipe flow, drops in pits and other situations.

A summary paper by O'Loughlin and Stack (2002) compared the different algorithms and could not find significant differences which suggested that no single method was superior. However, the information indicated that a viable algorithm can be developed, and that further testing and development is required for the methods to acceptably match the full range of configurations of inlet structures provided by the Hare (1983). The FHWA algorithm appears to provide a significant advance in the determination of head losses and pit pressure changes in stormwater conveyance networks. Comparisons with alternative algorithms and experimental data indicated that simpler methods may provide equivalent results for losses.

Determining Pit Pressure Losses in Practice

Determining pressure changes in practice is complex due to the many possible geometric configurations of inlet structures. Geometric configurations of pits can vary according to:

- number of pipes entering pits (0, 1, 2, 3 or more);
- horizontal change of direction at the pit;
- vertical drop in the pit between inlet and outlet pipes;
- ratios of incoming and outgoing pipe diameters;
- a number of secondary factors, including slopes of pipes, shape and size of inlet structure, depths of sumps in the structure below the invert of the outgoing pipe, streamlining (or benching) of the pit and the entrance to the outlet pipe, and location of the confluence of the incoming pipes.

Variances in flows are impacted by:

- magnitudes of flow and velocity;
- ratios of grate flow entering the top of structures compared to the outflow; and
- tailwater levels.

The design calculations typically need to be repeated to achieve converging values. When designing to satisfy a freeboard requirement, revised coefficients may lead to circular alteration of pit and pipe inlet capacities which requires the designer to intervene.

Initial Estimates of k_u Before Commencing Iterative Calculations

An analysis of the hydraulic grade line of a pipe requires an estimated value of k_u at each inlet structure. Some government authorities may provide suggested values and experienced designers are likely to have developed 'rule-of-thumb' methods for determining these initial estimates of k_u. Engineers are encouraged to use these methods in hydraulic design wherever the methods have proven to be effective.

Guidance for initial estimates of k_u is provided in Figure 9.5.14 for a range of common pit configurations. These are not absolute or recommended values for final analysis of a network and are only indicative starting points of iterations required to converge to a final value. These estimations assume shallow pipes with typical minimum covers and no increases in outlet pipe diameters. Deeper inlet structures may increase values of k_u and increases in outlet pipe diameters may reduce values of k_u

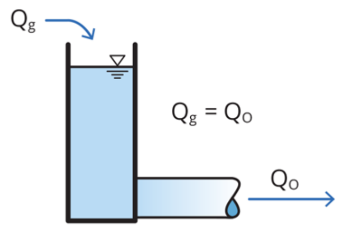
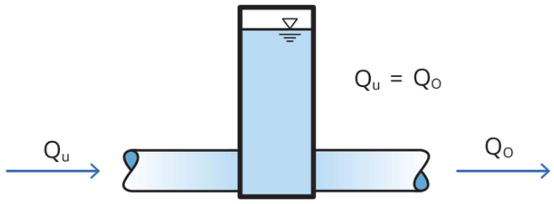
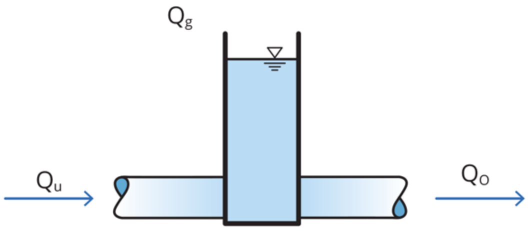
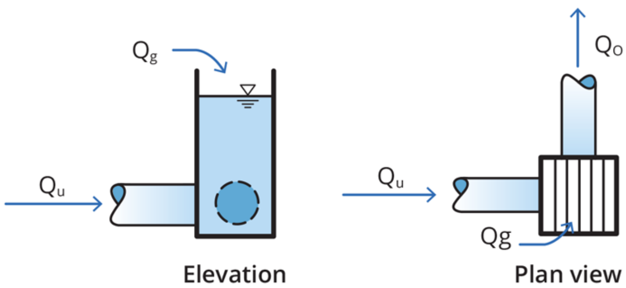
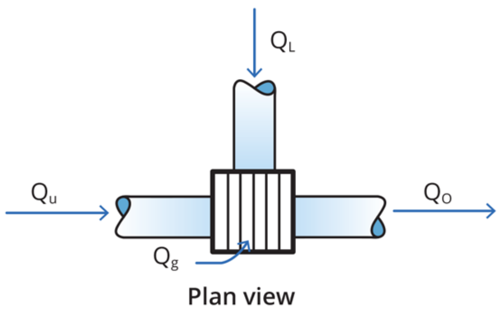
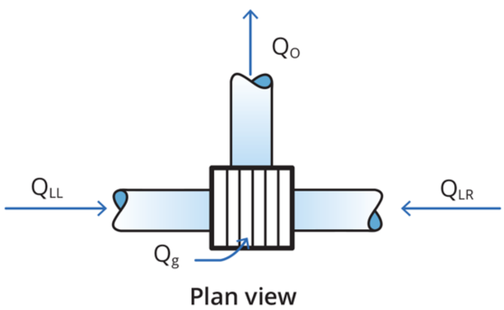
Pit Configuration	Initial k_u	Pit Sketches
First pit at the top of a line	4.0	
Well-aligned junction pit with straight through flow, no sidelines, no grate inflow	0.2	
Well-aligned pit With straight through flow, no sidelines, 50% grate inflow	1.4	
Pit with a 90° right angle direction change, no sidelines, 50% grate inflow	1.7	
Pit with a straight through flow, one or more sidelines	2.2	
Pit with a right angle direction change from two opposed inflow pipes	2.0	

Figure 9.5.14. Approximate Pressure Change Coefficients, k_u, for Inlet Structures

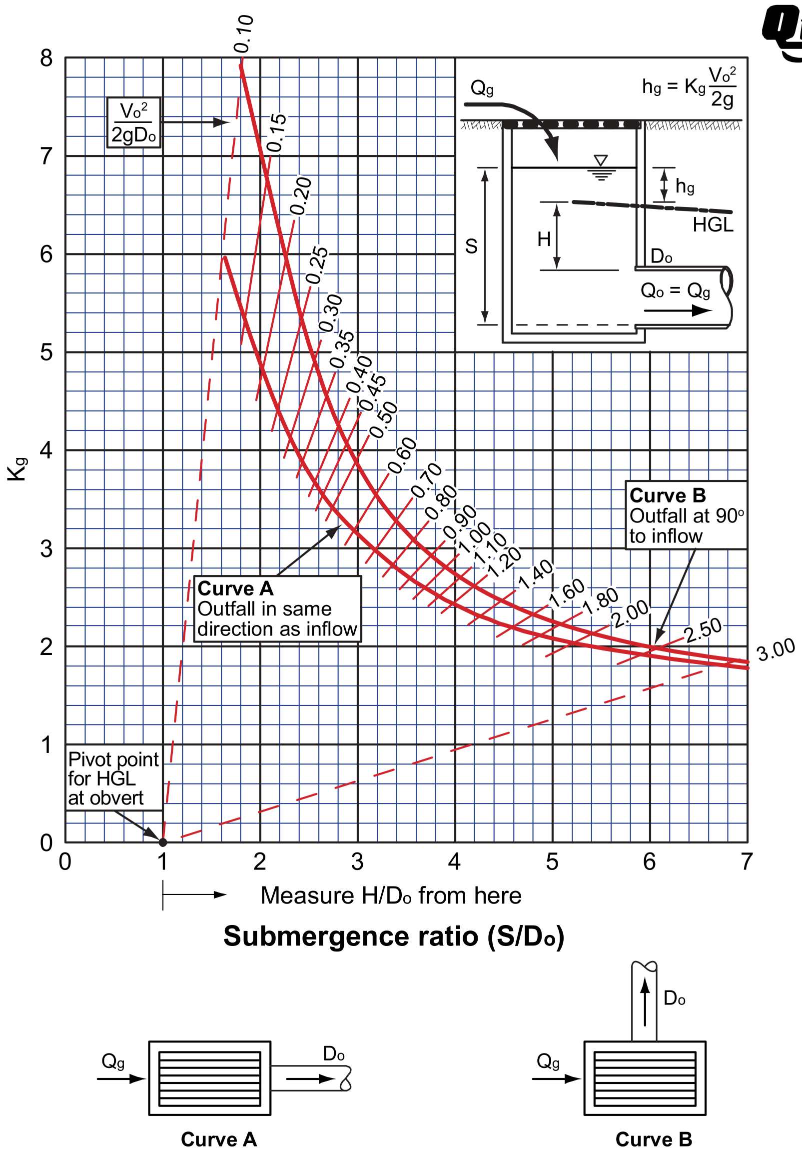
Simplified Approach

As discussed earlier, simplified design methods are available such as those presented by Mills and O'Loughlin (1998), Hare et al. (1990), and Argue (1986). Although these simpler methods may provide similar results to more complex semi-analytical methods, further laboratory research and development was recommended to account for the full range of pit configurations considered by the original Missouri Charts (Sangster et al., 1958) and by Hare (1980). Whilst simplified design methods may be considered for use during simple, non-critical pit and pipe network designs, use of Missouri Charts and Hare's results is preferred.

Recommended Approach

The Missouri Charts (Sangster et al., 1958) and the results from Hare (1980) remain widely accepted and are relevant to an estimated 85% of the possible configurations of inlet structures. The example charts presented in Figure 9.5.15 and Figure 9.5.16 are based on this information (QUDM, 2013). The first chart (Figure 9.5.15) was derived from the original Missouri Chart 2 with modification from the Department of Transport (1992) for an inlet structure with grate flow only. The pressure change coefficient k_u depends on the submergence ratio S/D_0 and iterative calculations are required.

The second example chart (Figure 9.5.16) was modified from the Missouri Chart 4 to include the results from(Hare, 1980). The inlet structure accommodates flows straight through the pit for a submergence ratio S/D_o of 2.5 and also considers inflows through grates. Here k_u depends on the ratio D_u/D_o and provides flow ratios Q_g/Q_o ranging from 0 to 0.5. A correction factor needs to be added from Table 9.5.2 when the submergence ratio S/D_o does not equal 2.5.



Pressure head change coefficients for rectangular inlet with grate flow only modified from DOT (1992)

Notes:

1. For a ***Side inlet,*** the inflow direction should be taken as the direction of flow in the kerb and channel.
2. Where the outflow direction is within 15 degrees of the direction of the direction of inflow, use ***Curve A***.
3. Where the outflow direction is greater than 15 degrees from the direction of inflow, use ***Curve B***.
4. $K_w = K_g$

Chart No. A2-3

Figure 9.5.15. Pressure Change Coefficient Chart (QUDM, 2013)

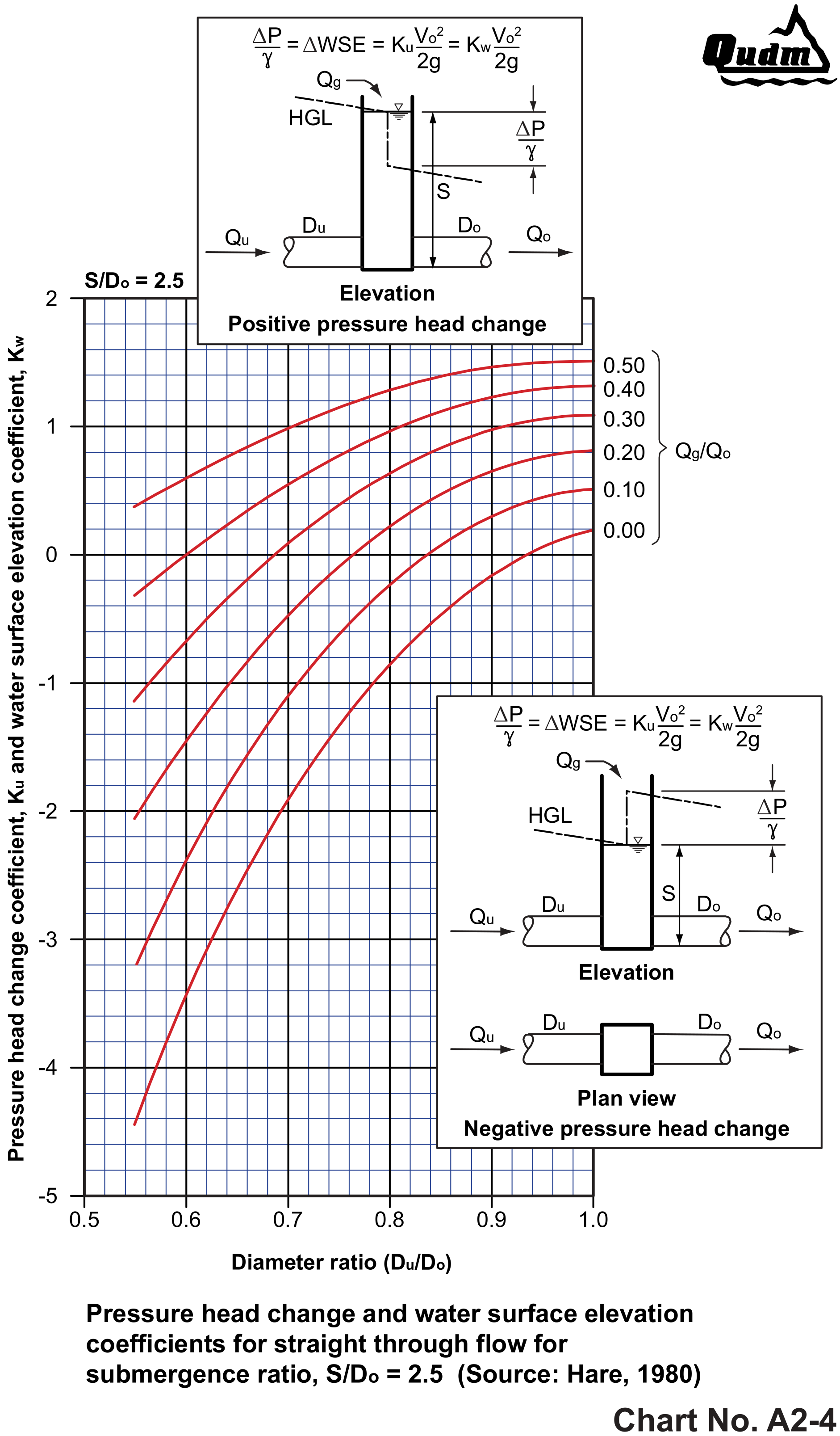


Figure 9.5.16. Pressure Change Coefficient Chart (QUDM, 2013)

Table 9.5.2. Correction Factors for k_u and k_w for Submergence Ratios (S/D_o) not Equal to 2.5 (QUDM, 2013)

S/D_o	Q_g/Q_o					
	0.00	0.10	0.20	0.30	0.40	0.50
1.5	0.00	0.11	0.22	0.33	0.44	0.55
2.0	0.00	0.04	0.08	0.12	0.16	0.20
2.5	0.00	0.00	0.00	0.00	0.00	0.00
3.0	0.00	-0.03	-0.06	-0.09	-0.12	-0.15
3.5	0.00	-0.04	-0.08	-0.12	-0.16	-0.20
4.0	0.00	-0.05	-0.10	-0.15	-0.20	-0.25

Additional influencing factors become apparent as configurations of inlet structure become more complex; such as interpolation coefficients for intermediate grate flow ratios, presence of deflectors and additional lateral or sideline pipes. The second chart (Figure 9.5.16) shows that k_u can be negative in situations where the outlet pipe is larger than the inlet pipe and "pressure recovery" occurs due to the lower downstream flow velocities than the upstream inflow velocities.

Large energy losses and pressure changes can be avoided by attention to simple rules in detailed design and construction. One principle is to ensure that jets of water emerging from inlet pipes do not impinge directly on pit walls. Wherever possible the stormwater jets from inflow should be directed into outlet pipes. Hare (1983) states that changes of flow direction should generally occur on the downstream face of pits, rather than at the upstream face or centre. Losses may be reduced by use of curved pipelines, precast bends and slope junction fittings at changes of flow direction. Typical loss factors for these fittings are:

- tee – k = 1.15 for energy loss expression $kV^2/2g$
- 90° double mitre bend – k = 0.47
- 60° double mitre bend – k = 0.25
- 45° single mitre bend – k = 0.34
- 22° single mitre bend – k = 0.12

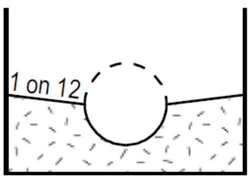
Benching

The recommended Missouri Charts do not include the effect of benching to reduce energy losses. Potential decreases in pressure change coefficients as a result of benching are provided in Figure 9.5.17, (Table 7.16.4 in QUDM (2013)).

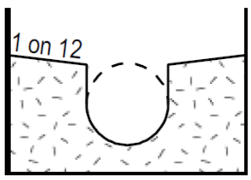
Potential decrease in pressure change coefficient as a result of benching

Access chamber type [3]	Potential decrease in pressure change coefficient (%)	
	Half-height benching [1]	Full-height benching [2]
Straight through	30	40
90° bend	20	40
Tee chamber with lateral inflow less than 50%	Nil	Nil
Tee access chamber with lateral inflow approximately 50%	Nil	10
Tee access chamber with lateral inflow approximately 100%	20	40

Notes:



Note [1]: (a) –
Half-height benching



Note [2]: (b) –
Full-height benching

Note [3]:
Results based upon testing of square pits.

Figure 9.5.17. Decrease in Pressure Change Coefficient as a Result of Benching (QUDM, 2013)

Computer Models

Various procedures have been implemented in computer software. Some unsteady flow computer programs allow for pressure losses in rather simplistic ways, such as increasing pipe friction factors to include estimated pressure losses. Other complex procedures employed by computer software include:

- iterative processes based on Missouri Charts, geometry and hydraulic results;
- semi-analytical algorithm based approaches; and
- numerical methods.

5.6. Conveyance Infrastructure

Urban conveyance networks collect rainfall runoff from urban surfaces (properties and adjacent roads) and utilise gutters, road surfaces, pipes, culverts and channels to convey stormwater to downstream infrastructure or receiving waters. This section discusses the design of conveyance infrastructure.

5.6.1. Hydraulic Models to Define Flow Characteristics

The complexity of conveyance networks requires that simple calculations based on energy gradients are often replaced by more complex procedures. Rainfall runoff is collected at multiple entry points (inlet structures) and accumulates throughout the conveyance network.

The necessary calculations combine these inflows and route them throughout a network by determining the water depths and velocities in the conveyance infrastructure. Simpler methods or models can do this for steady flows with unchanging flow rates whereas more complex models are required for unsteady and time-varying flow ates. Hydraulic grade lines (HGL) and energy grade lines (EGL) can be used to define flow depths, pressures and energies in conveyance networks as shown in Figure 9.5.18. Hydraulic models must allow for overflows when water levels exceed limits or pass over barriers. Additional information about hydraulic models is provided in Book 6 and Book 9, Chapter 6.

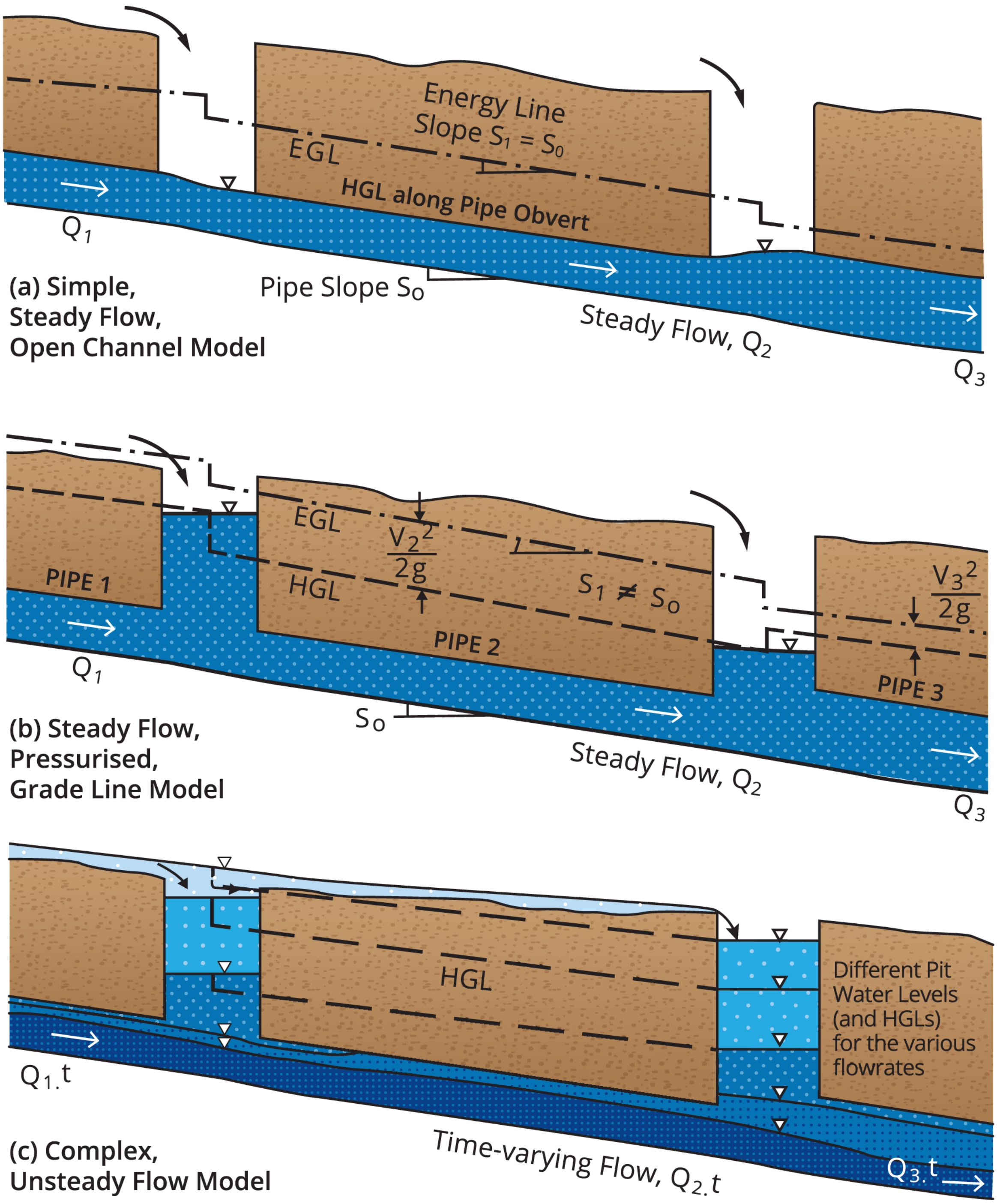


Figure 9.5.18. Schematic of Three Hydraulic Models of Conveyance Networks

Figure 9.5.18(a) demonstrates a simple model that accepts peak inflows derived from a hydrological model. It is assumed that steady flows occur in each pipe reach or link. Hydraulic grade lines are assumed to be located at the obvert (upper inside surface) of pipes and the flow condition is described as “flowing full but not under pressure”. Allowances for local losses are provided by a small drop (up to 90 mm, depending on change of flow direction) within inlet structures. The capacity of pipes can be calculated easily by applying a friction formula such as the Manning Equation and accounting for the grade of the pipe. The conveyance network is assumed to behave as a network of open channels and no allowance is made for upstream or downstream surcharges.

Figure 9.5.18(b) shows a second approach to hydraulic analysis that also assumes steady-state conditions where peak flows occur as pressure flows in pipes and the HGL is located above or along the pipe obvert. This method includes energy losses and pressure changes at inlet structures that are likely to be greater than open channel flow assumptions where water levels are below pipe obverts. Capacity of pipes is also dependent on downstream water levels which may create backwater effects on flows in pipes.

These methods accept peak flow from hydrological models and assume that peak flows occur simultaneously throughout the conveyance network. Flow rates are constant within each link and the calculated HGLs and EGLs represent upper envelopes of these flows. This process will usually estimate lower pipe capacities than unsteady flow assumptions.

Figure 9.5.18(c) presents unsteady flow processes that are created by the inflow to the conveyance network of full hydrographs typically generated in computer models and real rainfall depths and patterns. The simulations account for the changes in water levels and flow characteristics in pipes and the network throughout storm events. These processes include dynamic effects such as fast-travelling waves generated by changes in flow conditions that can create shock losses in the conveyance network. This model is applied using computers that process and solve finite difference computations. A steady flow system is assumed to be independent of time and only requires one set of calculations. However calculations in computer models of unsteady flows are repeated for many time steps and pipe reaches are divided into several sections during the calculation processes.

All three hydraulic models can be utilised for design and analysis. The first and simplest method can be used for design of small networks where downstream conditions may ultimately be varied to account for the actual behaviour of the conveyance network. These adjustments may not be possible for design of a fixed conveyance network, and the estimated capacities and impacts of conveyance network may be incorrect.

The assumed steady-state flows and a connected hydraulic grade line throughout a network of the second method is more suitable for basic design and analysis tasks. This method is likely to provide more efficient designs as it more closely reproduces real hydraulic behaviour and allows for surcharging of pits and pressure flows. This process may be used as a checking procedure by working backwards from the receiving water level towards the top of the catchment.

This model was presented by ARR 1987 (Pilgrim, 1987) as the preferred hand calculation method for hydraulic design of simple pipe networks. Calculations typically involve two iterations for a conveyance network. The first iteration commences at the top of the catchment, accumulating the flows arriving at each inlet structure, and allows for possible bypass flows at pits. The calculated flows through the conveyance network are used to determine the sizes of pipes and the invert levels at their ends whilst ensuring that HGLs do not rise above a limit, usually 0.15 m below the surface level of inlet structures. This design procedure involves a series of trials with increasing pipe sizes selected from the commercially available diameters. The smallest commercially available pipe diameters that meet the design requirements are typically selected.

The iteration of the calculations commence at the outlet using a set tailwater level and project the HGLs upward towards the top of the catchment by considering the HGL slope due to pipe friction and the local pressure changes at each inlet structure. The previously calculated flow rates, pipe diameters and water levels in pits can be used in design charts such as the Missouri and Hare charts to determine local pressure changes at pits (refer to Book 9, Chapter 5, Section 5). When the upstream process of calculations reaches an inlet structure with two or more pipe branches, the calculations progress separately and upstream in each branch.

This projection process can be employed for part-full pipe flows, and for pressurised and full-pipe flows. However, the straight water surface profiles assumed for part-full flows will not be exact. A more accurate procedure is to project water surfaces upstream using the gradually-varied flow methods commonly called backwater curve computations.

Some designers of conveyance networks are still using the simple, steady flow procedures. However the unsteady flow models produce more realistic behaviours in response to hydrographs and flow volumes that are essential for analysis volume sensitive systems that include volume management facilities (refer to Book 9, Chapter 4). Modelling using unsteady hydraulic (and hydrology) assumptions is the preferred method for detailed analysis of conveyance networks that need to respond to strict constraints and where realistic modelling of network behaviour is needed. This approach is also essential for analysis of existing conveyance networks to replicate an existing deficit in performance or to reproduce a known flooding problem. There are many software products currently available that can be utilised for these types of analysis and design (refer to Book 9, Chapter 6).

5.6.2. Overland Flow

Conveyance networks receive and include overland flows. Overland flow is conveyed as sheet flow across land surfaces or in an overland flow path within a channel or swale. Sheet flow is typically produced when rainfall exceeds the volume of depression storages and infiltration capacity of a catchment resulting in overland flows travelling towards a receiving watercourse or an inlet structure in a conveyance network. Overland flows can also be escaping floodwater when the capacity of a conveyance network or watercourse is exceeded.

Overland flow paths typically convey stormwater when the capacity of the minor system conveyance network is exceeded as bypass flows between inlet structures along a kerb and gutter in a street, along swales in rural or grassed areas, or sometimes undesirably through private property. Calculations for overland flows are similar to open channels in that they can be defined as a number of different channel sections with constant cross-sections and slopes. However a key difference between overland flow paths and open channels is that overland flow paths are typically limited to shallower flow depths to meet safe design criteria. Open channels typically convey stormwater at greater depths and flow rates.

Urban stormwater management may combine buffer strips or vegetated swales or bioretention with overland flow paths as cost effective methods to facilitate attenuation of flows and removal of pollutants.

Limitations of Depth and Width of Overland Flows

The depths, widths and velocities of overland flows should be limited to meet objectives for safety and erosion. A range of conditions may be applied when a cross-section of a road is to be used to convey major and minor flows and the limiting factor is deemed to be the most restrictive criteria. These criteria include risks to pedestrians, particularly children, and the importance of the road for transport purposes. The following conditions should apply when guidance from a consent authority is not available:

- The depth of stormwater flows at the kerb (d_g) should be limited to the lower side of a street to prevent uncontrolled overflows from entering properties. For streets with 150 mm high kerbs and a footpath with a substantial slope towards the gutter, a suitable limiting depth may be 200 mm or to the height of a water-excluding hump on a property driveway plus an appropriate freeboard. In addition a maximum width of flow should not be exceeded in the carriageway. Greater depths may be tolerated where a street is significantly lower than the land on both sides and in tropical areas with greater intensity rainfalls. A suitable freeboard should apply to floor levels of habitable rooms in properties adjoining the road.

- The product of depth and velocity ($d_g.V$), with V being the average velocity in the gutter, should not exceed 0.4 m^2/s for safety of pedestrians, 0.3 m^2/s for stability of parked vehicles (depending on size), or as directed by the consent authority (refer to Book 6, Chapter 7).

- Depths of stormwater flows should not exceed the height of the crown of the road during minor storms or where flows are to be contained on one side of a street. This includes locations that include ponding of stormwater such as at sag pits. Depending on the importance of the road (local, collector, arterial) and the importance of access, limits on width of flow of 2 to 2.5 m are typical.

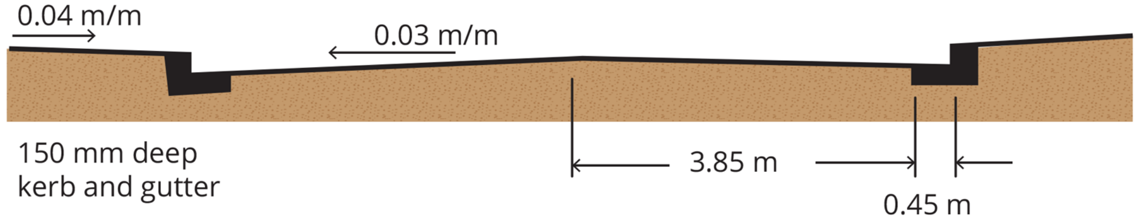
- Widths of flows may be limited to allow clear lanes in the centre of a road for passage of vehicles. Flow depths should not exceed the height of the crown of a road by more than 50 mm for major overland flow paths not considered part of the trunk drainage system and in new development areas.

Dimensions of Flow

The Manning Equation can be used to calculate flows in trapezoidal style overland flow paths. Sheet flow is commonly estimated using a version of the kinematic wave equation for flow distances up to 130 m and then sheet flows are then concentrated into some form of gully or defined overland flow path (NHI, 2013).

Equations for road gutters can be extended to calculate flows along full road cross-sections during major events. For a given flow rate, the normal depth corresponding to steady, established flow can be found by simple iterative calculations using a friction formula such as the Manning Equation. Although these assumptions may not be entirely valid, the errors involved may be generally acceptable.

Design charts of the flow capacities of roadway cross-sections can be prepared using Equation (9.5.8). Allowable zones are defined by various limiting conditions and criteria as shown by the example in Figure 9.5.19.



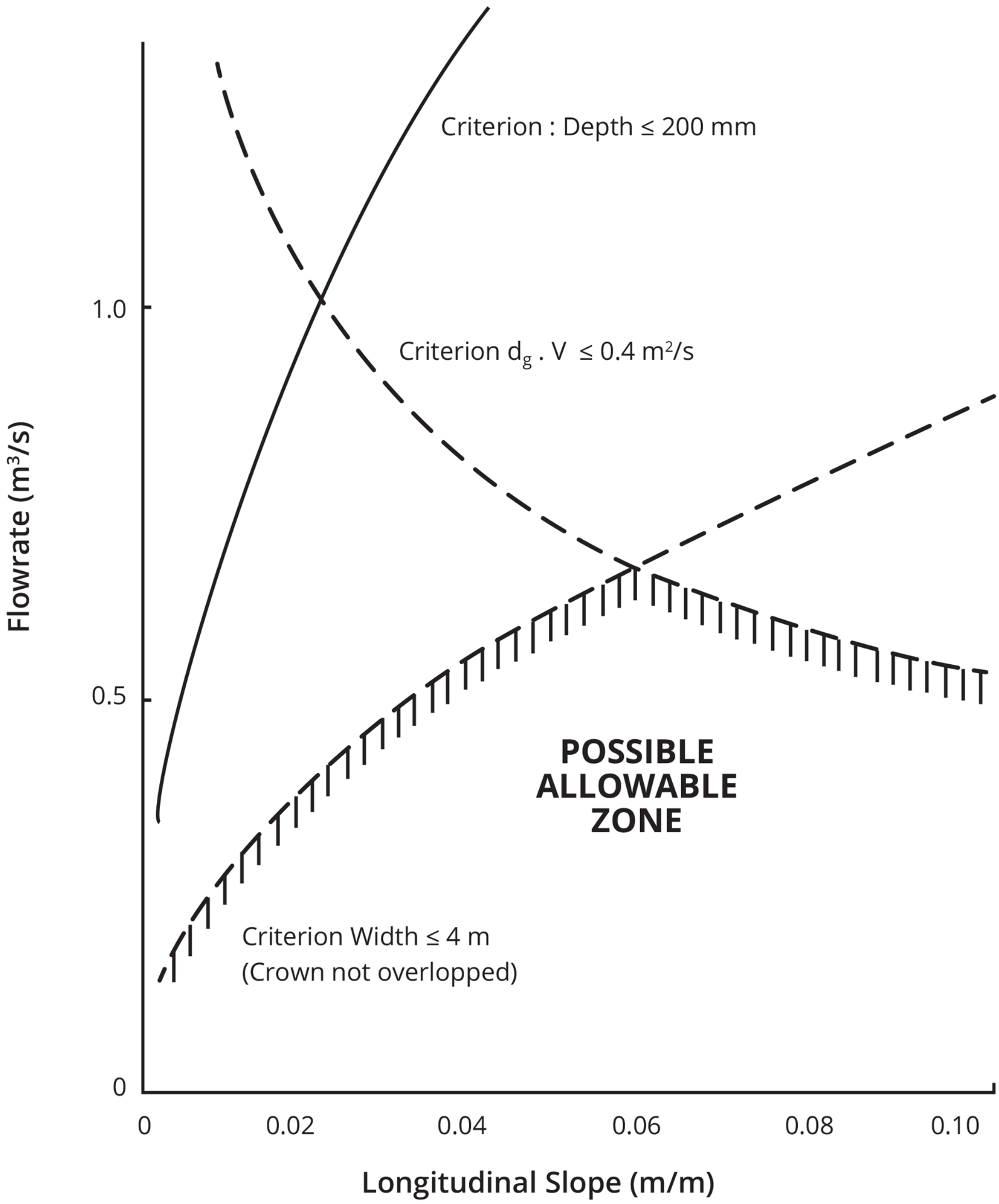


Figure 9.5.19. Flow Capacity Chart for One Side of an 8 m Carriageway with 3% Cross-fall

Gutter and Roadway Flow Equation

The following general equation developed by the U.S. Bureau of Public Roads (Searcy, 1969) is recommended to determine flows in streets. With reference to Figure 9.5.20(a), the equation is:

$$Q = Q_{ABC} - Q_{DBF} - Q_{DEF} - Q_{GEH} = 0.375F \left[\left(Z_g/n_g\right).\left(d_g^{8/3} - d_p^{8/3}\right) + \left(Z_p/n_p\right).\left(d_p^{8/3} - d_c^{8/3}\right)\right].S_0^{1/2} \quad (9.5.8)$$

where Q (m^3/s) is the total flow rate which is estimated by dividing the section as shown in Figure 9.5.20(a) and applying the equation by Izzard (1946) for a triangular channel with a single cross-fall:

$$Q = 0.375Fd^{8/3}S_0^{1/2}Z/n \quad (9.5.9)$$

Where:

F is a flow correction factor,

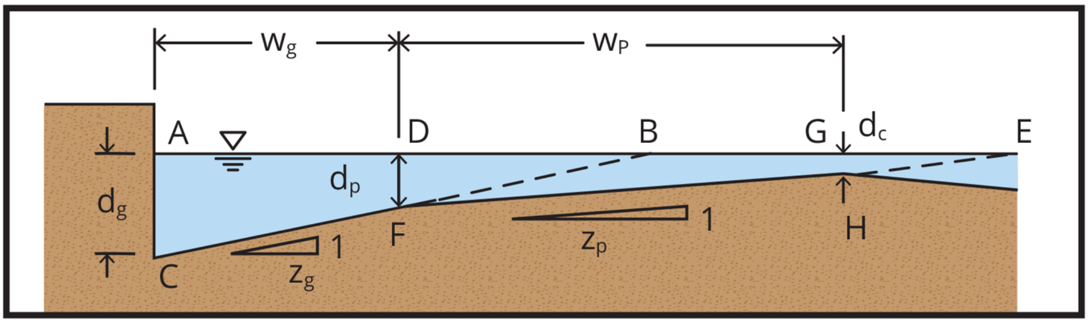
Z_g and Z_p are the reciprocals of the gutter and pavement cross-slopes (m/m),

n_g and n_p are the corresponding Manning's roughness coefficients,

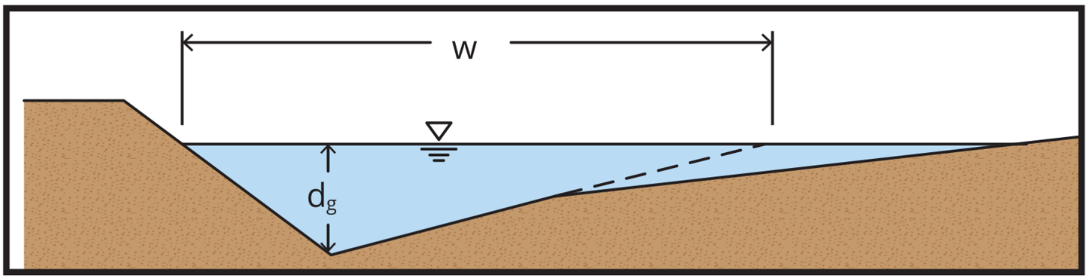
d_g and d_p are the greatest gutter and pavement depths (m),

d_c is the depth of water on the road crown, and

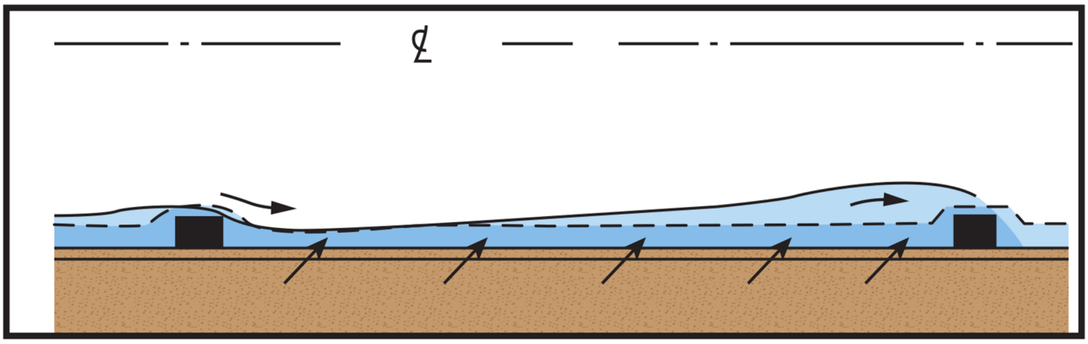
S_o is the longitudinal slope (m/m).



(a) Gutter and Roadway Profile with Vertical Kerb



(b) Gutter Profile with Sloping Kerb Face



(c) Plan View Showing Flow Spread

Figure 9.5.20. Gutter Flow Characteristics.

Equation (9.5.8) can be applied in simplified form when flows are contained in a gutter or on one side of a road. Clarke et al. (1981) estimated values for F of about 0.9 for simple triangular channels and 0.8 for gutter sections of the type shown in Figure 9.5.20(a). These assumptions may be used in the absence of more precise information. Typical values of Manning's n are 0.012 for concrete, 0.014 for asphalt, 0.018 for flush seal and 0.025 for stone pitchers (Dowd et al., 1980).

Consider the face of the kerb to be vertical in situations where the face of a kerb is relatively steep. Equation (9.5.8) can be applied to “lay-back” kerbs with sloping faces by assuming that zg is equal to w/dg as defined in Figure 9.5.20(b).

Open channel flow equations, such as the Manning Equation, can also be used to determine flows in lined gutters or unlined drains or swales.

Flow depths and widths for a specified flow rate can be determined using Equation (9.5.8). Velocities are estimated by dividing the flow rate by the corresponding flow area. Travel times for stormwater conveyance can be derived by dividing gutter length by flow velocity. Distributed lateral inflows as shown in Figure 9.5.20(c) can generate flow rates and characteristics such as width, depth and velocity that vary along a gutter. In this situation the average flow velocity occurs at about 60% of the distance along the gutter towards the inlet structure. Gutter flow calculations that use of the total flow arriving an inlet structure will overestimate velocities impacting on the structure.

Other Considerations

Gutter flow times depend on flow rates and it is necessary to specify a time in order to estimate a flow rate. A set of iterative calculations are required. In these calculations, a velocity or time is assumed, and a flow rate calculated. Then a check is undertaken to determine whether the total time of flow in the overland and gutter flow paths agrees with the original assumption.

A precise calculation of gutter flows must allow for concentrated inflows such as bypass flows from an upstream pit at the upper end of the gutter or an outflow from a large site at some point along the gutter. A representative design flow rate must be estimated to permit calculation of the average velocity and travel time.

Parked vehicles and driveways may interrupt and widen surface flows. The limited experimental evidence available suggests that these effects are localised. Allowance for this effect may be needed for streets where close parking of vehicles is likely but specific allowance does not appear necessary at other locations. The design process should account for possible future alterations to gutter and road profiles including resurfacing of roads. Effects of possible pit blockages must be assessed at locations where overflows may cause significant damage.

Aquaplaning or hydroplaning is also an important consideration, especially for highway drainage. This occurs when the tyre's inability to shed water from the contact patch is exceeded, resulting in a layer of water building between the tyre and road surface leading to a loss of traction that prevents the vehicle from responding to control inputs. Although aquaplaning is dependent on other geometric factors of road design, adequate sizing and placement of inlet structures and cross culvert drainage systems is also a significant factor in reducing the risk of sheet flow occurring on roads. For guidance on aquaplaning, or highway drainage in general, refer to sources such as Austroads (Austroads, 2013), road transport authority guidelines for each state and territory, the FHWA or UK Highway Agency.

It is also important to consider the longevity of an overland flow path and this is especially relevant for flow paths through private property. Blockages are likely to occur due to lack of maintenance, or by post construction modifications such as from garden beds and mulch, or by modifications designed to enclose domestic pets.

It is often necessary to locate structures within minor overland flow paths including property fencing, sound-control barriers and above ground services. When designing overland flow paths that may contain these types of structures it is important to consider the potential for flows to be redirected by these barriers.

5.6.3. The Hydraulic Grade Line (HGL) and Energy Grade Line (EGL)

The hydraulic (HGL) and energy grade line (EGL) concepts are derived from the Bernoulli Equation and assist with the analysis of complex flow problems. The HGL is determined by plotting the relationship for pressure head P/γ and height above an arbitrary datum z at key locations in a conveyance network using the following equation:

$$HGL = P/\gamma + z \tag{9.5.10}$$

Where P is pressure and γ is specific density of water.

Similarly, the EGL adds the velocity head $V^2/2g$ to the HGL to provide a relationship for EGL that can be derived at key locations in the conveyance network:

$$EGL = V^2/2g + P/\gamma + z \tag{9.5.11}$$

Where V is the average velocity in a conduit and g is gravity.

The vertical distance to point (such as the centre of a conduit) below the HGL represents the pressure head or pressure energy at a point. Negative heads or partial vacuums may occur at siphons and the conduit is above the HGL. The HGL coincides with the water surface for open channel flows, except at points such as brinks of weirs where non-hydrostatic conditions prevail. Water rises to the level of the HGL in an inlet structure (pit) that acts as a vertical riser.

The EGL is located above the HGL and represents the total energy (velocity + pressure + potential) available to the flow that is expressed as a height (metres) equivalent to flow energy per unit weight in joules (or newton-metres) per newton.

Grade lines typically slope downwards in the direction of flow in conveyance networks and slope represents energy losses due to pipe friction. The HGL and EGL are parallel for steady flows. The grade lines generally have a different slope to the pipe in closed conduits under pressure (with the HGL above the pipe). The grade-lines are parallel to open channels that are subject to steady and uniform flows since the friction loss equals the potential energy loss represented by the slope of the conduit.

Changes in the shape or direction of conduits create turbulence and local losses that are represented as sharp drops in EGLs. Significant energy losses are typically assumed to act at the centre of inlet structures in analysis of conveyance networks. The HGL is also assumed to change at the centre of inlet structures as illustrated in Figure 9.5.21. These assumptions differ from the actual location of losses at the entry and exit of inlet structures (pits).

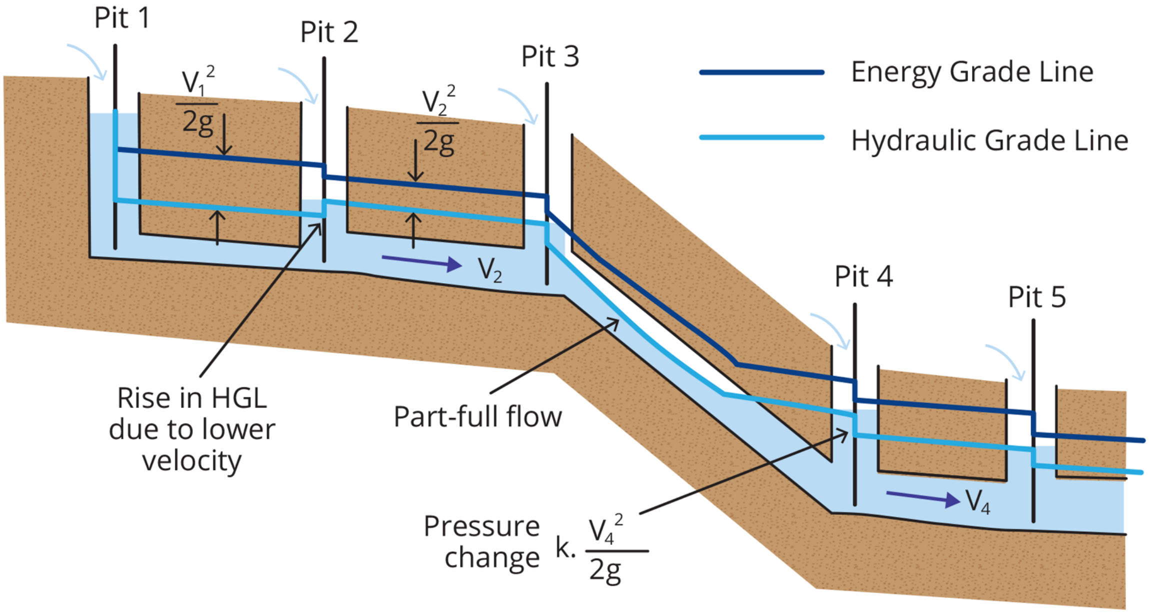


Figure 9.5.21. Flow Behaviour in a Surcharged Pipe System Showing Energy Grade Lines and Hydraulic Grade Lines

5.6.4. Flows Through Conveyance Networks

Local Losses

Changes in the shape or direction of conduits can create turbulence and local losses that are represented as sharp drops in the EGL. Losses occur at entrances and exits to pits, pipe bends, and at contractions, expansions, junctions, and valves in conveyance networks. Except for expansions and contractions of conduits, these losses have the following relationship:

$$h = k\,.\frac{V^2}{2g} \qquad (9.5.12)$$

where h is the loss in m, and k is a loss coefficient multiplied by the velocity head of the downstream flow.

The loss factor k is dependent on the geometry of entrances to a conduit. A square-edged entrance will usually have a factor of k_e = 0.5 and for a rounded entrance the factor is approximately 0.2. The factor at a pipe exit k_{exit} is usually 1.0 as it is assumed the entire kinetic energy of flows will be lost as the pipe discharges into a larger body of water or atmosphere.

The losses at bends depend on the radius of the bend and have a typical value of k_b = 0.5. Contractions in conduits (decreases in pipe diameter) are subject to low levels of losses with a typical factor k_c of 0.05. Expansions in conduits (increases in diameter) generate higher losses h_L that are dependent on the upstream V_u and downstream V_d velocities:

$$h_L = K_{\exp}\,.\frac{(V_u - V_d)^2}{2g} \qquad (9.5.13)$$

where k_{exp} is about 1.0 for abrupt pipe expansions.

Valves have variable loss factors which can become very large as the valve closes.

Full Flows in Conduits

The estimation of flow rates through conveyance networks that are flowing full is made by relating the available energy or head to the losses as expressed by the velocity head. The following calculation shows how flowrates can be determined from the available head and the assumed energy losses along a 300 mm pipe discharging from a reservoir as shown in Figure 9.5.22.

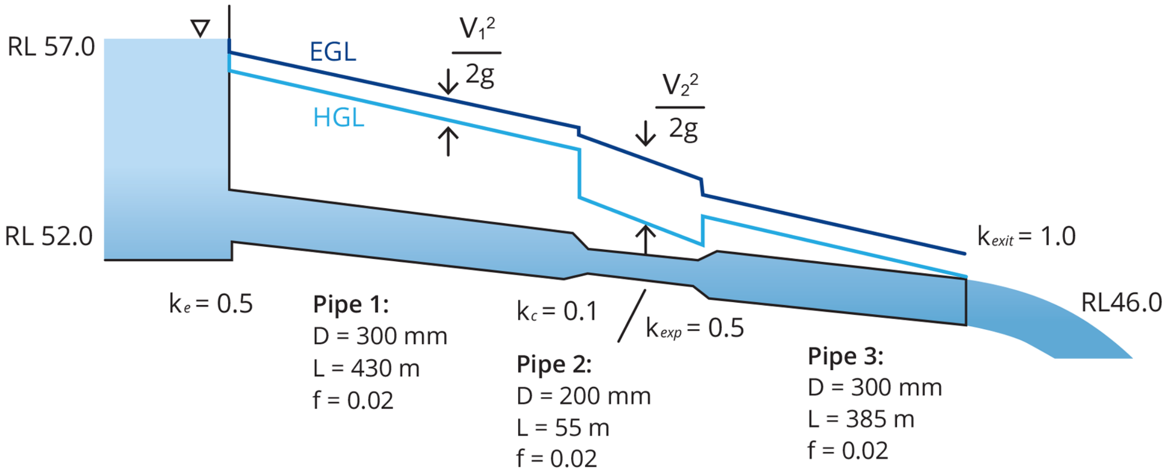


Figure 9.5.22. Example of Full Flows in a Pipe

The pipe diameter reduces from 300 mm to 200 mm at the middle of the pipe branch. The energy loss at the following expansion is assumed to be 0.5 times the velocity head in the downstream pipe and all friction values (f) in the Darcy-Weisbach Equation are set at 0.02.

The water level in the reservoir is 57.0 m above a height datum and the total head available is 57.0 - 46.0 = 11.0 m. The various losses are all functions of the velocity heads in the pipes. Since V_3 = V_1 and $V_2 = V_1 . \frac{A_1}{A_2} = V_1 . \left(\frac{D_1}{D_2}\right)^2$, the sum of the losses will be:

$$\left(k_e + \left(f . \frac{L}{D}\right)_1 + k_c + \left(f . \frac{L}{D}\right)_2 . \left(\frac{D_2}{D_1}\right)^4 + k_{\exp} + \left(f . \frac{L}{D}\right)_3 + k_{exit}\right)\frac{{V_1}^2}{2g} = 11 \text{ m} \quad (9.5.14)$$

$$\left(0.5 + \left(0.02.\frac{430}{0.3}\right) + 0.1 + \left(0.02.\frac{55}{0.2}\right).\left(\frac{0.3}{0.2}\right)^4 + 0.5 + \left(0.02.\frac{385}{0.3}\right) + 1.0\right)\frac{{V_1}^2}{2g} = 11 \text{ m} \quad (9.5.15)$$

Thus,

$$84.28 \cdot \frac{{V_1}^2}{2g} = 11 \text{ m},$$

$$\begin{aligned} V_1 &= \left(\frac{11.0 \times 19.60}{84.28}\right)^{0.5} \\ &= 1.60 \text{ m/s and} \\ Q &= V_1 A_1 \\ &= \pi/4 \cdot (0.3)^2 \cdot 1.60 \\ &= 0.113 \text{ m}^3/\text{s} \end{aligned} \tag{9.5.16}$$

The Manning Equation can also be used with friction losses expressed by $\left(2g\, n^2 \frac{L}{R^{4/3}}\right) \text{x} \frac{V^2}{2g}$ since slope of the energy gradeline is S = h^f/L.

Equations using conservation of mass, energy and momentum can be constructed to describe the state of an entire conveyance network that includes multiple pipes and inlet structures. These equations are solved to provide information about the pressures and velocities throughout a conveyance network which can be visualised as EGLs and HGLs. More complex partial differential equations are required to cope with unsteady flows that change with time.

Conduits Flowing Partially Full

Conduits that are flowing partially full in stormwater conveyance networks can exhibit complex behaviours. A maximum flow capacity is achieved when conduits are operating at less than full flows. However it is not good practice to design conduits with partial flows as disturbances may eliminate free surfaces in conduits and cause a transition to pressurized full flows that may lead to surcharges.

This assumes that flows in conduits are open channel flows with atmospheric pressure at the surface. Submergence at the entrance and tailwater levels affecting the outlet of conduits generates further complications. In addition, large air bubbles and air pockets can occur in conduits that operate in partially full conditions resulting in pressures that can be above or below atmospheric pressure. The theory of open channel hydraulics is addressed in Book 6.

Complex Procedures

A more complex and correct procedure for analysis of conveyance networks is to apply partial differential equations of unsteady flow varying in space (the distance along a conduit) x and time t that is defined as steps or intervals. These numerical models divide river, channel or pipe reaches into segments and define the transfer of mass and momentum between adjacent segments using the *Saint Venant Equations* for conservation of mass and momentum in unsteady flows as described in Book 6, Chapter 2. The equations must be solved iteratively using finite difference or finite element models and matrix calculations that may require longer computing times.

These more complex calculation processes are quite different from water surface projection methods such as the 'standard step' procedure. Nevertheless the same outputs are produced such the HGL levels at points along a conduit and at different times during a flow event. The equations allow for pipe friction and local losses, and also incorporate pressure changes at inlet pits and junctions.

Modelling of urban conveyance networks is typically carried out using a range of computer software packages that provide different levels of rigour or precision which involve trade-offs between speed and accuracy. However the designer should be aware of other important considerations such as stability. Unlikely high or low pressures, water levels, and flow rates are generated when iterative calculations become unstable. The usual way of achieving stable results with a computer model is to choose a shorter time step or adjustment of factors affecting the relative time steps in space and time. Small errors in volumes or flows (typically < 1%) can be accepted in order to achieve faster running times.

Priessmann Slot

Methods of analysis must allow for flows that change from partially full to full conduit flows and back again. Modelling procedures that account for unsteady flow regimes employ the Priessmann Slot assumption. This mixed flow problem is simplified by the addition of a hypothetical slot in the pipe which allows the depth of flow to exceed the pipe diameter and provides pressurized flow effects (Yen, 1986; Butler, 2004). The width of the slot must not be too wide to significantly impact on continuity and should be determined to ensure that the gravity wave speed equals the pressure wave speed.

The hypothetical slot allows the analysis of the conveyance network to be treated as an open channel flow problem. However, a limitation of this approach is that it cannot accurately simulate the formation and impact of air pockets or negative pressures results from shocks.

Outlet Structures

Regardless of whether flow within the conduit is full or part full, suitable transition is required at the end of a conveyance conduit, where flow discharges to the receiving environment. The transition structure, or outlet structure should accommodate potential for high velocity and/or turbulent flow. This can be achieved through armouring of the surface using material such as rock or concrete, along with gradual transition of geometry from that of the conduit, to that of the receiving channel or basin. Energy dissipation and/or flow dispersion can be achieved at the same time using appropriate outlet structure design. This is particularly necessary where stormwater settlement processes are expected in a receiving basin structure such as a bio-retention basin or constructed wetland.

The outlet structure may also represent an opportunity for removal of gross-pollutants prior to stormwater passing into a receiving channel or structure. This can be achieved using various forms of screens, baskets and mechanical filters. The impact of these structures, whether clear, partially blocked or fully blocked on the hydraulic performance of the conveyance infrastructure needs careful assessment at the design stage.

5.6.5. Culverts

The simplest conveyance network is a single-pipe culvert which is a common component of highway and railway networks that is located wherever an embankment crosses a waterway or drainage path. These transport crossings may only involve a single pipe (or multiple parallel pipes). However, the hydraulic calculations can be complicated. Culvert hydraulics are comprehensively described by Normann et al. (2005). The treatment of culvert hydraulics (or headwalls) is divided by two flow conditions:

1. Inlet controls – dependent on the orifice effect at the culvert entrance; and

2. Outlet controls – dependent on full, pressurised flow conditions through the pipe or on high tailwater levels.

Multiple culverts may be connected by pits or junctions in a similar manner as a large conveyance network (refer to Book 9, Chapter 5, Section 6)

Inlet Control

Inlet conditions for culverts are created by the vena contracta effects shown in Figure 9.5.23.

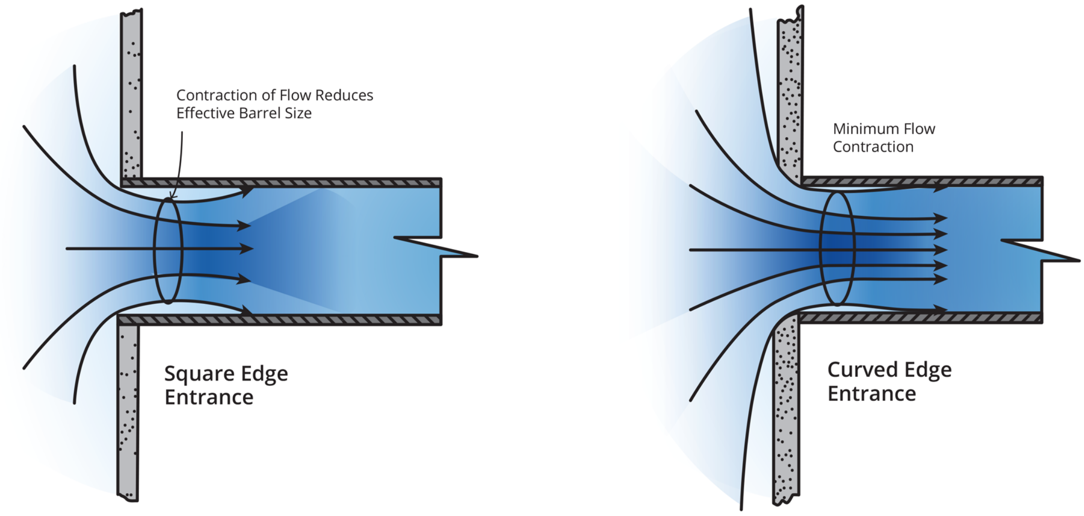


Figure 9.5.23. Vena Contracta or Contraction at a Culvert Entrance

The streamlines of flows entering a culvert cannot turn abruptly and the curvature of flows continues into the culvert creating a jet with a diameter less than that of the culvert. This process reduces the available cross-sectional area of flows and the overall flow rate. The ratio between the jet and the pipe diameters is 0.6 for a square-edged entrance. Values for other entrance types are shown in Figure 9.5.24.

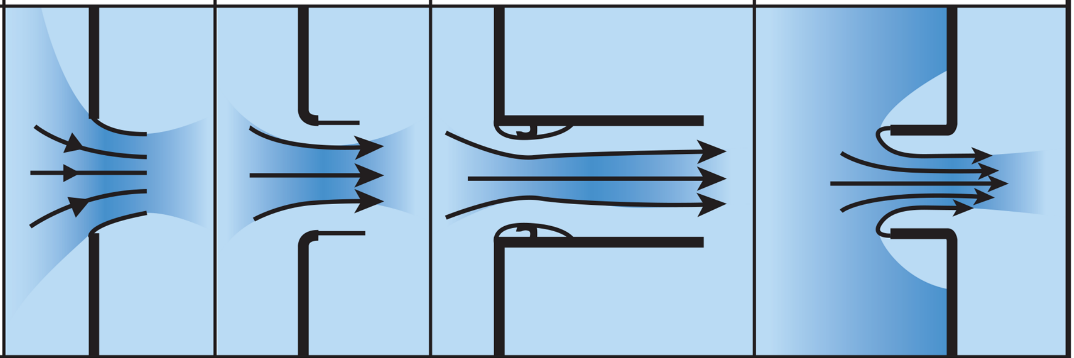
Orifices and their Nominal Coefficients				
	Sharp edged	Rounded	Short tube	Borda
				
C	0.61	0.98	0.80	0.51
C_c	0.62	1.00	1.00	0.52
C_u	0.98	0.98	0.80	0.98

Figure 9.5.24. Orifice Coefficients (Vennard and Street, 1982)

The correction coefficient for the reduced area is C_c and C_u is the factor for the velocity being less that the theoretical value of $V = \sqrt{2gh}$ where h is the pressure head on the orifice (m) and g is the acceleration due to gravity (m/s^2). The overall correction coefficient is $C = C_c.C_u$.

The general case of inlet control is presented in Figure 9.5.25 where it is observed that the culvert barrel has a greater capacity than the entrance as it is flowing partially full. As indicated, Figure 9.5.24 shows that the capacity of the culvert can be improved by modifying the entrance by rounding sharp edges and changing the streamlines. These improvements may be useful in situations when additional capacity is required.

The general equation governing orifice flow for a circular pipe is:

$$Q = VA = C\left(\frac{\pi D^2}{4}\right)(2gh)^{0.5} \qquad (9.5.17)$$

where C is the correction factor (dimensionless),

D is the pipe diameter (mm),

h is the head on the orifice, usually taken from the upstream water surface to the centre of the orifice (m), and

g is gravitational acceleration (9.81 m/s^2).

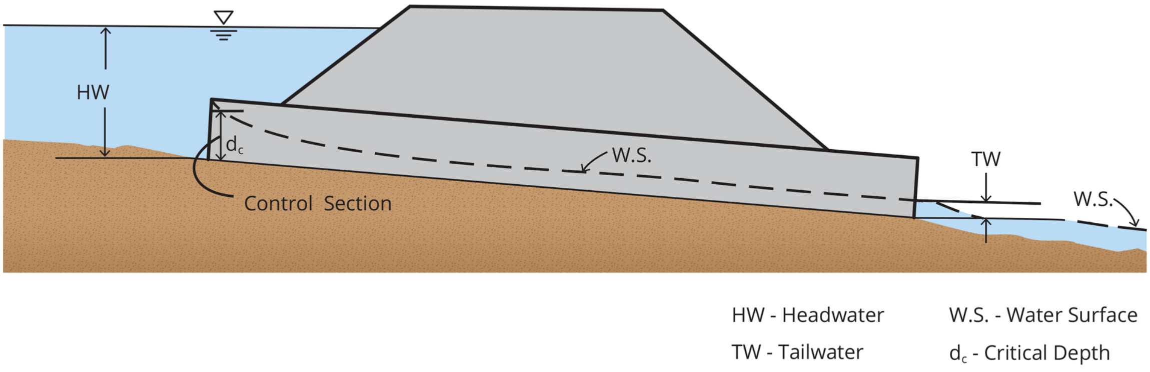


Figure 9.5.25. Example of Inlet Control (U.S. Department of Transport, 2005)

The hydraulics is more complicated when the entrance to the culvert is not completely submerged. This may involve three different states depending on the headwater height above the invert HW and the culvert diameter or height D:

- Partially full flow for HW < 0.8D is a weir type flow as water pours into the pipe;
- Partially full flow with 0.8 < HW < 1.2D is similar to weir flow; and
- Fully submerged inlet flow for HW > 1.2D is an orifice flow.

The stated limits of 0.8D and 1.2D are approximate. These three zones lead to the behaviour demonstrated in Figure 9.5.26 where the inlet control relationship changes depending on the headwater elevation. It is also possible to have two different flow rates at the same water elevation which depends on whether the culvert is operating as an inlet or outlet controlled system. These states can also depend on whether flows are increasing or decreasing.

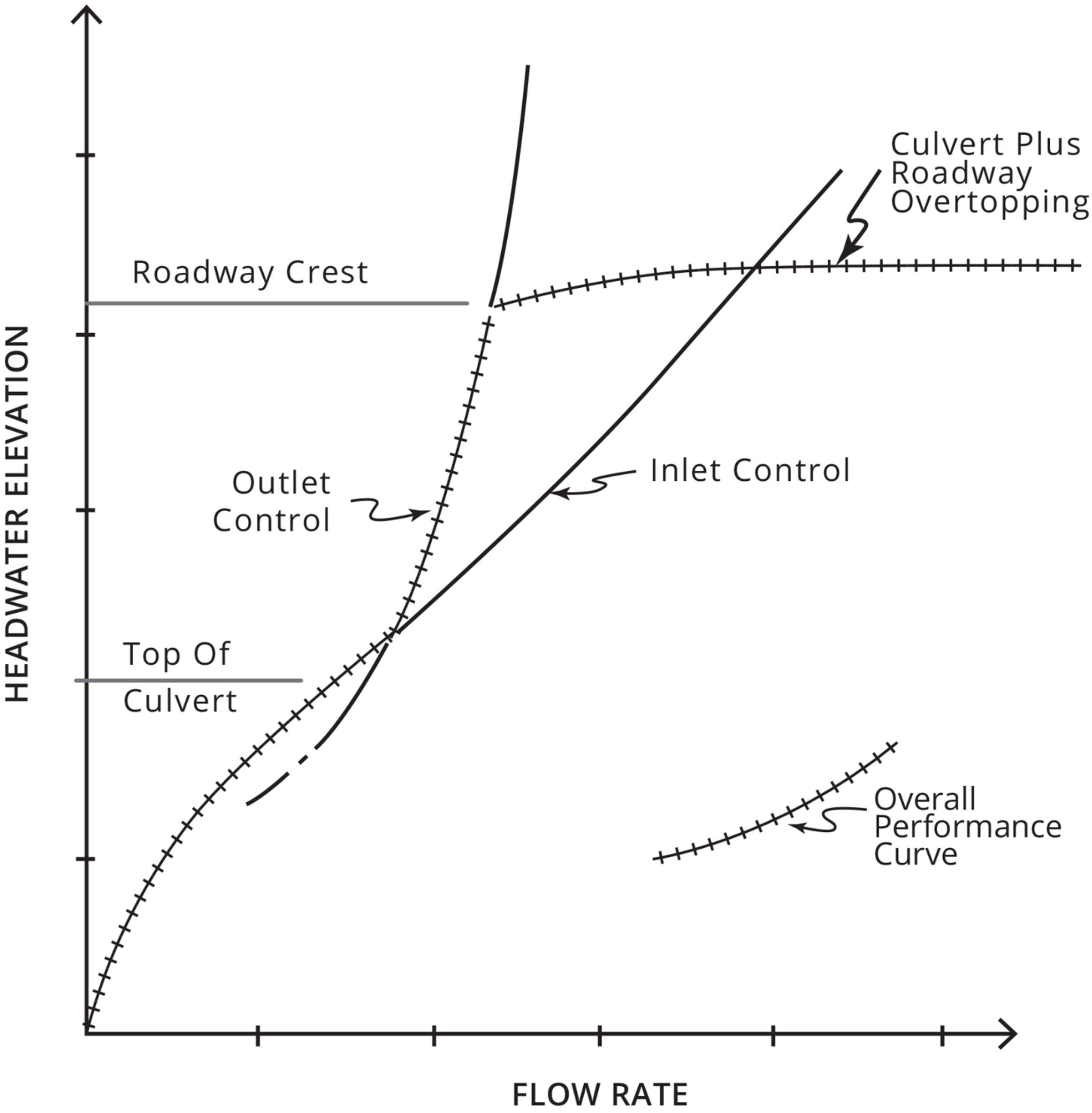


Figure 9.5.26. Inlet Control versus Elevation of Headwaters (U.S. Department of Transport, 2005)

A range of design aids are generally available in the form of nomographs used to calculate headwater levels for various situations involving circular, box and other types of culverts. A better approach is to use computer software to model culvert hydraulics.

Outlet Controls

Outlet control occurs when a culvert is not capable of conveying as much flow as the inlet can accept. The controlling section is generally at the culvert exit where subcritical or pressurised flow conditions are occurring or further downstream of the culvert due to tailwater conditions. Two outlet-controlled situations are provided in Figure 9.5.27. The difference between upstream headwater and the tailwater levels drives the flows through the culvert. Energy losses are added and equated to the available head.

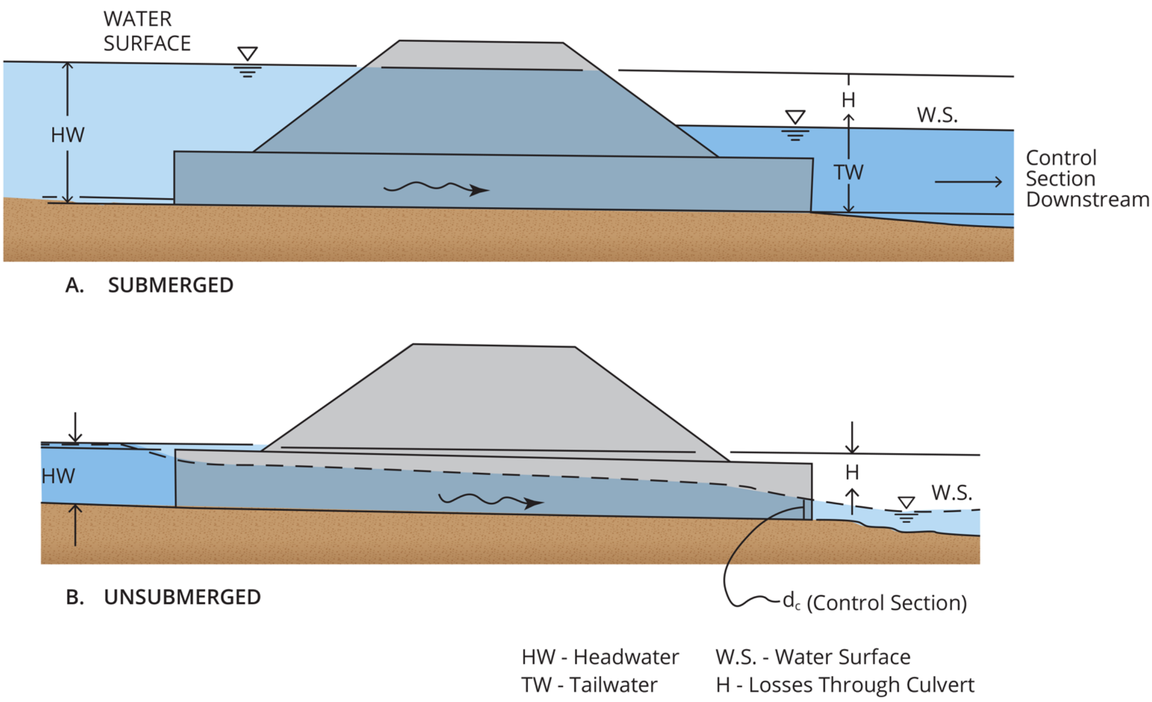


Figure 9.5.27. Example of Outlet Control Situations (U.S. Department of Transport, 2005)

These calculations involve backwards projection of the HGL that commences at the tailwater level if this submerges the outlet. Different computer models make various assumptions for free outfalls. It is assumed that the level will be half way between the pipe obvert and the critical depth, and it is necessary to determine that critical depth from nomographs or equations. However other computer models assume that it is the lower of (a) the critical depth and (b) the normal depth.

A weir equation is applied to allow for overtopping of road embankments:

$$Q = C_w L_w H^{1.5} \tag{9.5.18}$$

where C_w is a weir coefficient, depending on the weir shape (Figure 9.5.28),

L_w is the width or length of the weir perpendicular to the direction of flow, and

h is the height of water above the weir crest (m).

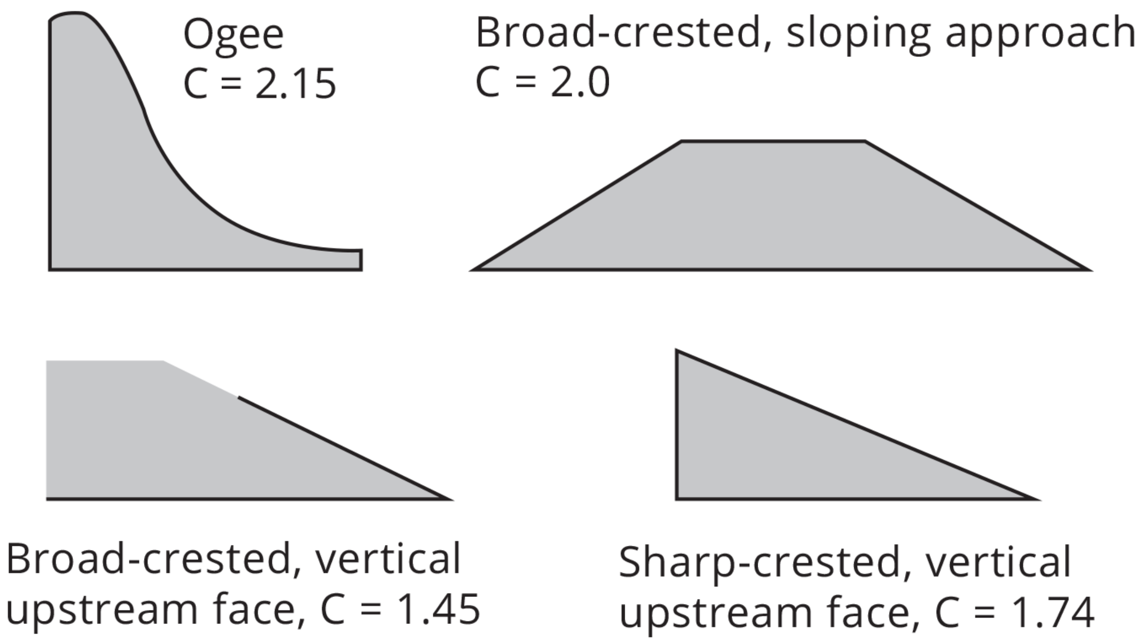


Figure 9.5.28. Shapes of Weir Crests (Laurenson et al., 2010)

Culvert and overflow weir outflows can be combined into a composite relationship as shown in Figure 9.5.26. This calculation should account for inlet and outlet controls and usually the most conservative relationship that provides the lowest flow rate for a given depth is accepted.

The real behaviour of a culvert is more complex and involves a phenomenon called 'priming'. As upstream water levels rise, culverts tend to remain under inlet control until they run full. As upstream water levels decline, culverts tend to remain at full flows in an outlet control configuration until there is a sudden reversion to inlet control and decline in headwater level.

Since culverts are often used as outlets for detention basins and conveyance networks. The relationships presented above can be applied to specify the elevation and discharge relationships needed for routing of flows through volume management facilities.

5.7. References

Argue, J.R. (1986), Storm drainage design in small urban catchments: a handbook for Australian practice, Special Report No.34, Australian Road Research Board.

Australian Road Research Board (1979), The hydraulic capacity of a side entry pit incorporating a deflector and grate, Pavement Surface Drainage Symposium, Sydney, Australia

Austroads (2013), Guide to Road Design Part 5A: Drainage-Road Surface, Networks, Basins and Subsurface, Austroads

Burgi, P.H. And Gober, D.E. (1977), Bicycle-safe grate inlets study, United States Federal Highway Administration, Environmental Division, 77(24), The Division.

Butler, D., Davies. J.W. (2004), Urban Drainage, Ed.2, London: Taylor & Francis.

Chang and Kilgore (1989), HYDRAIN V5.0 Federal Highway Administration's Research Report.

Chang, F.M., Kilgore, R.T., Woo, D.C. and Mistichelli, M.P. (1994), Energy losses through junction manholes, Report No. FHWA-RD-94-080, Federal Highway Administration.

Clarke, W.P., Strods, P.J. and Argue, J.R. (1981), Gutter/pavement flow relationships for roadway channels of moderate or steep grade, Proceedings of the First National Local Government Engineering Conference, Adelaide, pp: 130-137.

Coombes P.J. (2015), Transitioning drainage into urban water cycle management, Proceedings of the WSUD 2015 Conference, Engineers Australia, Sydney.

Coombes P.J., Babister M. and McAlister, (2015), Is the Science and Data underpinning the Rational Method Robust for use in Evolving Urban Catchments, Proceedings of the 36th Hydrology and Water Resources Symposium, Engineers Australia, Hobart.

Department of Transport, Queensland (1992), Queensland Urban Drainage Manual, Edition 1, September 1992.

Dowd, B.P., Loakim, R. And Argue J.R. (1980), The simulation of gutter/pavement flows on South Australian urban roads, ARRB Proceedings, 10(2), 145-152.

GKY and Associates Inc, (1999), User's Manual for HYDRAIN, Integrated Drainage Design Computer System: Version 6.1, Federal Highway Administration, US Department of Transportation, Washington D.C., accessible at <www.fhwa.dot.gov/bridge/hyd.htm>

Hare, C.M. (1980), Energy losses and pressure head changes at storm drain junctions, Thesis, University of Technology, Sydney.

Hare, C.M. (1983), Magnitude of Hydraulic Losses at Junctions in Piped Drainage Systems, Civil Engineering Transactions, Institution of Engineers, CE 25 (1).

Hare, C.M., O'Loughlin, G.G. and Saul, A.J. (1990), Hydraulic Losses at Manholes in Piped Drainage Systems, Proc. International Symposium on Urban Planning and Stormwater Management, Kuala Lumpur.

Hare, C.M., O'Loughlin, G.G. and Saul, A.J. (1990), Hydraulic Losses at Manholes in Piped Drainage Systems, Proceedings of the International Symposium on Urban Planning and Stormwater Management, Kuala Lumpur.

Izzard, C.F. (1946), Hydraulics of runoff from developed surfaces. In Proceedings of the 26th Annual meeting of the Highway Research Board, National Research Council, Washington, DC EEUUA, pp: 129-150.

Jens, S.W. (1979), Design of urban highway drainage, Publication No. TS-79-225, Federal Highway Administration, Washington, D.C.

Laurenson, E.M., Mein, R.G. and Nathan, R. (2010), RORB - Version 6 Runoff Routing Program, User Manual, Monash University Department of Civil Engineering and Sinclair Knight Merz

Marsalek, J. (1982), Road and bridge deck drainage systems, Report RR228, Research and Development Branch, Downsview: Ontario Ministry of Transportation and Communications.

Mills, S.J. and O'Loughlin, G.G. (1986) Workshop on Piped Urban Drainage Systems, Swinburne Institute of Technology and University of Technology, Sydney (first version 1982, latest 1998)

Mills, S.J. and O'Loughlin, G. (1998), Workshop on urban piped drainage systems, Swinburne University of Technology and University of Technology, Sydney.

NHI (2013), Urban Drainage Design Manual, HEC-22, ed. 3, National Highway Institute, U.S. Department of Transportation, Federal Highway Administration.

Department of Main Roads NSW (1979), Model analysis to determine hydraulic capacities of kerb inlets and gully pit gratings, NSW: Department of Main Roads.

National Capital Development Commission (1981), Waters of the Canberra Region, Technical Paper 30, National Capital Development Commission.

New South Wales, Department of Housing (1987), Road Manual, Sydney.

Normann, J.M., Houghtalen, R.J. and Johnston, W.J. (2005), Hydraulic Design of Highway Culverts, U.S. Department of Transportation, Federal Highway Administration, FHWA-NHI-01-020, Hydraulic Design Series No.5, Ed.2.

O'Loughlin, G. and Stack, B. (2002), Algorithms for pit pressure changes and head losses in stormwater drainage systems, Global Solutions for Urban Drainage, pp: 1-16.

Parsell, R. (1992), Generalised Equations for Estimating Pressure Change Coefficients at Stormwater Pit Junctions, Proceedings of the International Conference on Urban Stormwater Management, Institution of Engineers, Australia, Sydney.

Pilgrim, DH (ed) (1987) Australian Rainfall and Runoff - A Guide to Flood Estimation, Institution of Engineers, Australia, Barton, ACT, 1987.

QUDM (2013), Queensland Urban Drainage Manual. Department of Energy, Water and Energy Supply, Queensland Government.

Sangster, W.M., Wood, H.W., Smerdon, E.T. and Bossy, H.G. (1958), Pressure Changes at Storm Drain Junctions, Engineering Series Bulletin No. 41, Engineering Experiment Station, University of Missouri.

Searcy, J.K. (1969), Drainage of highway pavements, U.S. Department of Transportation, Washington D.C.

Stein, S.M., Dou, X., Umbrell, E.R. and Jones, J.S. (1999), Storm Sewer Junction Hydraulics and Sediment Transport, US Federal Highway Administration Turner-Fairbanks Highway Research Centre, VA

U.S. Department of Transport, Federal Highway Administration (2005), Hydraulic Design Series Number 5 - Hydraulic Design of Highway Culverts.

Vennard, J.K. and Street, R.L. (1982), Elementary Fluid Mechanics, 6th Edition, Wiley, New York.

Victoria Country Roads Board (1982), Road design manual. Chapter 6., Drainage. Kew, Victoria.

Walsh C.J. (2004), Protection of in-stream biota from urban impacts: minimise catchment imperviousness or improve drainage design?, Marine and Freshwater Research, 55: 317-326.

Walsh, C.J., Booth, D.B., Burns, M.J., Fletcher, T.D., Hale, R.L., Hoang, L.N., Livingston, G., Rippy, M.A., Roy, A.H., Scoggins, M. and Wallace, A. (2016), Principles for urban stormwater management to protect stream ecosystems, Freshwater Sci., 35: 398–411.

Weeks B., Witheridge G., Rigby E., Barthelmess A. and O'Loughlin G. (2013), Blockage of hydraulic structures, Australian Rainfall and Runoff revision Project 11, Stage 2 Report, Engineers Australia.

Yen, B.C. (1986), Hydraulics of sewers, Advances in Hydroscience, Urban Storm Drainage, 14: 1–115.

Chapter 6. Modelling Approaches

Peter Coombes, Steve Roso, Mark Babister

The authors collaborated with Mikayla Ward and Sophia Buchanan to produce the Brownfield and Greenfield case studies.

Chapter Status	Final
Date last updated	14/5/2019
Minor edits	27/08/2024

6.1. Introduction

Urban stormwater management responds to an increasing number of performance objectives including to mitigate property damage, avoid risks to human life, enhance the amenity of urban settlements, and protect surrounding environments (refer to Book 9, Chapter 3, Section 3 and Book 9, Chapter 5, Section 2). This involves consideration of the full spectrum of rain events, from frequent to rare (refer to Book 9, Chapter 3), from the perspective of flooding, water quality, provision of infrastructure, protection of environments, and enhancing amenity of urban areas. The assessment of urban stormwater behaviour, performance against objectives and associated design tasks, involves complex analytical problems that are better resolved using a computer-based model system.

A computer model involves use of software or a complex spreadsheet. Compared with hand calculation, computer models permit rapid numerical calculation across large spatial and temporal domains, while facilitating testing of multiple suites of parameters and inputs (refer Book 9, Chapter 3, Section 4). This in turn allows the model to be calibrated to best represent the real world conditions that are under assessment. Models can be a useful tool to assist our thinking, and can be readily documented and reviewed, ultimately leading to better assessments and design outcomes.

Reliable estimates are nevertheless conditional upon best practice application of the computer model. It is important to remember that models are only tools to guide our thinking about design and management. The purpose of this chapter is to provide guidance on the selection and application of modelling approaches within urban catchments, having regard to the techniques described in other books of ARR. The chapter is structured as follows:

- Book 9, Chapter 6, Section 2 describes tasks that are characteristic of urban modelling.
- Book 9, Chapter 6, Section 3 discusses current trends in urban modelling. This may assist with planning a long-term strategy for technology adoption, research, and training. A general description of the types of computer models commonly applied in urban stormwater practice is provided as an aid for model selection.
- Book 9, Chapter 6, Section 4 provides a framework for application of computer models to urban stormwater catchments. This discussion includes guidance for each segment of the catchment, from the watershed, through the urban stormwater network, and into the receiving waterway.

This chapter is not intended to duplicate content in other chapters. Where relevant detail is available elsewhere, references to other books and chapters are provided.

In the context of this chapter, an 'urban model' can be defined as a conceptual or computer-based modelling system that performs hydrologic, hydraulic, water balance, or water quality calculations, across a catchment significantly disturbed by urban development and associated infrastructure. This modelling system may operate across all the significant scales of urban areas from allotment to neighbourhood to precinct to region. Urban infrastructure of most direct relevance includes increased impervious surfaces, modification to natural conveyance areas (e.g. pits, pipes, and open channels), and volume management infrastructure (e.g. rainwater tanks, bioretention, and basins).

Emerging urban stormwater analysis and solutions are based on a systems approach that incorporates multiple linked scales (Book 9, Chapter 3). The USEPA (2008) highlights that past practice of designing individual items of stormwater infrastructure at a single centralised scale has been inadequate for managing urban flooding and water quality in waterways. Stormwater management needs to be designed as a system that integrates structural and non-structural attributes of design with site characteristics and performance objectives.

More recently, the USEPA (2008) established that green infrastructure solutions distributed at multiple scales throughout urban catchments partially disconnected impervious surfaces. They also contributed to improved stormwater quality and avoided flood damages (Atkins, 2015). These insights are consistent with earlier Australian applied research finding that both the peak flows and volumes of stormwater runoff are required for the design of stormwater infrastructure (Goyen, 1981), and the local scale was the basic building block of cumulative urban rainfall runoff processes (Goyen, 2000).

Many methods for modelling stormwater runoff are based on regional scale assumptions and processes. However, inclusion of local scale processes in analysis improves knowledge of within catchment outcomes and whole of catchment responses.

It is suggested that a catchment with less than 10 percent impervious surfaces, or with less than 10 percent of the natural conveyance areas modified, would not be considered an 'urban catchment'. In which case, the advice in this chapter may have less relevance. However, each catchment is different, some natural or rural catchments contain sub-catchments that are urbanised (for example, in semi-urban areas). The relevance of this chapter to a specific modelling investigation needs to be determined by the reader through application of judgement and experience.

6.2. Urban Modelling Tasks

Typical urban modelling tasks are introduced in this section to establish context for subsequent discussion. In particular this section focusses on those modelling tasks that are not typically required when modelling rural and natural catchments. This section should be read in conjunction with Book 5, Book 6 and Book 7 where the reader can find information about modelling tasks and assumptions that are common across all catchment types (i.e. urban, semi-rural, rural, and natural).

There are some important differences between modelling of urban catchments compared with modelling other types of catchments. Urban areas can include:

- A larger proportion of impervious surfaces (refer Book 9, Chapter 6, Section 2).
- Stormwater conveyance infrastructure. This includes a network of inlet structures and non-natural flow paths that provide for greater concentrations and velocities of flow (refer Book 9, Chapter 6, Section 2).

- Numerous hydraulic structures. This includes infrastructure for waterway crossings, temporary storage of volume, water harvesting and treatment of runoff (refer Book 9, Chapter 6, Section 2, Book 9, Chapter 6, Section 2 and Book 9, Chapter 6, Section 2).

- A greater variety of land uses at different scales with different connectivity to catchment outlets (refer Book 9, Chapter 6, Section 4).

The density of land uses and associated infrastructure within an urban catchment also changes with time. The urban modelling process must therefore consider the information needs of the stakeholder and ensure the temporal scenarios being modelled are relevant.

There are also differences relating to the availability and use of model input data. Modelling in urban areas has intensive requirements related to representation of urban form, land uses, and stormwater infrastructure. Therefore, collection and collation of input data can become a significant component of the overall urban modelling task (refer Book 9, Chapter 6, Section 2).

6.2.1. Impervious Surface Estimation

One of the defining characteristics of an urban catchment is the presence of impervious surfaces such as roads, buildings, footpaths, and driveways. These surfaces have an associated reduced infiltration loss and decreased lag in hydrologic response in comparison to pervious surfaces (i.e. landscaping, lawns, open space) or natural catchments. Book 4, Chapter 2, Section 7 provides further discussion of the effects of impervious cover on runoff from urban areas.

Hydrologic modelling of urban areas requires an estimate of the proportion of impervious surfaces across each catchment and sub-catchment to be modelled. As described in Book 5, there are two main types of impervious surfaces that exist within urban areas:

1. Impervious areas which are directly connected to the conveyance network or urban waterway – referred to as Directly Connected Impervious Areas (DCIA).

2. Impervious areas which are indirectly connected to the conveyance network, typically where impervious surface runoff flows over pervious surfaces before reaching the conveyance network (e.g. a roof that discharges onto a lawn). These are referred to as Indirectly Connected Impervious Areas (ICIA). Alternatively, the responses of these impervious surfaces are disconnected from sub-catchment outlets by volume management measures (refer Book 9, Chapter 4).

These two configurations of impervious surfaces provide different hydrologic responses with Directly Connected Impervious Areas contributing to runoff more quickly than Indirectly Connected Impervious Areas (refer Book 5, Chapter 3, Section 4).

For large urban catchments, isolating the separate hydrologic effects of these two types of impervious surfaces is challenging. Book 5, Chapter 3, Section 4 instead describes a concept referred to as Effective Impervious Area (EIA) that encompasses the combined hydrologic effect of both directly and indirectly connected impervious areas. The estimated EIA value for a catchment is calculated and then applied to hydrologic calculations using the adopted modelling software.

The approach described in Book 5, Chapter 3, Section 4 involves estimation of EIA via linear regression of site stream flow gauge and rainfall data. In situations where there is insufficient available data to allow this technique to be used, the ratio of EIA to Total Impervious Area

(TIA) has been established for a collection of gauged catchments that allows EIA to be estimated based on an estimate of TIA (Refer Book 5, Chapter 3, Section 4).

TIA is a measurable catchment feature that is typically estimated using GIS methods (refer Book 5, Chapter 3, Section 4). The selection of a technique for estimation of TIA will depend on catchment scale, data availability, accuracy requirements, and whether the catchment scenario being investigated relates to an existing or future condition.

From Book 5 the recommended ratio of EIA/TIA for the majority of urban catchments sits within the range of 50% and 70%. For example, if the TIA for an urban catchment was measured to be say 55% then the EIA for that same catchment would be somewhere between 27.5% and 38.5% of the total catchment area.

However, when the EIA approach is used, it is important that the characteristics of the catchment under investigation are compared to those of the catchments that have been used to establish the recommended EIA/TIA ratio. Different catchments have different stormwater management standards and land use patterns that may alter the overall degree of connectivity between impervious surfaces and the drainage network serving the catchment. Where there is higher connectivity, the EIA is also expected to be higher.

For some catchment investigations where there is strong connectivity between the impervious surfaces and the downstream drainage system, the measured TIA value may be the more suitable impervious surface value to be used for hydrologic modelling purposes. For example, the analysis of a sealed carpark surface, where the entire impervious area is directly connected to surface inlets, is more appropriately undertaken using a TIA estimate.

Also, where the scale of the catchment is small, for example an individual parcel of land or a small development site, the use of TIA values in conjunction with a sub-catchment definition that reflects actual stormwater connectivity may be more appropriate. To avoid over estimation, designers should only use TIA for small scale catchments when they are satisfied that all the impervious flow is directly connected. The effect of any volume management infrastructure should also be explicitly reflected in these model simulations.

Consideration also needs to be given to the overall need for accuracy when deriving estimates of impervious cover. The majority of techniques applied by designers typically under or over-estimate actual impervious cover by between 10 and 20 percent (Roso et al., 2006).

Predicted peak discharges and runoff volumes are sensitive to error in impervious cover when modelling low rainfall events with both event based and continuous simulation models. Roso et al. (2006) observed that a difference in impervious surfaces of +/- 10 percent from actual conditions, can result in typical errors of 13% in peak discharge and 25% in runoff volume. These errors decrease in situations where rainfall depths are higher and infiltration losses less significant.

It is also noted that where a catchment has significant impervious cover, the variability of runoff is reduced in comparison to a similar pervious catchment since infiltration losses have less influence.

Additional discussion of configurations of impervious surfaces is provided in Book 9, Chapter 6, Section 4. Local observations or information about connectivity of impervious surfaces should be applied in models wherever possible.

6.2.2. Conveyance Infrastructure

Urban areas typically contain a significant amount of stormwater conveyance infrastructure, including numerous stormwater inlet structures feeding a network of other conveyance infrastructure such as street gutters, pipes, open channels, roadways, and overland flow paths through open spaces. These are linked together to form a continuous and distributed network from source to receiving waterway (refer Book 9, Chapter 5).

While natural waterway conveyance is increasingly sought as a design objective for new urban areas, traditionally urban drainage systems have been designed to transfer runoff quickly and within a minimum corridor, often partly underground. This containment of flows within conduits that have artificial linings and unnatural slope leads to faster average flow velocity, greater volume, and significantly altered flood hydrographs compared to those from comparably sized rural and natural catchments.

In order to accurately represent the hydrologic and hydraulic behaviour of an urban area, the influence of conveyance infrastructure on routing and flow behaviour should be included within the adopted urban flood model. For most applications, the model should be capable of describing the effect of conveyance infrastructure on flow characteristics such as flow depth, velocity, direction, surface level, and the hydraulic grade line showing hydraulic losses including their position and size. Other important information includes the split of flow between the minor and major flow path, maximum flow widths in gutters, maximum allowable flow velocity in pipes, the location and direction of any diversions and breakouts, and the extent of property inundation.

The effect of conveyance infrastructure on these flood characteristics varies across the different types of urban flood models.

Some models reflect the performance of conveyance infrastructure explicitly, which requires that the designer input a detailed physical definition of conduits and their hydraulic characteristics. The typical data that must be collected and input to these models include:

- Conduit type;
- cross-sectional dimensions (e.g. pipe diameter, channel width and depth, profile);
- length;
- slope, or sufficient elevation data to allow slope to be calculated using length; and
- hydraulic parameters (e.g. mannings 'n', viscosity).

This information must be gathered for each relevant piece of conveyance infrastructure that is part of the network being investigated. In some cases, this data may be readily accessible in an asset database. In other circumstances this data may require collection via ground survey. When data cannot be obtained due to inaccessible structures, assumptions regarding the network geometry may be required.

A schematic representation of the overall conveyance network, including connectivity between inlets, conduits, and junctions is then constructed within the model.

These models can be data intensive, but they also have potential to provide detailed and accurate descriptions of flood behaviours.

Depending on the type of user interface and pre-processor associated with the adopted model software, some of these data requirements may be automatically harvested from other raw input data. For example, a three dimensional surface model may be used to establish roadside gutter profile and slope automatically. Even so, some information will be required such as in this case the plan position of the roadside gutter.

For large urban areas, particularly those that have become densely developed over a long period of time, the task of collecting and collating all the dimensions of all conveyance infrastructure can be a major undertaking. Further complexity and effort arises since each inlet structure has a potential hydrologic sub-catchment that must be defined and input to the model. It is also possible that the size of the sub-catchment may change as flow rate increases. An exception is a rainfall-on-grid model approach where sub-catchment definition may not be required, but even still, substantial effort is required to ensure each inlet structure is capturing a realistic amount of runoff.

In some cases the burden of this infrastructure definition task can be reduced through use of simplified models and assumptions that do not explicitly model the performance of all conveyance infrastructure items. For example, the capacity of underground drainage may be an assumed proportion of the total runoff hydrograph or in some cases totally ignored. This approach can be acceptable if the capacity of the underground system is small relative to the size of floods being investigated. In this case the model construction may instead focus on a more accurate definition of surface-based conveyance infrastructure and overland flow paths.

Other models can provide flood estimates using an even more implicit description of conveyance infrastructure. For example, rating curves and stage hydrographs may be used for selected locations in conjunction with run-off routing hydrologic estimates. In this case less physical data needs to be collected.

Any decision to simplify the description of conveyance infrastructure within a model needs to be made recognising the accuracy requirements of the investigation and the risks associated with any limitations that may be introduced. It is important that the impacts of simplifying models and associated assumptions are fully understood. This is further discussed in Book 9, Chapter 6, Section 3.

6.2.3. Waterway Crossings

Waterway crossings are urban infrastructure for the purpose of allowing access across a natural or man-made waterway. The most commonly encountered waterway crossings comprise of causeways, culverts and bridges that are constructed as part of a vehicular, rail or pedestrian transport system.

Waterway crossings can have considerable hydraulic impact for floods within the range where the crossing structure causes the cross-sectional area of the waterway to be substantially reduced. In these circumstances additional energy is required to pass flow through and/or over the structure causing increased pressure head upstream of the crossing (afflux). Afflux is flow dependant and will change across the range of potential flood discharges. This afflux can cause a significant storage volume to be engaged upstream of embankments which can therefore also heavily influence downstream flood behaviour.

A comprehensive description of hydraulic behaviour at waterway crossings and other hydraulic structures is found in Book 6, Chapter 3.

As well as causing afflux locally around the structure, the hydraulic behaviour associated with waterway crossings can also have an impact on:

- Floodplain storage and hydrograph attenuation;
- tail water levels for upstream drainage;
- cross-catchment diversion of flow; and
- bed scour and local stream morphology.

These impacts are not necessarily confined to those that are in the immediate vicinity of the investigation site or study area and may impact areas upstream or downstream. A comprehensive urban flood investigation should therefore consider the impact of each existing or proposed waterway crossing in the catchment (and adjoining catchment in the case of cross-catchment diversion) and whether they could have an impact on local flood behaviour.

Once the relevant waterway crossings have been identified, the urban modelling task is then to suitably define the crossing structure within the model. This will normally include the physical dimensions and shape of the waterway opening beneath the crossing deck and any obstruction caused by associated railings, embankments, and utility services. Models may also assist with identifying locations where bed shear stress increases are likely and the design of scour protection measures (refer to Book 6, Chapter 3).

Consideration also needs to be given to blockage potential of the overall structure and which blockage scenarios may be required in order to fully describe potential flood behaviour. Book 6, Chapter 6 provides further detail regarding blockage considerations.

As with conveyance infrastructure, some types of urban models may estimate the flood behaviour impacts of the waterway crossing in an implicit manner through use of rating curves and stage hydrographs. The impact of any such simplifications and assumptions on model accuracy needs to be considered when selecting an appropriate model platform for the investigation.

6.2.4. Volume Management Infrastructure

Volume management infrastructure comprises of discrete facilities, primarily for the purpose of controlling peak discharge and volume. They can be located at almost any point within a drainage network and are linked by conveyance infrastructure and/or natural waterways. A comprehensive description of typical volume management infrastructure facilities is found in Book 6, Chapter 4.

The hydrologic and hydraulic impact of these facilities can be significant and will vary according to the design of the facility and size of the flood. For the urban designer, the task associated with this infrastructure is the physical description and schematisation of the facility within the model. This will normally include:

- Storage characteristics and how the volume stored varies with depth; and
- outlet characteristics and how the outlet influence depth and volume of water stored in the facility

The way this model task is completed will depend on the type of model being used, but most commonly involves entering a form of definition table describing storage volume with depth along with details regarding the physical dimensions and elevations of the outlet structure.

Depending on the intended purpose of the urban modelling task, consideration should also be given to antecedent conditions, whether the storage is partly utilised prior to the onset of the storm burst and whether there is potential for blockage of the outlet structure at some point in time and to what extent.

The hydrologic and hydraulic impact of a volume management facility may be distant from its physical location (upstream or downstream). The designer must consider inclusion of all volume management facilities that could potentially impact the investigation site. Also, as proposed storage volumes increase, the critical storm duration and pattern may correspondingly change, necessitating the inclusion of additional rainfall scenarios into the suite of model tests.

6.2.5. Water Quality Treatment Performance

An increasingly common urban modelling task is the assessment of the water quality treatment performance associated with a water treatment facility such as those described in Book 9, Chapter 4.

The facilities that perform this function are often co-located or are an integral part of a volume management infrastructure facility. Where this is indeed the case then similar model inputs are required such as the basin storage and outlet characteristics. However, a different model platform may be necessary since the treatment process targets smaller storms and occurs over longer time periods. For example, event based hydrologic models may not be a suitable basis for these assessments. Instead a continuous simulation-based model would be more suitable.

In addition, further information is required to define the treatment characteristics of the facility. These are mostly based on empirical relationships that simply associate the performance of the facility with its size or alternatively retention curves that relate inflow and outflow concentrations of pollutants. The pollutants of most interest are gross pollutants, nutrients (Total Nitrogen and Total Phosphorus), and Total Suspended Solids.

6.2.6. Data Collection and Collation

A well organised data collection and collation process is essential in the modelling process. It not only ensures that the modelling is fit for purpose, but it documents the sources of data and how the data was interpreted and used in the model. Models often evolve as improvements are made or processes are changed to better represent different components. This task is much simpler if a good data management process has been used.

It is important that the data management system properly documents the source of the data, the format, and the date of acquisition. Book 1, Chapter 4 provides comprehensive advice on the use of data. A key challenge in urban catchments is that many urban drainage components cannot be put directly into a model but need to be schematised. Examples include converting a basin drawing into a stage storage table or representing a complex pit system. It is important that the data management system properly documents this process so the interpretation and schematisation is properly documented and can be reviewed or refined later. While data can be classified in many ways, there are three broad types of data:

- Model inputs such as rainfall and temporal patterns that change between events;
- model components such as pipes, storages, terrain information and land use data; and
- observed data such as observed peak flood levels and flows

The digital age has changed many aspects of data collection with data often being easier to find but often the original data sources are unclear with merged data sets representing the largest part of this problem. This same problem exists in the model development process where many data sets are interpreted and merged. While most urban catchments are ungauged recent observed flood data can often be found on social media and older historical flood information can be found in scanned historical records and newspapers.

6.3. Model Selection

There is a wide range of conceptual modelling approaches, software platforms and systems available to the urban designer. Each platform has different capabilities and strengths. It is not the role of this Guideline to recommend specific conceptual modelling approaches, software packages or prescribed flood estimation methods. However, the guidance contained in this chapter does seek to classify the available options into categories and highlight the current strengths and weaknesses of each to support a decision on the adoption of an appropriate platform or estimation procedure for the task at hand. The authors are mindful that the science and practice of urban stormwater management will continue to evolve, and new models and data will become available. The guidance in this chapter should not be perceived to be excluding new and innovative approaches.

6.3.1. Overall Trends in Urban Modelling

The last 30 years has seen fundamental changes in the way urban stormwater assessment and design tasks are undertaken. It is reasonable to assume that similar change will occur over the next 30 years. Recognising that the decision to adopt a specific urban catchment model platform can have significant implications for personal research and training, this section provides introductory level discussion about these trends. It is expected this will support more informed choices related to adoption of a model platform, either for a specific investigation project or for a longer-term strategic assessment program.

6.3.1.1. Computing Power

In response to the overall computing requirements of society, urban modelling designers now have access to faster computers with enormous numerical computation capability. This has arisen through improvements to computer processors (CPUs) including 64-bit computing and multi-core processing. More recently the use of Graphics Processing Units (GPUs) has led to further substantial processing improvements. New opportunities are also arising with the advent of high-performance computing services, including on the cloud. The transition from hand calculations to widespread availability of computing power to assist in designs is a major change in stormwater management practice since ARR 1987 (Book 9, Chapter 3, Section 3).

As these computing advances have occurred, urban modelling software platforms have been adapted to harness some of the available computational speed increases. This permits the modelling designer to consider:

- Increasing the physical size of the model domain. For example, model a larger urban catchment;
- increasing the spatial and temporal resolution of the model to allow for finer grained numerical calculations that account for location and connectivity of different land uses;
- longer time-series of rainfall;
- a greater number of catchment scenarios;

- tighter integration of hydrologic and hydraulic computation;
- more model iterations to support improved calibration and sensitivity analysis; and
- less conceptualisation and closer alignment to complex physical processes.

It can be expected that computational capabilities will continue to increase into the future and that urban modelling software platforms will continue to be refined and improved to harness more of the available capacity.

Currently, computing power is such that it is reasonable to expect that most urban hydrologic model simulations, even relatively complex ones, can be undertaken within seconds or minutes. It can therefore be assumed that pure hydrologic investigations are already unconstrained by computing power regardless of the choice of model platform.

Computing power is still somewhat of a constraint for hydraulic simulations. Some of the more complex finer resolution or larger domain hydraulic model simulations can take hours or days per simulation. This may constrain the design of an urban hydraulic modelling investigation and also means that due care must be taken when selecting a hydraulic model platform. A hydraulic platform and method should be chosen that has computational efficiency to match the problem at hand. Models with very long run times should be carefully managed as they usually preclude comprehensive testing, checking or calibration.

The future will permit very large multi-catchment spatial domains to be modelled at the finest level of temporal and spatial resolution necessary, with sufficient speed to allow simultaneous and exhaustive exploration of hydrologic and hydraulic scenarios.

This trend may outpace our ability to improve the underlying science and gather sufficient quality input data, and to respond with more informed design and management solutions. Consideration will also need to be given to whether the ultimate outcomes of investigations are improved by aggressively pursuing the full capabilities of available computing power.

In other words, at some point in the future, further improvements to computing power may cease to provide any material value to urban modelling designers. Substantial further research and data collection, for a range of urban catchment scales, is necessary to ensure theory is able to keep pace with computing power.

6.3.1.2. Alignment to Physical Hydrologic and Hydraulic Processes

The underlying methods that are applied using computer-based models have experienced a trend away from conceptual and simplified deterministic techniques to methods that more closely align with the actual physical processes that are occurring.

Some examples of this trend are:

- A move away from isolated storm bursts with a single pattern, towards consideration of pre-burst rainfall and more complete storm bursts including an ensemble of equally likely but different temporal patterns. This leads to more robust design and resolves some of the issues that arise when trying to maintain probability neutrality between rainfall and flood (refer Book 3 and Book 5). The future will see this trend continue with designs becoming increasingly based on complete storms and continuous recorded or synthetic rainfall sequences.
- The use of direct rainfall, also referred to as ‘rainfall-on-grid’ approaches which attempts to explicitly resolve the accumulation of runoff progressively down the catchment, removing

the need to pre-identify flow paths and sub-catchments. This is a useful way to ensure flow paths are not inadvertently omitted from an investigation. With further research and software development this approach may in time also eliminate the need for hydrologic models to undertake surface routing. At this stage however, there is inadequate evidence that a direct rainfall approach should be relied upon for this purpose with many parameters being scale and approach dependent (refer Book 5).

- The hydraulic models applied in practice have increasingly changed from one-dimensional to two-dimensional representations of the floodplain surface. This allows a more realistic definition of potential flow paths which in turn improves the representation of flood behaviour (refer Book 5 and Book 7).

With continuation of this trend it can be anticipated that model platforms will eventually converge on more accurate representations of rainfall runoff and flood processes, requiring different model inputs, parameters, and application techniques. Again, this will only occur with adequate research and software development effort and data collection for a range of urban catchment scales.

6.3.1.3. Statistical Approaches

There has been increasing awareness and understanding of the need to consider the joint probability of model assumptions and physical processes (Kuczera et al., 2006). This has given rise to techniques such as Monte-Carlo sampling and ensembles of rainfall patterns to reduce potential probability distortions and gain better appreciation of model uncertainty (Book 3). For simple urban models or where the design objectives have limited sensitivity to model results these approaches may not be warranted.

These approaches should be considered where better appreciation of natural variability and uncertainty is required. This may include sensitive urban areas, a major waterway crossing, large flood mitigation proposal or hydrologic design of regional scale water quality infrastructure.

Machine learning algorithms are also being used for the prediction of stream flow using statistical information drawn from historic rainfall and stream gauge data, providing an alternative approach to hydrologic modelling.

6.3.1.4. Accumulation of Longer Periods of Recorded Data

With the passage of time, longer periods of recorded data have become available to allow refinements of design rainfall, losses, and more informed model calibration (Book 2, Chapter 3, Section 4). Over time this will allow a better understanding of model performance and uncertainty, particularly within those catchments where data has been recorded. There will be diminishing situations where models are left uncalibrated for the want of historic data.

6.3.1.5. The Internet and Spatial Information Systems

Since the 1987 version of ARR, the internet has emerged to become a ubiquitous part of life. The internet provides urban modelling designers a new potentially more effective method for:

- Accessing and disseminating research, including international practice;
- gathering model input data;
- processing of simulations (using cloud processing technology); and

- storing and communicating information arising from model investigations.

Furthermore, modelling software platforms that have traditionally been tied to a single computer, are now able to be offered as internet-based services. Into the future other new applications will be found for the internet that cannot be fully anticipated at this time but will likely support further improvements in the application of urban models.

In parallel to the internet, an associated trend that has also emerged is a deeper interest and reliance on Geographical Information Systems (GIS). These systems are used for the storage, handling and display of physical catchment data, catchment parameters and infrastructure data.

Spatial information systems have become an important support technology for the application of urban models, with most platforms leveraging these tools for pre-processing and post-processing of data, storage of data, data display, data enhancement and the preparation of information products for stakeholders.

6.3.1.6. Information Needs of the End User

The information needs of the end user have become more complex. A greater number of aspects are of interest. For example, the extent, depth, and level of floodwaters are now typically supplemented by velocity, combinations of velocity and depth (hazard), volume and timing. Enhanced datasets are also now prepared such as risk and planning controls. These results are often required at many additional locations distributed across urban catchments rather than at selected locations at the bottom of catchments.

Urban modelling designers should consider how the model software platforms they use can be used to accommodate these growing information needs.

6.3.2. Types of Urban Models

Notwithstanding the potential future trends in urban modelling described above, today's industry designers already have a greater array of model platforms and estimation options than available in the past. However, each option differs in the quality of spatial representation they are capable of achieving, as well as the capability with which they can represent different physical flood processes. Accordingly, some models or methods may or may not be suitable for a specific urban modelling task.

Some of the more common types of models and methods are listed in Table 9.6.1. For each type, a generic classification of its capability is also provided. This is a snapshot in time of the capability of these models and will change with time. This classification is based on the examples in Table 9.6.2. A subsequent section describes the performance of these models at different spatial scales.

Table 9.6.1. Common Types of Urban Models

Focus	Urban Model Type	Estimation Capabilities (also refer Table 9.6.2)				Example Model Platforms (where relevant)
		Runoff Generation and Surface Routing	Channel and Storage Routing	Structure Hydraulics	Other specific capabilities or limitations	
Hydrology	Rational Method	Limited	None	None	Peak flow only – scalar quantity, single lumped catchment, requires 'Time of Concentration' assumption, only suitable for small catchments. It has best capabilities where there is no storage present.	RATHGL, PCdrain
Hydrology	Time Area Method, Extended Rational Method	Moderate	None	None	Suitable for small catchments only. Can be extended as a collection of linked sub-catchments.	ILSAX, DRAINS
Hydrology	Runoff Routing	Strong	Moderate	Limited	Full event hydrograph, empirically derived lag parameters, non-linear routing capabilities. Structure hydraulics can be moderately capable for discrete structures but not for continuous conveyance networks.	RORB, RAFTS, WBNM, URBS, HEC-HMS
Hydrology	Continuous Simulation	Strong	Moderate	Limited	Continuous multi-year runoff sequence, comprehensive infiltration loss models. Limited capability for rare to very rare floods unless utilised with replicates of conditioned synthetic continuous rainfall (such as DRIP)	XP-RAFTS, MUSIC, PURRS, Systems Framework
Hydrology and Hydraulics	Hydrology coupled to 1D hydraulic model	Moderate	Moderate	Strong	Not always emulating full capability of the underlying hydrologic model	DRAINS, PCdrain, XP-SWMM

Focus	Urban Model Type	Estimation Capabilities (also refer Table 9.6.2)				Example Model Platforms (where relevant)
		Runoff Generation and Surface Routing	Channel and Storage Routing	Structure Hydraulics	Other specific capabilities or limitations	
Hydrology and Hydraulics	Direct Rainfall ('rainfall-on-grid')	Limited	Moderate	Strong	Does not require pre-defined flow paths. Sensitive to topographic data pre-processing and surface roughness assumptions. Not suitable for 'greenfield' subdivision drainage design.	TUFLOW, MIKE21, SOBEK, ANUGA
Hydrology and Hydraulics	Runoff routing coupled to two-dimensional hydraulic model	Moderate	Strong	Strong	Requires pre-defined understanding of flow paths in order to establish initial model. Requires input and output procedure between two model software packages.	RAFTS with MIKE21, WBNM with TUFLOW, XP STORM with TUFLOW, DRAINS with TUFLOW
Hydraulics	One-dimensional hydraulic model	None	Moderate	Strong	Simple channel or pipe behaviour only. Limited where complex flood storages exist.	HEC-RAS, MIKE11, SOBEK
Hydraulics	Two-dimensional hydraulic model	None	Strong	Strong	Complex flow behaviour including breakout and diversion. Flow transitions and hydraulic jumps. Principally surface flow.	TUFLOW, SOBEK, ANUGA, MIKE21, HEC-RAS 2D, RMA, RiverFlow2D
Hydraulics	Pipe network models	None	Moderate	Strong	Specialist models for underground drainage networks, storage routing performance best where flow is contained within the minor system.	SWMM, XP-STORM, DRAINS, PC drain, MIKE URBAN

Focus	Urban Model Type	Estimation Capabilities (also refer Table 9.6.2)				Example Model Platforms (where relevant)
		Runoff Generation and Surface Routing	Channel and Storage Routing	Structure Hydraulics	Other specific capabilities or limitations	
Water Quality	Water quality model	Moderate	Limited	Limited	Additional capabilities related to pollutant generation and removal. Hydraulic capabilities can be extended by coupling to 1D hydraulic model. Runoff generation less suited to event based flood estimates.	MUSIC, EPA-SWMM

Table 9.6.2. Generic Classification of Model Estimation Capability

Flood process	Limited Capability	Moderate Capability	Moderate Capability
Runoff generation and surface routing	Average intensity or burst Cursory treatment of infiltration losses Surface characteristics not fully represented	More complete storm Infiltration losses Surface characteristics partially represented	Full storm or rainfall sequence Infiltration losses Spatial distribution of rainfall Surface characteristics well represented (including surface wave speed)
Channel and storage routing	Channel characteristics not represented No explicit calculation of flood storage and its attenuation effects	Channel characteristics partially represented Storage behaviour partially represented including attenuation effects and spatial influences	Channel characteristics and flood wave speed well represented Storage behaviour well described including complex hydraulic behaviour and attenuation effects

Flood process	Limited Capability	Moderate Capability	Moderate Capability
Structure hydraulics	Basic hydraulic structures only Rating tables Manning’s formula for open channels.	Small range of hydraulic structures Basic topographic representation	Wide range of hydraulic structures Resolves shallow water equations (1D or 2D or both)

When selecting a particular model or technique, the designer should in the first instance look to match the estimation capabilities of the model, whether they be ‘limited’, ‘moderate’ or ‘strong’, with the nature of urban modelling problem that is being investigated.

For example, if channel routing and structure hydraulics are not aspects of the problem that need to be investigated, then the model selected need not have any capabilities in these areas. Equally, if it is expected that a particular problem will require significant capabilities in (for example,) runoff generation, then a model with ‘strong’ capabilities in this area should be considered.

Where the estimation capabilities are identified in Table 9.6.1 as ‘limited’, significant caution must be adopted. As a minimum they should be applied by, or under the direct guidance of, a designer who fully understands the limitations of these approaches. The tolerance for error in the results should be considered and if greater accuracy is required then an alternative more capable model platform applied.

As always, the level of experience of the designer is a significant factor. Someone with significant experience and familiarity with a specific model may be able to extend its capabilities to a level that achieves an acceptable level of estimation accuracy that is beyond its normal capabilities if deployed by an average or less advanced user.

6.3.3. Model Scale

Urban models are constructed at different spatial scales depending on the size of the overall catchment to be analysed and the nature of the performance objectives being sought. Typically the smallest catchment a designer will consider is that of a single small parcel of land with a single dwelling. The type of model assessments that are normally undertaken at this scale include model calculations to assist with design of internal drainage systems and small volume management facilities (e.g. rainwater tanks and OSD).

At the other end of the spectrum of potential scale, an urban model may be constructed to represent all the stormwater catchments spanning an entire suburb or even a small city. These larger models are often used for the purpose of regional flood mapping, establishing flood levels for development purposes or the design of large-scale stormwater and road crossing infrastructure.

When evaluating which type of model to adopt for a particular urban modelling project, the spatial scale of interest is an important factor to consider since some particular models may not be capable of competently representing all the complexity that is encountered at the scale of interest.

Consider four example spatial scales with each physical footprint increasing by an approximate order of magnitude as shown in Table 9.6.3 below.

Table 9.6.3. Typical Urban Model Scales

Lot	Site	Neighbourhood	Precinct
A small parcel of land with 1 or 2 buildings.	A large parcel of land with multiple buildings. Sometimes a small number of 'lots' combined.	Many parcels of land each with at least one building. Many 'lots' and potentially some multi-building complexes.	Hundreds of parcels of land each with at least one building. A large number of 'lots' and multi-building complexes combined. Several neighbourhoods.
e.g. single detached dwelling or duplex up to 1,000m^2 in area	e.g. large townhouse complex covering an area up to 1 hectare	e.g. a residential subdivision stage or a neighbourhood covering an area up to 10 hectares	e.g. a small suburb covering an area of 100 hectares

As a model's spatial scale increases from 'lot' through to 'precinct' the more likely that the catchment being modelled will contain a greater range of features of relevance to stormwater behaviour such as:

- Public roads acting as overland flowpaths
- A larger variety of different land uses and associated connectivity
- Large capacity conveyance infrastructure
- Large basins and volume management infrastructure
- Urban waterways
- Urban waterway crossings

In conjunction with this increase in the number of stormwater features, it follows that the potential number of rainfall runoff processes encountered in a larger scale model will also increase. In this context flood generation processes include damaging floods as well as much smaller floods that are relevant to yield and water quality assessment.

Table 9.6.4 below provides a list of the flood generation processes encountered at each of the four spatial scales described above. This listing is non-exhaustive and only provided to demonstrate that there is a larger number of potential flood generation processes that can be expected to occur as spatial scale increases from 'lot' scale to 'precinct' scale (growing from approximately 8 to 32 in the example listing provided in Table 9.6.4).

Some degree of simplification of these flood generation processes normally occurs when preparing an urban flood model. The flood generation processes listed in Table 9.6.4 have different levels of importance and influence when trying to decide whether any simplifications are possible. Each process has been indicated in Table 9.6.4 by one of two different symbols as follows:

A very important flood generation process. A model constructed at this scale should have the capability to competently address this flood process.	1

A flood generation process that is less important. This process may be omitted or simplified if accuracy of model estimates is not critical	2

Further discussion regarding model simplification is included in Book 9, Chapter 6, Section 3.

Table 9.6.4. Example Flood Generation Processes at Different Model Spatial Scales

Example Flood Generation Processes	Lot	Site	Neighbourhood	Precinct
Overland flow routing across surface of lot	2	2	2	2
Conveyance capacity of roof gutters and downpipes	2	2	2	2
Routing through internal underground drainage	2	2	2	2
Runoff generation from impervious surfaces within lot (e.g. roof)	1	1	2	2
Runoff generation from pervious surfaces within lot (e.g. garden)	1	1	2	2
Conveyance capacity of internal underground drainage	1	1	2	2
Routing through temporary and/or permanent storage connected to dwelling (source control)	1	1	2	2
Storage outlet behaviour including use of stored water for internal and external private demand (source control)	1	1	2	2
Overland flow routing between multiple lots		2	2	2
Routing through open surface drains and driveways		2	2	2
Routing through inter-allotment drainage		2	2	2
Runoff generation from impervious surfaces within common areas (e.g. common driveway)		1	2	2
Runoff generation from pervious surfaces within common areas (e.g. landscape areas)		1	2	2
Conveyance capacity of open surface drains and driveways		1	2	2
Conveyance capacity of inter-allotment drainage		1	2	2
Capacity of inlets to the internal underground system and potential bypass		1	2	2
Routing through temporary and/or permanent storage within common area (source control)		1	2	2
Storage outlet behaviour including use of stored water for external demand within common areas (source control)		1	2	2

Example Flood Generation Processes	Lot	Site	Neighbourhood	Precinct
Overland flow routing across the sub-catchment surface			2	2
Routing through roadside gutters and table drains			2	2
Routing through underground drainage and trunk drainage			1	2
Routing through major overland flow paths			1	2
Conveyance capacity of roadside gutters and table drains			1	2
Runoff generation from impervious surfaces (neighbourhood scale)			1	2
Runoff generation from pervious surfaces (neighbourhood scale)			1	2
Capacity of inlets to the road drainage system and potential bypass			1	2
Capacity of inlets to the trunk underground drainage system and potential bypass			1	2
Conveyance capacity of underground drainage and trunk drainage			1	2
Conveyance capacity of major overland flow paths			1	2
Routing through temporary and/or permanent storage within public areas (neighbourhood control)			1	2
Storage outlet behaviour including use of stored water for external demand within public areas (neighbourhood control)			1	2
Runoff generation from impervious surfaces (precinct scale)				1
Runoff generation from pervious surfaces (precinct scale)				1
Routing through large open channels and urban waterways				1
Conveyance capacity of large open channels and urban waterways				1
Performance of culverts and bridges including impact of blockage and diversion				1
Routing through temporary and/or permanent storage within public areas (regional control)				1
Storage outlet behaviour including use of stored water for external demand within public areas (regional control)				1

2. Opportunity for model simplification (refer Book 9, Chapter 6, Section 3)

Most model platforms have some limitations on which processes they can represent. A decision will be required at the commencement of model preparation as to whether the selected model and the available data are capable of achieving the required level of accuracy and reliability.

As a result of the expected increase in the number of flood generation processes with scale, if a catchment investigation requires investigation across a large spatial scale, then the designer can expect that a model or method with ‘strong’ estimation capabilities across multiple flood process areas will be necessary (refer Table 9.6.1 and Table 9.6.2).

For example, the Rational Method, with ‘limited’ runoff generation and surface routing capabilities, is not likely to be suitable for a ‘precinct’ scale estimate of peak flow as it cannot adequately simulate the array of flood processes that are encountered, even in the simplest of catchments. However, it may be suitable at a ‘lot’ scale in circumstances where storage routing is not critical.

If volume management infrastructure forms part of a solution, or if an understanding of potential impacts on downstream flooding are required, then a ‘strong’ hydrologic estimation method such as a runoff-routing model should be used. For most urban modelling at this point in time, a runoff-routing model coupled to a two-dimensional hydraulic model or pipe network model will provide the strongest estimation capabilities across a wide range of model scales.

The resolution of model inputs and boundary conditions also needs to be considered. There is little value in developing a high-resolution model with coarse lumped inflows or considering the performance of a complex system using a single temporal pattern.

6.3.4. Flood Magnitude

The capability of each type of model also varies with magnitude of the flood being considered. For the smallest of floods, including frequent storms and runoff events, the model’s capabilities should include consideration of infiltration losses including for some applications the recovery of soil moisture profiles during inter-event periods and baseflow. The importance of this capability may change depending on the level of impervious cover within the catchment, becoming decreasingly important as impervious cover increases.

These capabilities are principally the domain of runoff-routing and continuous simulation models. Other processes that effect total runoff volume such as harvesting and use of rainwater may also be important considerations for smaller flood magnitudes. Figure 9.6.1 indicates the likely range of effectiveness for the different types of hydrologic models against flood magnitude on x-scale and model scale on y–scale.

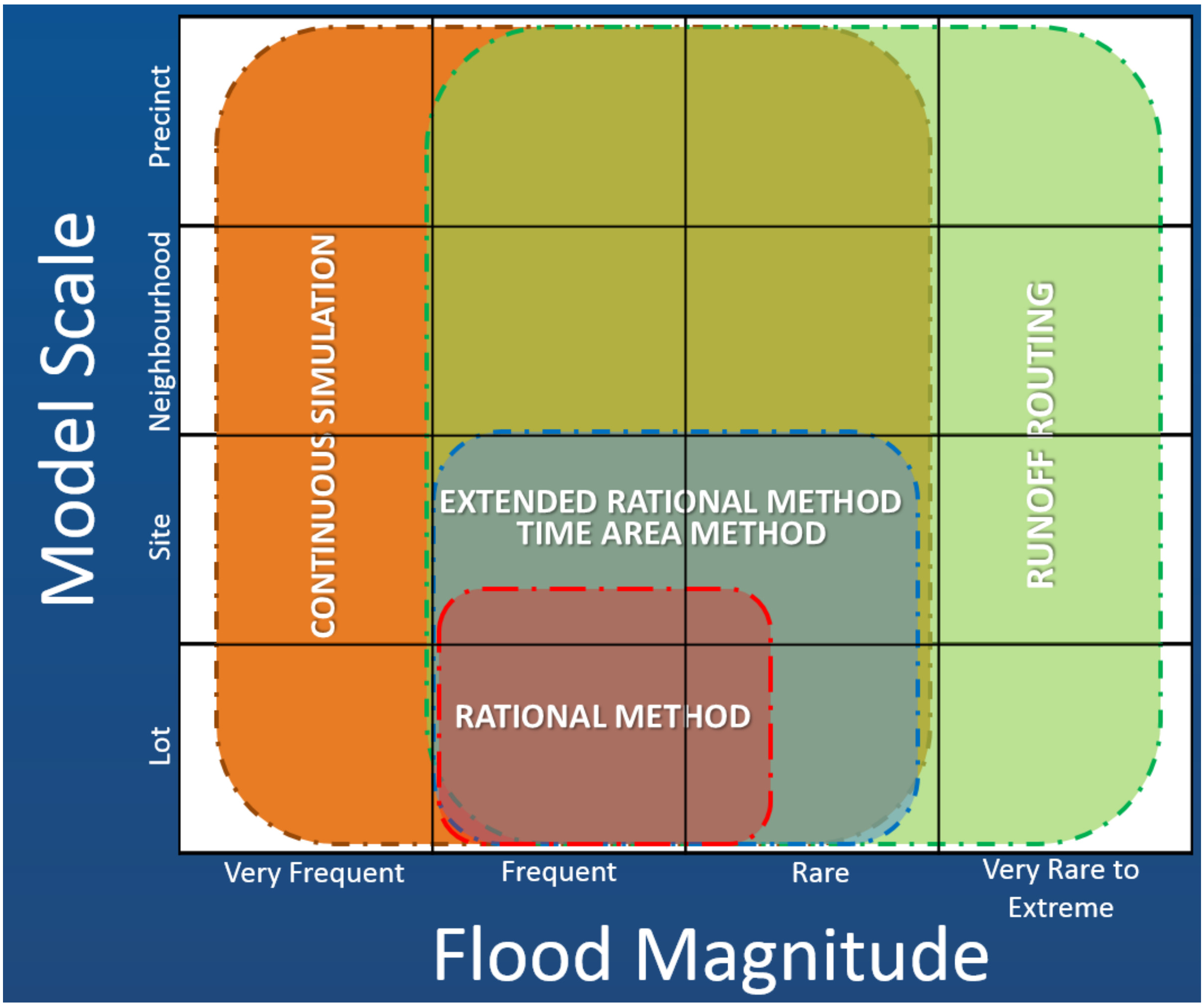


Figure 9.6.1. Types of Urban Hydrologic Models and their Likely Application Range

As the magnitude of flooding that is of interest increases, different hydrologic model requirements emerge since the importance of antecedent soil moisture and rainfall diminish. Typically, runoff-routing models applied using discrete rainfall bursts or more complete storms would be used.

For the companion hydraulic calculations during small floods, and where flooding is confined to the pipe network or a simple channel, a pipe network model and/or 1D channel hydraulic model will normally be adequate. Even some hydrologic model packages have the capability to undertake basic hydraulic calculations.

For hydraulic calculations associated with large floods that exceed the normal capacity of a channel, or where substantial overland flows develop, a 2D hydraulic model may have more utility since the likelihood of complex flow patterns increases.

Further detailed information regarding hydraulic models is included in Book 6, Chapter 4.

6.3.5. Choosing a Model

A stepwise process is suggested below to assist with identifying the types of models that may be suitable for a specific urban modelling problem. Consideration of each step in the flowchart shown in Figure 9.6.2 will help to progressively reduce the number of candidate model types that may apply.

Figure 9.6.2. Stepwise Flowchart for Selecting an Urban Stormwater Model

Where there are multiple options arising from this process, the simplest model, capable of the necessary calculations should be favoured. Other model selection criteria include, availability of sufficient input data and parameter research, output data capabilities, availability of other required functionality (e.g. water quality calculation), cost, and designer familiarity with the model. A hydraulic model involves a more explicit representation of flow routing and how storage is represented in the catchment. Generally, a hydraulic model will be required where there is a need to understand both flow and flood levels. one-dimensional pipe and channel models only provide this information at key locations but are well suited to 'greenfields' subdivision design, while a two-dimensional model provides a detailed spatial representation of surface stormwater processes and may be more suited to brownfields investigations.

6.3.6. Model Simplification

In conjunction with selecting a type of model that has the necessary estimation capabilities and is well suited to the model scale of interest and associated smaller scale influences, consideration must also be given to the degree of model simplification that might be appropriate.

When modelling at small spatial scales it is simpler to closely represent each flood process and its associated physical features and drainage connections explicitly. As spatial scale increases it is sometimes possible to adopt some model simplifications to manage data requirements and the general complexity of the modelling task. For example, when building a ‘precinct’ scale model it may be possible to omit or simplify ‘lot’ scale processes. However, models should not be simplified unless that consequences of spatial averaging, deterministic assumptions and judgements is well understood. Where simplification is undertaken, efforts should be made to fully understand the impacts of simplification and limits on validity of the model outputs. For example, by comparison of results against a more detailed sub-model or results generated by an alternative model.

Experience and careful judgement are required when choosing to omit or simplify those processes that are suggested as being less important. In general, the omission or simplification of such a process should only occur when the investigation does not demand highly reliable estimates, for example, for preliminary sizing of structures or where flood risks are low. Table 9.6.4 indicates those flood generation processes that may be less important and therefore could be considered as an opportunity for simplification at each different spatial scale.

6.3.7. Model Resolution

Closely related to consideration of model simplification is the interrelated consideration of model resolution. Resolution can in this context have multiple aspects.

Firstly there is spatial resolution of the model. For a hydrologic model this will relate to the minimum size of sub-catchments. For a hydraulic model this will relate to the density of sampling of the ground surface.

The adopted spatial resolution of a model will govern the density of reporting locations i.e. where model results are output by the model software. It may also influence model accuracy. Through experience a designer will develop an understanding of the optimum model spatial resolution for each type of model and to what degree spatial simplification can be tolerated.

Then there is the temporal resolution of the model and the ability to extract output time series that are fit for purpose. For example, the temporal resolution necessary for regional water supply planning may be lower than required for calculation of stormwater harvest yield from a small catchment. In this case a degree of temporal simplification to daily or monthly data may be acceptable for a regional water supply planning task.

Again, through experience a designer will develop an understanding of the optimum model temporal resolution and to what degree temporal simplification can be tolerated.

6.4. Application to Urban Modelling

Stormwater management is subject to ongoing evolution and change. There has been substantial change to the practice and science of stormwater management since 1987 as discussed in Book 9, Chapter 3. This version of ARR combines 30 years of additional data with evolving science and professional capability to accommodate changes in professional and community aspirations. This process has provided a range of new methods, data and resources that can assist the designer to address the local challenges of managing stormwater runoff in urban areas.

Drainage networks (also discussed in Book 9, Chapter 5) are now considered to be part of more comprehensive stormwater management approaches (refer Book 9, Chapter 3) that respond to multiple water cycle objectives including protecting waterways, mitigating flood risks, provision of water resources, managing the quality of stormwater runoff and enhancing the amenity of urban areas. These approaches respond to a need to manage urban water balances (discussed in Book 9, Chapter 2) and to also incorporate a range of storage measures (refer Book 9, Chapter 4) that aim to manage flooding, stormwater quality and provide additional water resources.

This section provides a framework for application of modelling approaches to urban stormwater catchments. The framework provides guidance for key segments of catchments from the behaviour of land uses within sub-catchments that flow to inlet structures, through urban stormwater networks, and into the receiving waterway.

A range of approaches are now available to determine the configuration of measures in a linked stormwater management system than may include a conveyance network, volume management strategies and non-structural measures. These methods can range from simple procedures to detailed computer modelling. The application of new rainfall data and methods to modelling approaches is discussed with reference to the different approaches to the design of stormwater management measures and systems.

6.4.1. Urban Modelling Frameworks

An increasing range of modelling frameworks and approaches are available to urban designers (refer Figure 9.6.1). The urban stormwater design process, as outlined in Book 9, Chapter 5, Section 3 (refer Figure 9.5.3), should be modified to respond to the characteristics of a particular project. Selection of a modelling framework will depend on the purpose of the analysis, scale and complexity of the project, availability of data and the consequences of failure, and includes:

- Hydrological models that translate rainfall into stormwater runoff and evaluate behaviour of storages;
- hydraulic models that evaluate or design the transfer of stormwater flows through networks of infrastructure and across land surfaces;
- hydrology models that include simple pipe hydraulics or one-dimensional hydraulic models;
- linked hydrology and hydraulic models that include detailed two-dimensional surface flows with hydrodynamic conveyance networks;
- rainfall-on-grid models;
- continuous simulation of rainfall runoff and physical processes to evaluate behaviour of integrated solutions and account for antecedent conditions, water quality and associated performance issues; and
- approximate empirical relationships or peak runoff assumptions used to design and evaluate components of urban catchments.

We should be mindful that all models are an approximation of reality that can be used to enhance our understanding about the likely stormwater behaviours for particular urban scenarios. The different hydrological and hydraulic models can be classified by their outputs of peak flowrates, hydrographs, flood depths or continuous sequences of stormwater runoff. These models can also be distinguished by the methods used to route rainfall runoff towards inlet structures in urban conveyance networks or stormwater volume management measures. Models can also be described by different spatial detail such as lumped, semi-distributed or distributed inputs (Figure 4.2.5, Book 4, Chapter 2, Section 6). Lumped catchment models approximate the behaviour of the catchment using single average inputs and assumptions. Semi-distributed models employ a range of sub-catchments with different attributes and assumptions. In contrast, spatially explicit details are included in distributed models – this detail may include the range of different land uses and properties in an urban model or a grid of equal size and shape used throughout the model. An emerging type of distributed hydrology and hydraulic model is the direct rainfall or rainfall-on-grid methods (refer Book 6, Chapter 4, Section 7).

Empirical relationships can be utilised to determine peak flows from small catchments and are applied to the design of roof gutters, downpipes, and infrastructure to manage stormwater runoff from properties in accordance with standards such as AS/NZS 3500.3. These approximate methods include nominal “deemed to comply” infrastructure specifications or generally require information about catchment area and slope, and utilise assumed runoff coefficients, time of concentration and design rainfall intensity in a lumped catchment design process.

The probabilistic or the urban Rational Method is a more detailed approximate method that is utilised to generate peak flowrates for use in the design of pipe networks within small properties and for small sub-catchments. This framework of analysis differs from simple empirical relationships by including equivalent or effective impervious areas, accumulation of flow rates and the areas of different land uses. The method uses rainfall intensity derived from Intensity Frequency Duration (IFD) data, assumed runoff coefficients and time of concentration to derive stormwater peak flows.

The design approach associated with urban Rational Method is often based on lumped sub-catchment inputs to inlet structures which require the resolution of partial area effects on the timing of cumulative peak discharges throughout a conveyance network. A lumped sub-catchment process combines all land uses, including the area of pervious and impervious surfaces (full area), with an estimated time of concentration to derive peak flows at the outlet of a sub-catchment which is the inlet to a conveyance network. A partial area effect is, for example, where the runoff from impervious surfaces (partial area) arrives at the outlet before runoff from pervious surfaces reach the outlet at less than the full area travel time. These methods may be used to analyse the capacity of individual pipes or peak flows from small catchments but cannot simulate actual flow behaviour throughout conveyance networks and urban stormwater management systems (Pilgrim, 1987).

The simple nature of the urban Rational Method cannot account for the complexity of contemporary urban catchments and modern stormwater management approaches, the temporal and spatial variability of storm events, and variations in antecedent or between storm event processes. Approximate methods, such as Rational Method, should only be applied within a catchment where more detailed analysis of rainfall runoff observations have defined the parameters (for example, runoff coefficient and time of concentration) for use in the method (Phillips et al., 2014; Coombes et al., 2015a). However, Goyen (2000) established that derivation of runoff parameters at the regional scale or bottom of a catchment may not necessarily describe local processes in sub-catchments. Local information is also needed to determine urban runoff parameters.

Runoff or hydrograph routing methods are commonly associated with computer models that include internal processes that incorporate different land uses with separate pervious and impervious surfaces. The process includes depression storages and losses with lag times to generate separate hydrographs of runoff for each land surface. These runoff routing methods typically employ event based rainfall inputs (Book 4, Chapter 3, Section 2) of selected Annual Exceedance Probability (AEP) and duration of peak burst rainfall (refer to Book 9, Chapter 6, Section 4). An objective of this process is to achieve probability neutrality between rainfall inputs and generated runoff for urban catchments.

These runoff routing methods may utilise single or multiple design storms and associated temporal patterns to determine regimes of excess rainfall that is then routed through hydraulic models that range from simple pipe hydraulics to full two-dimensional hydrodynamic processes. A key limitation of event based modelling approaches is the need for assumptions about joint probability of antecedent conditions (such as soil moisture and available storage in volume management solutions) and the characteristics of storm events (Kuczera et al., 2006). In addition, event based methods have traditionally only simulated runoff from burst rainfall and have not considered that runoff is also generated by pre-burst and post-burst rainfall (refer to Book 9, Chapter 6, Section 4). The magnitude of rainfall runoff in urban catchment may be under-estimated by event based processes unless pre-burst rainfall is also counted in rainfall event based models.

The limitations of rainfall event based models, and dramatic increases in the capacity and utilisation of computers has fostered the use of continuous simulation (Book 4, Chapter 3, Section 3 and Book 9, Chapter 3) models that can account for continuous physical, conceptual and statistical processes in urban catchments. These methods have traditionally utilised real or synthetically generated rainfall sequences to understand the yield from water supply catchments and the behaviour of water and wastewater distribution networks. These methods are also used to estimate the behaviour of stormwater quality solutions in urban catchments (Fletcher et al., 2001). However, continuous simulation can also be employed to account for the interactions between climate processes, human interventions or behaviours and stormwater runoff from urban catchments (Coombes and Barry, 2015). Pluviograph rainfall records with intervals of less than an hour (often 6 minute intervals) are used in continuous simulation of rainfall runoff from urban catchments.

The continuous simulation method involves simulation of a rainfall runoff model over a time period of sufficient length to account for all of the important interactions between rainfall and catchment processes to produce an urban flood frequency analysis. Sufficient lengths of observed rainfall are usually not available to provide adequate information about rare runoff events and synthetic rainfall sequences are often required for continuous simulation models (Book 4, Chapter 3, Section 3; Book 2, Chapter 7,). Use of continuous simulation with synthetic rainfall inputs may require calibration of the rainfall model and the continuous runoff routing model (Book 4, Chapter 3, Section 3). However, all models require calibration and verification.

An alternative use of continuous simulation is to derive the probability distribution of initial conditions prior to storm events such soil moisture storage, and available storage in rainwater tanks and bioretention facilities (Coombes and Barry, 2008a; Hardy et al., 2004). These probability distributions of initial conditions are then utilised in event based runoff routing models to determine runoff from urban catchments. Note that these types of probabilistic inputs are associated with complete storm events and will need to be applied in event based models using complete storm events or combinations of pre-burst and burst rainfall.

Direct rainfall or rainfall-on-grid models combine hydrological and hydraulic processes to generate rainfall runoff and hydraulic routing in a single model. Rainfall is applied to each grid in a two-dimensional hydraulic model to generate overland flows and discharges in conduits (Book 6, Chapter 4, Section 7). This method can provide more realistic representation of catchment storages and surface runoff processes including cross catchment flows. A fine grid of good quality topographic, losses and roughness data is required, and topography information will need to be edited to include key infrastructure such as street gutters, hydraulic structure, conveyance networks and road crowns (Hall, 2015). Rainfall-on-grid models should be calibrated to local historical spatial flood levels or flow data. Use of regional rainfall runoff parameters is not suitable for direct rainfall methods that are driven by local processes.

There may also be a need to vary roughness parameters (such as Manning's n) with flow depth (for example, Zahidi et al. (2017); Khrapov et al. (2015); Muglera et al. (2011)) and carefully assign loss parameters in each grid (Babister and Barton, 2012). The results at local and sub-catchment scales may be unexpected as all flow paths are identified. The method is subject to a range of potential challenges including mathematical instabilities, unrealistic flows and large errors created by losses, variable roughness, long runtimes and shallow flow depths. These powerful direct rain methods are subject to ongoing research and model results should be interpreted with caution. It is imperative that designers check that catchment response with an alternative model and volume of runoff is consistent with loss model used (refer to Book 9, Chapter 6, Section 4). If a rainfall excess model is used this represents the volume of runoff that appears at the catchment outlet not rainfall applied to the model so depression storage needs to factored into losses.

6.4.2. Choice of Rainfall

Most hydrology and hydraulic models require rainfall inputs to estimate stormwater runoff and associated flood responses. The investigations underpinning the this guideline incorporated 30 years of additional data and science (Book 2, Chapter 1) to develop improved design rainfall frameworks. There was also a need to incorporate climate change processes into design rainfall frameworks (Book 2, Chapter 2, Section 4). Design rainfalls are simpler and different to real or observed rainfall. More advanced design rainfalls that assume storm bursts and spatial uniform temporal patterns cannot capture that actual variability of observed rainfall. This insight motivated a change in practice from simple average rainfall intensity or single rainfall burst approaches to ensemble and Monte Carlo methods to better capture the natural variability of rainfall.

The design of stormwater infrastructure and understanding of runoff for urban areas involves decisions at multiple scales. This insight can be combined with ensembles of design rainfall patterns to determine the appropriate rainfall inputs as shown (for example) by the Box and Whisker plot of peak runoff (discharge) to the catchment outlet in Figure 9.6.3.

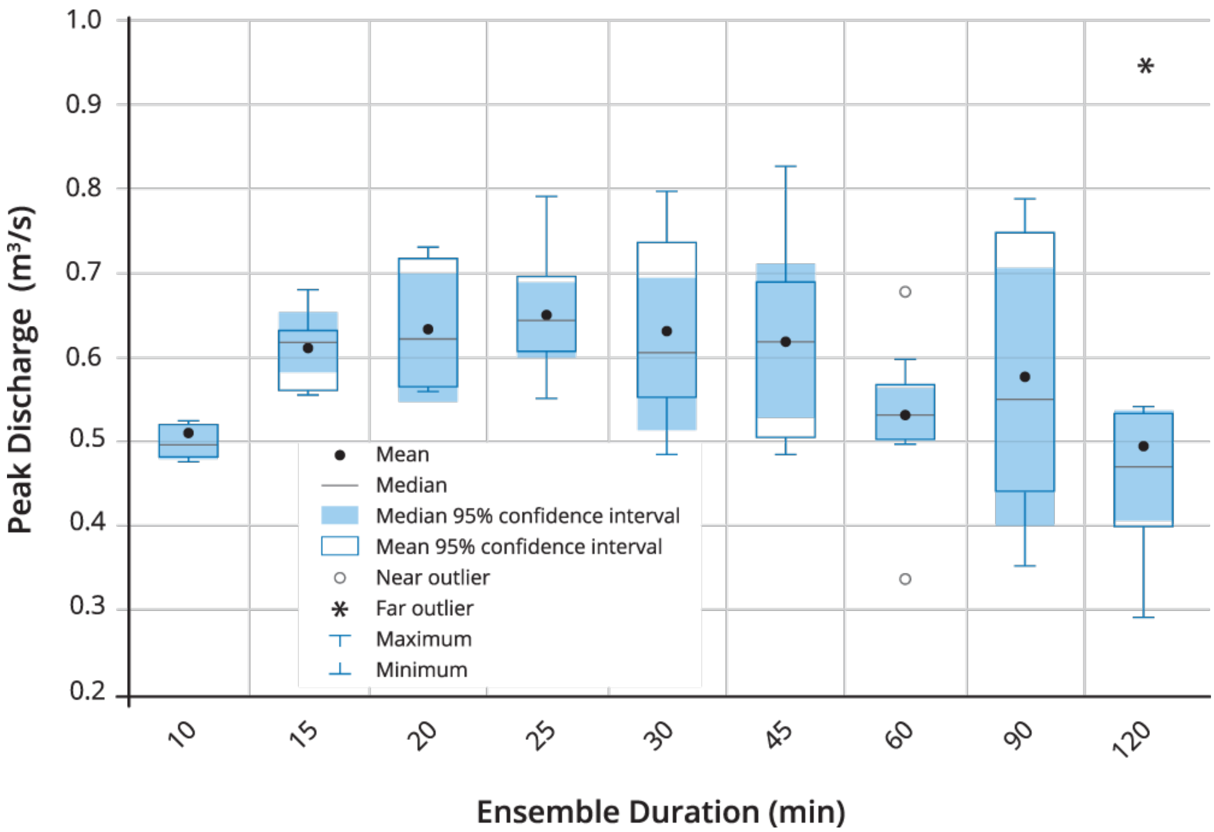


Figure 9.6.3. Example of a Box and Whisker Plot of Peak Stormwater Runoff Utilised to Select the Critical Storm Burst Ensemble and Other Design Information

Figure 9.6.3 indicates highest average and median peak discharge is generated by the ensemble of 10 storm bursts of 25 minute duration at the catchment outlet. A small number of higher values of peak runoff also occur in the 45 minute (maximum value) and 120 minute (far outlier value) durations which could be used to test the potential maximum hazard of surface flows. Conveyance infrastructure within the catchment should be designed using ensembles of storms with durations up to and including 25 minutes to account for impacts of smaller duration storms upstream of the outlet. Different design ensembles may apply in situations that incorporate within catchment storage solutions and at different locations in the urban catchment.

This improved approach to design rainfall inputs to models is particularly important for urban catchments that are significantly different to rural catchments because they generate runoff from majority of rainfall as shown in Figure 9.6.4.

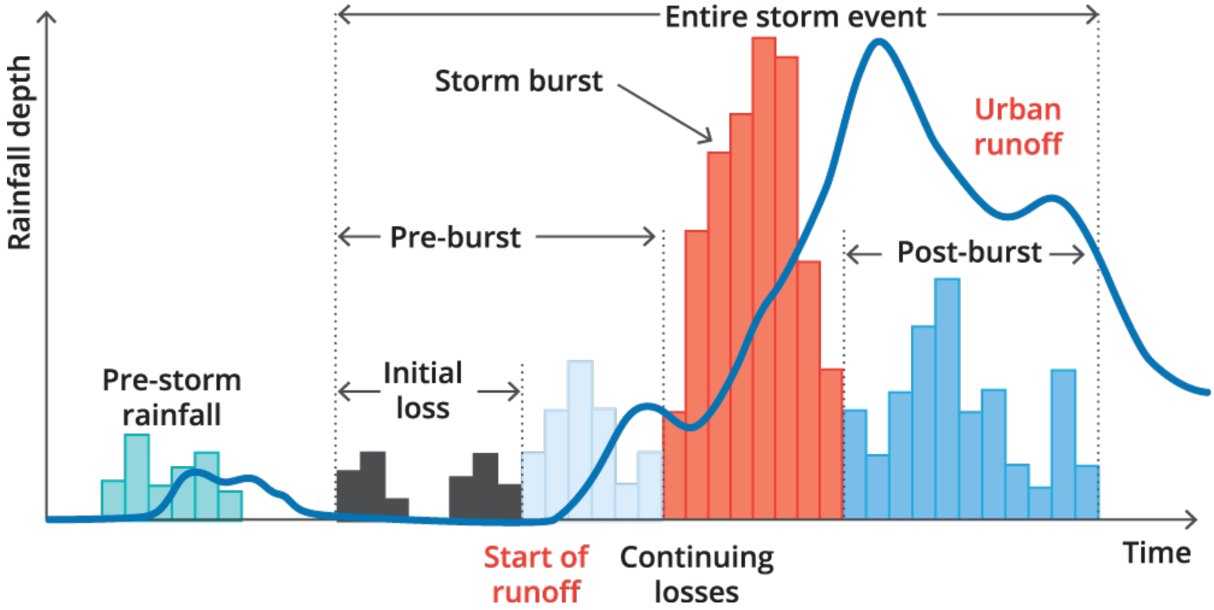


Figure 9.6.4. Rainfall Runoff Processes in Urban Catchments

Figure 9.6.4 demonstrates that urban runoff can be generated by pre-burst, burst and post-burst proportions of complete storms (entire storm event). There are many different configurations of pre-burst, burst and post-burst rainfall in real rainfall events that should be considered in analysis of urban hydrology. Urban designs based on a single burst pattern of rainfall or peak rainfall assumptions can overlook substantial runoff rates and volumes which may adversely impact on the performance of inlet structures in conveyance networks, volume management measures, roads and overland flow paths (Coombes et al., 2015b).

A range of updated rainfall products are available from the ARR Data Hub (Babister et al., 2016), including new spatially distributed IFD, Areal Reduction factors (ARF), design temporal patterns for burst rainfall, hydrological losses, and pre-burst rainfall – as summarised in Table 9.6.5.

Table 9.6.5. Summary of Updated Design Rainfall Processes

Input	ARR 1987	Pre Update	ARR 2016
IFD	Paper maps	BoM web page	Updated BoM web page. Book 2, Chapter 3.
ARF	Figure 2.7 from US data	FORGE work (except NSW)	New equations derived using Australian data. Book 2, Chapter 4.
Temporal patterns	Single temporal pattern of design burst rainfall based on Average Variability Method (AVM)	AVM, filtered for embedded burst	Ensemble of real storms. Book 2, Chapter 5.
Spatial pattern	Centroid	Spatially distributed IFD	Spatially distributed IFD

Input	ARR 1987	Pre Update	ARR 2016
Climate change			Factors available from Book 1, Chapter 6 and the ARR Data Hub.
Losses	State based advice, sometimes based on data	Calibrated in the hydrologic Model.	Calibrated losses. Uncalibrated models use losses available from Book 5, Chapter 5 and the ARR Data Hub.
Pre-burst	Allegedly incorporated into advice	Mixed	Estimates provided on ARR Data Hub. Use 60 minute pre-burst rainfall with burst rainfall ensembles of durations less than 60 minutes

The different rainfall inputs to hydrology and hydraulic models are discussed in Book 2. The updated IFD design rainfall data is available from the BoM website. Derivation of the IFD data using the additional rainfall records is outlined in Book 2, Chapter 3, Section 4 and the application of the updated IFD design rainfalls is presented in Book 2, Chapter 3, Section 9.

ARF are available from the ARR Data Hub and is discussed in Book 2, Chapter 4. Design rainfalls (IFD) only apply at a point in a catchment. When estimates of rainfall runoff are required for catchments with areas greater than 10 km^2, the design rainfall intensities at a point are not representative of the areal average rainfall intensity for the entire catchment. The ARF is the ratio between the design values of areal average rainfall and point rainfall, for same duration and Annual Exceedance Probability (AEP). Application of ARF is outlined in Book 2, Chapter 4, Section 3.

Most runoff-routing methods utilise design temporal patterns to determine the timing of rainfall falling on catchment and generate hydrographs of runoff. The traditional use of a single average temporal pattern has been found to be inadequate for hydrological analysis due to the variability of natural rainfall patterns (Book 2, Chapter 5) and of the characteristics of urban catchments (Book 9, Chapter 3). The application of design temporal patterns as outlined in Book 2, Chapter 5, Section 9. Ensembles of design temporal patterns that are more likely to capture these natural and human variabilities are available from the ARR Data Hub. It is noted that two different ensemble patterns are provided, point rainfall patterns for catchments with areas up to 75 km^2 and areal rainfall patterns for catchments with areas greater than 75 km^2.

Climate change has the potential to alter the frequency and severity of rainfall events, storm surge and floods by altering rainfall IFD relationships, rainfall temporal patterns, continuous rainfall sequences, antecedent conditions and baseflow regimes (Book 1, Chapter 6; Book 2, Chapter 2, Section 4). Climate change factors are presented as changes in average temperature and associated increases in rainfall intensity and losses for selected global emission pathways in the ARR Data Hub[1].

The ARR Data Hub provides regional rural losses for complete storms and pre-burst rainfall. In urban areas, the median values of local losses should be utilised wherever possible. The average initial losses from urban impervious surfaces is less than 1 mm (Book 4, Chapter 2, Section 7) and ranges from 1 mm to 4 mm for urban effective impervious areas (Book 5, Chapter 3, Section 4). In most cases, storm burst loss is equal to median storm loss less pre-burst rainfall.

Rural and regional loss assumptions should not be a default assumption for urban areas and a hierarchy for selecting urban losses is highlighted as follows:

- Use local losses based on GIS investigations, local knowledge and observations. Losses derived at a regional scale are not local losses- use local losses in small scale models. Note that a well-constructed model with adequate spatial scale should account for effective impervious area and connectivity effects

- Regional losses (Book 5, Chapter 3, Section 4 and Book 5, Chapter 3, Section 5): Impervious area losses: IL: <1 mm, CL: 0 mm/hr; Effective Impervious Area: IL: 1-2 mm, CL: 0 mm/hr; Pervious area ≈ rural losses

- Rural losses: Urban losses are some proportion of rural losses

Continuous simulation of rainfall runoff processes is aided by the increased availability of continuous (also known as pluviograph or instantaneous) rainfall from the Australian Bureau of Meteorology (BOM). However, as discussed in Book 9, Chapter 6, Section 4, longer synthetic continuous rainfall records are usually required to understand the impacts of rarer runoff events. Development and availability of synthetic continuous rainfall sequences are discussed in Book 2, Chapter 7. Additional discussion of synthetic continuous rainfall records that incorporate regional layers (surfaces) of spatial observed climate observations is also provided by Coombes and Barry (2015) and Coombes and Barry (2018). This guideline also provides software to generate multi-site continuous synthetic rainfall (Multi-site Rainfall Simulator) at http://arr.ga.gov.au/.

Radar rainfall (refer Cecinati et al. (2017)) can be used to interpolate between point rainfall observations for use in hydrology and 2D hydraulic models. There have been many studies that have developed methods to correct errors in radar rainfall but some residual errors are intrinsic to radar rainfall that should be resolved by spatial and temporal comparison to point rainfall observations.

6.4.3. Runoff From Properties

Stormwater runoff from roofs and properties, at the lot scale, is the basic building block of urban stormwater catchment behaviour ((Goyen and O'Loughlin, 1999a; Stephens and Kuczera, 1999) and Book 9, Chapter 3). Runoff from properties involves a complex interaction of roofs, yards, paved areas, gardens, and adjoining roads and footpaths as shown for a residential property in Figure 9.6.5.

[1] This section was written before the latest climate change guidance in Book 1, Chapter 6 (2024). A minor change to the text has been made to reflect the change in guidance.

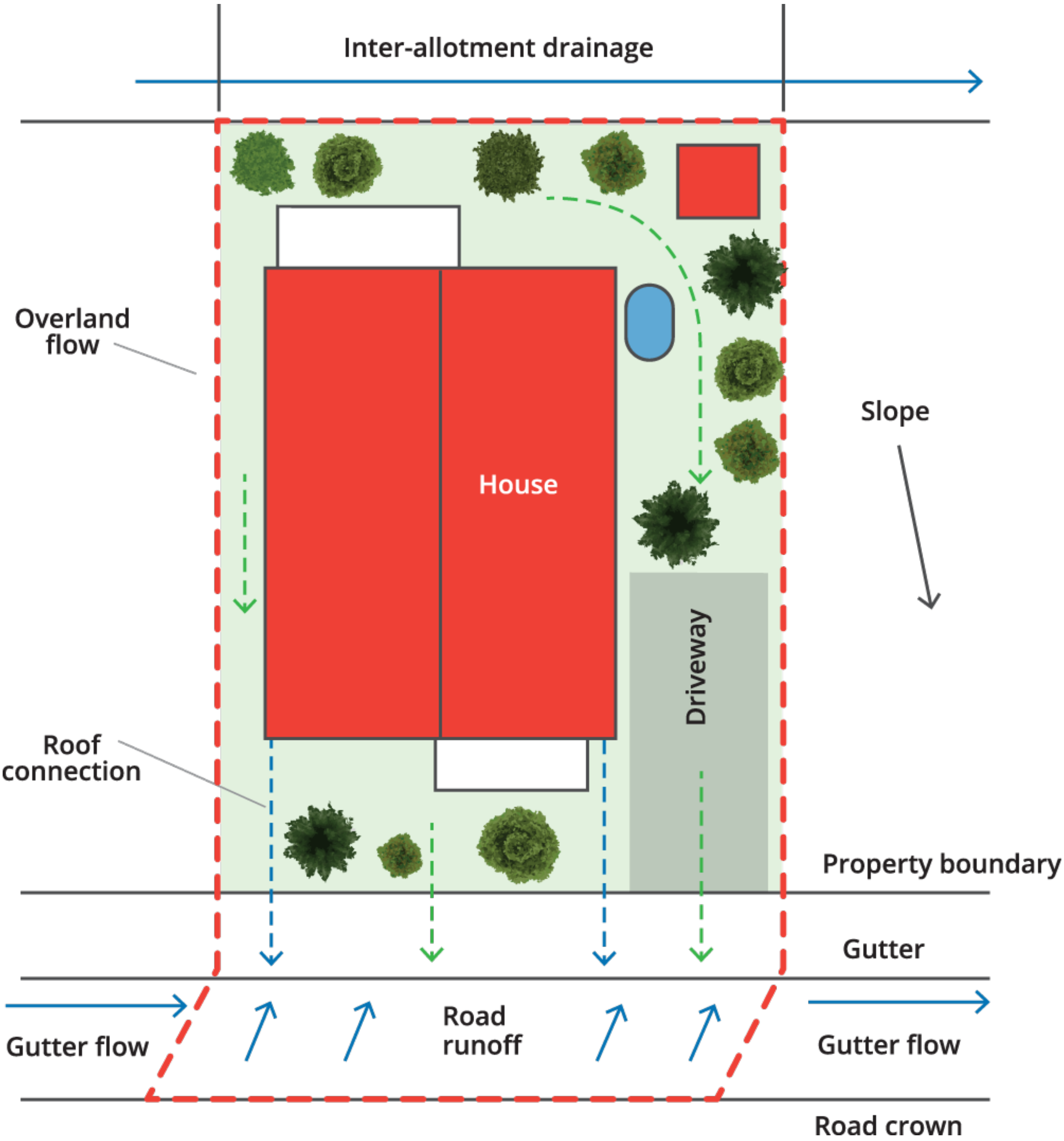


Figure 9.6.5. Stormwater Runoff from Roofs and Properties – Lot Scale Effects

Figure 9.6.5 demonstrates the pathways of stormwater runoff from different surfaces within a property. These runoff processes are dominated by directly connected impervious surfaces, indirectly connected surfaces and pervious surfaces. Rain falling on impervious roof surfaces flow into roof gutter storages which discharge via downpipes into pipes connected to the street gutter or pipe network. Runoff from impervious driveway surfaces and adjacent road surfaces discharge to street gutters. These impervious surfaces facilitate highly efficient translation of rainfall into runoff, are subject to small depression storage losses, and are mostly directly connected to street gutters. Rain falling on pervious yard areas is partially retained in depression storages and infiltrates into soil profiles prior to generation of runoff from residual rainfall. These types of pervious surfaces are relatively inefficient at generating runoff and are often indirectly connected to street gutters or pipe networks. Urban properties can also include impervious areas that discharge stormwater to pervious surfaces or storages (for example, rainwater tanks, onsite detention and raingardens) that partially disconnect these surfaces from street gutters.

Runoff from impervious surfaces may also arrive at street gutters more rapidly than runoff from pervious surfaces. In many situations, pervious surfaces may not generate runoff for frequent rainfall events. These runoff behaviours are influenced by the configuration of property assets (including building form), topography and stormwater management measures. In situations where allotments slope away from roads, runoff from roofs and impervious surfaces may be directed to an inter-allotment conveyance (easement drainage) network. Local authorities will often specify locations of stormwater discharges from properties – this is known as a legal discharge point. Subsoil drains are sometimes used on properties to lower water tables around buildings or in waterlogged areas and discharge stormwater from properties.

Property scale influences are fundamental to urban stormwater runoff. However, there has been limited testing at this scale (Stephens and Kuczera, 1999), and designs of roof and property drainage are not clearly defined (Jones et al., 1999). A major challenge for simulation of urban stormwater runoff is the behaviour of individual properties and accumulation of these property behaviours throughout urban catchments (Goyen (2000), Coombes (2015); Book 9, Chapter 3). The cumulative impacts of properties on the behaviour of catchments are defined by the timing, volume and rate of stormwater runoff from each property. The runoff behaviour of properties can also be altered by a range of onsite stormwater management approaches including disconnection of roof downpipes from street gutters, raingardens, landscaping, rainwater tanks, infiltration measures, onsite detention and green spaces (refer Book 9, Chapter 3 and Book 9, Chapter 4). Local authorities can apply restrictions on the flow rate, quantity and quality of stormwater that discharges from a property to encourage onsite management of stormwater to avoid or reduce downstream impacts (Chocat et al., 2001; Patouillard and Forest, 2011; Walsh et al., 2012; Everard and McInnes, 2013).

Calibration or verification of urban stormwater modelling frameworks at the catchment scale does not imply that the sub-catchment or local behaviours in models are also correctly described (Goyen and O'Loughlin, 1999a; Stephens and Kuczera, 1999; Kuczera et al., 2006; Coombes, 2015). Attention to local detail in stormwater design is required to ensure that potentially overlooked local processes do not generate local failures or excessive infrastructure or unexpected downstream consequences. The problems generated by approximated local behaviours can become worse in areas subject to increasing urban density and infill development. Kemp and Myers (2015), for example, found that increases in urban density of 18% generated 16% increase in runoff volumes and a 300% increase in expected flood damages for 20% AEP storm events.

Simple methods for design of roof gutters, downpipes and property drainage are provided in Australian Standards (for example, AS/NZS 3500.3), by suppliers of roofing materials, government authorities and the Plumbing Code of Australia. These approaches include nominal and general methods. Nominal methods apply to single dwellings on properties with land areas up to 1,000 m^2 by providing “deemed to comply” specifications of infrastructure (configuration, minimum pipe sizes, depth of cover over pipes and slopes).

Design calculations are provided for more complex land uses and larger properties. These guidelines highlight the need to avoid ponding against buildings, flows into buildings and management of overland flows from adjoining properties. Large residential, commercial and industrial properties and car parks include more complex and dendritic stormwater management systems (for example Figure 9.6.6).

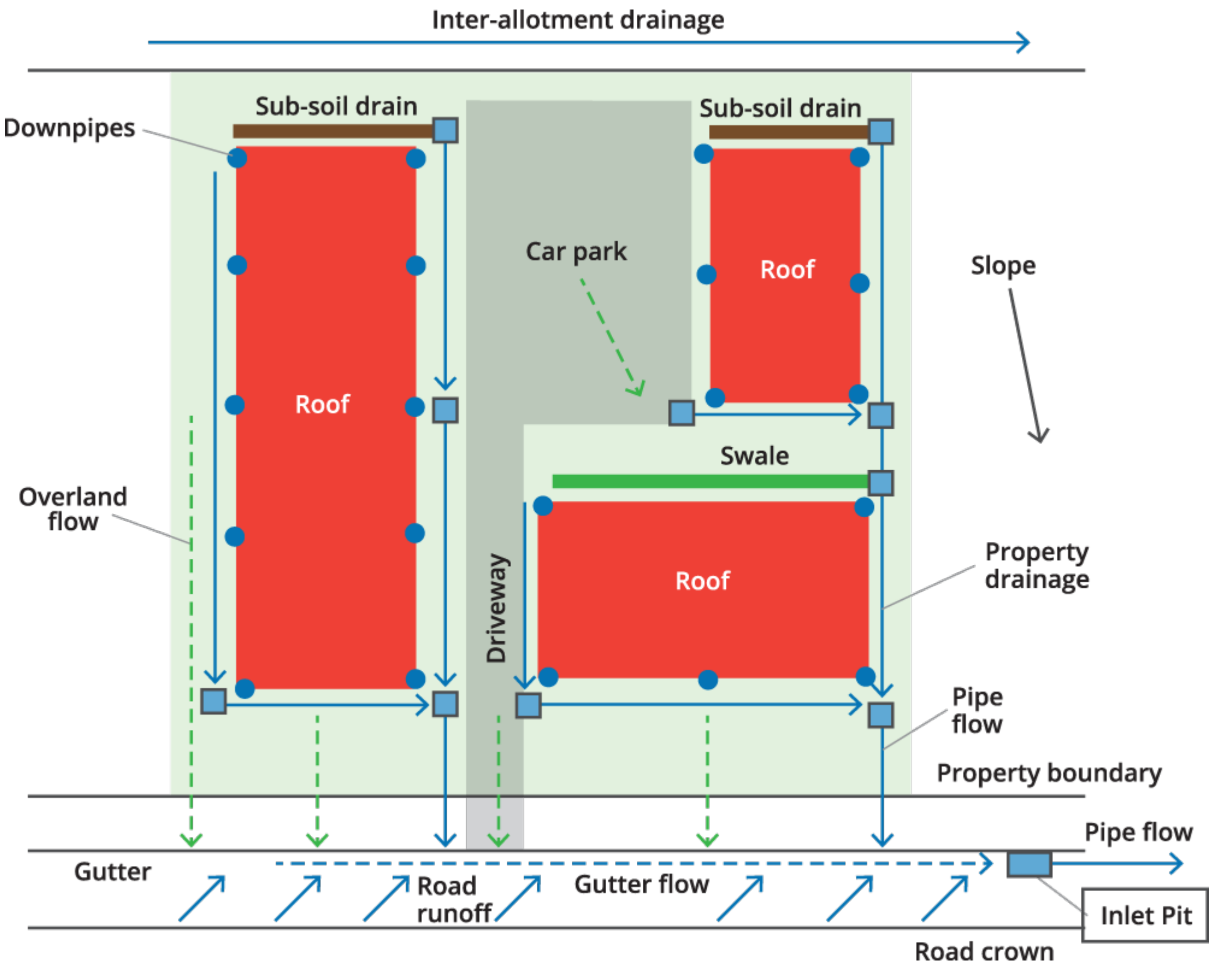


Figure 9.6.6. Stormwater Management System for Larger Properties with Complex Land Uses

Figure 9.6.6 shows that stormwater management schemes within properties may combine multiple pathways of stormwater runoff from different surfaces that have variable levels of connection to the street gutter or inlet pit in the street conveyance network. The performance of these networks may be affected by in-pipe attenuation effects, volume management measures, and substantial variations in the timing and magnitude of runoff to sub-catchment outlets. These outflows from properties are surface flows, or direct inflows to pipe networks in streets or inter-allotment conveyance networks.

Approximate or general design methods are based in rules derived from simple Rational Method assumptions and utilise catchment areas (roofs, paved surfaces and gardens), proportions of imperviousness, slopes, assumed times of concentration with associated average rainfall intensities and runoff coefficients to generate maximum or peak flow rates. A five minute time of concentration and associated rainfall intensity was commonly assumed in design processes for roof and property drainage. Performance standards for roofs have been defined by choice of rainfall intensity of a 5% AEP for roof gutters and of a 1% AEP for box gutters. Design of conveyance networks within properties aim to avoid surcharges and overland flows for 1 EY in low density areas and up to 5% AEP for important land uses (such as hospitals and aged care facilities) that may be vulnerable to greater risk or inconvenience. The volume, pattern and timing of stormwater runoff are not considered in these approaches which may lead to under-performance of stormwater management measures included unexpected surface flows on properties.

Field measurements suggest that travel time to street gutters from residential properties is two minutes or less (Stephens and Kuczera, 1999; Coombes, 2002). The assumption of five minute time of concentration in ARR 1987 (Pilgrim, 1987) was based on the lowest available time interval of IFD rainfall at the time. Revised IFDs available from the BoM provide values for rainfall intensity that commence at a one minute duration which permits use of finer detail in design and to account for shorter flow times to outlets. Observations by Stephens and Kuczera (1999), Goyen (2000) and Coombes (2002) indicate that initial losses from roof gutter systems range from 0 mm to 1 mm and continuing losses range from 0% for metal roofs to 20% for dry tile roofs. Average depression storage losses of impervious surfaces can range from 1 mm to 10 mm and average losses from pervious surfaces range from 2 mm to 20 mm.

Goyen and O'Loughlin (1999b) highlighted that spatial and temporal patterns of rainfall losses and their magnitude have significant impacts on peak stormwater runoff. Larger scale and more general estimates of losses are provided in Book 3, Chapter 3. Wherever possible, local information on losses should be incorporated in analysis of stormwater runoff and associated designs of infrastructure.

More detailed hydrograph routing methods may be required for larger properties with complex land uses to design infrastructure for given performance standards, and to understand the behaviour of the stormwater management system. The need to manage inflows of groundwater and surface runoff to basements on some properties will also require volume based analysis to understand the extent of flooding and to design pump out infrastructure. Argue (2004) provides a range of simple methods for including volumes in the small scale design processes that are known as “regime in balance” and accounting for “emptying times” of storages.

Stormwater management strategies for larger or more complex properties should be designed or analysed using event based hydrograph routing methods that utilise storm burst patterns and pre-burst rainfall as inputs. The pre-burst rainfall, rainfall intensities and patterns of storm bursts for a given location can be downloaded from the ARR Data Hub http://data.arr-software.org/ and included in models of stormwater runoff. These rainfall inputs are provided in most proprietary software packages.

This modelling process includes details of different surfaces within sub-catchments that influence stormwater runoff to inlet structures within the property stormwater management network. The analysis should include the characteristics of pervious and impervious surfaces – such as initial and continuing losses, sub-catchment areas, slopes and details of overland flow paths. This approach is similar to the design and analysis process for public stormwater conveyance (street drainage) networks.

Use of ensembles of storm burst rainfall will ensure that the stormwater management system for a property is tested by a range of equally likely storm patterns and volumes of rainfall. This will permit a more complete understanding of potential surface flow paths within the property and in the adjacent street gutter, and the impacts on downstream infrastructure. However, use of complete storms or inclusion of pre-burst rainfall with the burst rainfall patterns will assist with defining the likely magnitude of overland flow behaviours at the property. Initial losses in the analysis may need to be set to zero if the magnitude of pre-burst rainfall is greater than the capacity of depression storages on the property. At some locations, the residual pre-burst rainfall may also contribute to additional runoff and overland flows within the property. These approaches can be combined in a range of computer modelling packages.

The ability of peak flow or event based models to describe runoff behaviours are limited in situations where the joint probability of antecedent conditions and storm events is not well defined (refer to Book 4, Chapter 3, Section 3) and there are continuous responses to complete storm events. These limitations apply to stormwater strategies that include volume storage measures, rainwater or stormwater harvesting, and water quality solutions.

In these situations, continuous simulation using real local rainfall or synthetic rainfall sequences can be utilised to test the continuous interactions between key components of the stormwater management. The results from continuous simulation can be directly interrogated to understand key performance criteria such as annual average reduction in water demand, stormwater runoff and nitrogen loads created by rainwater harvesting and raingardens. Alternatively, continuous simulation can provide distributions of available storage in volume management measures (such as rainwater tanks, infiltration measures and bioretention devices) or soil profiles prior to storm events versus frequency of storm events that can be used in event based analysis (Coombes and Barry, 2008b; Hardy et al., 2004) as shown, for example, in Figure 9.6.7.

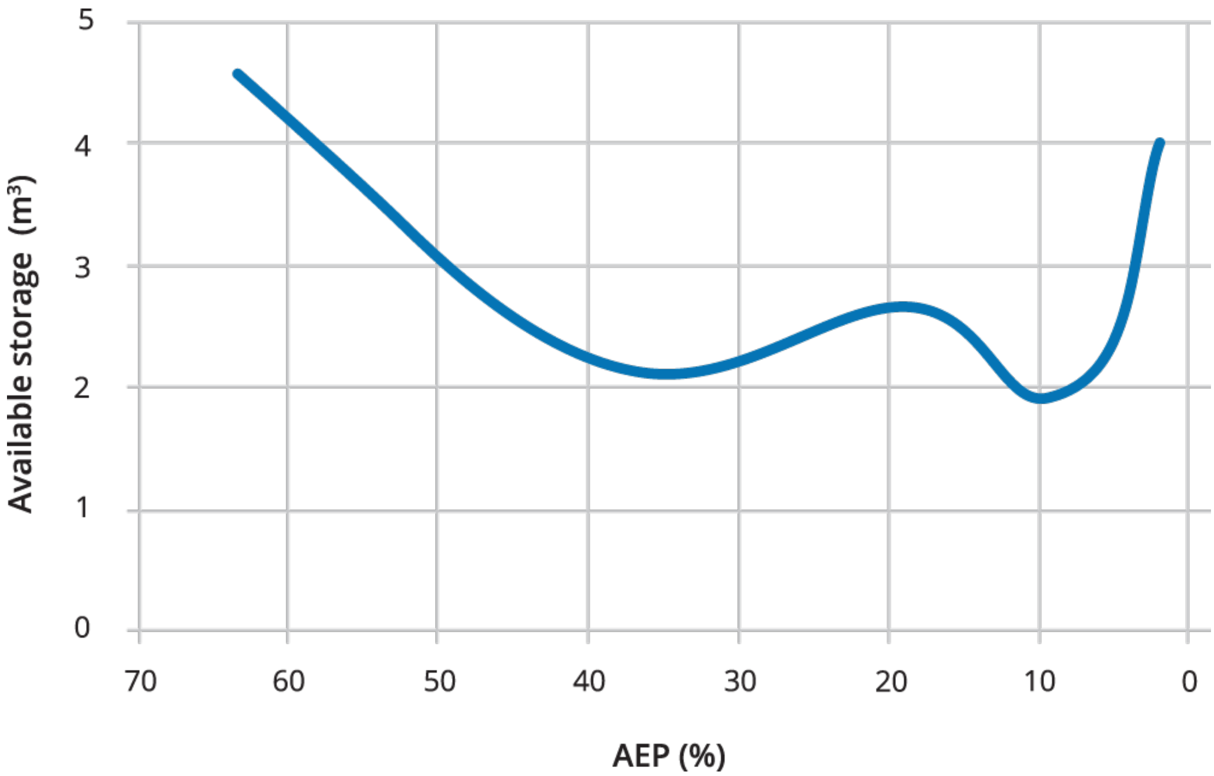


Figure 9.6.7. Example Distribution of Available Storage Prior to Storm Events versus Annual Exceedance Probability (AEP) of Storm Events

Figure 9.6.7 (for example,) demonstrates the average retention storage available in rainwater tank (capacity of 5 m^3 collecting runoff from a 100 m^2 roof area and supplying household indoor and outdoor uses) prior to storm events of a given AEP that was derived using continuous simulation. This type of information can be used in event based models to determine stormwater peak flows and runoff volumes. These results will vary significantly with different land uses, building form and throughout Australia.

6.4.4. Sub-Catchment Runoff to Inlet Structures

Sub-catchments define an urban area that discharges stormwater runoff to an inlet structure within a stormwater management network. There is further discussion of conveyance networks in Book 9, Chapter 5, Section 1 to Book 9, Chapter 5, Section 3 and of inlet structures in Book 9, Chapter 5, Section 5. The configuration and characteristics of the urban area within a sub-catchment will define the hydrological response that produces stormwater inflows to a stormwater network. These surface flows define the performance of an inlet structure as inflows to a conveyance network and as surface bypass flows. An example of a simple urban sub-catchment is provided in Figure 9.6.8.

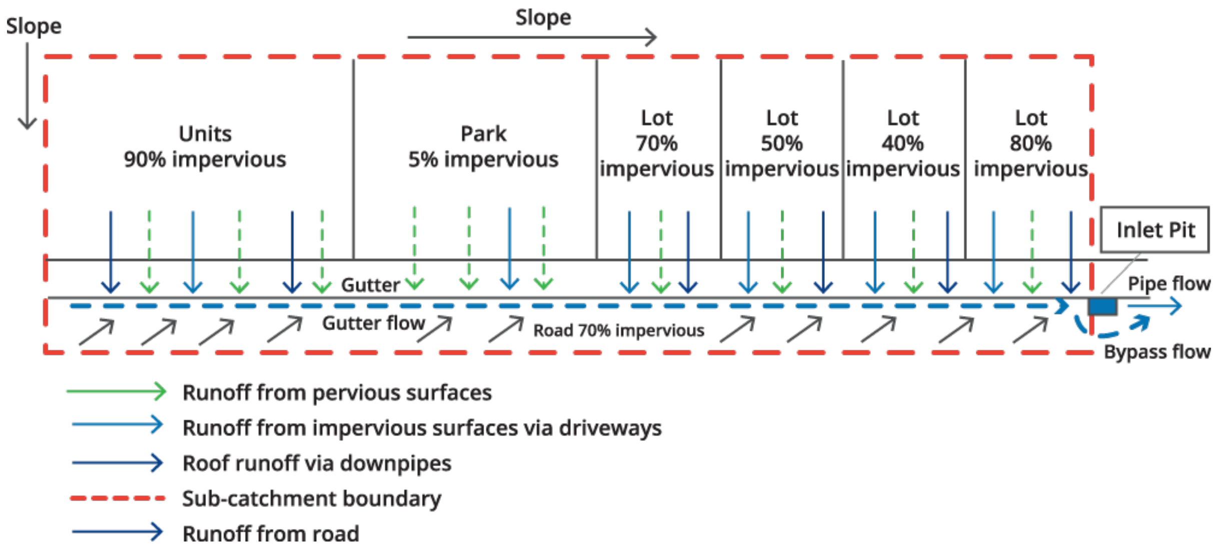


Figure 9.6.8. Example of a Simple Urban Stormwater Sub-Catchment

Figure 9.6.8 highlights that an urban sub-catchment may contain a range of different land uses, including (for example) unit and detached residential dwellings on properties, a park and part of a road. These land uses incorporate different surfaces, including roofs, paved areas (impervious), garden and grassed areas (pervious) that produce different regimes of stormwater runoff.

The behaviour of the sub-catchment surfaces can be estimated using lumped catchment approximations which are based on sub-catchment area, the total impervious area (TIA) and a travel time (or time of concentration) for a critical rainfall duration to the inlet structure (refer Book 5, Chapter 2, Section 2 and Book 9, Chapter 5, Section 5). For example, a sub-catchment area of 4,250 m^2 with an impervious proportion of 56% and time of concentration which depends on rainfall intensity, slope and distance to inlet structure. In the absence of other data, these types of approximations could be used in simple calculations or in computer models. However, it is preferable to construct analysis of urban sub-catchments using local details which can be sourced from site inspection, survey plans and inquiry using GIS.

Impervious or pervious surfaces can be directly connected or disconnected to inlet structures in conveyance networks. These surfaces may be also distant from the inlet structure or near the inlet. Thus the level of connectedness and distance of impervious areas from inlet structures should also be considered in analysis of stormwater runoff in urban areas. In addition, the analysis should account for surfaces that discharge to inlets via rapid conveyance mechanisms, such as street gutters, and for other surfaces that may discharge to inlets via slower conveyance processes such as across pervious surfaces (green spaces) or via storages.

An urban sub-catchment often includes depression storages, and a mosaic of different surfaces, runoff rates, storages and cumulative connectivity. The order of actions in the connectivity of different types of surfaces with storages to the inlet can dramatically change travel times and peak flows. Roofs may discharge via pipes to street gutters that facilitate rapid transfer of runoff volume to inlets. Runoff from road surfaces to the gutter may arrive at a similar or earlier time (refer Figure 9.6.8).

It is unlikely that lumped catchment approximations will provide reliable estimates of stormwater runoff from urban sub-catchments that include a range of different land uses and catchment storages with variable connectivity to inlets. Use of lumped catchment with TIA approximations may generate over-estimations of stormwater peak flows. Distributed methods of analysis may be more appropriate for ungauged catchments, where there are storages within catchments or for analysis using more robust runoff routing in computer models.

Urban sub-catchments include Directly Connected Impervious Areas (DCIA), Indirectly Connected Impervious Areas (ICIA) and pervious areas as described in Book 5, Chapter 3, Section 4. Limited regional investigations suggest that a combination of these effects produced Effective Impervious Areas (EIA) which are 55%-65% of the TIA of urban sub-catchments. Estimates of indirectly connected areas are further impacted by interactions between impervious and pervious areas, by storage in sub-catchments with Water Sensitive Urban Design (WSUD) measures and are influenced by Antecedent soil Moisture Conditions (AMC).

Other impervious surfaces may discharge via driveways to the street gutter which produces a different time for stormwater runoff to reach the inlet structure. Pervious surfaces also discharge to the street gutter, partially via impervious surfaces, to the inlet. Thus the timing of the arrival of runoff volumes to the inlet is dependent on these many different configurations and characteristics within the sub-catchment. So the performance of the inlet structure and the magnitude of surface bypass flows are dramatically affected by these considerations. These complex processes can be better described by semi-distributed (link-node) and distributed (grid) computer models (Book 5, Chapter 2, Section 4; Book 6, Chapter 4, Section 7) that explicitly combine these details with pre-burst rainfall and ensembles on burst rainfall patterns.

Regional analysis of a small number of urban catchments provides estimated initial losses of 1 – 3 mm for EIA and 20 – 30 mm for indirectly connected areas in sub-catchments (Book 5, Chapter 3, Section 5). Estimated median continuing losses were 2.5 mm/hour in South East Australia and 1 – 4 mm/hour elsewhere. These event based regional values should only be used in the absence of local data. It is essential that assumptions about losses in stormwater models are based on assessment of local conditions. The magnitude of losses is also impacted by AMC which is altered by garden watering in urban areas and by available storage in volume management measures throughout the sub-catchment. It is unlikely that event based models can fully account for these effects. Sensitivity checks, Monte Carlo processes and continuous simulation can be utilised to include the variation in AMC and available storage within urban sub-catchments.

Urban drainage was historically designed using peak flows derived using peak rainfall intensity or peak rainfall bursts in accordance with the assumption that peak flowrates only affect conveyance infrastructure. Many urban drainage networks are operating below anticipated service levels due to a range of impacts including increased density of urban areas. Analysis by Coombes et al. (2015a) indicates that the absence of stormwater runoff volumes in design processes based on peak runoff assumptions may partially explain under-performance of some urban drainage networks. The performance of inlet structures and therefore drainage networks can also be affected by the volume of stormwater arriving at the structure, variations in rainfall temporal patterns and by pre-burst rainfall that was not included in the design process. The uncounted volumes of stormwater runoff in peak flow and storm burst assumptions can become additional and unexpected overland or bypass flows in urban systems.

6.4.5. From the Inlet to the Outlet

Rainfall runoff from sub-catchments accumulates as inflows to conveyance networks or as surface flows throughout urban catchments that discharge towards an outlet (Book 9, Chapter 5). A network of conveyance infrastructure may incorporate pipes, open channels, roadways and open space. These networks often include water quality, volume management and flow control infrastructure (refer to Book 9, Chapter 4) that are incorporated in sub-catchment scale processes (such as source and neighbourhood controls: see Figure 9.5.1) or as regional controls at the outlet.

Analysis and design of stormwater management and flooding in urban areas was historically based on separate hydrology and hydraulic processes, and is focused at the network scale. A key objective of these processes was determination of flows in conveyance infrastructure such as pipes and open channels to avoid surcharges and bypass flows at inlets (refer to Book 9, Chapter 5, Section 5) to avoid nuisance, property damage and risk to life (refer to Book 9, Chapter 5, Section 2 and Book 6, Chapter 7). These urban conveyance networks include significant surface flows, usually along roads and through open spaces, from sub-catchments into and throughout conveyance networks. These flows from sub-catchments to inlets and within conveyance networks were determined as a hydrological process as an input to hydraulic models of conveyance networks (see Book 9, Chapter 5, Section 6and Book 5, Chapter 2).

The conveyance network is a framework of sub-catchment inputs. Urban stormwater design typically employed pipe network hydraulic models that utilise peak inflows or hydrographs as inputs (refer to Figure 9.5.18). More advanced one-dimensional models were also available that can be applied to simulation of conveyance networks (refer to Book 6, Chapter 4, Section 6; Book 5, Chapter 6 and Book 9, Chapter 5).

Overland or surface flows are a key consideration in analysis and design of urban stormwater management infrastructure. The dominant urban hydraulic response to rare rainfall events (such as 1% AEP) is often overland flows on roads and across open space. Emerging methods of analysis and design of urban stormwater involve combined hydrology and hydraulic models to better understand surface flows throughout urban catchments. These methods include coupled one and two-dimensional models, and direct rainfall (rainfall-on-grid) models. Book 6, Chapter 4, Section 7, and Babister and Barton (2012) provide detailed discussion about these approaches.

The flowrates, depth and area of surface flows in urban catchments are highly sensitive to different temporal patterns and volumes of rainfall (Babister and Barton, 2012). Similarly, Goyen (1981), Goyen (2000) and Coombes et al. (2015a) found that the performance of conveyance infrastructure also varies with temporal patterns and volumes of rainfall. It is recommended that ensembles of ten temporal patterns of design rainfall are used for investigation of the hydrology and hydraulic processes in urban areas. The separation of hydrologic and hydraulic routing is often blurred in analysis of urban areas which fosters complicated decisions around the use of hydrologic inputs and their interaction with hydraulic models. An overview of the difference approaches to rainfall inputs provided by this guideline is compared to ARR 1987 approach in Figure 9.6.9.

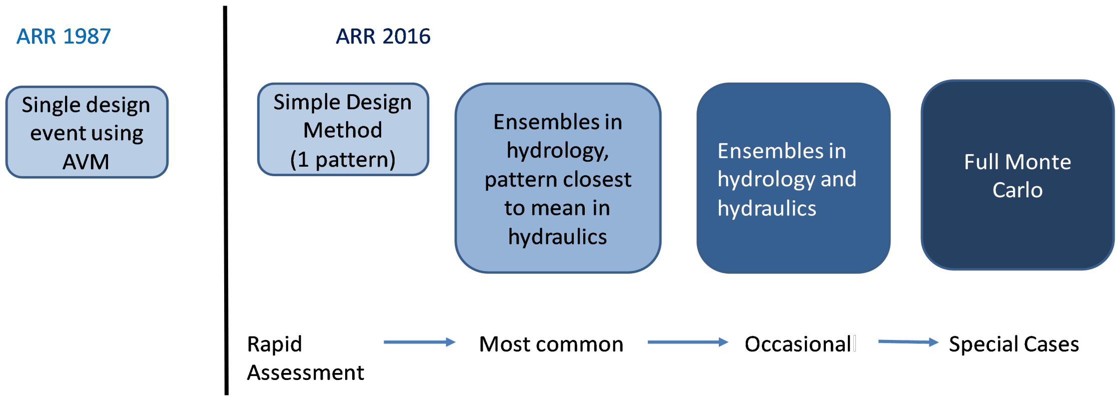


Figure 9.6.9. Changes in Design Modelling Techniques for Urban Areas

Figure 9.6.9 highlights that this guideline provides ensembles of 10 temporal patterns for each region that is a departure from the single event process supported by ARR 1987. These rainfall inputs can be used in hydrology and hydraulic modelling as required for different design and assessment tasks (refer to Book 2, Chapter 4 for further detail). The rapid assessment approach is not recommended for design of urban conveyance networks and the Monte Carlo processes can be used in special cases. It is expected that rainfall ensembles in hydrologic simulations, and in hydrologic and hydraulic simulations would be commonly utilised in urban conveyance networks. The process of using rainfall ensembles in hydrology is outlined in Figure 9.6.10.

Figure 9.6.10 shows that the inputs to analysis of the conveyance network include IFD information from the BOM, ensembles of rainfall temporal patterns, regional losses, pre-burst rainfall and Areal reduction factors from the ARR Data Hub. Wherever possible, local losses derived in accordance with Book 9, Chapter 6, Section 4 and Book 9, Chapter 6, Section 4 should be used in preference to regional losses for urban areas. These inputs are used in a hydrology model to generate ensembles of peak flows throughout the urban catchment for various storm durations and the required quantiles or AEPs of storm events. Mean peak flows are derived for key locations in the catchment and the rainfall temporal pattern that produces peak flows closest to the mean peak flows are utilised in the hydraulic model. This approach may be better suited to models with longer run times as considerable time can be expended determining critical durations in both hydrology and hydraulic models.

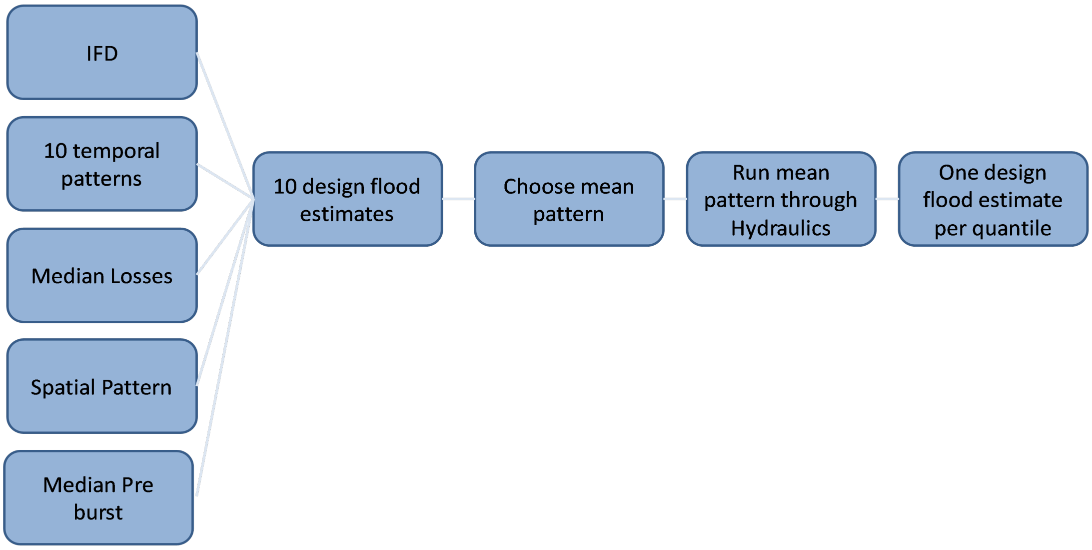


Figure 9.6.10. Design Process that Utilises Rainfall Ensembles in Hydrology to Select the Rainfall Pattern Closest to Mean Peak Flows for use in Hydraulic Analysis

The processes outlined in Figure 9.6.10 produce a single estimate of flood depth for each selected quantile or AEP. It is important to highlight that the critical rainfall duration and temporal pattern estimated using the hydrology model is likely to be different to the critical rainfall and temporal pattern relevant to the hydraulic simulations. These differences between critical hydrology and hydraulic inputs can have substantial impacts on the design of infrastructure and understanding of surface flows.

In situations where the hydraulic impacts of the design processes are significant, rainfall ensembles can be used in the hydrologic and hydraulic simulations as outlined in Figure 9.6.11.

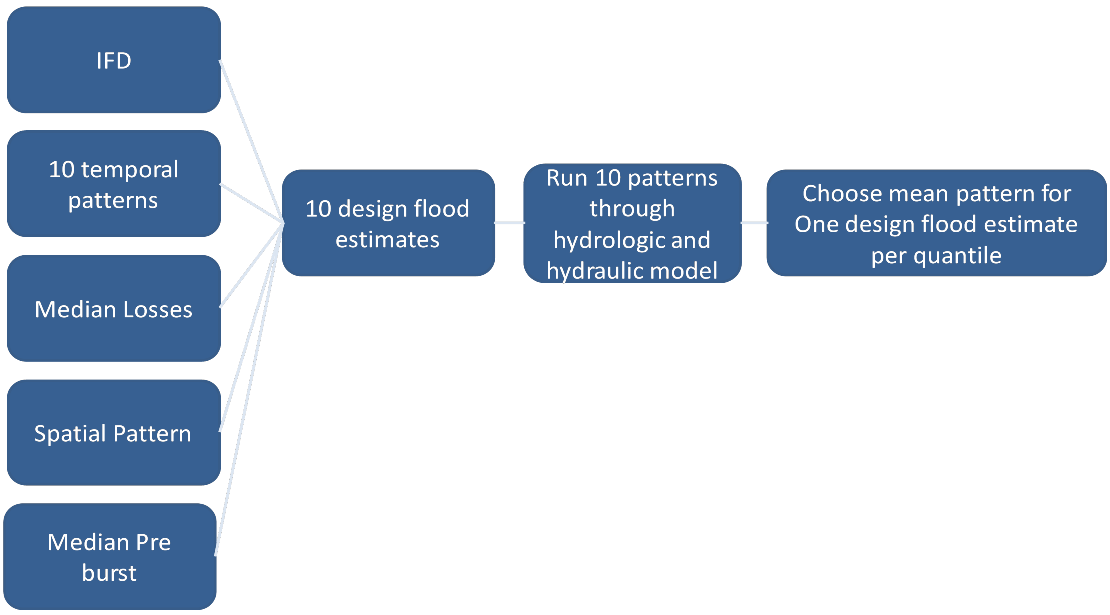


Figure 9.6.11. Design Process that Utilises Rainfall Ensembles in Hydrology and Hydraulic Simulations to Select the Mean Pattern for Analysis of Flooding

Figure 9.6.11 outlines that process for utilising ensembles of rainfall patterns in hydrology and hydraulic models. This process is better suited to situations where there are shorter model run times, critical flooding considerations and for coupled hydrology and hydraulic models. The processes outlined in Figure 9.6.10 and Figure 9.6.11 may also need to be applied to understand critical rainfall durations and patterns at key internal locations within catchments.

A brownfield case study based on the Woolloomooloo catchment in Sydney demonstrates the use of ensemble temporal patterns of rainfall and effects on the performance of hydraulic models used for design or assessment of conveyance networks (see Ward et al. (2018)). The catchment area is approximately 1.6 km^2 and has been heavily urbanised with limited open spaces or pervious areas. The catchment is also characterized by undulating terrain and contains known depression storages. The catchment drains to the harbour through a pit and pipe network, with the streets acting as overland flow paths (Figure 9.6.12).

Figure 9.6.12. The Woolloomooloo Catchment in Sydney

This guideline supports a number of modelling techniques and Table 9.6.1 and Table 9.6.2 provide guidance on selection of modelling approaches. Use of coupled 1D/2D and direct rainfall models were necessary to understand within catchment surface flows and flooding. The potentially short model run times and need to understand local flooding supports use of rainfall ensembles in both hydrology and hydraulics models. This study combined a well-known hydrology model with a popular 2D and 1D hydraulic model that relies on second order finite-difference schemes to simulate the hydrodynamics of floodplains and waterways.

This case study discusses three modelling options that were designed to account for within catchment overland flows and flooding:

- Use of a hydrology model to generate overland flows from small sub-catchments for use in a coupled 1D/2D hydraulic model. Individual properties, roofs and small area land surfaces were assigned as sub-catchments (refer to Book 9, Chapter 6, Section 4) in the hydrologic model to capture the rainfall concentration phase of stormwater runoff into the hydraulic model. This approach is necessary to understand within catchment flooding.

- A concentrated direct rainfall model where rainfall is applied to polygons of different land surfaces separated by perviousness and connectivity to the hydraulic 1D/2D model. These concentrated land surfaces also account for rainfall losses.

- Direct rainfall-on-grid where rainfall, after accounting for initial and continuing losses, was applied to all active grid cells. A fixed grid of $2m^2$ was employed in the hydraulic model.

Direct rainfall methods are known to trap volumes of rainfall in depressions and in areas with high roughness throughout 2D hydraulic models. The value of a carefully constructed direct rainfall model is the ability to identify sub-catchment flow paths, contributing areas and storage. However, the designer must ensure that catchment storages or initial losses are not doubled counted in simulations by the addition of regional loss assumptions. Given that there is a paucity of research into the accuracy of direct rainfall models, it is recommended that results of direct rainfall methods are compared with traditional methods by examination of the characteristics of hydrograph produced by both methods (Babister and Barton, 2012). A suitable method of representing buildings and good quality topography data is also required to produce accurate urban stormwater runoff behaviours. A mass balance or volume error check is also recommended.

The historical process of determining rainfall loss parameters using ARR 1987 assumptions, including soil type and antecedent moisture content parameters (AMC) from the ILSAX model, is provided in Table 9.6.6 for comparison.

Table 9.6.6. ARR 1987 Rainfall Loss Parameters

Parameter	Value
Paved Area Depression Storage (Initial Loss)	1.0 mm
Grassed Area Depression Storage (Initial Loss)	5.0 mm
SOIL TYPE	3
Slow infiltration rates. This parameter, in conjunction with the AMC, determines the continuing loss	
AMC	3
Description	Rather wet
Total Rainfall in 5 Days Preceding the Storm	12.5 to 25mm

This guideline provides a range of up-to-date parameters for use in analysis. The catchment is located within the East Coast South temporal pattern region. Temporal patterns for the East Coast South region and Intensity Frequency Duration (IFD) rainfall depths were downloaded from the ARR Data Hub website. This information is combined to construct ensembles of 10 rainfall patterns for the required flood quantiles (AEP). This case study focuses on the 1% AEP storm. The initial and continuing storm losses of 28 mm and 1.6 mm/hour for rural areas, and median pre-burst rainfall of 1.1 mm associated with a one hour 1% AEP storm event can also be downloaded from the ARR Data Hub.

It is recommended that varied rainfall losses are applied to different types of surfaces in the catchment. These surfaces include urban pervious areas such as parks, and impervious areas such as roads, median strips and building roofs. The identified impervious areas were split up into Effective Impervious Area (EIA) and Indirectly Connected Impervious Area (ICIA).

Effective Impervious Area represents the portion of a catchment area that has an impervious response. Due to the highly urbanised nature of the catchment this portion was identified as 75% of the total impervious area. The remaining area that is not classified as Effective Impervious Area is Indirectly Connected Impervious Area (25%). Building roofs were identified separately as Indirectly Connected Impervious Area as the down pipes were not assumed to directly discharge into the storm water pipes. The information from the ARR Data Hub is modified by loss values for urban catchments that are provided in Book 5, Chapter 3 and in Book 9, Chapter 6, Section 4 as summarised in Table 9.6.7.

Table 9.6.7. ARR 2016 Rainfall Loss Parameters for Urban Areas

<table>
<tr><th>Urban Area</th><th>Storm Initial Loss (mm)</th><th>Continuing Loss (mm/hr)</th></tr>
<tr><td>Effective Impervious Area</td><td>1 – 2 mm</td><td>0</td></tr>
<tr><td>Indirectly Connected Area</td><td>60 to 80% of rural catchment losses</td><td>For south eastern Australia, a typical value of 2.5mm/h, with a range of 1 to 3 mm/h, would be appropriate. This value should be adjusted based on engineering judgement and reviewing the catchment characteristics such as soil types, interaction of indirectly connected impervious areas with pervious areas.

For other areas, adopt a range of 1 to 4 mm/h.</td></tr>
<tr><td>Urban Pervious Area</td><td colspan="2">Traditionally, designers have adopted similar loss values for these areas as for those they would adopt in rural areas.</td></tr>
</table>

In event based modelling approaches, it is important to subtract pre-burst rainfall from local losses associated with impervious and pervious surfaces as follows:

Burst initial loss = Storm initial losses – Pre-burst rainfall (for Burst initialloss ≥ 0)

For example, the burst initial loss for effective impervious area is 1.5 – 1.1 = 0.4 mm. The adopted burst losses for the urban surfaces are presented in Table 9.6.8. Note that in a situation where pre-burst rainfall is greater than the storm initial losses, the residual pre-burst rainfall should be included in the analysis.

Table 9.6.8. Adopted ARR 2016 Rainfall Loss Parameters

Urban Surface	Burst Initial Loss (mm)	Continuing Loss (mm/hr)
Effective Impervious Area	0.4	0
Indirectly Connected Area	16.1	1.6

Urban Surface	Burst Initial Loss (mm)	Continuing Loss (mm/hr)
Urban Pervious Area	26.9	1.6

Hydraulic and associated flood behaviour is influenced by the hydraulic resistance due to topography and urban form. The selection of appropriate roughness coefficients is critical to the success of this approach (see Book 6, Chapter 4). Depth varying Manning's "n" roughness parameters were selected for each land use to account for shallow overland flow depths across urban surfaces. Some hydraulic modelling packages provide this capability in accordance with emerging research into depth varying roughness (for example, Zahidi et al. (2017), Khrapov et al. (2015), Muglera et al. (2011)).

Analysis of the performance of urban conveyance networks is critically dependent on potential blockage of inlet structures (Book 6, Chapter 6; Book 9, Chapter 5, Section 5) and the need to address safety design criteria (see Book 6, Chapter 7; Book 9, Chapter 5, Section 3). Assessment of potential blockage of inlet structures should also consider data from local authorities about maintenance programs and local flooding (Weeks et al., 2013). The assumed blockage factors for inlet pits subject to runoff from 1% AEP rainfall events were derived from Book 9, Chapter 5, Section 5, from local government historical records, and maintenance programs (see Table 6.6.1), from Weeks et al. (2013) and are presented in Table 9.6.9.

Table 9.6.9. Assumed Capacity of Inlet Pits for 1% AEP Rain Events

Sag Inlet Pit	
Kerb Inlet	80%
Grated Inlet	50%
Combination	Assume Grate 100% blocked
On-grade Inlet Pit	
Kerb Inlet	80%
Grated Inlet	60%
Combination	90%

The critical rainfall duration for the catchment was derived using the ensembles of rainfall temporal patterns in the combined hydrology and 2D hydraulic model to reveal the highest mean and median flood elevations at key locations as shown for in Figure 9.6.13.

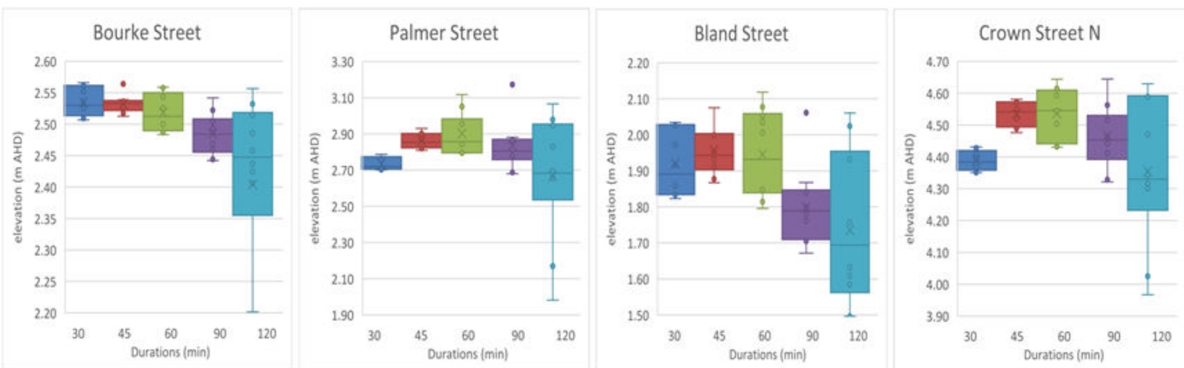


Figure 9.6.13. Use of Ensembles of Storm Bursts (1% AEP) in the Hydraulic Model to Select Critical Duration

Figure 9.6.13 reveals that the use of ensembles of rainfall in the hydraulic model indicates that different critical rainfall durations apply throughout the catchment. The results from Figure 9.6.13 were used with consideration of the characteristics of the catchment to select the critical storm duration of 60 minutes. The impact on stormwater runoff from using the single storm burst pattern from ARR 1987 is compared to use of an ensemble of ten storm burst patterns (1% AEP) from this guideline for part of the Wooloomooloo catchment in Figure 9.6.14. This graph presents ten hydrographs of stormwater runoff in the trunk drainage system at Bourke Street confluence.

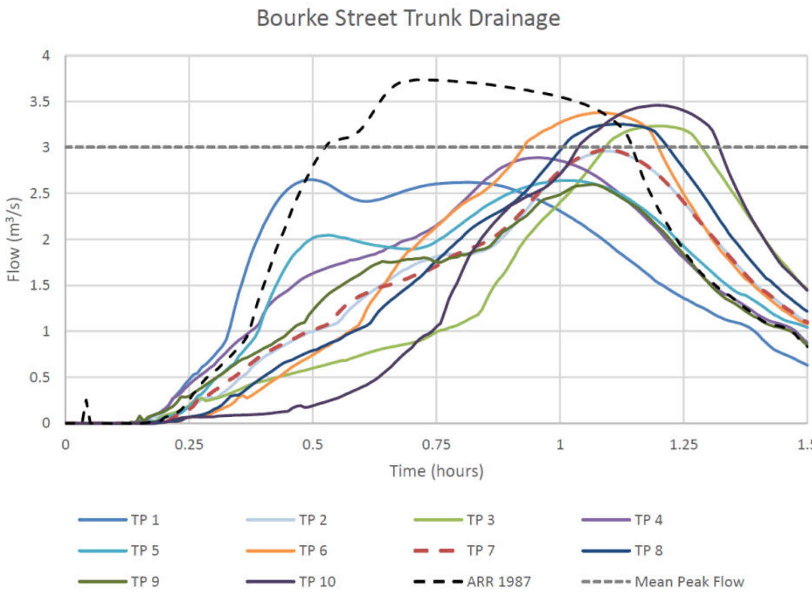


Figure 9.6.14. Example of Runoff from ARR 1987 Single Storm Burst and Ensembles of Storm Bursts from this guideline (1% AEP)

Figure 9.6.14 demonstrates that a single pattern of burst rainfall from ARR 1987 produces a different hydrograph shape, volume and peak runoff at the catchment outlet to the ensemble of storm burst patterns from this guideline. This difference is driven by the 30 years of additional data and science available to this guideline that has allowed the derivations of more spatially relevant rainfall and temporal patterns. The variability of the equally likely storm burst patterns from the ARR ensembles facilitates testing of catchment characteristics for generation of maximum runoff.

The direct rainfall method applies rainfall directly to all grid cells and the scale of routing is at every 2 m by 2 m grid cell. In this approach the depth of flow is shallow and rainfall can get stuck on the model grid. To maintain the area of rainfall applied to the grid, the buildings were nulled (removed) from the actual grid and rainfall was scaled up to account for the lost building areas.

The concentrated direct rainfall method applied rainfall to polygons of different local surfaces such as buildings and parks. This process permits the specification of the area, initial and continuing losses that are applied to each land use polygon. Separate attributes are applied to roofs to account for the different connectivity to concentrated stormwater flows.

A manual volume check should be undertaken on all direct rainfall model configurations. The volume of water leaving the model through the downstream boundary should be equal to the amount of water that was applied (via direct rainfall and inflows across external boundaries), less losses and storages within the model. The upper portion of the catchment (area of 52.8

Ha) was assessed to maximize the volume of water that drains from the catchment at the last time step. The characteristics of the upper catchment are shown in Table 9.6.10.

Table 9.6.10. Characteristics of the Upper Catchment used in the Volume Check

Type	Catchment Area (m^2)	IL (mm)	CL (mm/hr)
100% Pervious	22,508	26.9	1.6
100% Impervious	102,607	0.4	0.0
EIA	133,909	0.4	0.0
ICIA	34,549	16.14	1.6
ICIA (Buildings)	234,630	16.14	1.6
AVERAGE LOSS		**11.4**	**0.9**
TOTAL (m^2)	**528,202**		

The model run was extended to allow all stormwater to drain from the catchment by extrapolating the outflow curve towards zero. Inflow volume was calculated as the cumulative depth of rainfall less initial and continuing losses multiplied by the area of the catchment. Flows extracted from the hydraulic model 1D results can also be converted into volumes. A flow line along the upstream catchment divide together with outflow boundaries were used in the 2D hydraulic model to also account for the volume of overland flows leaving the catchment. These results can be presented as a cumulative depth graph or as a pie chart (refer to Figure 9.6.15)

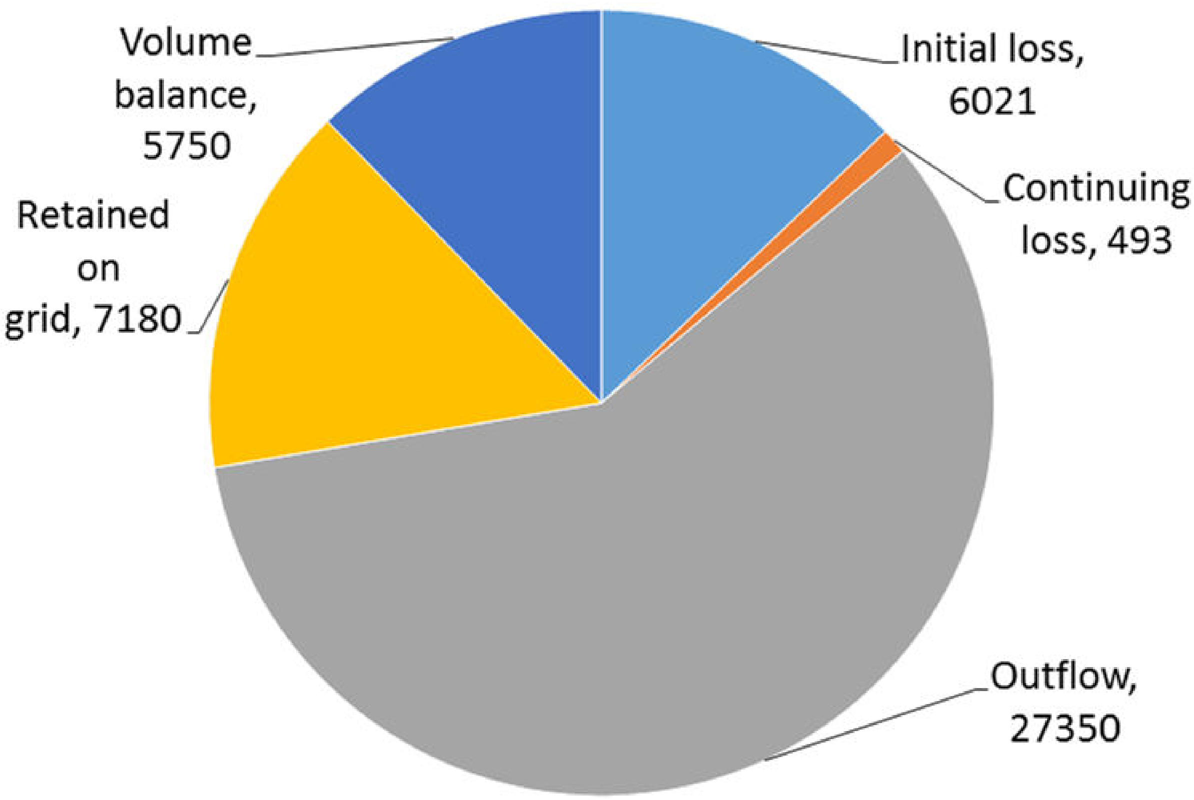


Figure 9.6.15. Upper Catchment Volume Check for Direct Rainfall Model (Prior to Corrections)

Figure 9.6.15 shows 5,750 m^3 (14%) of rainfall was retained in the model (11 mm) which is described as the volume balance. An acceptable error or additional retention of stormwater is less than 5% which indicates a need to reduce initial losses used in the direct rain model. Accounting for volumes of depression storage in the catchment topography by decreasing initial rainfall losses will increase in overall pipe and overland outflows. These results indicate that the catchment topography includes depression storages that capture 23.1 mm of rainfall. The results from coupled hydrology and 1D/2D hydraulic model with traditional loss assumptions revealed rainfall losses of 24.3 mm. The concentrated direct rainfall and direct rainfall methods can also be evaluated using sensitivity testing of initial conditions as follows:

- No accounting for rainfall lost to depression storage;
- Accounting for depression storage loss by reducing the initial loss. Apply direct rainfall with initial loss, less the average depth on grid;
- Accounting for depression storage using a restart file, which reapplied the conditions from the last time step to the model. Direct rainfall applied with the initial conditions adopted from the final time step of the initial simulation.

The outflow depths in the standard direct rainfall simulations changed from 52 mm to 61 mm by using a restart file and in the standard direct rainfall simulations changed from 52 mm to 56 mm by reducing the assumed initial losses in the models.

The hydrograph outputs of overland flows at selected locations (refer to Figure 9.6.12) at Riley Street near the park (top pane) and at Crown Street North (bottom pane) are shown in Figure 9.6.16. It is clear that overland flow is under-represented in the uncorrected direct rainfall models as compared to traditional coupled 1D/2D models.

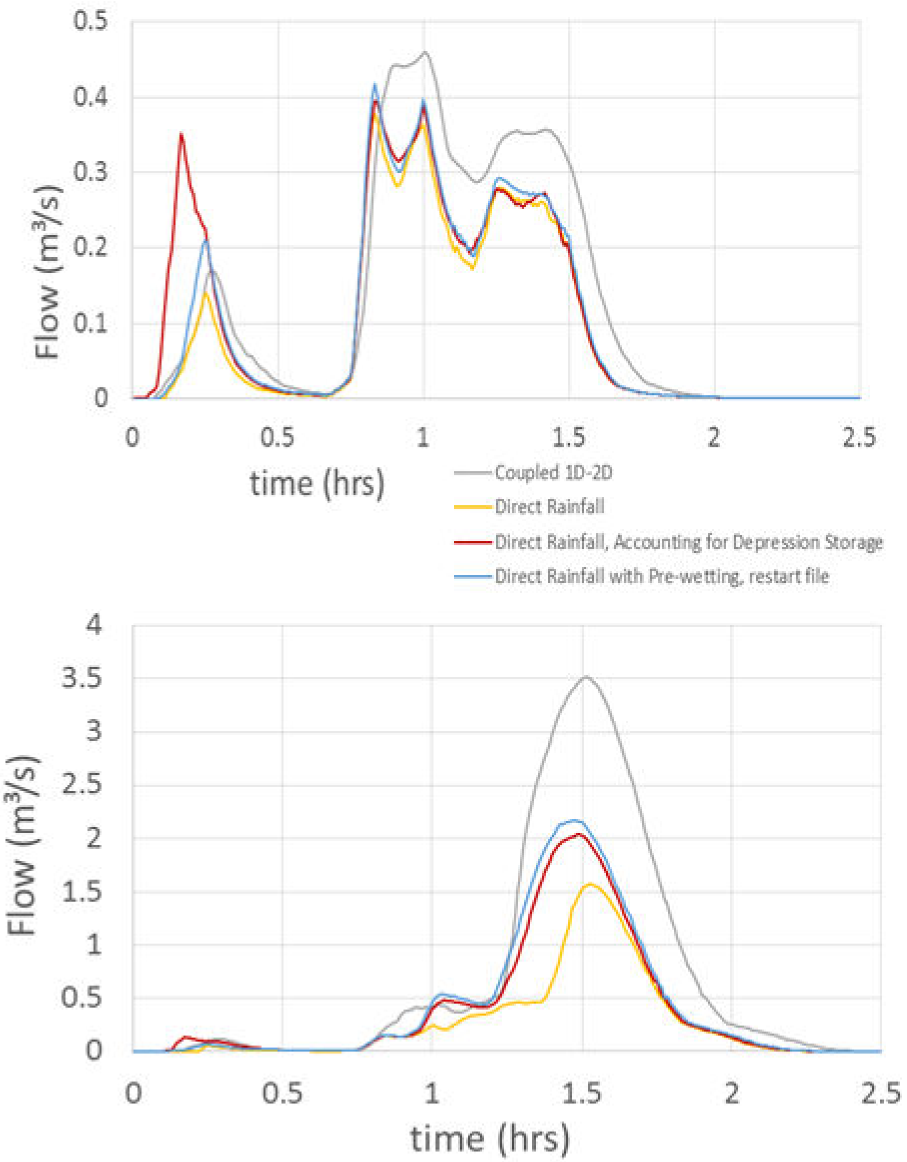


Figure 9.6.16. Comparison of Treatment of Initial Conditions in Overland Flows Generated by Coupled Direct Rainfall Models Near the Top of the Catchment (Top Pane: Riley Street) and Near the Bottom of the Catchment

Figure 9.6.16 demonstrates that the uncorrected direct rainfall models produce variable under-estimation of surface flows, as compared to a traditional coupled 1D/2D model, that is dependent on location and attributes of sub-catchments. Techniques that account for depression storage or pre-wetting of the catchment surfaces using restart files can improve the comparative performance of direct rainfall models. However, the residual differences in surface flows highlight that 2D models and in particular direct rainfall models should also be verified using historical records of local flood depths. Surface flows are a significant proportion of the responses from urban catchment as shown in Figure 9.6.17.

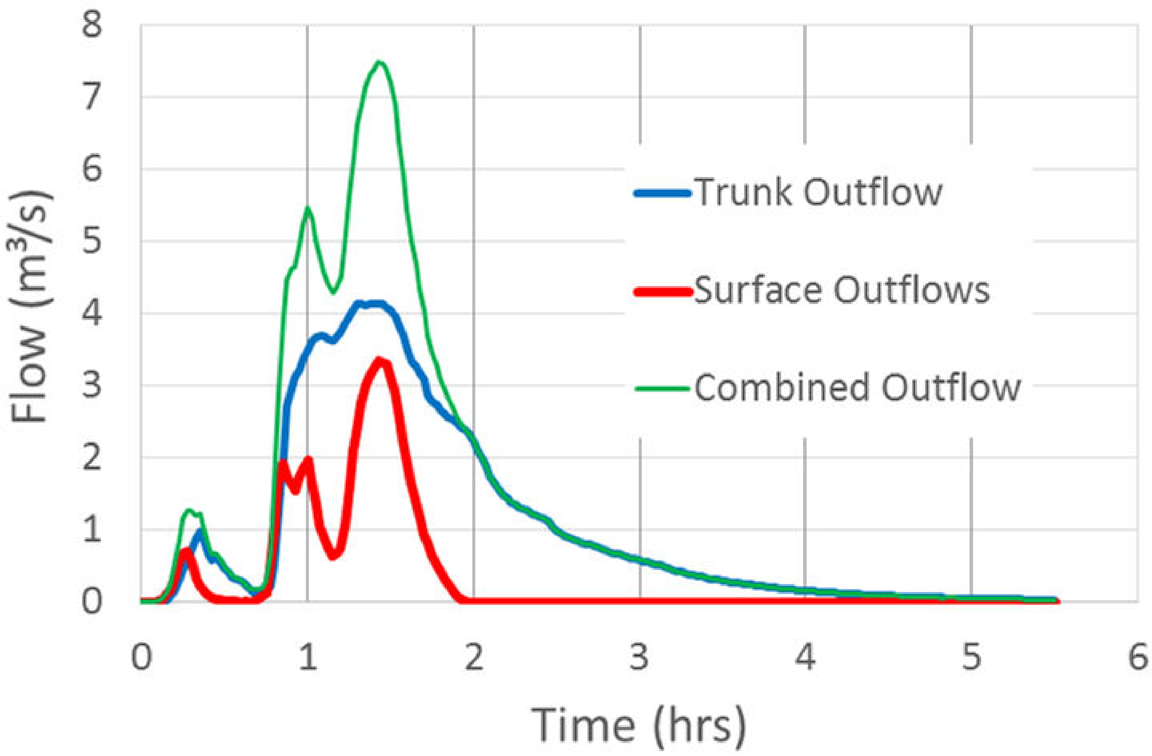


Figure 9.6.17. Outflow Hydrographs Catchment Showing the Significance of Surface Flows

This case study demonstrates practical application of the ARR ensemble temporal patterns on an urban catchment that is dominated by overland flow. The pattern that best represents the mean response has been selected based on flood elevation rather than flow. It is clear from the results of this analysis that a volume check of direct rainfall approaches should be undertaken in accordance with recommendations of Babister and Barton (2012) and the results should be verified using historical records of spatial flooding. A significant amount the rainfall excess is not generating runoff because rainfall is trapped on the terrain grid. This trapped rainfall excess represents an effective overestimation of the catchment loses with associated underestimation of surface flow and should be factored into the losses so that the correct amount of rainfall excess is generated. This can be carried out by either pre-wetting parts of the catchment or adjusting the assumed initial losses or a combination of both.

6.4.6. Downstream

Outflows from urban sub-catchments and conveyance networks interact with regional storage controls and water quality measures (refer to Book 9, Chapter 4 and Book 9, Chapter 5), discharge to urban waterways (See Book 9, Chapter 2and Book 9, Chapter 3) and to receiving waters such as estuaries, rivers, bays and oceans. The methods outlined in Book 6, Chapter 5 may need to be applied to interactions of rainfall and storm surge processes in estuaries, bays and oceans to account for combined impacts on urban flooding.

The complexity of urban areas also fosters the need to consider the joint probability of the different factors such intersection of urban runoff with regional flows in rivers or water levels in regional storages and water quality measures, which may be correlated or independent of each other. Methods to account for joint probability are provided in Book 4, Chapter 4. The urban designer should also consider climate change impacts on urban flooding as outlined in Book 8, Chapter 7, Section 7 and Book 1, Chapter 6.

The connectivity between design of urban conveyance and a volume management facility, setting the rural base case for design targets, application of climate change and assessment of downstream impacts on a sensitive waterways is combined in a greenfield example (Coombes and Barry, 2018). This conceptual design example is located near Ballarat in Victoria and includes an objective of no increase in peak flows in the downstream natural waterway to mitigate impacts of the urban development on erosion of the stream. The pre-development catchment is shown in Figure 9.6.18 and the proposed development is presented in Figure 9.6.19.

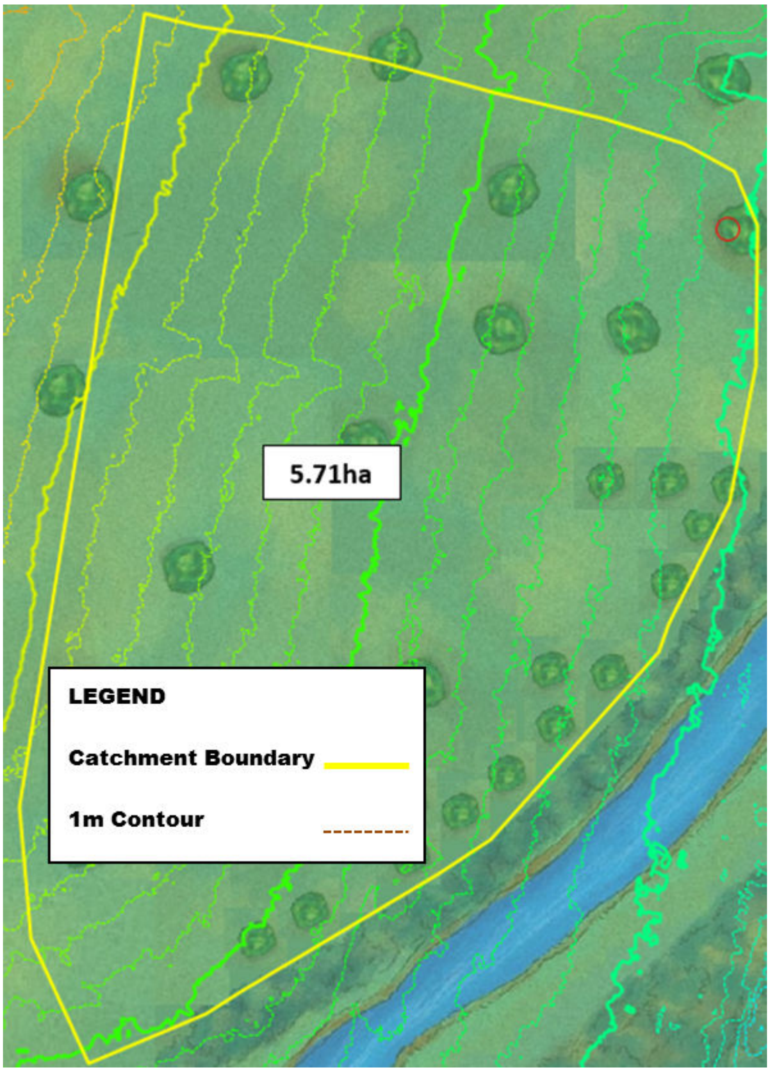


Figure 9.6.18. Catchment prior to development

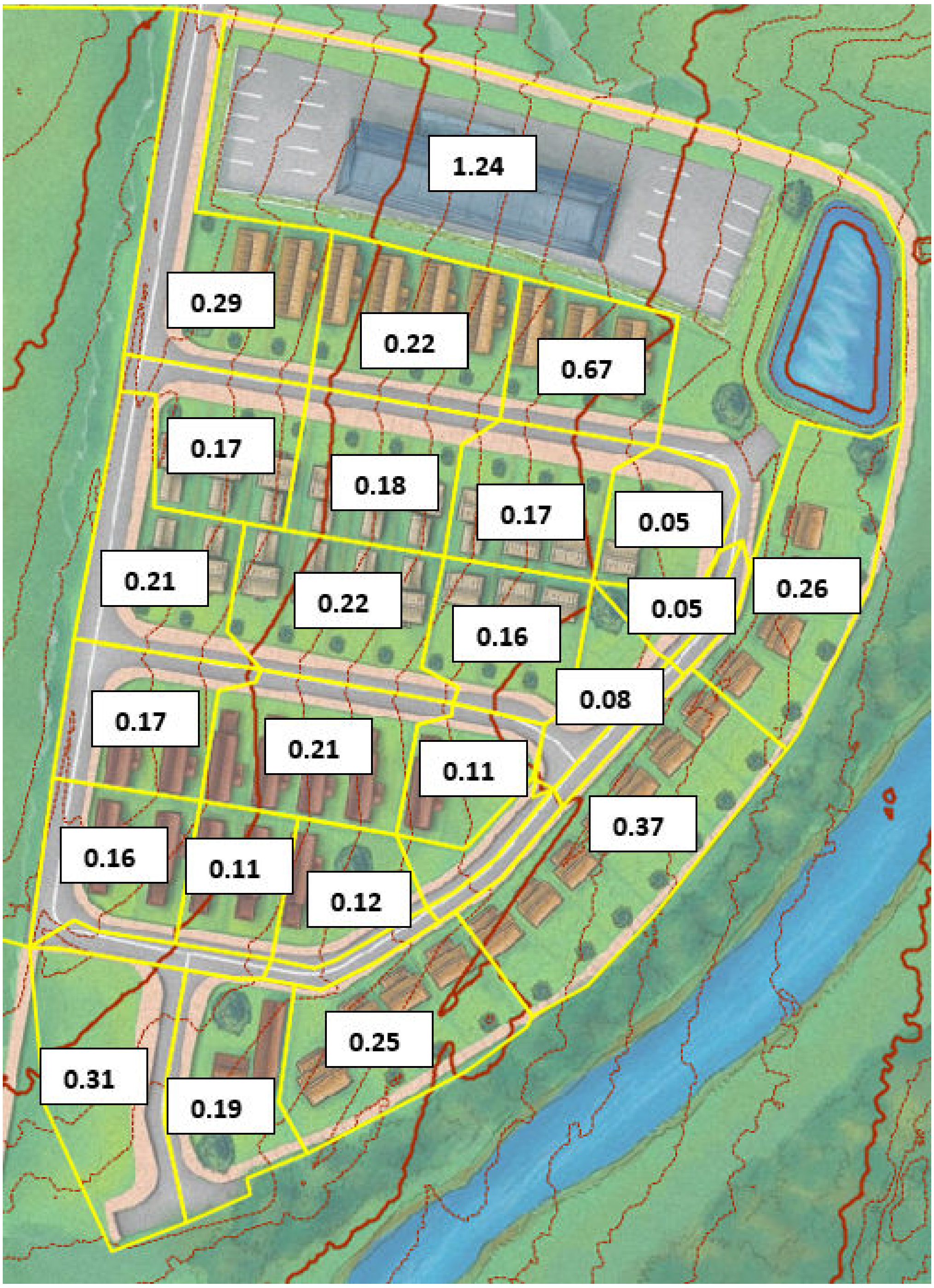


Figure 9.6.19. Developed Catchment

An estimate of pre-development peak flows was required to set the design peak flow targets for the proposed urban development. The Regional Flood Frequency Estimation Model (RFFE) available from http://rffe.arr-software.org/ was utilised to estimate rural peak flows with uncertainty as shown in Figure 9.6.20 which is based on gauged flows from multiple regional gauges (Figure 9.6.21). The use and limitations of the RFFE is described in Book 3, Chapter 3. Whilst the example catchment size is less than the currently recommended minimum and the RFFE is subject to improvement, this process provides a good starting point for defining the rural flow target. The rural flows from the RFFE might also be combined with statistical analysis of observed flows in a nearby catchment using FLIKE (refer to Book 3, Chapter 2, Section 8) to improve regional flow estimates. These improved regional peak flow results from the nearby catchment can be used to calibrate a hydrology model and the parameters transferred to the design catchment as explained by Coombes et al. (2016). Patil and Stieglitz (2012), for example, outline methods of transferring parameters from gauged catchments to ungauged catchments.

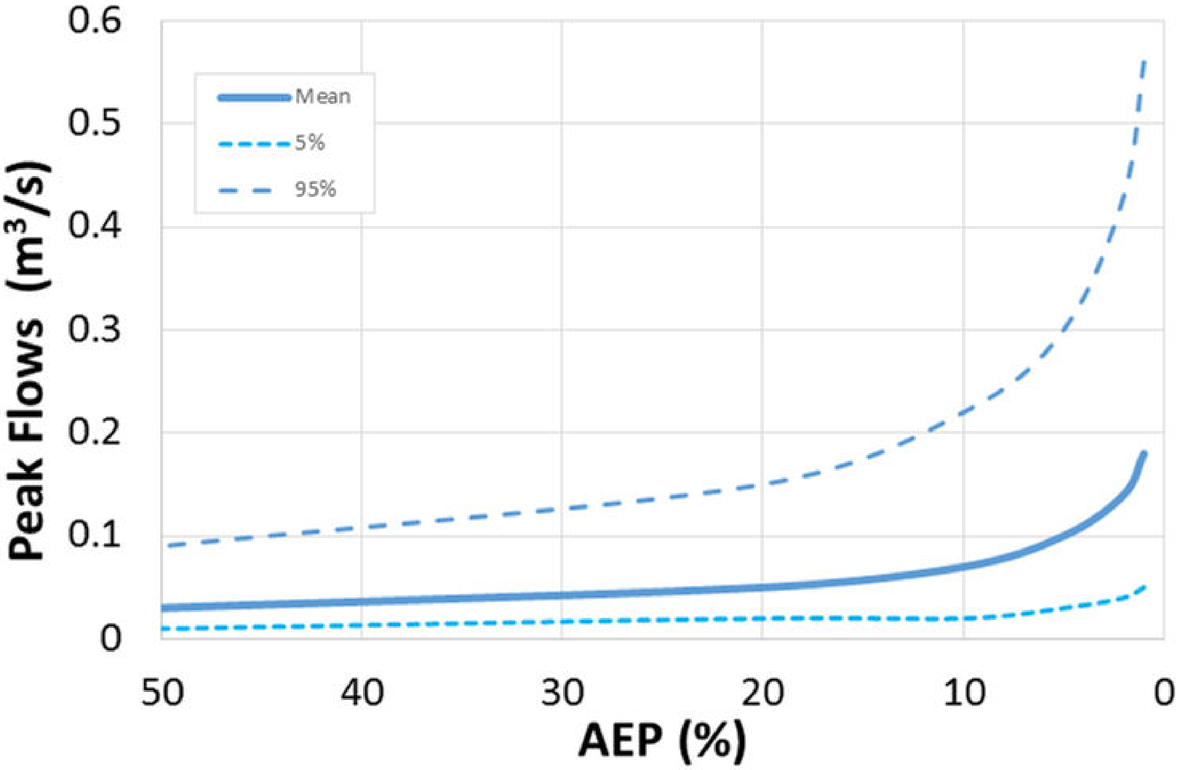


Figure 9.6.20. Estimated Rural Peak Flows using the RFFE

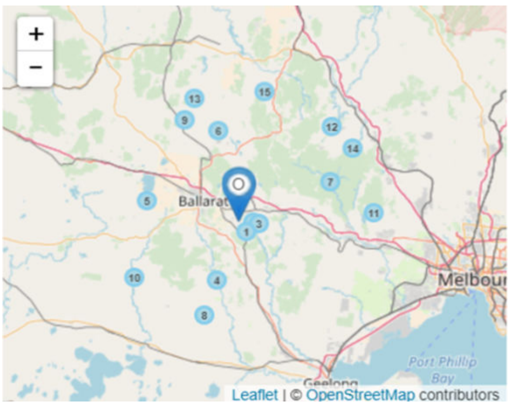


Figure 9.6.21. Regional Flow Gauges used in the RFFE Estimate of Rural Peak Flows

The next step in the design process involved selecting the project location in the ARR Data Hub and downloading hydrology and rainfall information, including local design rainfall IFD and ensembles of temporal patterns. Most proprietary models will download this information and set up the ensembles of rainfall inputs. Estimated regional rural losses for initial losses (IL) of 25 mm and continuing losses (CL) of 4.3 mm/hr were also downloaded from the ARR Data Hub.

A model with combined hydrology and hydraulic capacity was used with initial estimates of IL = 25 mm and CL = 4.3 mm/hr, design burst rainfall ensembles and pre-burst rainfall to estimate local rural losses that were calibrated to rural flows sourced from the RFFE as shown in Figure 9.6.22. The critical duration was found to be 1.5 hours as defined by highest mean peak flows for 50%, 10% and 1% AEP events as shown in Figure 9.6.23, Figure 9.6.24 and Figure 9.6.25. Median pre-burst rainfall for 90 minute storm durations were also selected from the ARR Data Hub for 50% AEP: 4.1 mm; 10% AEP: 3.3 mm and 1% AEP: 1.1 mm. The pre-burst rainfall was included in the hydrology model and spread over the hour prior to burst rainfall and the calibration processes aimed to find values of IL and CL that produced simulated rural peak flows that were similar to RFFE peak flows for the 10% AEP events. This process enabled an estimate of local rural initial losses of 16 mm and continuing loss of 5 mm/hr for an assumed Mannings roughness coefficient (n = 0.075).

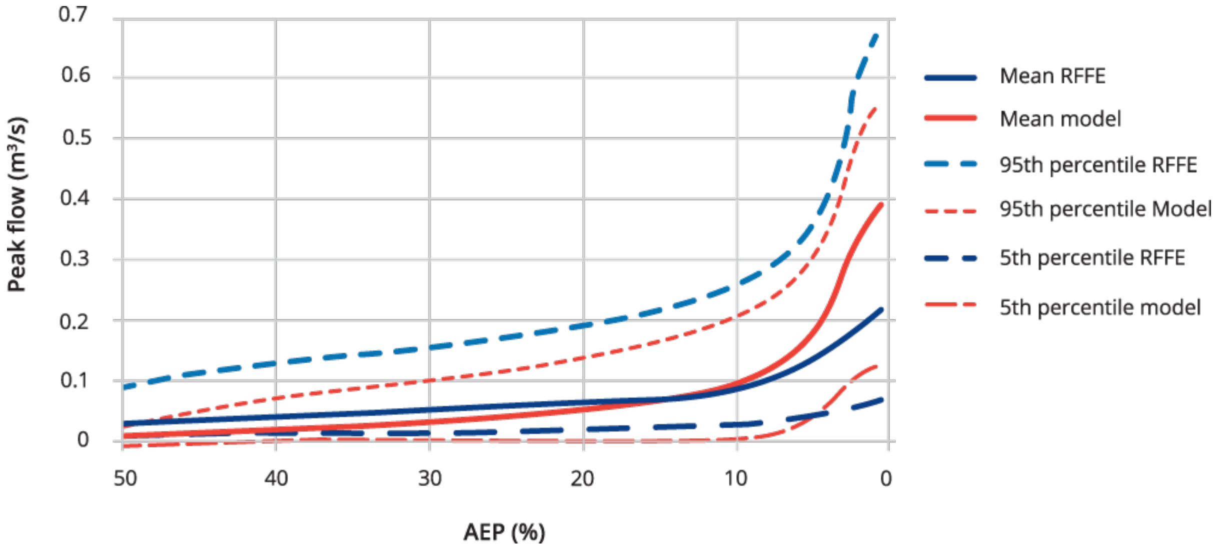


Figure 9.6.22. Calibration of Rural Flows to RFFE Flow Estimates

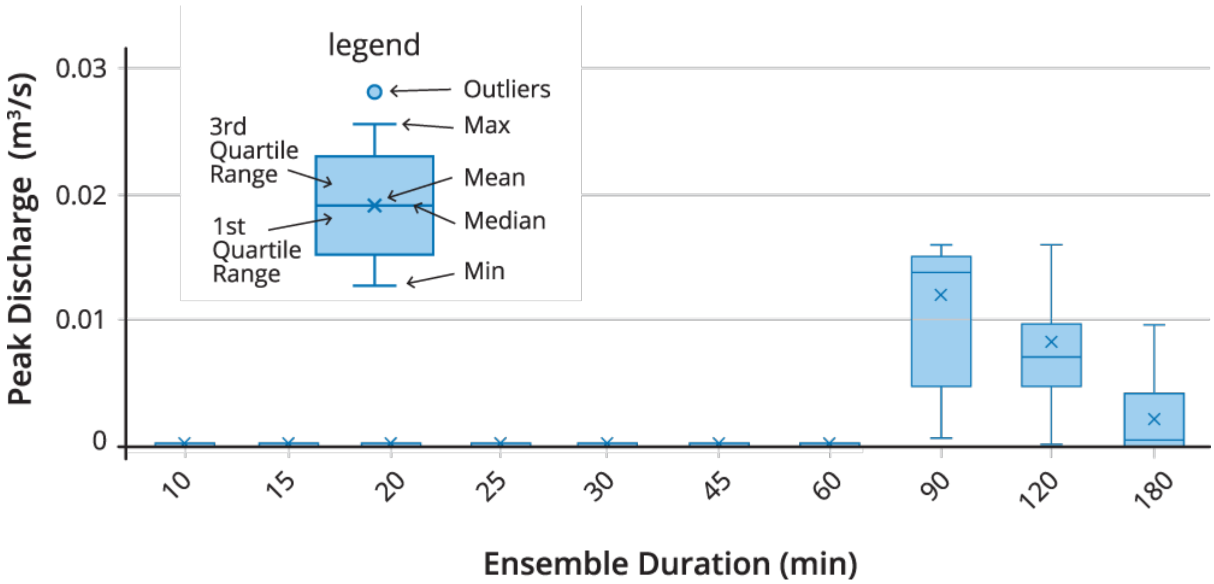


Figure 9.6.23. Pre-Development Peaks Flows for 50% AEP Events

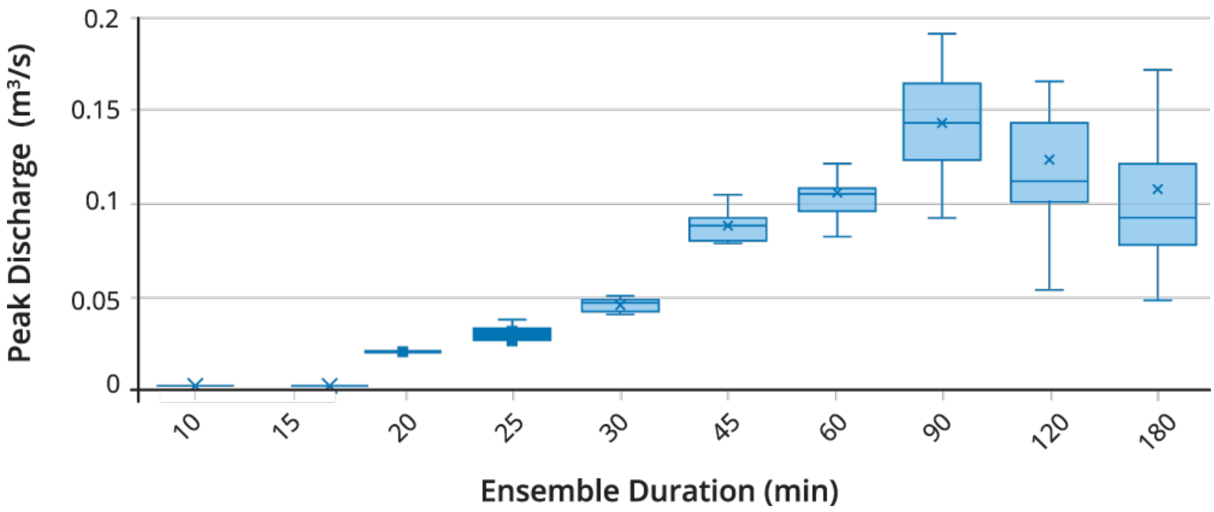


Figure 9.6.24. Pre-Development Peak Flows for 10% AEP Events

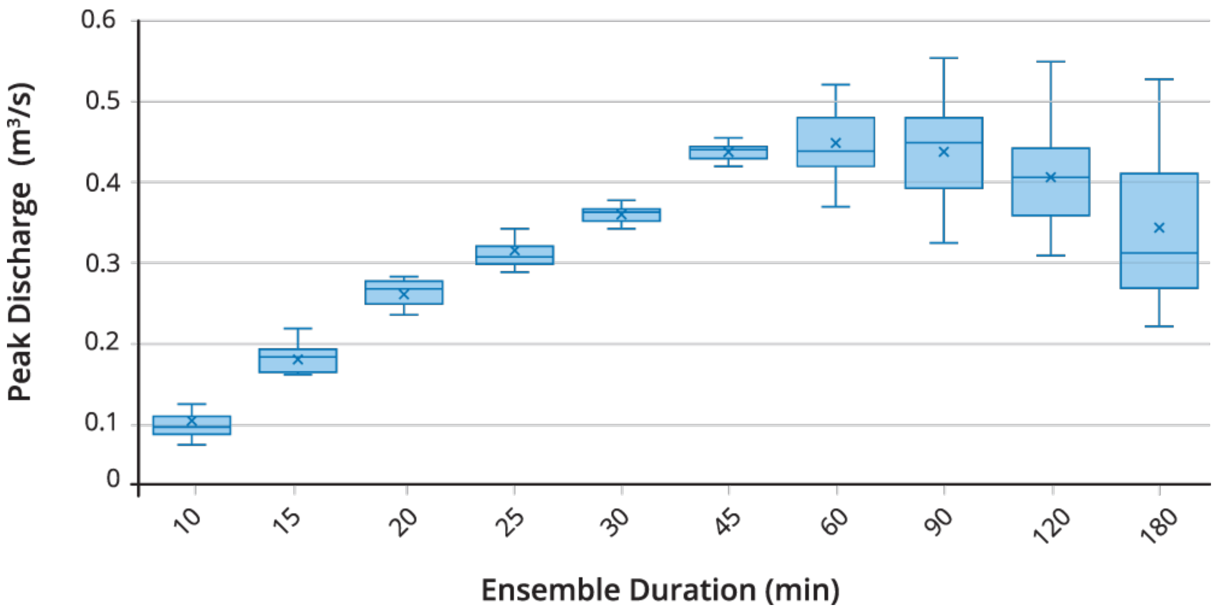


Figure 9.6.25. Pre-Development Peak flows for 1% AEP Events

The mean maximum pre-development peak flows for the 50%, 10% and 1% AEP were found to be 0.011 m^3/s, 0.14 m^3/s and 0.45 m^3/s respectively. These values were used as the peak flow targets for the urban development. The altered land surfaces (impervious and pervious areas of roads and properties) associated with the urban development was included in the hydrology model. The loss values for the urban catchment from Book 5, Chapter 3 and Book 9, Chapter 6, Section 4 were assigned as follows:

- Effective Impervious Area: IL =1.5 mm, CL = 0 mm/hr
- Pervious Area = rural losses

Indirectly connected impervious area assumptions were not required because the spatial detail of land uses with associated connectivity were included in the hydrology/hydraulics model. The hydrology of the urban catchment was simulated for all design rainfall ensembles to determine a critical duration of 10 minutes for the 10% AEP flows relevant to the design of pit and pipe conveyance infrastructure (refer to Book 9, Chapter 5). These simulations were completed prior to design of infrastructure to determine the relevant critical duration and design storm for use in the design process. Pre-burst rainfall for a one hour duration was selected from the ARR Data Hub (50% AEP: 2.2 mm, 10% AEP: 2.2 mm, 1% AEP: 0.8 mm) for use with the 10 minute duration design rainfall ensembles relevant to the design of the pit and pipe conveyance infrastructure. The pre-burst rainfall was distributed across an hour prior to the burst rainfall.

The hydrographs from the simulation using ensembles of 10% AEP design burst rainfall with pre-burst rainfall was examined to select the design storm closest to mean peak flow for design of conveyance infrastructure as shown in Figure 9.6.26. Urban peak flows from all design rainfall durations for 50%, 10% and 1% AEP events are presented in Figure 9.6.27, Figure 9.6.28 and Figure 9.6.29 respectively.

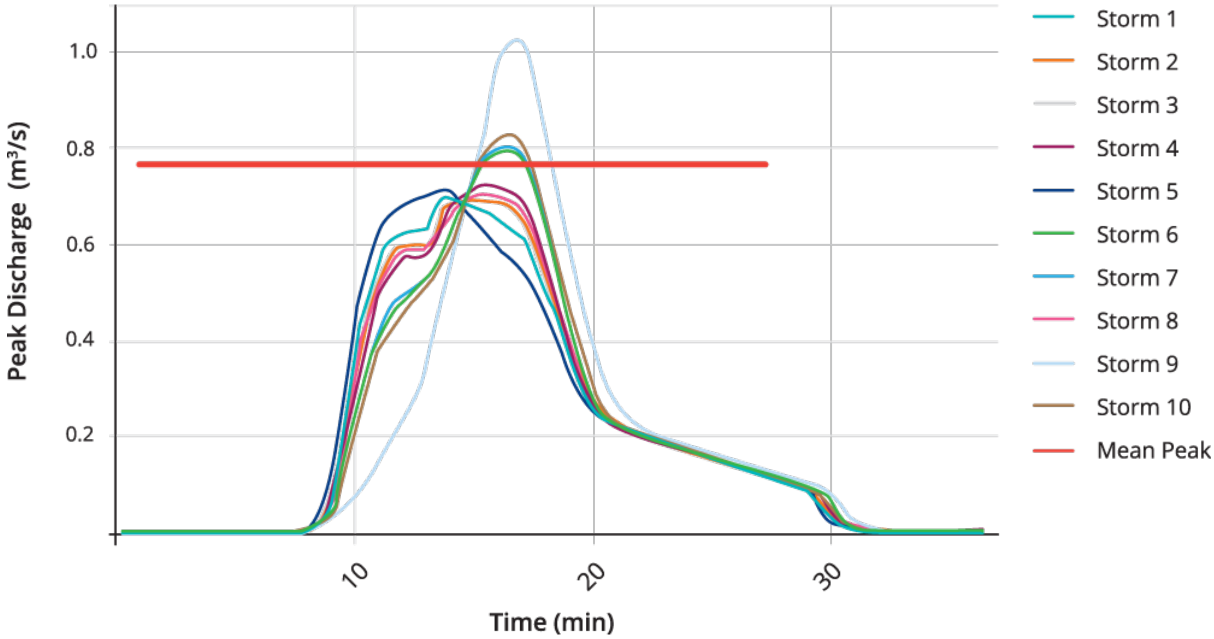


Figure 9.6.26. Selection of the 10% AEP Design Storm for Preliminary Infrastructure Design

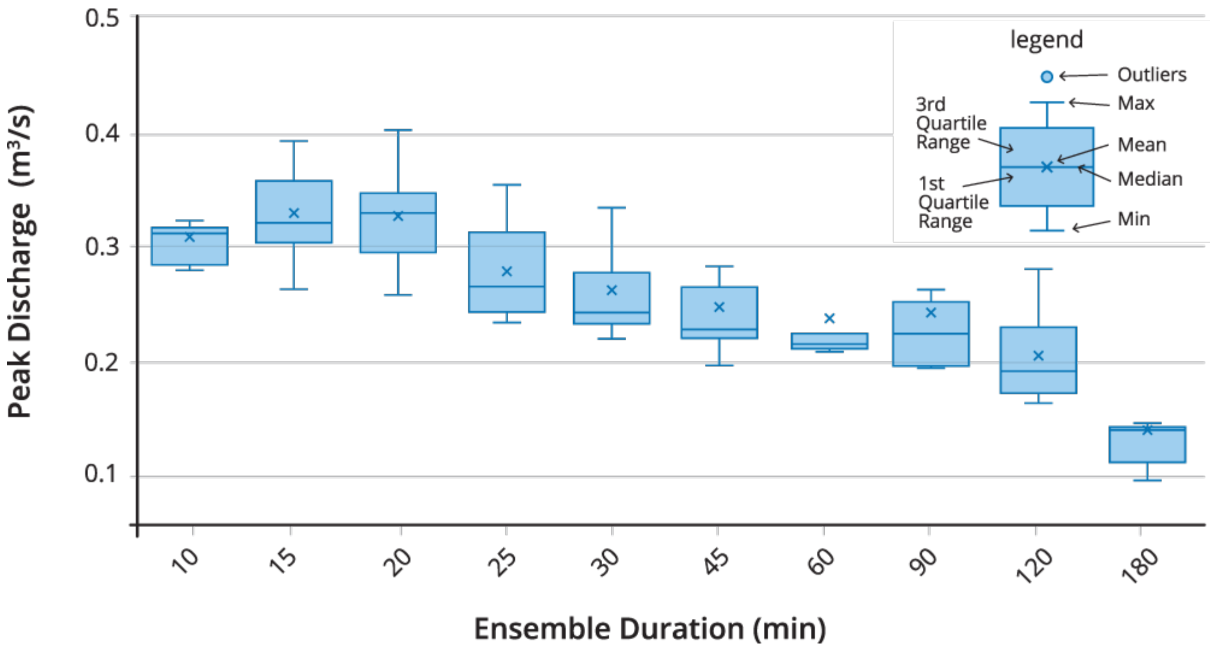


Figure 9.6.27. Development Peak Flows for 50% AEP Events

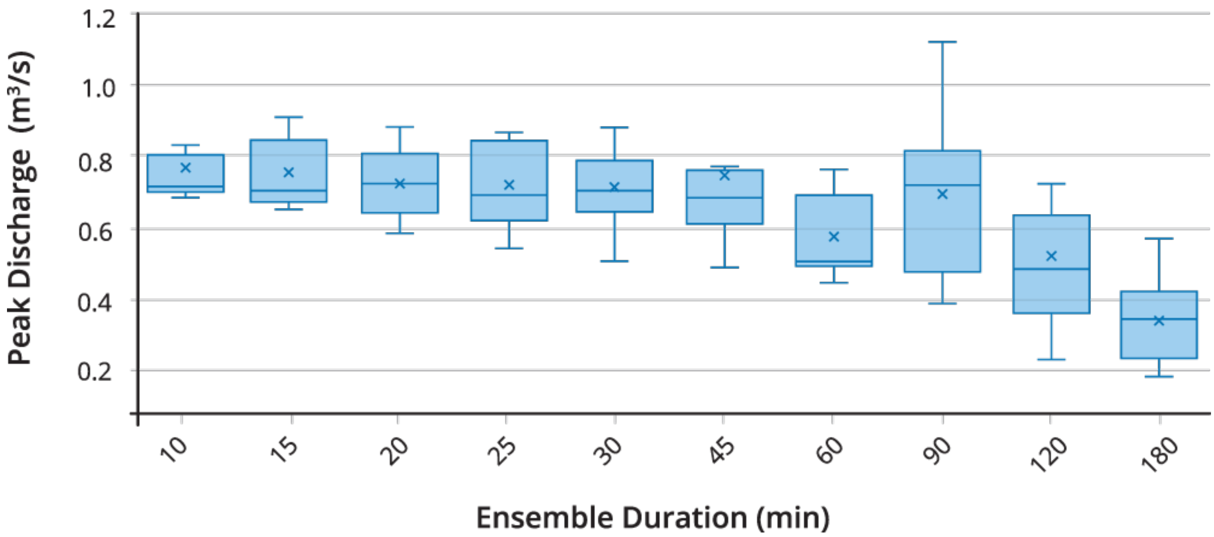


Figure 9.6.28. Development Peaks Flows for 10% AEP Events

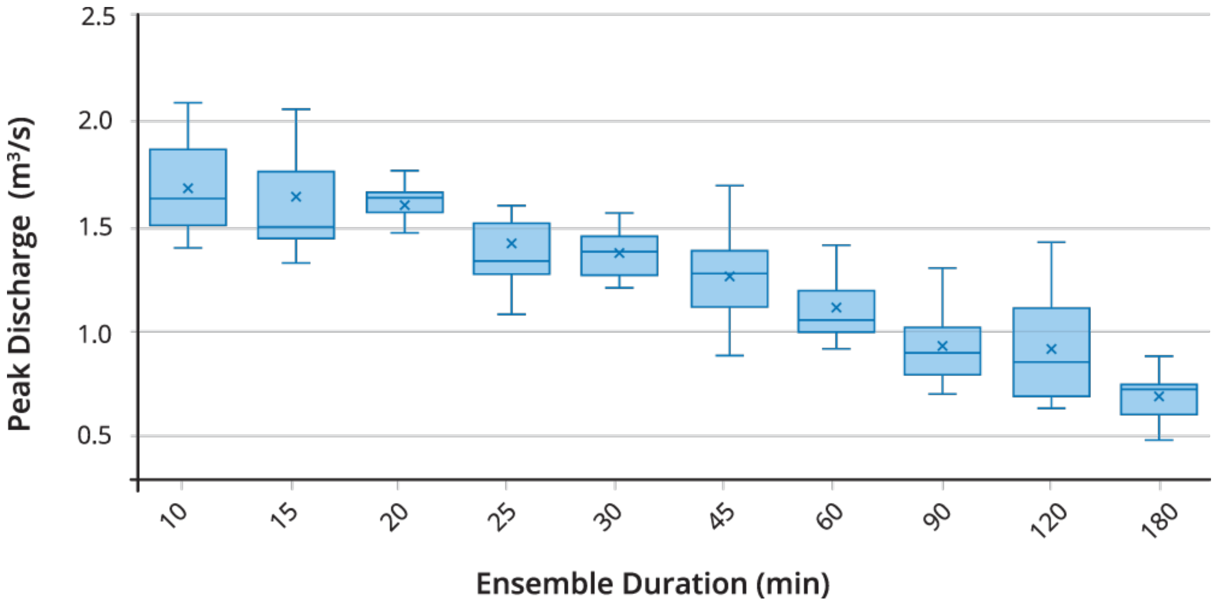


Figure 9.6.29. Development Peak Flows for 1% AEP Events

The preliminary design of the conveyance network (Book 9, Chapter 5) shown in Figure 9.6.30 was sized using storm 7 (Figure 9.6.26) for the 10% AEP event with a design pre-burst rainfall of 2.2 mm. Inlet pit relationships from Book 9, Chapter 5, Section 5 were applied to the design process. Pit inlet capacities for on grade pits (Figure 9.5.12) and sag pits (Equation (9.5.1) to Equation (9.5.4)) were derived using Book 9, Chapter 5, Section 5. Design blockage of on grade and sag pits was derived from Table 9.5.2 and pit energy losses were defined using Book 9, Chapter 5, Section 5.

A hydrology/hydraulics model was use to sized pipes in the conveyance network with objectives of maintaining 150 mm freeboard to grates of inlet pits and less than two metre flow width on roads. The design of the conveyance network was then checked using ensembles for 10% AEP design storm events with pre-burst rainfall for 10, 15, 20 and 30 minute durations.

The safety of surface flows were also checked by simulating the performance of the conveyance network using design rainfall ensembles for 1% AEP burst events with pre-burst rainfall for 10, 15, 20 and 30 minute durations. In accordance with Book 9, Chapter 3, Section 4 and Book 9, Chapter 5, Section 6 (also see Book 7, Chapter 6), the design aimed to limit surface water depths to less than 200 mm and less than 50 mm at road crowns. These objectives also included limiting depth velocity product to less than 0.4 and aimed for freeboard to floor levels of greater than 300 mm.

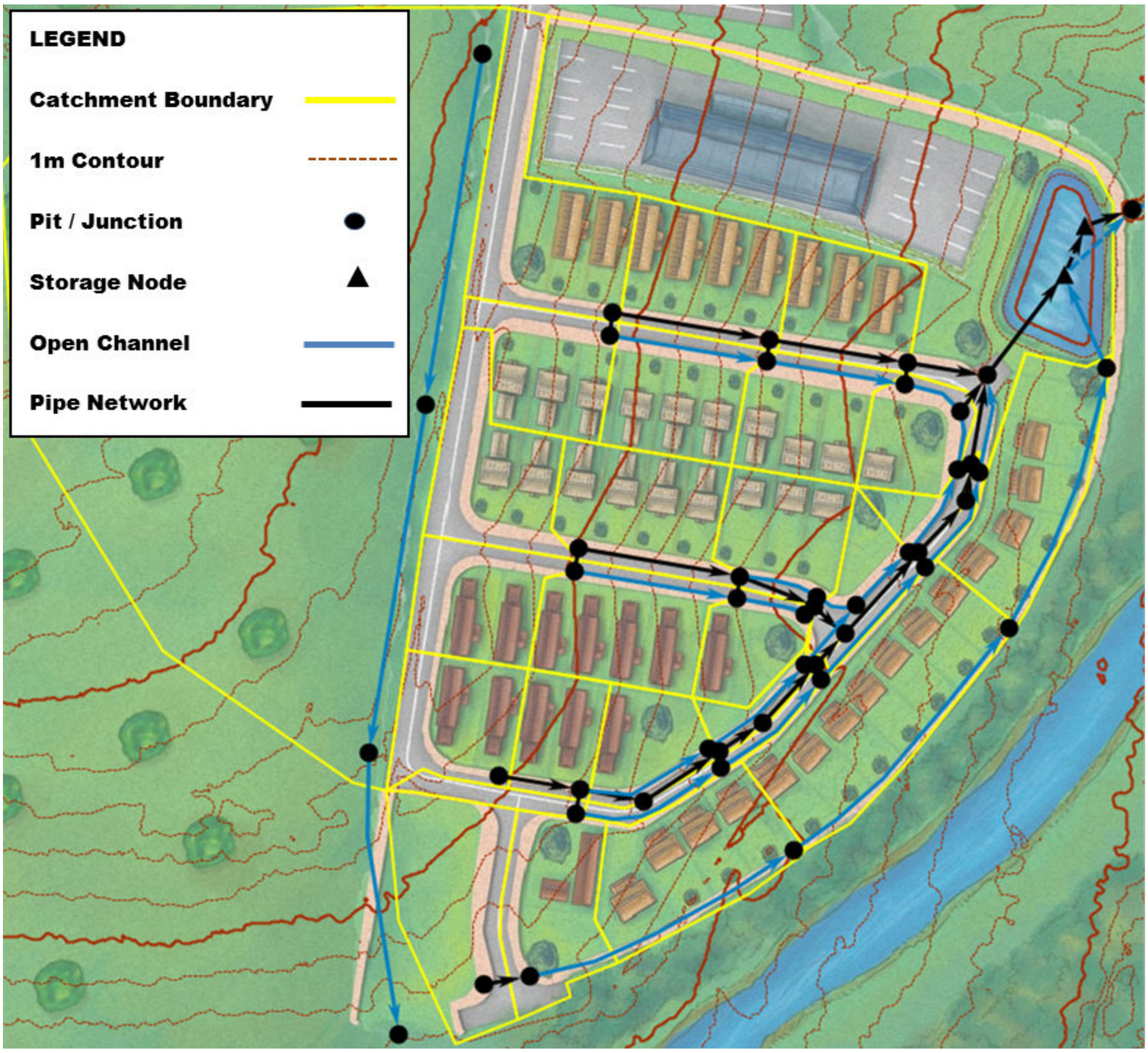


Figure 9.6.30. Overview of the Planned Conveyance Network in the Urban Development

A storage basin was then designed to manage flooding and impacts on downstream waterway (refer to Book 9, Chapter 4) by mitigating the 50%, 10%, 1% AEP peak flows to meet the rural target defined above. Storage volume and outflow arrangements were utilised to achieve this (refer to Figure 9.6.31). The design of the basin included a freeboard of 300 mm from 1% AEP maximum depth and an emergency spillway designed for full blockage of 1% AEP rainfall events (refer to Book 6, Chapter 6 for blockage discussions).

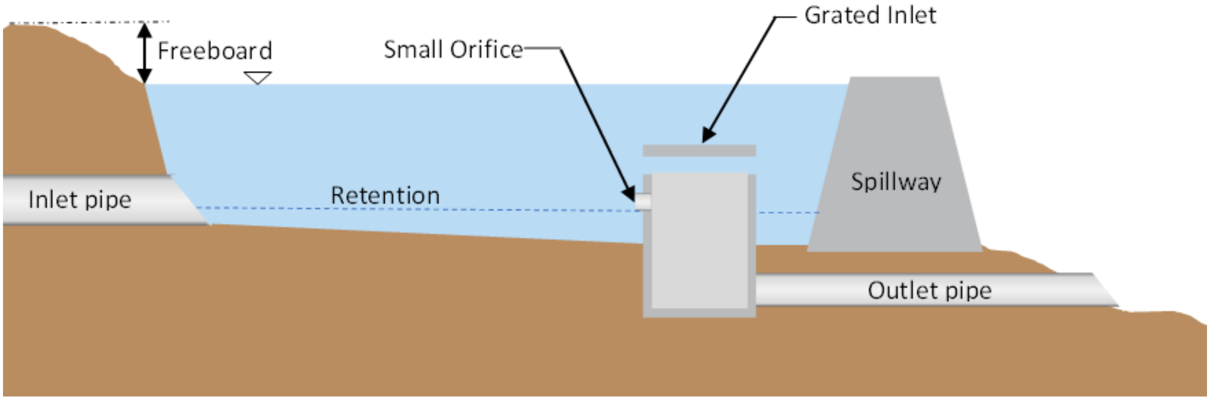


Figure 9.6.31. Overview of the Planned Storage Basin Below the Urban Development

A trial basin design was undertaken using the hydrology/hydraulics models and ensembles of 1.5 hour duration design rainfall with pre-burst rainfall. The design of the basin was then tested and modified using ensembles of design rainfall with pre-burst rainfall for all durations to ensure the rural peak flow targets were met and the maximum basin depth was not exceeded. The final results for peak flows discharging from the development via the basin are shown in Figure 9.6.32, Figure 9.6.33 and Figure 9.6.34 for 50%, 10% and 1% AEP rainfall events. Water levels in the basin for all 1% AEP rainfall durations are provided in Figure 9.6.35.

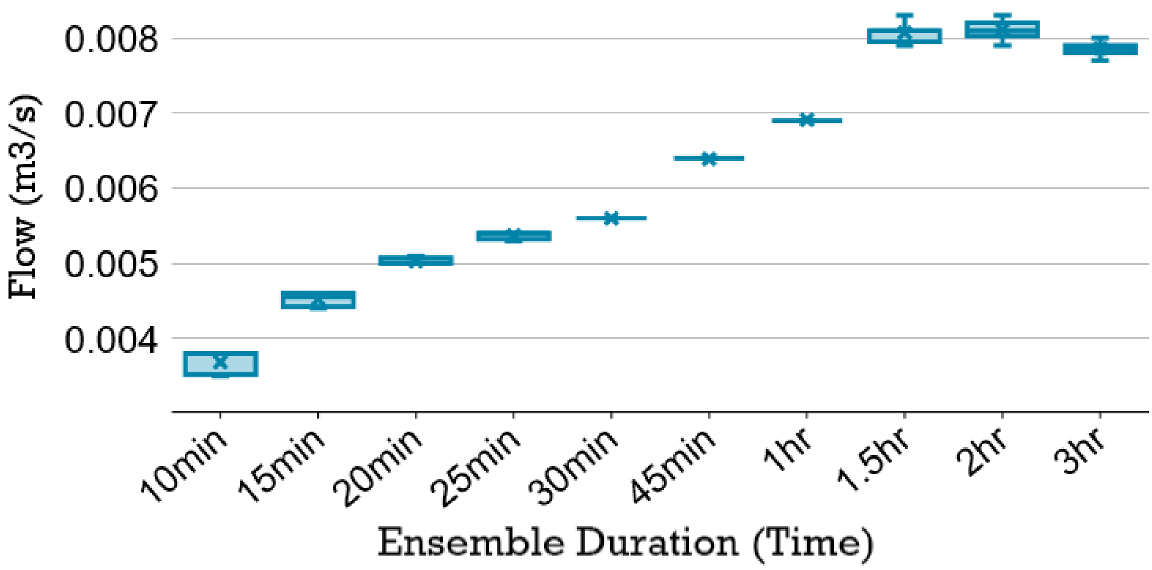


Figure 9.6.32. Peak flows from the Basin for 50% AEP Events

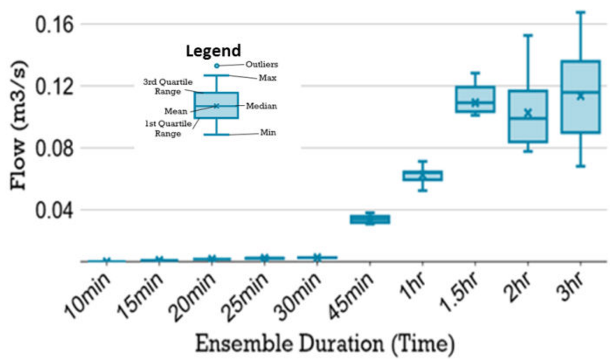


Figure 9.6.33. Peak Flows from the Basin for 10% AEP Events

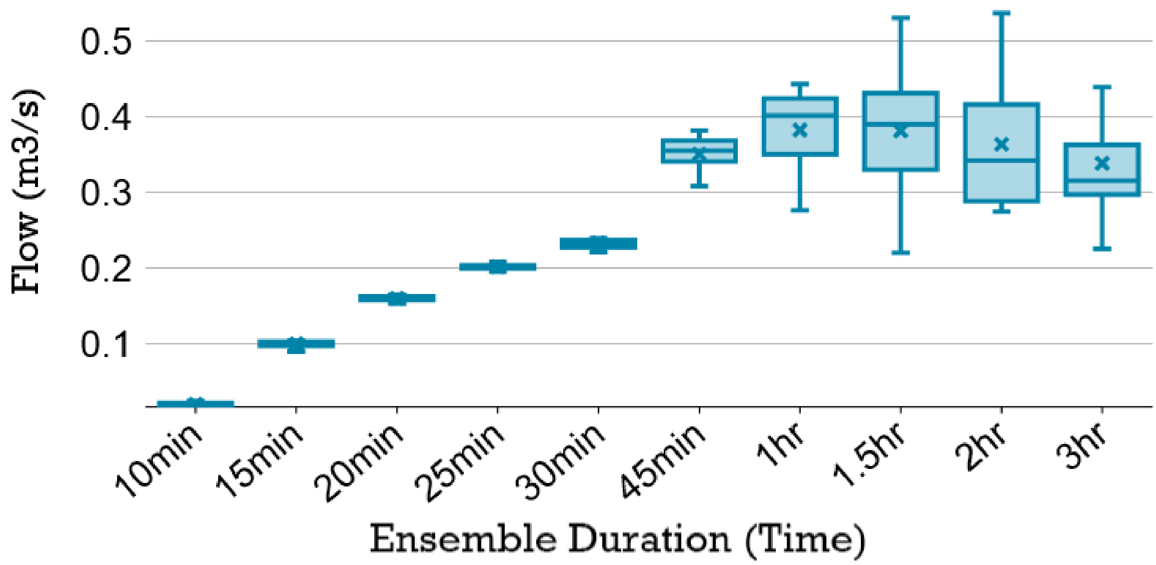


Figure 9.6.34. Peak Flows from the Basin for 1% AEP Events

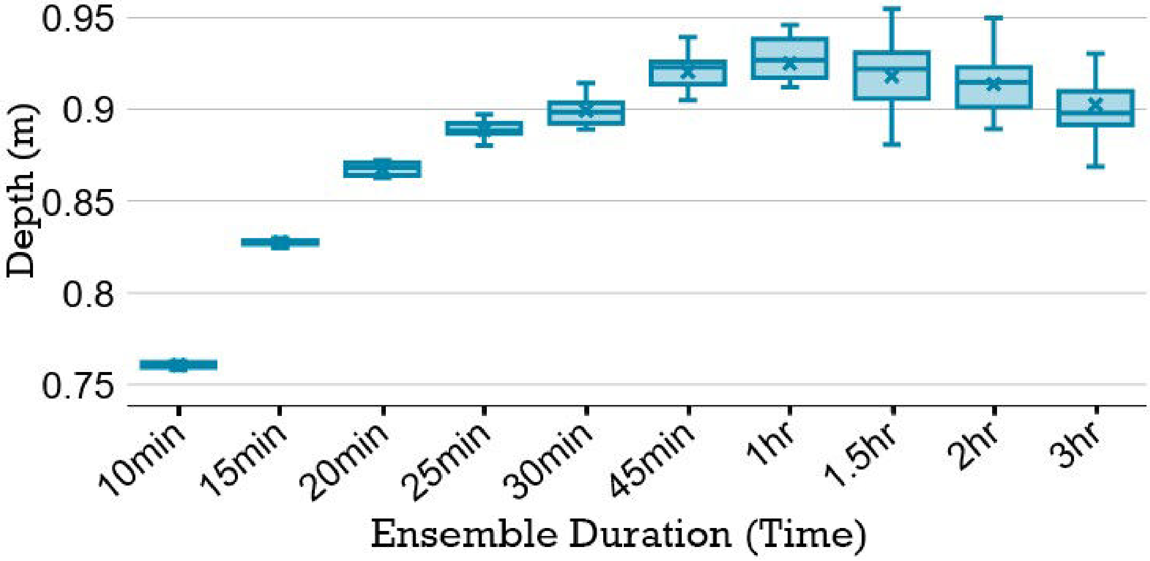


Figure 9.6.35. Peak Water Levels the Basin for 1% AEP Events

Figure 9.6.32 to Figure 9.6.34 show that the mean peak flows from the basin were less than the rural flows with critical durations ranging from one to three hours. A one hour critical duration of peak water levels in the basin was also observed from the analysis (refer to Figure 9.6.35). These result highlight that critical durations of stormwater runoff can vary throughout catchments and across different types of the infrastructure.

This example was developed using the methods in Book 1, Chapter 6 (2019). The methods have evolved since time of writing but the principles are the same[2]. A design life for the infrastructure and consequence level for climate change impacts was selected. A design life of 100 years was assumed for the basin with medium consequences of failure due to impacts on the waterway and surrounding rural properties.

This assessment was utilised to extract data from ARR Data Hub for the RCP 8.5 value for 2090 which indicated an expected 16.1% increase in peak rainfall[3]. This expected increase in peak flows was used to alter the increase in peak rainfall. This expected increase in peak flows was used to alter the increase in peak rainfall (Please note the Data hub value for Ballarat has changed as of May 2019 to 16.3% to reflect changes to the predicted temperatures from Climate Change Australia). This expected increase in peak flows was used to alter the relevant design rainfall ensembles and the hydrology/hydraulic model was rerun to test the impact of climate change on peak water levels in the basin and on roads. Designers should also utilise emerging research to incorporate that most up to date climate change assessments. For example, Wasko and Sharma (2015) outline greater potential for increased rainfall intensities in urban areas.

The impact of applying the expected 2090 climate change effects on design rainfall on peak water levels in the basin and at a critical location on the road is shown in Figure 9.6.36. Increases in peak water depths are experienced in the basin and on the road. The increased runoff into the basin is managed by the emergency spillway and peak water levels are

[2] This section was written before the latest climate change guidance in Book 1, Chapter 6 (2024). A minor change to the text has been made to reflect the change in guidance.

[3]Please note the Data hub value for Ballarat has changed as of May 2019 to 16.3%. This reflects changes to the predicted temperatures from Climate Change Australia

acceptable. However, peak water levels on the road exceed the design objectives and the designer should highlight this situation to the consent authority for further consideration.

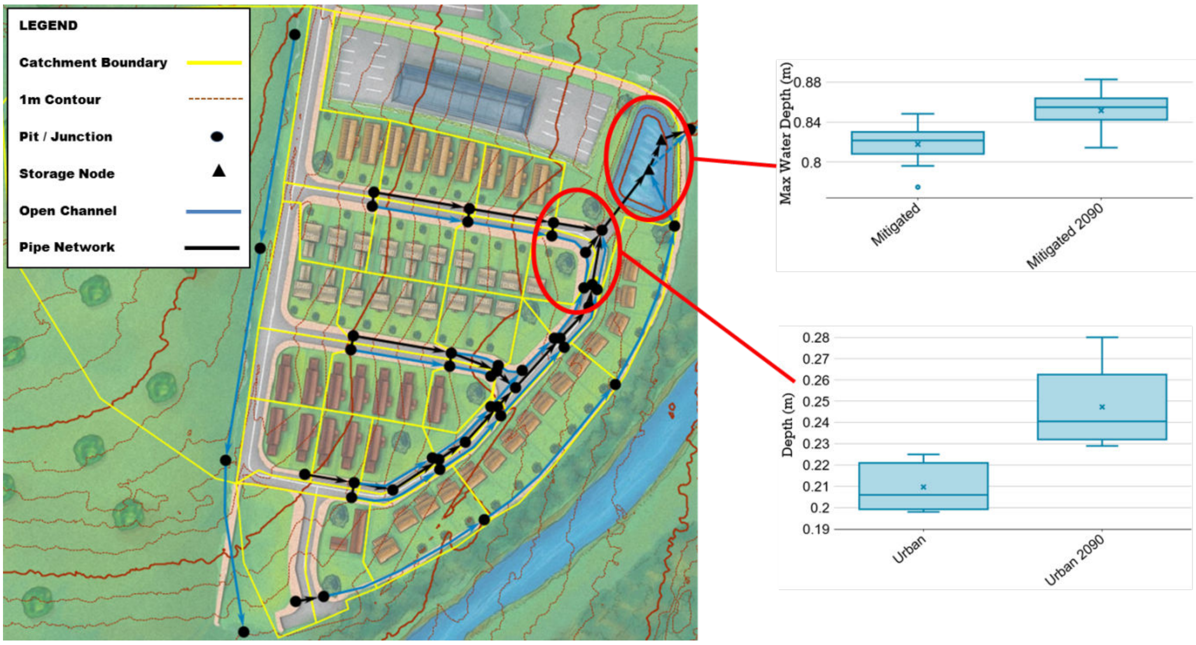


Figure 9.6.36. Peak Water Levels in the Basin and on Roads for 1% AEP Events Subject to Climate Change

6.5. References

Argue J.R., (2004), Water Sensitive Urban Design: basic procedures for 'source control' of stormwater - a handbook for Australian practice. Urban Water Resources Centre, University of South Australia, Adelaide, South Australia, in collaboration with Stormwater Industry Association and Australian Water Association. ISBN 1 920927 18 2, Adelaide.

Atkins (2015), Flood loss avoidance benefits of green infrastructure for stormwater management. USEPA.

Babister, M. and Barton, C. (eds) (2012), Two dimensional modelling in urban and rural floodplains. Australian Rainfall and Runoff Revision Project 15. Engineers Australia.

Babister, M., Trim, A., Testoni, I. and Retallick, M. 2016. The Australian Rainfall and Runoff Datahub, 37th Hydrology and Water Resources Symposium Queenstown NZ

Cecinati F., Rico-Ramirez M.A., Heuvelink G.B.M. and Han D. (2017), Representing radar rainfall uncertainty with ensembles based on time variant geostatistical error modelling approach, Journal of Hydrology, 548, 391-405.

Chocat, B., Krebs, P., Marsalek, J., Rauch, W. and Schilling, W. (2001), Urban drainage redefined: from stormwater removal to integrated management. Water Science Technology, 43(5), 61-8, PubMed PMID: 11379157.

Coombes P.J. (2015), Transitioning drainage into urban water cycle management, Proceedings of the WSUD 2015 Conference, Engineers Australia, Sydney.

Coombes P.J., (2002), Rainwater tanks revisited – new opportunities of integrated water cycle management. PhD Thesis. The University of Newcastle. NSW. Australia. 2002.

Coombes, P.J. and Barry, M.E. (2015), A systems framework of big data for analysis of policy and strategy, Proceedings of the 2015 WSUD and IECA Conference, Sydney.

Coombes, P.J., and M.E. Barry, (2018) Using surfaces of big data to underpin continuous simulation in ssystems analysis, WSUD2018 Conference, Engineers Australia, Perth

Coombes, P.J. and Barry, M.E. (2008a), Determination of available storage in rainwater tanks prior to storm events. Hydrology and Water Resources Symposium. Engineers Australia. Adelaide.

Coombes, P.J. and Barry, M.E. (2008b), Determination of available storage in rainwater tanks prior to storm events, Proceedings of the 31st Hydrology and Water Resources Symposium, 15-17 April, Adelaide.

Coombes P.J., Colegate M., Barber L. and Babister M. (2016), A modern perspective on the hydrology processes of two catchments in regional Victoria, 37th Hydrology and Water Resources Symposium, Engineers Australia, Queenstown, New Zealand.

Coombes, P., Babister, M. and McAlister, T. (2015a).Is the Science and Data underpinning the Rational Method Robust for use in Evolving Urban Catchments, Proceedings of the 36th Hydrology and Water Resources Symposium Hobart 2015.

Coombes P.J., Babister M. and McAlister, (2015b), Is the Science and Data underpinning the Rational Method Robust for use in Evolving Urban Catchments, Proceedings of the 36th Hydrology and Water Resources Symposium, Engineers Australia, Hobart.

Everard M., and McInnes R. (2013), Systemic solutions for multi-benefit water and environmental management. Science of the Total Environment. 461: 170-179.

Fletcher T.D., Wong T.H.F., Duncan H.P., Coleman J.R. and Jenkins G.A. (2001), Managing the impacts of urbanisation on receiving waters: a decision-making framework. Third Australian Stream Management Conference: The Value of Healthy Streams, Brisbane, pp: 217-224.

Goyen A.G. (1981), Determination of rainfall runoff model parameters. Masters Thesis, NSW Institute of Technology, Sydney.

Goyen, A.G. (2000), Spatial and Temporal Effects on Urban Rainfall / Runoff Modelling. PhD Thesis. University of Technology, Sydney. Faculty of Engineering.

Goyen, A.G., O'Loughlin, G.G. (1999a), Examining the basic building blocks of urban runoff. Urban Storm Drainage - 8th International Proceedings. Sydney, 30 Aug.-3 Sept. 1999, 3: 1382-1390.

Goyen, A.G., O'Loughlin, G.G. (1999b), The Effects of Infiltration Spatial and Temporal Patterns on Urban Runoff. Water 99: Joint Congress; 25th Hydrology & Water Resources Symposium, 2nd International Conference on Water Resources & Environment Research; Handbook and Proceedings, pp: 819-822.

Hall J. (2015), Direct Rainfall Flood Modelling: The Good, the Bad and the Ugly. Australasian Journal of Water Resources, 19(1), 74-85.

Hardy, M.J., Coombes, P.J. and Kuczera, G.A. (2004), An investigation of estate level impacts of spatially distributed rainwater tanks, 2004 International Conference on Water Sensitive Urban Design, Engineers Australia, Adelaide, Australia.

Jones R.F., O'Loughlin G.G. and Egan P.E. (1999), roof and property drainage design methods for Australia and New Zealand, Institution of Engineers Australia, Sydney, pp: 793-800.

Kemp D. and Myers B. (2015), A verification of the hydrological impact of 20 years of infill development in an urban catchment, 36th Hydrology and water resources symposium: the art and science of water, Engineers Australia, pp: 379-386.

Khrapov S.S., Pisarev A.V., Kobelev I.A., Zhumaliev A.G., Agafonnikova E.O., Losev A.G. and Khoperskov A.V. (2015), The Numerical Simulation of Shallow Water: Estimation of the Roughness Coefficient on the Flood Stage, Advances in Mechanical Engineering, Article ID 787016.

Kuczera G.A., Lambert, M., Heneker, T., Jennings, S., Frost, A. and Coombes, P.J. (2006), Joint probability and design storms at the crossroads. Australian Journal of Water Resources, 10: 63-79.

Mugler C., Planchon O., Patin J., Weill S., Silvera N., Richard P. and Mouche E. (2011), Comparison of roughness models to simulate overland flow and tracer transport experiments under simulated rainfall at plot scale, Journal of Hydrology, 402(1-2), 25-40.

Patil, S. and Stieglitz, M. (2012), Controls on hydrologic similarity: role of nearby gauged catchments for prediction at an ungauged catchment, Hydrology and Earth Systems Science, 16: 551-562.

Patouillard C., and Forest J., (2011), The spread of sustainable urban drainage systems for managing urban stormwater: a multi-level perspective analysis. Journal of Innovation Economics and Management. (8), 141-161.

Phillips, B., Goyen, A., Thomson, R., Pathiraja, S. and Pomeroy, L. (2014), Project 6 - Loss models for catchment simulation - urban catchments, Stage 2 Report. Report prepared for Australian Rainfall and Runoff Revision, Engineers Australia.

Pilgrim, DH (ed) (1987) Australian Rainfall and Runoff - A Guide to Flood Estimation, Institution of Engineers, Australia, Barton, ACT, 1987.

Roso S., Boyd M.J. and Chisholm L.A. (2006), Assessment of Spatial Analysis Techniques for Estimating Impervious Cover. 30th Hydrology and Water Resources Symposium, Launceston, TAS

Stephens M.L. and Kuczera G.A. (1999), Testing the time area urban runoff model at the allotment scale, Institution of Engineers Australia, Sydney, pp: 1391-1398.

United States Environmental Protection Authority (USEPA) (2008), Urban stormwater management in the United States, National Research Council. The National Academies Press, DC.

Walsh, C.J., Fletcher, T.D. and Burns, M.J. (2012), Urban stormwater runoff: a new class of environmental flow problem. PLOSone DOI: 10.1371/journal.pone.0045814.

Ward, M., Babister M., Retallick M., and Testoni I, (2018), Ensemble assessment and gridded surface response, 38th Hydrology and Water Resources Symposium, Engineers Australia, Melbourne.

Wasko, C. and Sharma, A. (2015), Steeper temporal distribution of rain intensity at higher temperatures within Australian storms, Nature Geoscience, 8(7), 527-529.

Weeks B., Witheridge G., Rigby E., Bathelmess A. and O'Loughlin G. (2013), Blockage of hydraulic structures, Australian Rainfall and Runoff revision Project 11, Stage 2 Report, Engineers Australia.

Zahidi I., Yusuf B., Cope M., Mohamed T.A. and Shafri H.Z.M. (2017), Effects of depth-varying vegetation roughness in two-dimensional hydrodynamic modelling. International Journal of River Basin Management, DOI: 10.1080/15715124.2017.1394313.

Australian Rainfall & Runoff

ENGINEERS AUSTRALIA

11 national Circuit
BARTON ACT 2600

www.arr.org.au

9 781925 848366