Australian Rainfall & Runoff

A GUIDE TO FLOOD ESTIMATION

BOOK 8 - ESTIMATION OF VERY RARE TO EXTREME FLOODS

Australian Government

ENGINEERS AUSTRALIA

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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However, many of the practices recommended in the 1987 edition of ARR have become outdated, and no longer represent industry best practice.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.

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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.

What is new in ARR 2019?

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 is considered to be final.

BOOK 8

Estimation of Very Rare to Extreme Floods

Estimation of Very Rare to Extreme Floods

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Chapter 1. Introduction

Rory Nathan, Erwin Weinmann

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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.

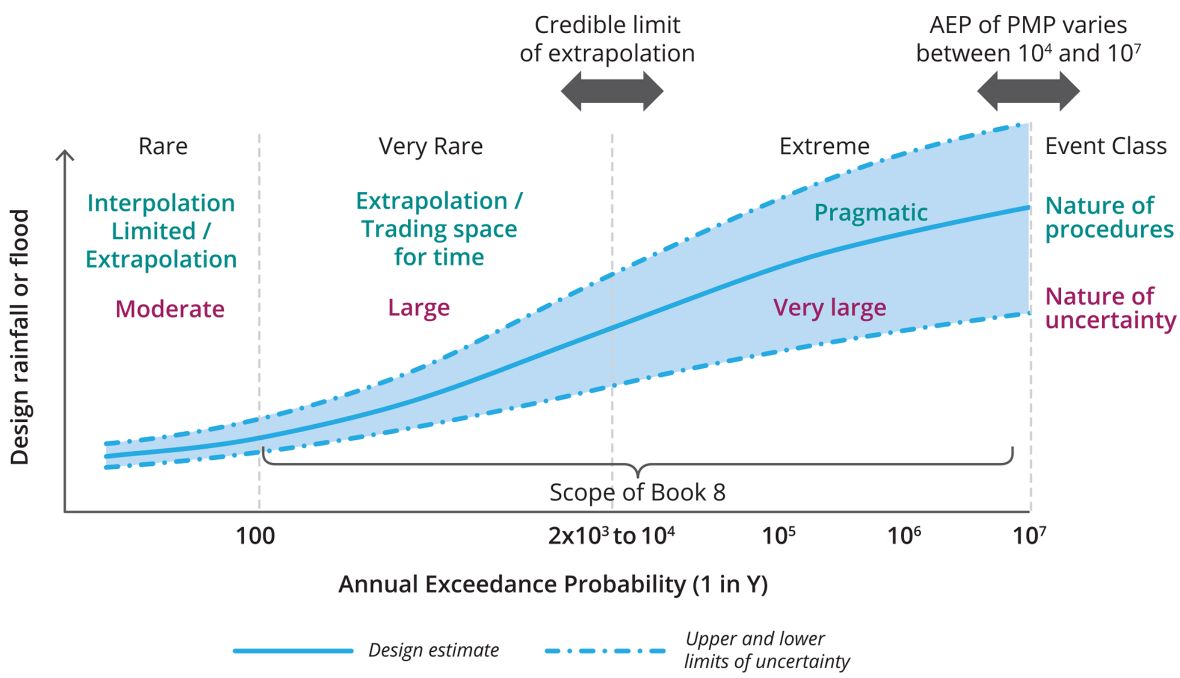

Figure 8.1.1. Design Characteristics of Notional Event Classes

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

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Standards Australia (2004), Bridge Design, Part 2 - Design Loads. AS 5100.2 AP-G15.2/04. Standards Australia, Sydney.

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Chapter 2. Procedures for Estimating Very Rare to Extreme Floods

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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 extrapolatio n**	**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.

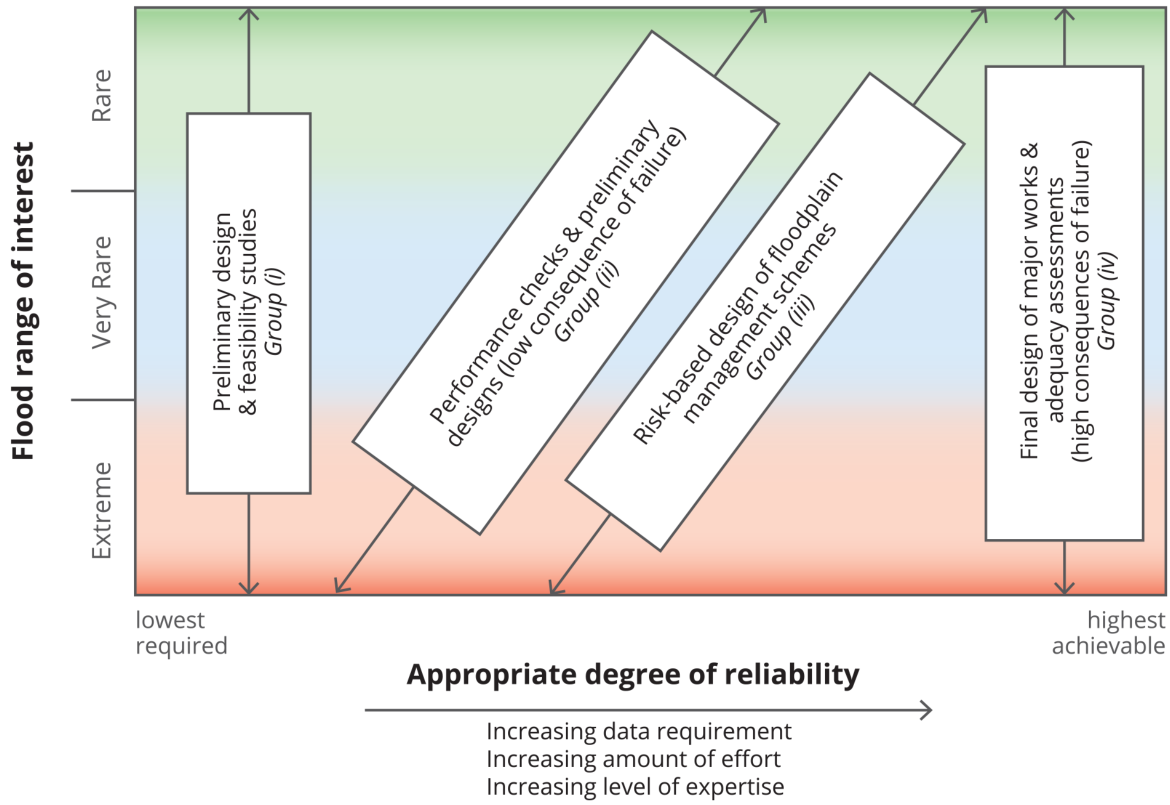

Figure 8.2.1. Qualitative Indication of the Range of Flood Magnitudes and the Relative Degree of Reliability Required for Different Applications

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

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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).

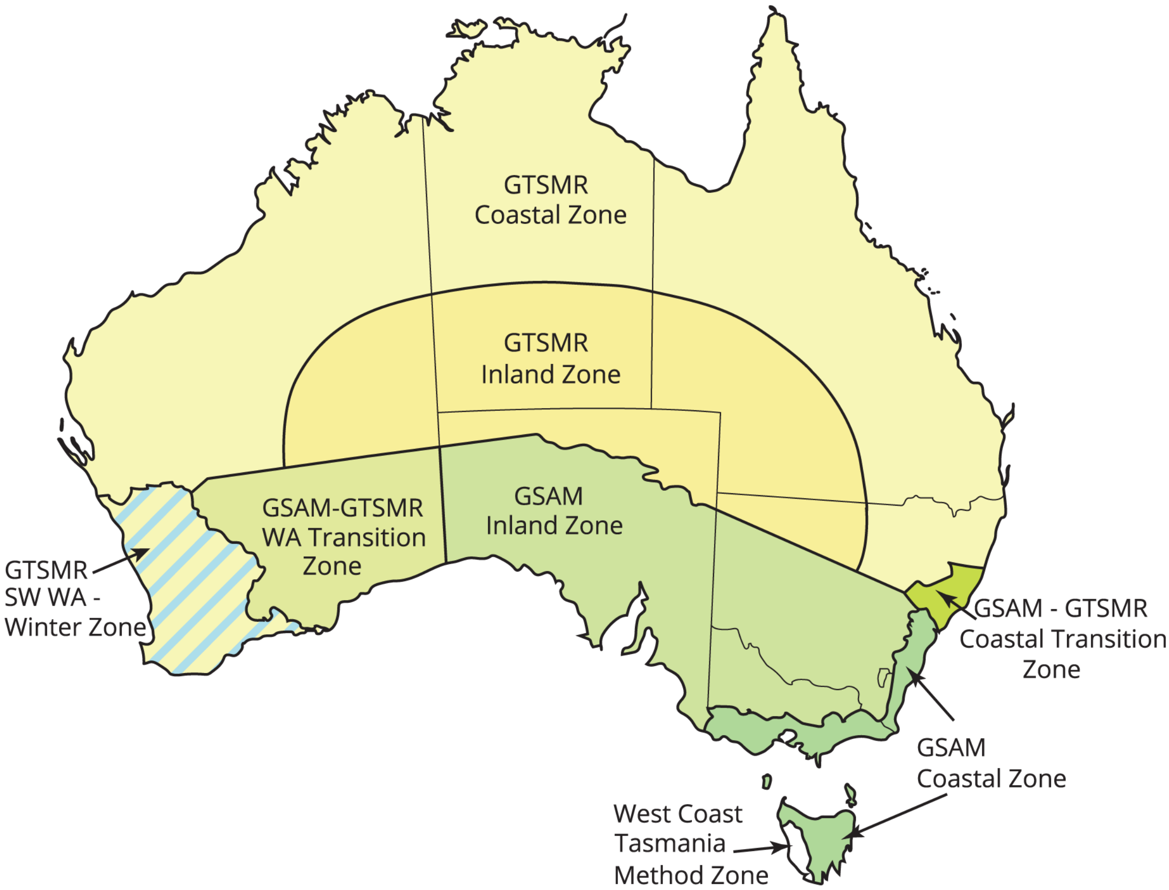

Figure 8.3.1. Generalised Long-Duration Probable Maximum Precipitation Method Zones (Bureau of Meteorology, 2006)

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.

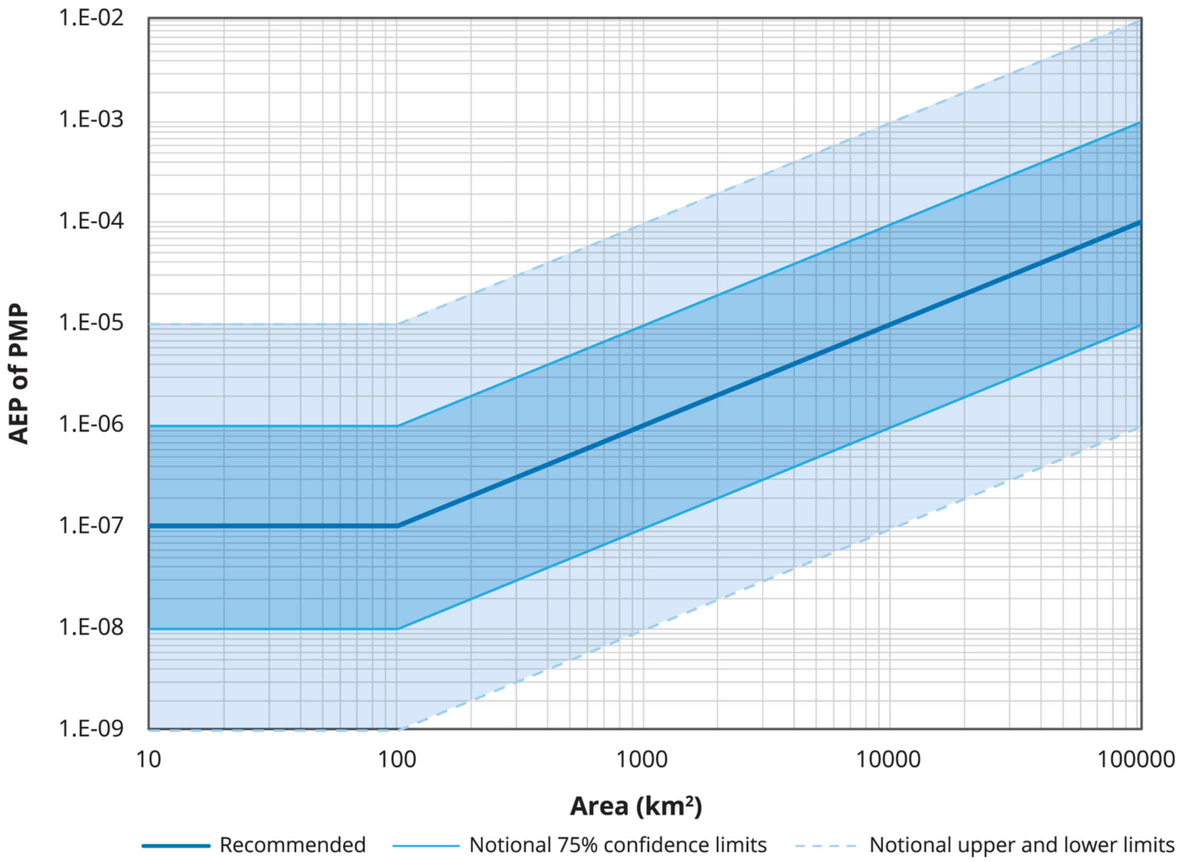

Figure 8.3.2. Recommended Regional Estimates for the AEP of PMP

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.

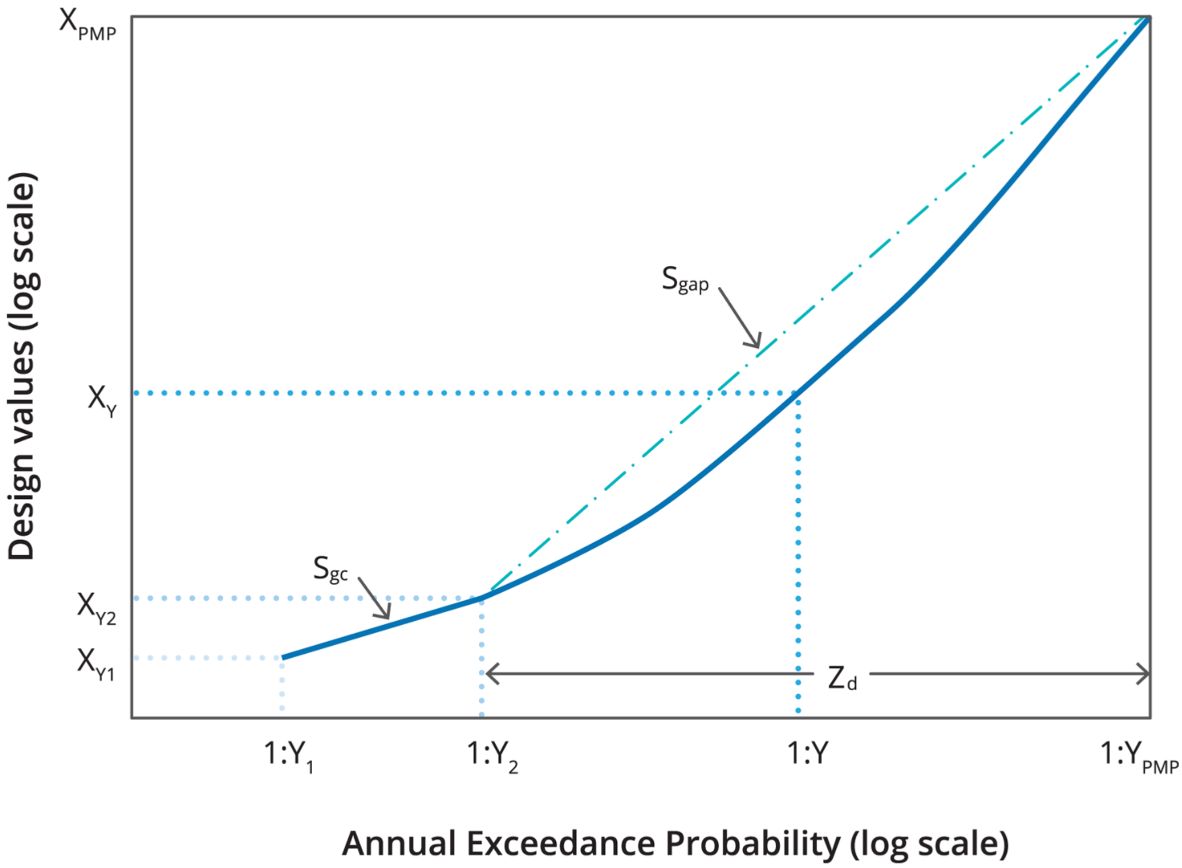

Figure 8.3.3. Schematic Illustration of Interpolation Procedure

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.

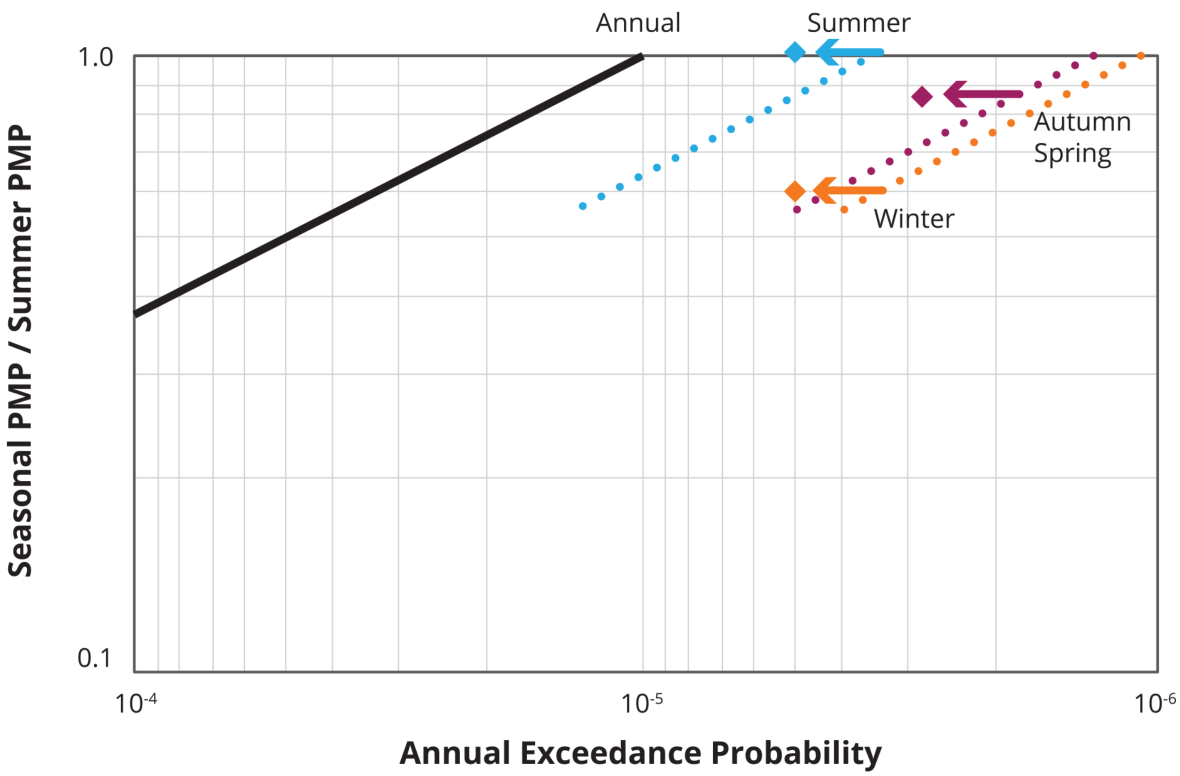

Figure 8.3.4. Hypothetical Frequency Curves for Seasonal and Annual Design Rainfalls Based on the AEP Assigned to the Annual PMP (adapted from Laurenson and Kuczera (1998), using Four Seasons of Relative Lengths 0.33, 017. 0.33, 0.17 and Relative Seasonal PMP Depths of 1.0, 0.85, 0.6, 0.85, for Summer, 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

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Chapter 4. Estimation of Rainfall Excess for Very Rare to Extreme Events

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Chapter Status	Final
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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:

$$\log(L_Y) = \log(L_1) + \{\log(Y) - \log(Y_1)\}\frac{\log(L_2) - \log(L_1)}{\log(Y_2) - \log(Y_1)} \qquad (8.4.1)$$

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		

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Chapter 5. Selection, Configuration, and Calibration of Hydrograph Models

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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

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Chapter 6. Derivation of Design Floods

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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.

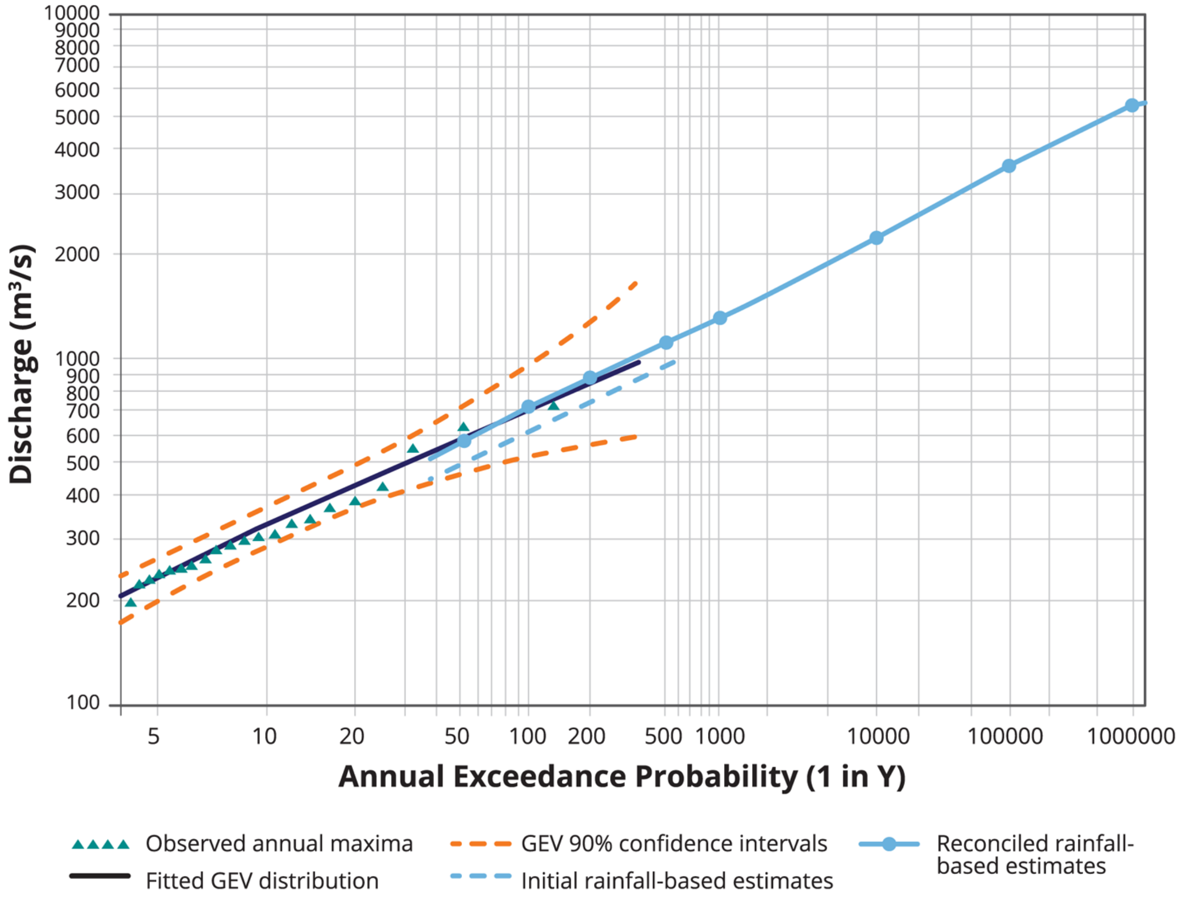

Figure 8.6.1. Illustration of Derived Frequency Curve Based on Reconciliation with Flood Frequency Quantiles

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.

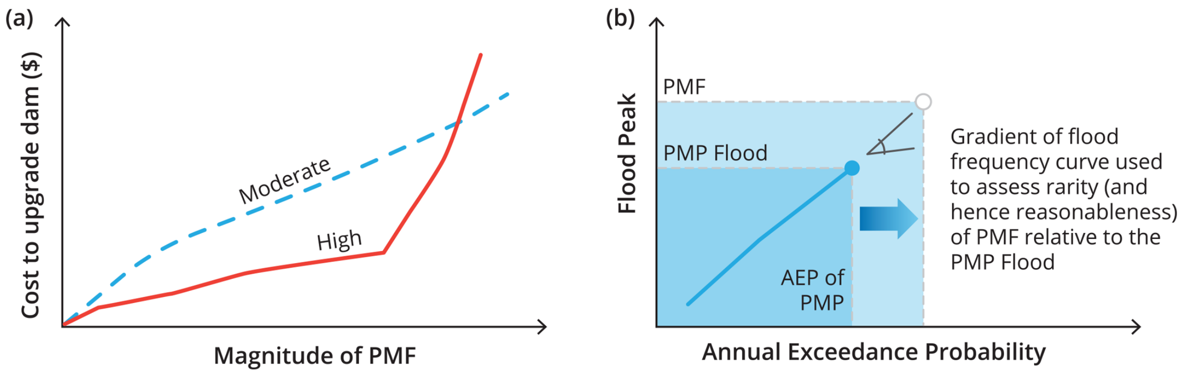

Figure 8.6.2. (a) Differential Importance of "Reasonableness" in PMF Assumptions on Dam Safety Decision-Making, and (b) Use of Simple Extrapolation to Infer Degree of Reasonableness

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.

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Chapter 7. Special Design Considerations

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Chapter Status	Final
Date last updated	14/5/2019

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.

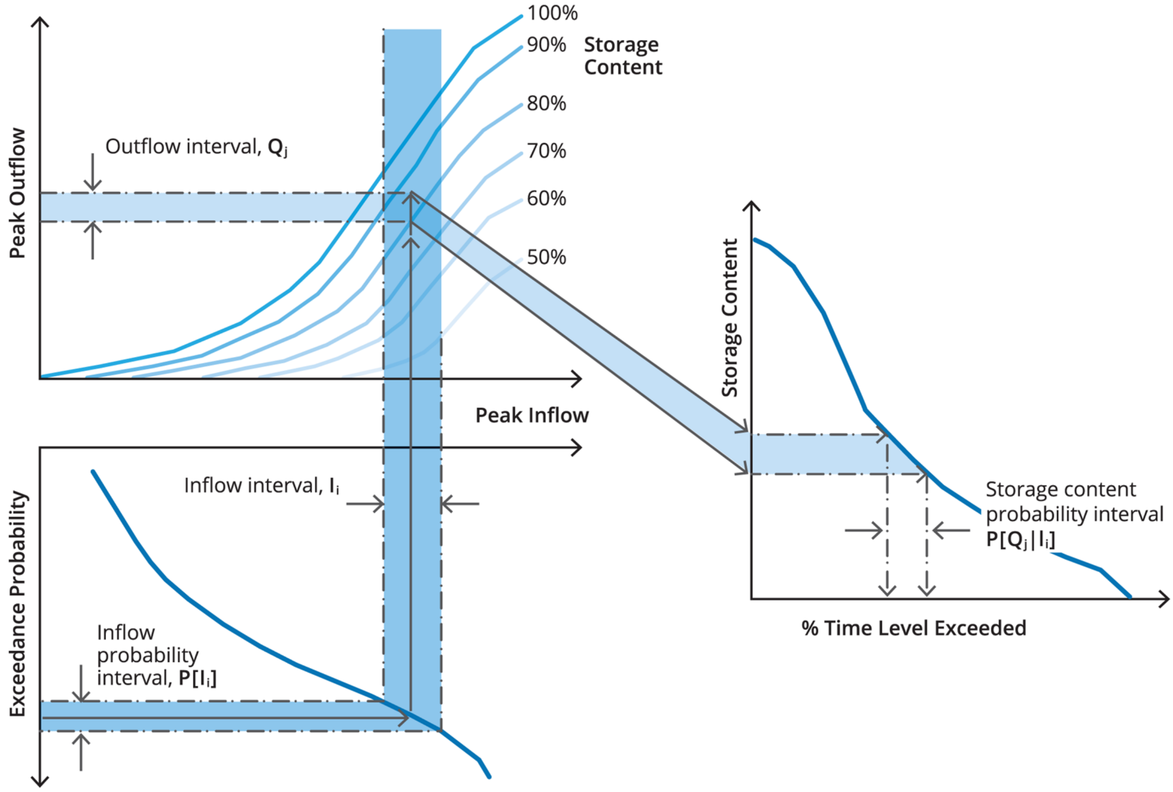

Figure 8.7.1. Schematic Illustration of the Determination of the Probability Interval of Storage Volume as a Function of Inflow and Outflow

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.

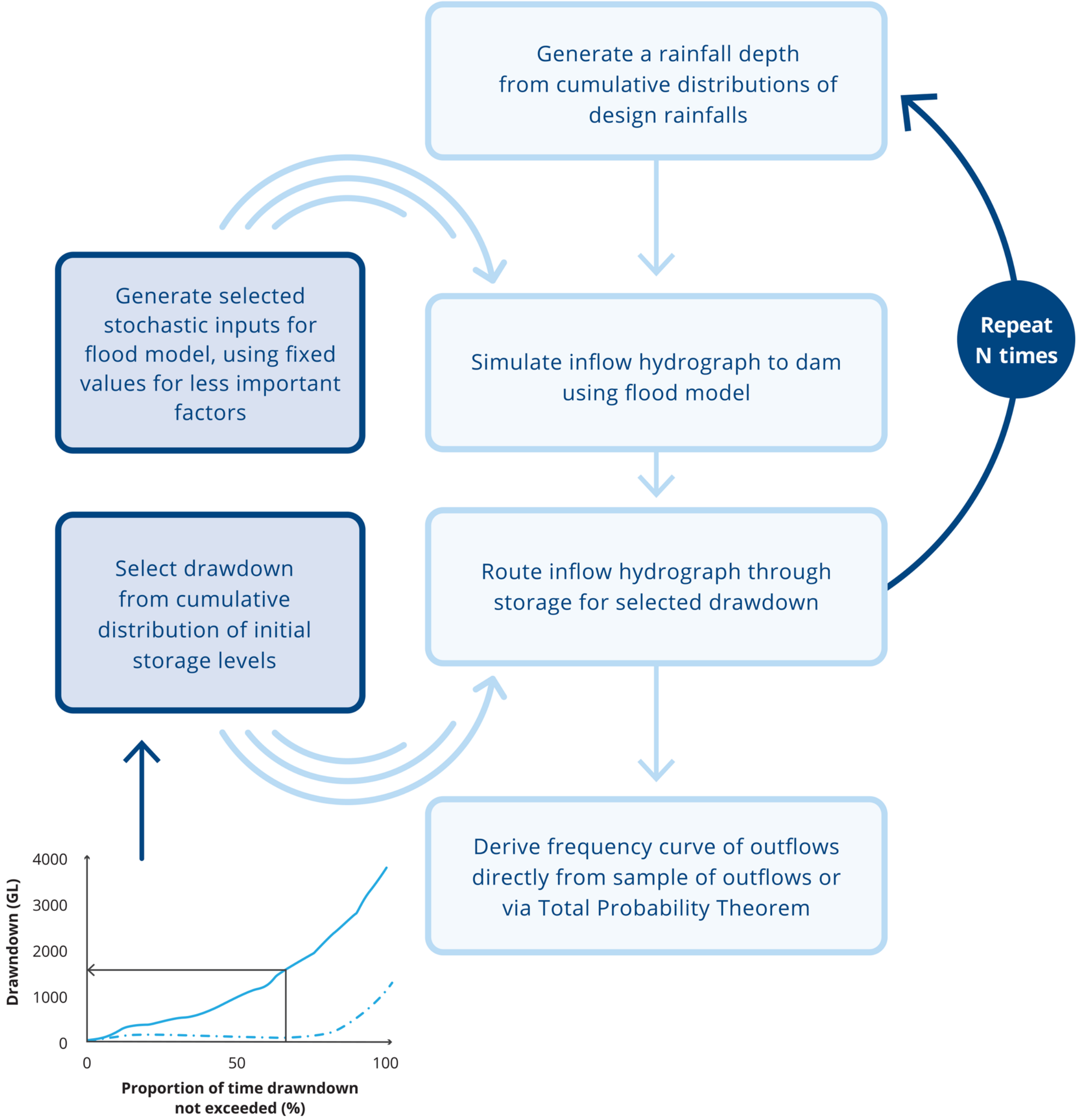

Figure 8.7.2. Illustration of Monte Carlo Framework to Derive Outflow Frequency Curve

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.

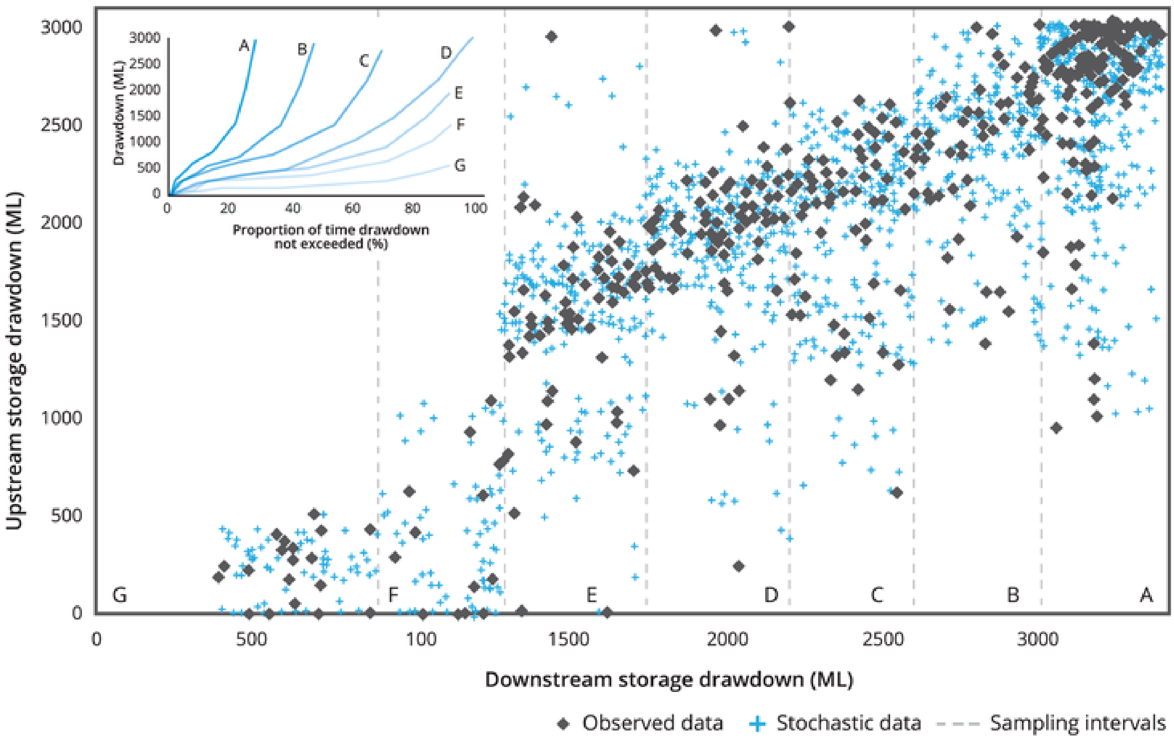

Figure 8.7.3. Illustration of Conditional Empirical Sampling in Which the Storage Volume in an Upstream Dam is Correlated with the Volume in a Downstream Dam

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.

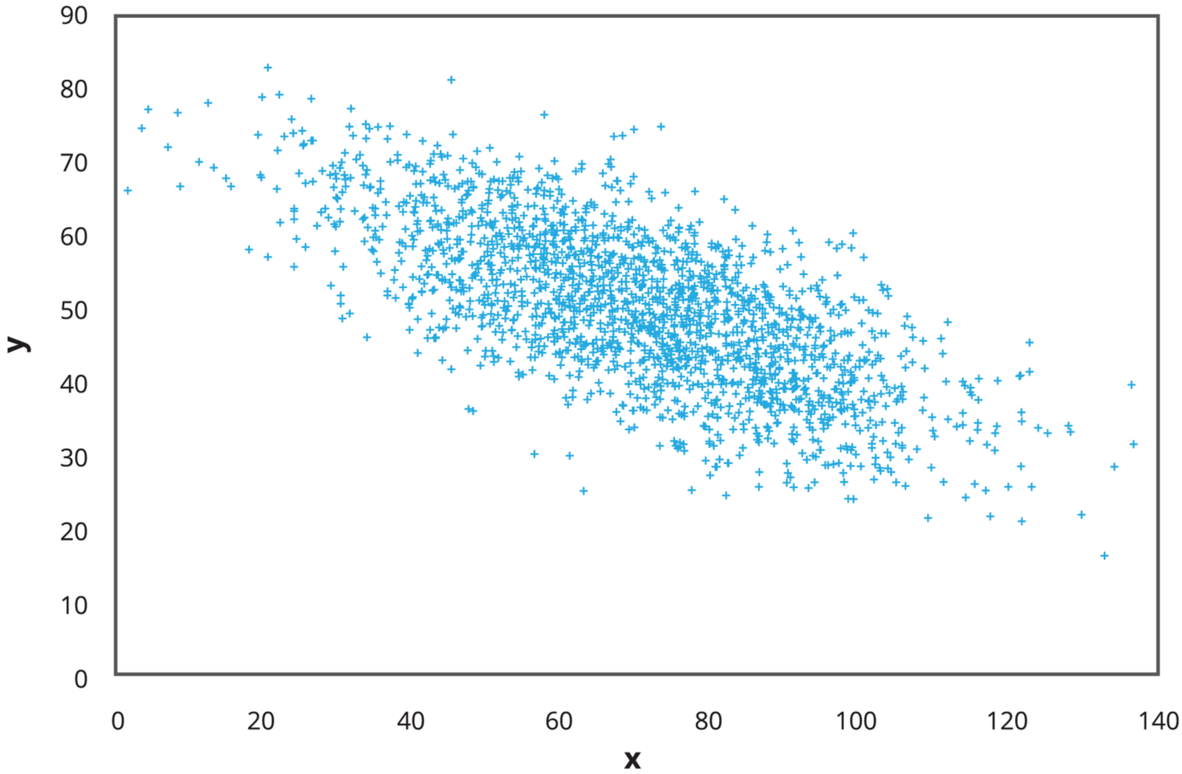

Figure 8.7.4. Illustration of the Generation of Variables with a Correlation of 0.7 Based on Normal Distributions

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:

$$\mu_{(y|x)} = \mu_x + \rho \frac{\sigma_y}{\sigma_x}(x - \mu_x) \tag{8.7.1}$$

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 ≥ 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.

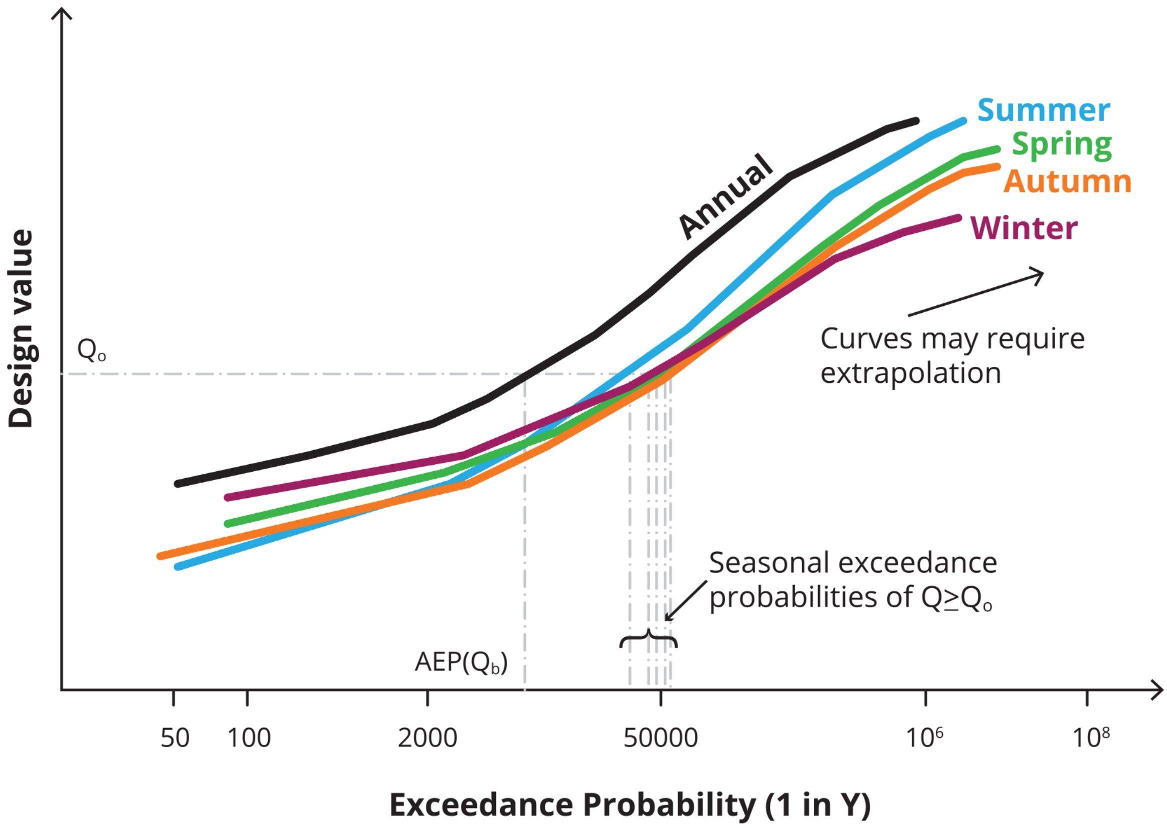

Figure 8.7.5. Schematic Diagram Illustrating the Conversion of Seasonal Exceedance Probabilities into Annual Estimates

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 is provided in Book 1, Chapter 6, however, it is worth noting that at present no allowance is made for climate change in estimates of the PMP. 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.

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Chapter 8. Worked Examples

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Chapter Status	Final
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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:

$$ARF = \min\left\{1.00, \left[1.00 - 0.4\left(A^{0.14} - 0.7\log_{10}D\right)D^{-0.48} + 0.0002(A)^{0.4}D^{0.41}\left(0.3 + \log_{10}(Y)\right)\right]\right\} \quad (8.8.1)$$

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:

$$ARF = \min\left\{1.00, \left[1.00 - 0.1\left(A^{0.14} - 0.879\right) - 0.029\left(A^{0.233}\right)\left(1.255 - \log_{10}(D)\right)\right]\right\} \quad (8.8.2)$$

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}- $2.28\text{x}10^6$

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.

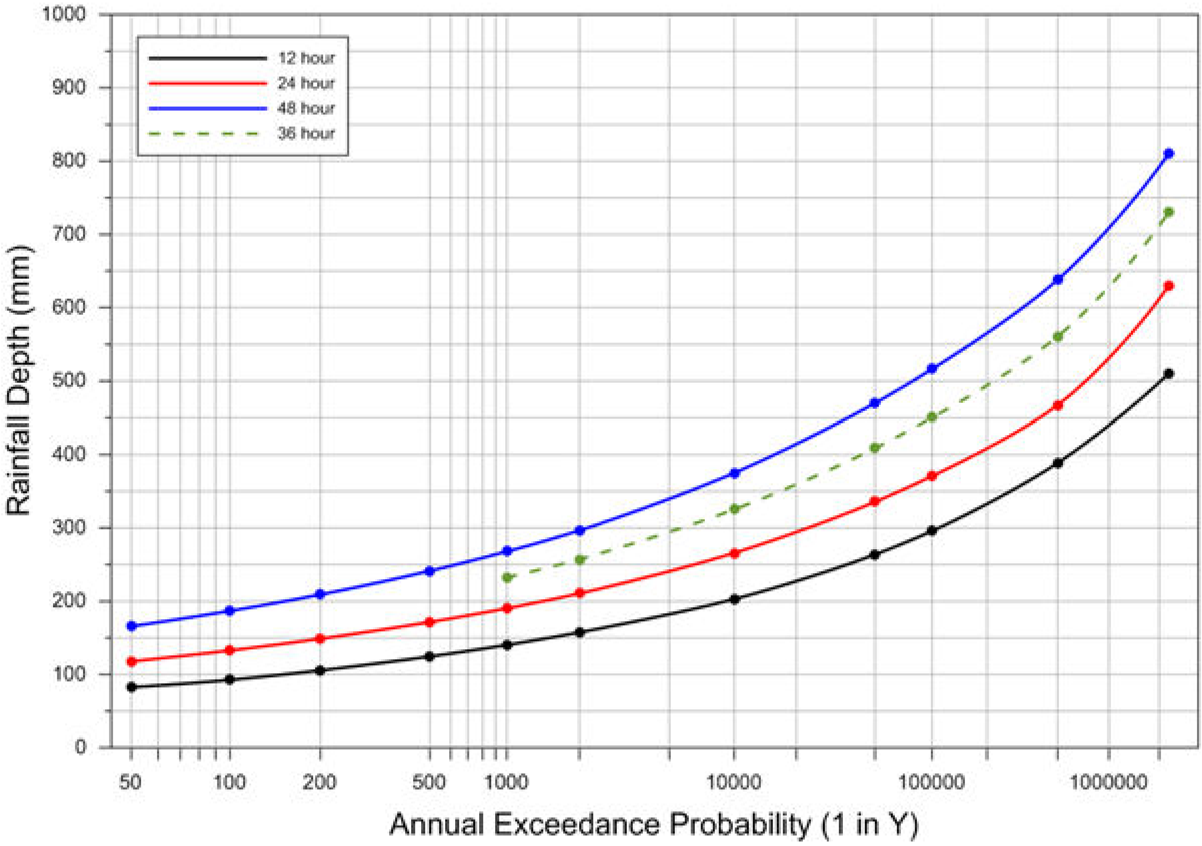

Figure 8.8.1. Example Rainfall Frequency Curves

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:

$$\begin{aligned}\log(36\text{ hr 1 in 100 AEP depth}) &= \frac{\log(36)-\log(24)}{\log(48)-\log(36)} \times (\log(268.0)-\log(190.4)) + \log(190.4) \\ &= 2.367\end{aligned} \quad (8.8.3)$$

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.

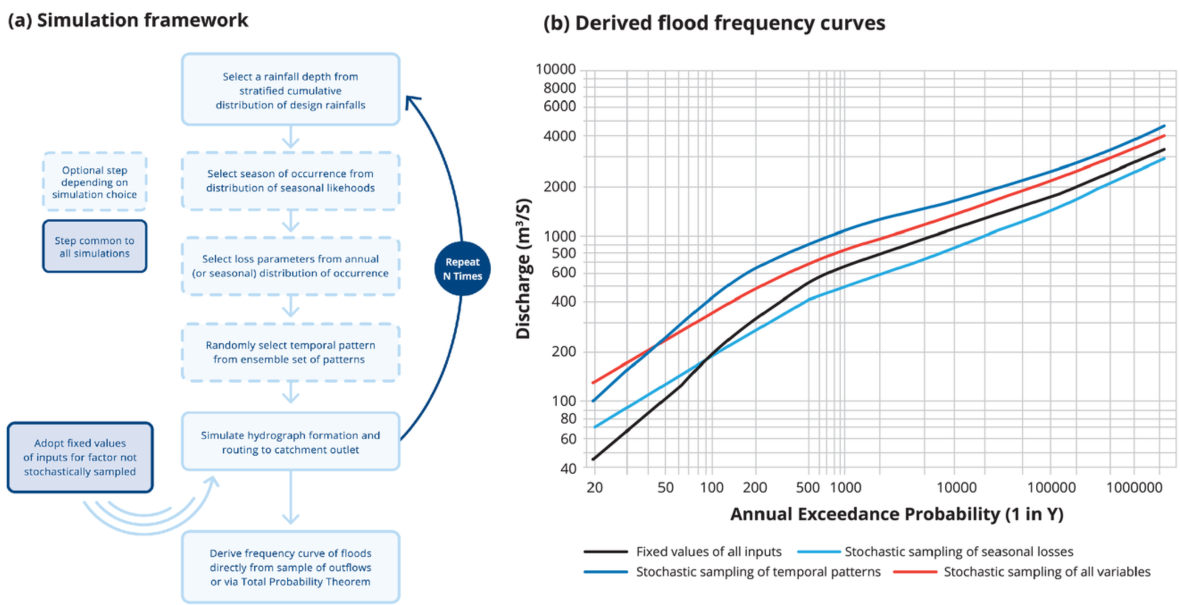

Figure 8.8.2. (a) Simulation Framework used to Generate Floods for Selected Stochastic Inputs and (b) Resulting Flood Frequency Curves

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.

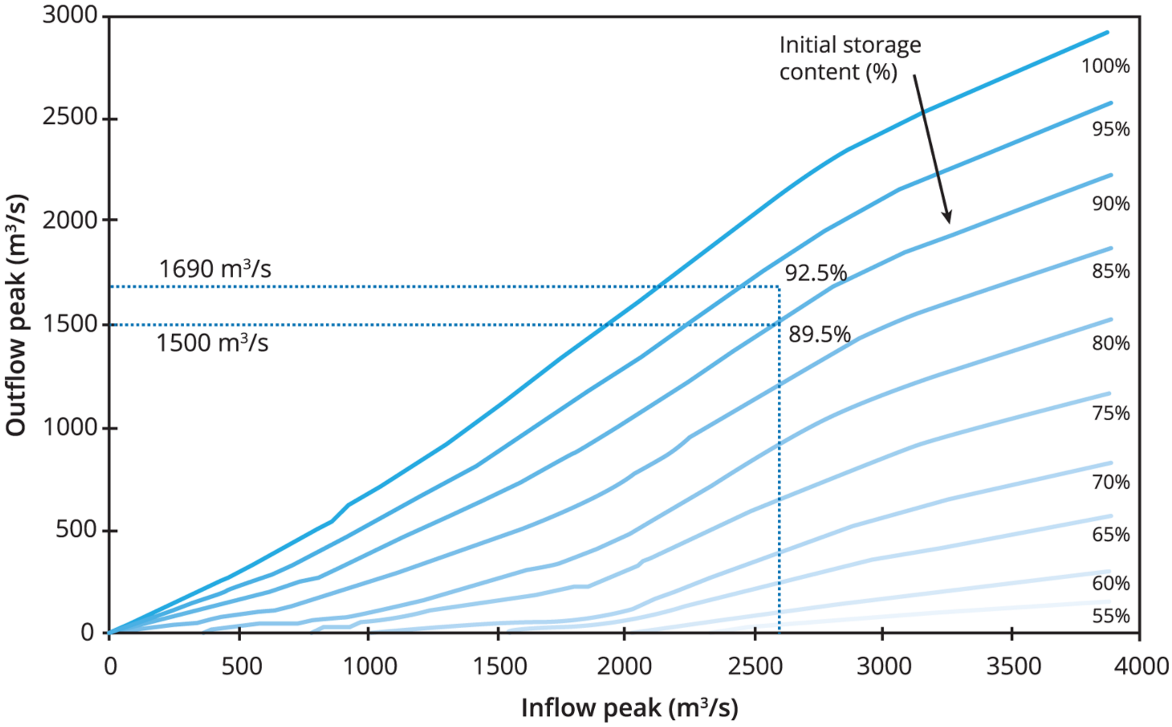

Figure 8.8.3. Inflow-Outflow-Storage Volume Relationship

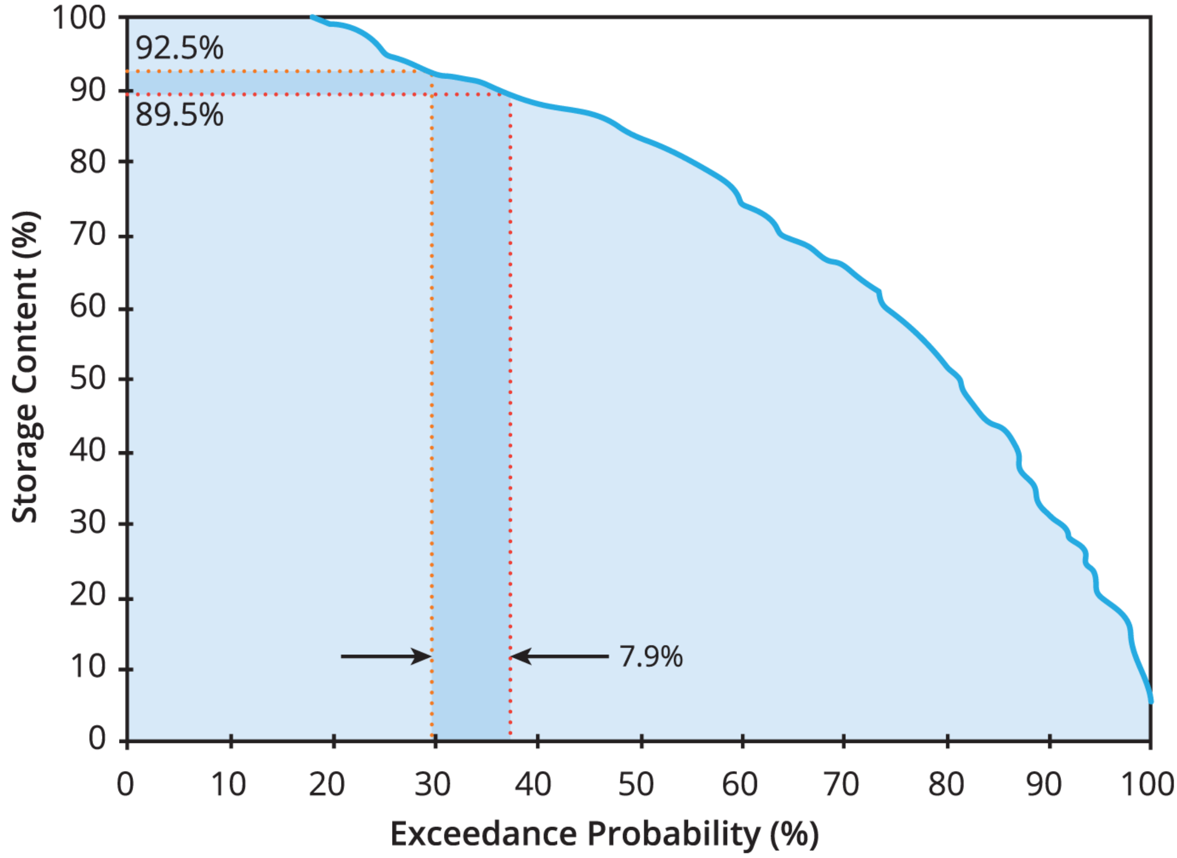

Figure 8.8.4. Probability Distribution of Initial Storage Volume

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.

Outflow Class Intervals (m^3/s) \ 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		
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

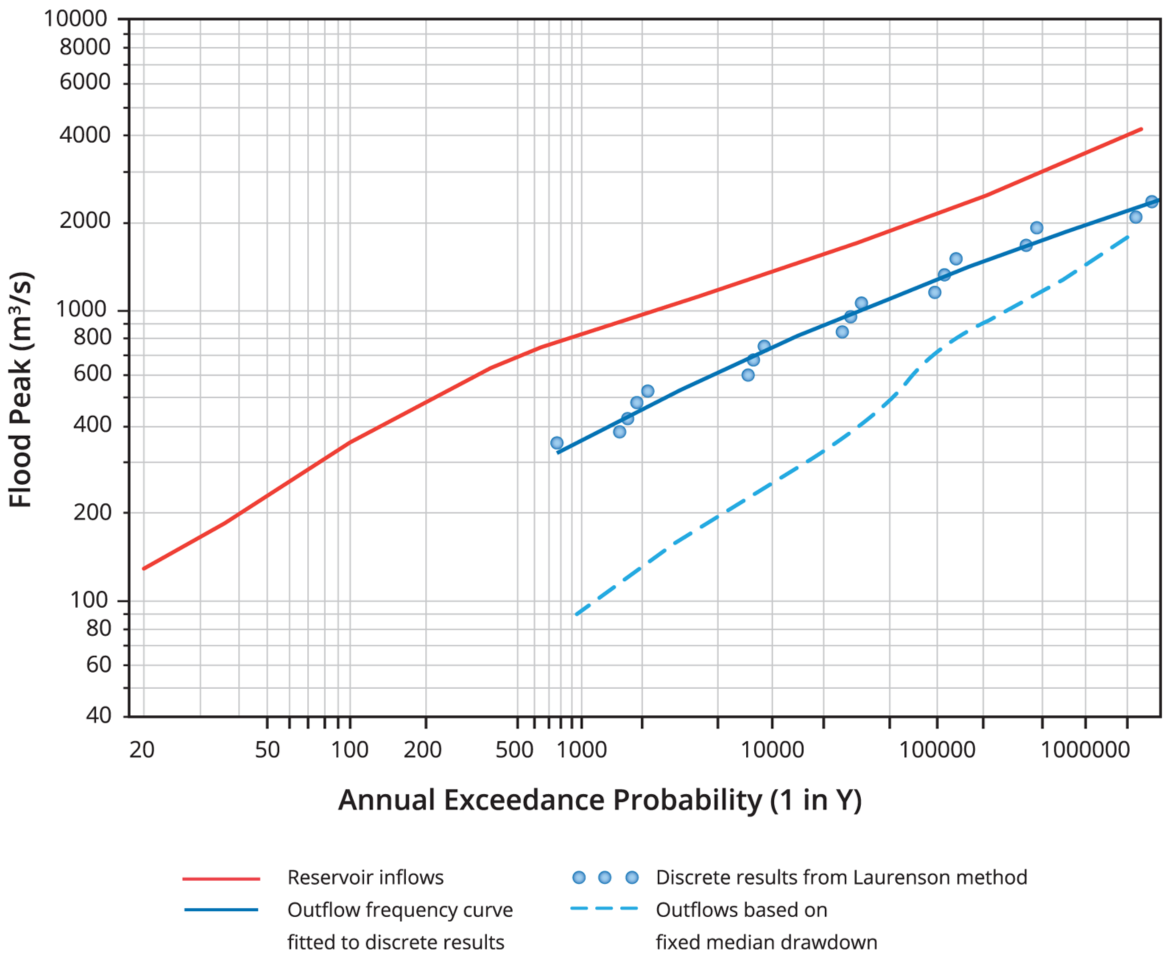

Figure 8.8.6. Outflow Frequency Curves Obtained using Joint Probability Analysis and a Median Level of Drawdown

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.

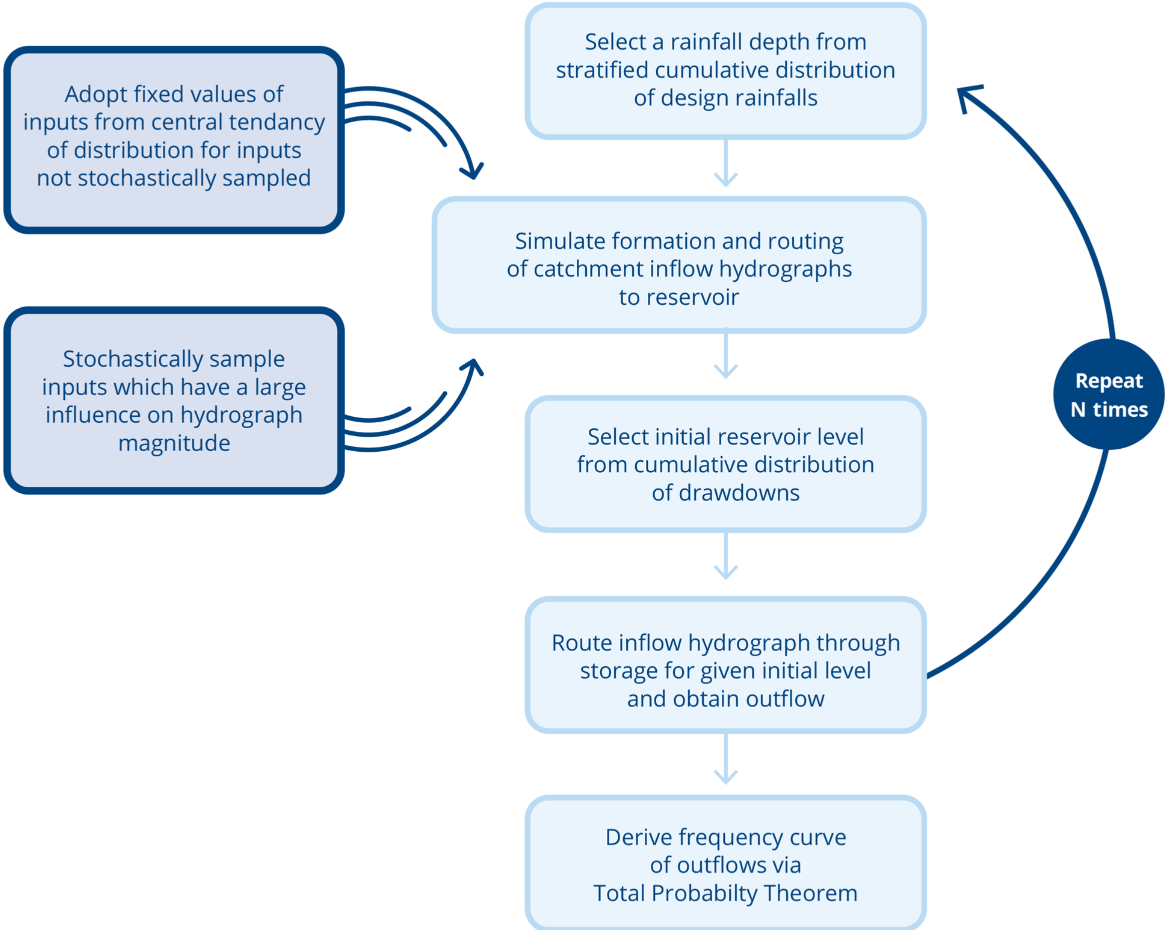

Figure 8.8.7. Simulation Framework to Derive Outflow Frequency Curve Based on Variable Initial Starting Level in Reservoir

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.

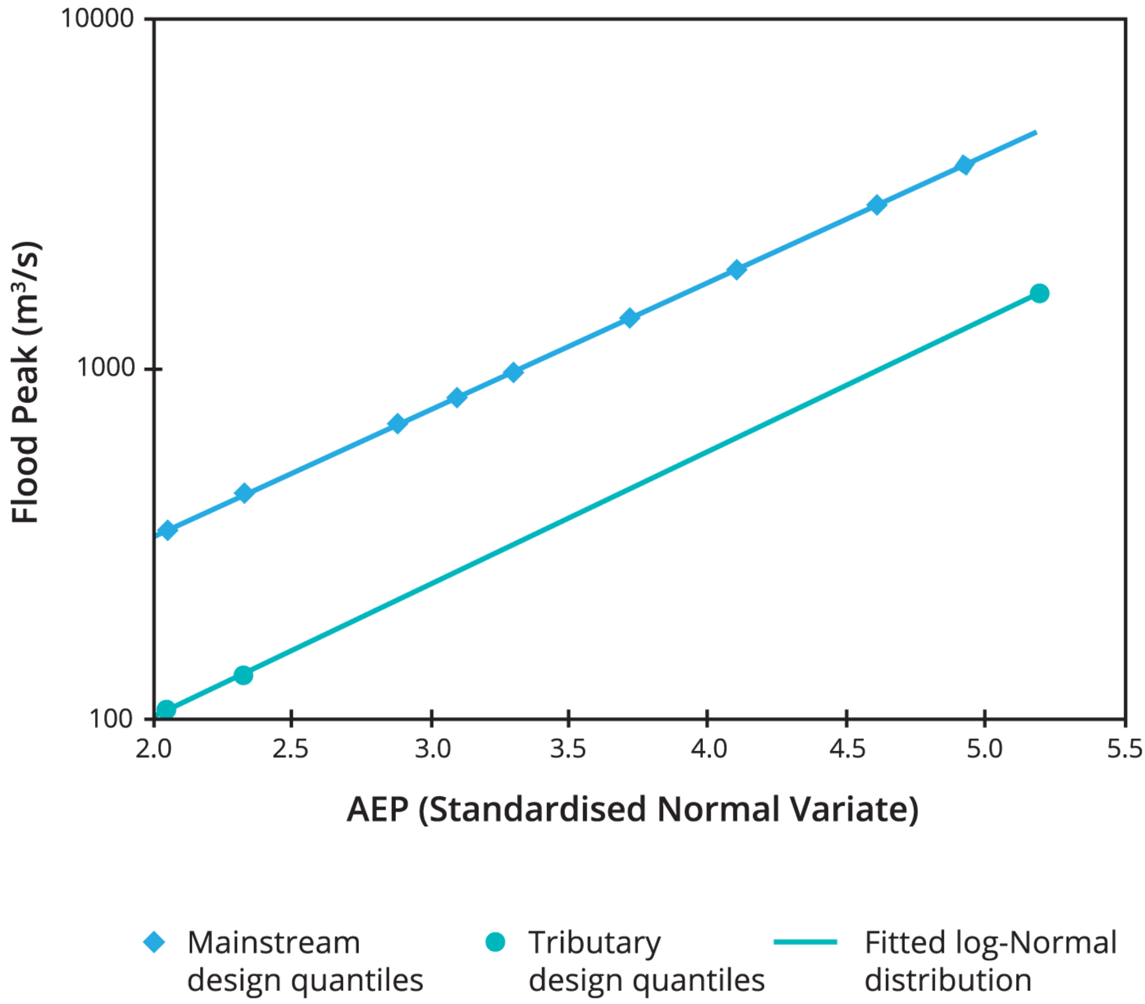

Figure 8.8.9. Fitted log-Normal Flood Frequency Curves for Mainstream and Tributary Design Flows

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.

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