INFRASTRUCTURE NSW

HAWKESBURY-NEPEAN VALLEY REGIONAL FLOOD STUDY

FINAL REPORT

VOLUME 1 – MAIN REPORT

JULY 2019

HAWKESBURY-NEPEAN VALLEY REGIONAL FLOOD STUDY

FINAL REPORT

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FOREWORD

Context

The Hawkesbury-Nepean Valley has the most significant flood risk exposure in NSW, if not Australia. To better manage this risk, the NSW Government released *Resilient Valley, Resilient Communities*: Hawkesbury-Nepean Valley Flood Risk Management Strategy (Flood Strategy) in May 2017.

The Flood Strategy is a comprehensive long-term framework for the NSW Government, local councils, businesses and the community to work together to reduce and manage the flood risk in the valley.

A key output of the Flood Strategy is the Hawkesbury-Nepean Valley Regional Flood Study (Regional Flood Study) – a technical document describing the existing flood behaviour of the main Hawkesbury-Nepean River from Bents Basin near Wallacia downstream to Brooklyn Bridge, and the backwater flooding associated with river flooding.

Purpose

The high flood risk in the Hawkesbury-Nepean Valley means that having access to the best available flood information is essential. The last regional flood studies were prepared in the mid-1990s. There have been significant changes to science and technology, as well as some changes to the valley landscape since then. There is also a need to consider the potential impacts of climate change on flooding.

The Regional Flood Study was developed to provide contemporary flood risk information for the valley.

Adopting best practice

The Regional Flood Study is underpinned by best practice approaches and up-to-date flood guidelines as specified in:

- *Floodplain Development Manual: the management of flood liable land* (NSW Government, 2005), and associated guidance
- *Australian Rainfall and Runoff: a guide to flood estimation* (Ball et al., 2016)
- *Managing the Floodplain: a guide to best practice in flood risk management in Australia* (AIDR, 2017).

This study identifies the full range of flood behaviour and flood hazard in the floodplain to inform decision-making consistent with the intent of the *Floodplain Development Manual* and *Managing the Floodplain*.

How should the Regional Flood Study be used?

The Regional Flood Study will be used to:

- help local communities understand their flood risk - providing accessible flood information including new flood maps to show flood extents and depths

- inform:
    - the continual improvement of emergency management and evacuation plans which require a detailed understanding of how floods behave and vary
    - regional land use and road planning, particularly the development of a new regional land use planning framework and road evacuation master plan
    - local land use planning, including through local environmental plans, development control plans and other council flood plans and policies
    - more accurate pricing of flood risk to help asset owners make decisions about flood risk mitigation measures, and for the insurance industry to more accurately price insurance premiums.
- underpin the ongoing assessment of regional flood mitigation options.

This Regional Flood Study supersedes previous regional studies for the Hawkesbury-Nepean River. It adopts a 'Monte Carlo' modelling approach, generating thousands of potential events to mimic the variability of actual floods in the valley. The new study has also used the best available guidance and information to assess the impacts of climate change on flooding.

The study provides the best source of information for defining the limits of flood prone land (up to the probable maximum flood or PMF) and for regional flood evacuation planning.

This Regional Flood Study covers a large geographic area and focuses on regional scale flooding. It does not include shorter-duration local catchment flooding or overland flow inundation.

Where more detailed flood modelling has been undertaken for locations in the floodplain, these studies may provide a better understanding of local flood behaviour. For example, Penrith City Council's *Nepean River Flood Study*[#] uses a more detailed model that may be more suited for the setting of flood planning levels within its study limits.

The best available information may come from a range of different studies, which may overlap in footprint. For a complete understanding of potential flood behaviours, councils and other users should consider the interaction of regional and localised flooding. What is most fit for purpose may depend on location and the decision being made.

Where the Regional Flood Study has revealed significant changes in flood levels, such as at Wallacia, more detailed investigations are required ahead of any decision to amend existing flood plans or policies.

[#] *The Nepean River Flood Study was adopted by Penrith City Council on 26 November 2018.*

Independent review

The Regional Flood Study was reviewed by a highly experienced, independent flood expert to check the validity and accuracy of the data, method and results.

The review found that the methodology was appropriate for the regional scale of the study and adopted some of the most complex hydrological methods available. This represented some of the most rigorous assessment that has been undertaken in Australia.

The reviewer reconfirmed the need to develop a more detailed, 'two-dimensional' hydraulic model to better represent the complex behaviour of flows in the floodplain.

The former Office of Environment and Heritage (now part of the Department of Planning, Industry and Environment) also provided comment.

Local councils in the Hawkesbury-Nepean Valley floodplain provided comment in relation to their areas of interest.

Key outputs

The Hawkesbury-Nepean Valley Regional Flood Study is a technical report comprising three volumes, as follows:

- Volume 1 – Main report
- Volume 2 – Appendices
- Volume 3 – Map book
    - Parts A/B – Flood levels, extents and depths
    - Part C – Provisional flood hazard
    - Part D – Hydraulic categorisation.

The results of the Regional Flood Study are also being used to develop a new, interactive online flood mapping tool to make it easier for people living and working in the valley to understand their flood risk.

In summary

While major floods are rare, their consequences can be catastrophic. Maintaining contemporary flood information, as detailed in the Regional Flood Study, is critical for building increased awareness and resilience in the Hawkesbury-Nepean Valley – one of the most flood-exposed regions of Australia.

This study has generated, for the first time, a large range of potential flood characteristics that better reflect the variability of floods that can occur in the floodplain.

Simon Draper
CEO, Infrastructure NSW

July 2019

EXECUTIVE SUMMARY

Project overview

The last regional flood studies for the Hawkesbury-Nepean Valley were prepared in 1996/1997. Since then there have been advances in the science of flood modelling and changes in the valley landscape. The understanding of the potential impacts of climate change on flooding has also improved. The development of contemporary information about flood behaviour is essential for understanding and managing flood risk in the valley, one of the most exposed floodplains in Australia.

In NSW, councils have primary responsibility for managing flood risk in their areas. In the Hawkesbury-Nepean Valley from Wallacia to Brooklyn, eight local government areas exercise this responsibility. The regional dimensions of flooding in the valley commend a coordinated approach to flood investigations. For this reason, the NSW Government commissioned specialist floodplain management firm, WMAwater Pty Ltd, to prepare a new Hawkesbury-Nepean Valley Regional Flood Study (Regional Flood Study).

This Regional Flood Study is a technical document describing the flood behaviour of the main Hawkesbury-Nepean River from Bents Basin near Wallacia downstream to Brooklyn Bridge, and associated backwater flooding, for existing conditions and under projected climate change. It does not include local catchment flooding or local overland flow inundation. The modelling of infrastructure options to mitigate downstream flooding is beyond the scope of this flood study and is addressed separately.

Study area

The Hawkesbury-Nepean Valley consists of a sequence of floodplains interspersed with incised sandstone gorges. The most upstream floodplain in the study area is around Wallacia. Below this the Nepean River joins the Warragamba River to discharge into another floodplain at Penrith and Emu Plains. The floodplain becomes constricted at Castlereagh although this is not a gorge on the same scale as others in the valley. The major Richmond-Windsor floodplain is located below Yarramundi. The river then enters the lower Hawkesbury River below Wilberforce and a series of incised sandstone gorges that extend around 100 kilometres from Sackville to the ocean at Broken Bay.

Due to its history the river has two names: the Nepean River upstream of the junction of the Grose River at Yarramundi, and the Hawkesbury River downstream to the coast.

History overview

Australia’s longest historical flood record at Windsor, oral history from Aboriginal people, and geomorphological and geological clues in the landscape all point to a long history of floods in the Hawkesbury-Nepean system. The largest known flood in modern times occurred in 1867, but there is evidence of a palaeoflood at least eight metres higher having occurred thousands of years ago. The historical flood records point to a pattern in which there are multidecadal periods with frequent and large floods, interspersed with similarly lengthy periods with infrequent and small floods. The relatively long historical flood record at Windsor and Penrith provided the opportunity for a detailed flood frequency analysis.

Method

This Regional Flood Study updates the 1996 Flood Study (WMA, 1996), which at the time was the most extensive flood study ever carried out in Australia. The 1996 Flood Study included a detailed analysis of primary flood data and used the most up-to-date technology at the time. It forms a foundation for this revised work.

This new study uses best practice and the latest techniques in flood estimation to define flood behaviour in the Hawkesbury-Nepean Valley. As part of this study, the previous flood frequency analysis was updated using current techniques and 22 years of additional rainfall and flow data, albeit with no additional major floods over that period. The flood frequency analysis was used to calibrate the hydrologic model and to verify the flow-frequency distribution derived from the Monte Carlo simulations (see below).

A hydrologic model (RORB) was developed to calculate flood flows resulting from rainfall events. This was calibrated and verified using seven historical floods including two large events (April/May 1988 and August 1990). Calibration sites included four stream gauging stations located upstream of Warragamba Dam, Warragamba Dam and various stations downstream.

A quasi two-dimensional hydraulic model (RUBICON) was developed to calculate peak flood levels resulting from the flood flows. This was calibrated and verified using ten historical flood events. The model also reproduces the 1867 flood profile down the river with the limited data available for that event.

A Monte Carlo modelling framework was established to better replicate the observed variability in actual flood events. Real flood events exhibit an enormous degree of variability, most of which is determined by where and when rain falls. Flood events are also influenced by how wet the catchment is before the event and – in the case of the Hawkesbury-Nepean floodplain – the levels in Warragamba Dam prior to an event. Fully understanding this variability is important for managing flood risk to life and property.

To better account for this variability, design flood estimation in Australia is moving from a single event per quantile (such as the 1 in 100 chance per year flood) to Monte Carlo modelling, where thousands of events are simulated with variable inputs. For the current study, the variability in each of the following key input variables was estimated from observed events, and a Monte Carlo framework was established to model flood events based on randomly sampling each variable from within the range of possible inputs:

- rainfall intensity and frequency
- spatial pattern of rainfall – where in the catchment rain falls
- temporal pattern of rainfall – when in the event rain falls
- initial loss – rain ‘lost’ at the beginning of an event through infiltration into the soil
- pre-burst rainfall – rain that occurs before the most intense burst of the storm
- dam drawdown – the level of Warragamba Dam before the start of an event
- relative timings of tributary inflows
- tides.

As depicted in the figure below, the variables from the Monte Carlo analysis were inputs to the hydrological model, and the resultant flows, together with the other variables including relative timings of tributary inflows and tides, were inputs into the hydraulic model. This was used to assess flood behaviour.

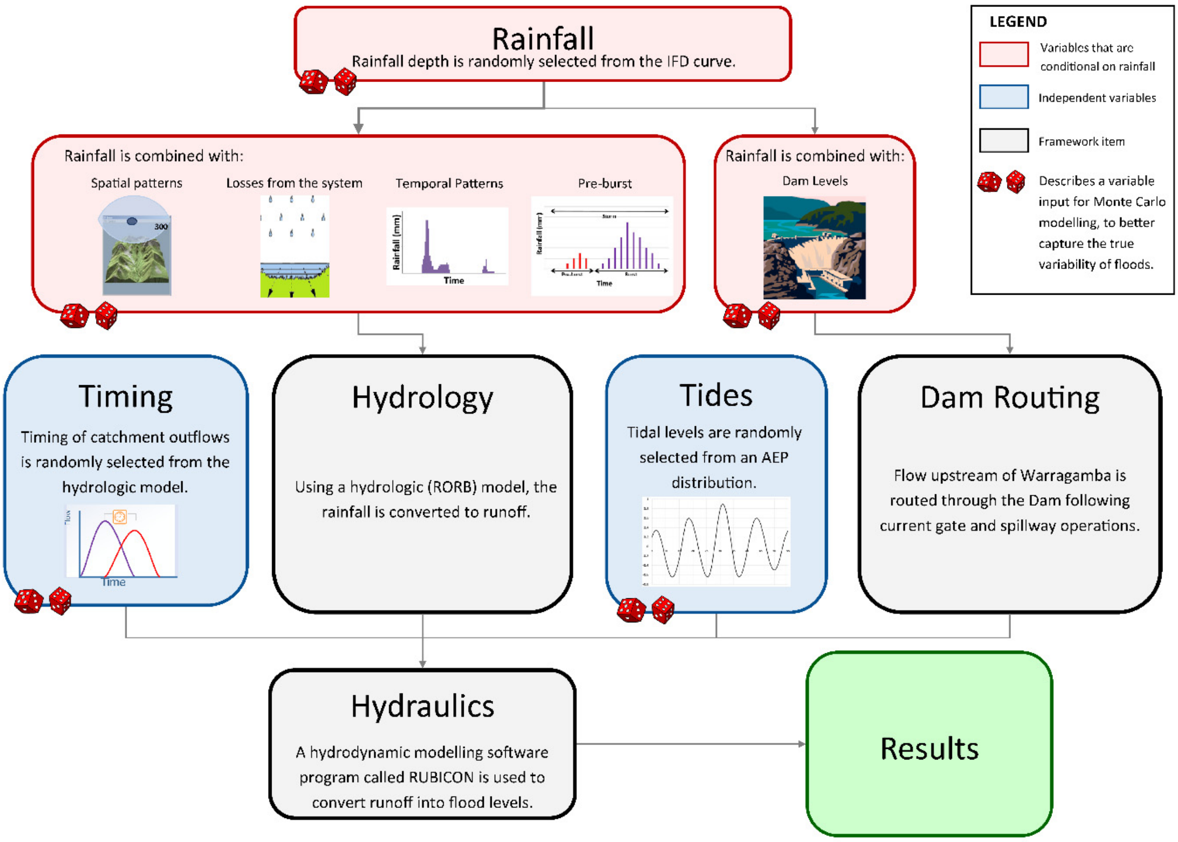

Figure 1 Flood modelling methodology

Design floods

The Regional Flood Study calculates flood levels, extents, depths, provisional flood hazard and hydraulic categories for a series of design events, where the design events are representative of the frequency quantiles from the Monte Carlo modelling. The design events included are the 1 in 5, 1 in 10, 1 in 20, 1 in 50, 1 in 100, 1 in 200, 1 in 500, 1 in 1000, 1 in 2000, 1 in 5000 AEP events and the probable maximum flood (PMF). For the purposes of this study, the PMF is estimated using the probable maximum precipitation (PMP). Compared to the previous 1996 Flood Study, the current study resulted in minor changes to the 1 in 100 AEP flows for Warragamba and Windsor, but a reduction of 12 per cent for Penrith.

Summary of results

Table 1 summarises the design flood quantile levels at Penrith, North Richmond, Windsor and Wisemans Ferry, and compares them to previous regional flood studies from 1996/1997.

Compared to the previous regional studies, this Regional Flood Study found that:

- the level of the 1 in 5 AEP event has decreased across the valley because the new study allows for the possibility that Warragamba Dam could be below its full water supply level at the beginning of the flood event and would be able to hold back inflows from smaller floods
- the 1 in 100 AEP flood level at Penrith has decreased slightly from previous studies due to the refined flood frequency methodology and longer flood record
- at Wisemans Ferry, flood events between 1 in 20 AEP and 1 in 200 AEP have increased by up to 0.7 metres as a result of updated data and the new modelling approach
- peak flood levels for the PMF have increased at several sites because of new approaches to modelling this extreme event, and updated information.

For the Wallacia floodplain, this new flood study identifies the very large flood height range (over 21 metres) between the 1 in 100 AEP and PMF events. This results from the potential for floodwaters from both the Warragamba and Nepean Rivers to back up due to the constrictive effects of the gorges between Wallacia and Penrith. More detailed investigation of the interaction of these rivers is required ahead of any decision to amend existing flood plans or policies.

Provisional flood hazard and hydraulic categories were defined for a range of events. Nationally-accepted flood hazard categories for the 1 in 100 AEP event indicate that the majority of the floodplain is considered unsafe for vehicles and people, with buildings requiring special engineering design and construction, or being vulnerable to failure.

Using the Monte Carlo approach, the Regional Flood Study also generated outputs on rate of rise, time to rise, rate of fall, time to fall, time above critical levels and travel time for key locations in the floodplain. This new information is important for assessing risk to life and to inform emergency response planning.

Climate change rainfall increase and sea level rise

There is strong evidence that increases in global temperatures will lead to an increase in the intensity of rare rainfall. This Regional Flood Study has assessed the impacts on flooding of both climate change induced rainfall increases and sea level rise using the best available information.

The study found that increases in rainfall intensity result in a significant increase in flood levels. A 9.1 per cent rainfall increase under climate change would raise the current 1 in 100 AEP flood level at Windsor by 0.71 metres. The current 1 in 100 AEP flood level at Windsor would be reached more frequently with a 9.1 per cent rainfall increase under climate change – becoming a 1 in 65 AEP event. Sea level rise impacts on the 1 in 100 AEP are largely confined to the lower reaches of the river.

Conclusions and limitations

This Regional Flood Study provides an update to the publicly available flood information for the Hawkesbury-Nepean Valley which was previously provided more than 20 years ago. The study will be used by a range of stakeholders including councils within the valley and the NSW Government to inform flood planning and emergency management. The outputs of this Regional Flood Study will provide contemporary information on flood risk important for increasing community awareness of their flood risk and building resilience.

The application of the Monte Carlo modelling approach better replicates the variability of floods in the Hawkesbury-Nepean Valley. Therefore, it provides the best source of information for defining the limits of flood prone land (PMF) and to inform regional land use and evacuation planning.

Where more detailed flood modelling within the limits of the Regional Flood Study has been undertaken, these more detailed studies may provide a better understanding of local flood behaviour in their study areas. For example, the *Nepean River Flood Study* prepared for Penrith City Council uses a more detailed model that may be more suitable for the setting of flood planning levels within its study limits.

This Regional Flood Study describes the flood behaviour dominated by Hawkesbury-Nepean riverine flooding and its backwater effects between Bents Basin and Brooklyn Bridge. Local catchments are modelled to have the same duration rainfall event as that which causes the highest flood levels in the main river. Shorter duration events are likely to result in higher flood levels within tributaries such as South Creek. Local flood studies will also be required in these locations. Local councils should be consulted to ascertain whether information is available to understand the combined flood risk at particular locations in the valley.

Table 1. Comparison of peak flood levels for design quantiles compared with previous flood studies

1 in X chance per year flood	Penrith (Victoria Bridge)			North Richmond Bridge			Windsor Bridge			Wisemans Ferry (Webbs Creek Ferry site)		
	1997 study[1] (1996 study[2])	Current study	Change	1997 study[1] (1996 study[2])	Current study	Change	1997 study[1] (1996 study[2])	Current study	Change	1997 study[3]	Current study	Change
	m AHD	m AHD	m	m AHD	m AHD	m	m AHD	m AHD	m	m AHD	m AHD	m
5	20.1	19.6	-0.5	12.5	11.4	-1.1	11.1	9.9	-1.2	3.2	2.8	-0.4
10	21.6	21.3	-0.3	14	13.7	-0.3	12.3	11.9	-0.4	NA	3.7	NA
20	23.4	23.3	-0.1	15.3	15.4	0.1	13.7	13.7	0	4.4	4.8	0.4
50	24.9	24.8	-0.1	16.4	16.5	0.1	15.7	16.1	0.4	5.6	6.2	0.6
100	26.1	25.8	-0.3	17.5	17.6	0.1	17.3	17.3	0	6.7	7.2	0.5
200	26.9	26.5	-0.4	18.9	18.6	-0.3	18.7	18.4	-0.3	7.5	8.2	0.7
500	27.5	27.1	-0.4	20.4	19.8	-0.6	20.2	19.6	-0.6	NA	9.5	NA
1000	28.6 (28.0)	27.5	-1.1	22.1 (21.5)	20.7	-1.4	21.9 (21.3)	20.6	-1.3	NA	10.5	NA
2000	NA	28.4	NA	NA	21.9	NA	NA	21.7	NA	NA	11.4	NA
5000	NA	29.4	NA	NA	22.8	NA	NA	22.6	NA	NA	12.8	NA
PMF	32.1 (30.9)	32.8	0.7	26.5 (25.6)	26.8	0.3	26.4 (25.5)	26.7	0.3	16.3	14.5	-1.9

1. Webb, McKeown & Associates (1997). Note, these design flood levels allow for Warragamba Dam's auxiliary spillway, which was completed in 2002.
2. Webb, McKeown & Associates (1996). Note, these older design flood levels do not allow for Warragamba Dam's auxiliary spillway, which was completed in 2002, and assumed that the dam does not fail in the PMF event. Should the dam fail, the PMF levels at Penrith, North Richmond and Windsor were modelled to peak at 35.6m AHD, 29.0m AHD and 28.9m AHD, respectively.
3. Australian Water and Coastal Studies Pty Ltd (AWACS) (1997), Tables 10.1 and 10.3.

HAWKESBURY-NEPEAN VALLEY REGIONAL FLOOD STUDY

TABLE OF CONTENTS

VOLUME 2: APPENDICES

LIST OF TABLES

Appendix A

Appendix B

Appendix C

Appendix D

Appendix F

Appendix G

Appendix H

Appendix J

Appendix K

Appendix L

Appendix M

Appendix N

LIST OF DIAGRAMS

LIST OF FIGURES – MAIN REPORT

LIST OF FIGURES – APPENDICES

Appendix A

Appendix B: Historical hydrologic figures

Appendix C: Historical hydraulic figures

Appendix E: Stage-frequency curves

Appendix F: Rate of rise scatter plots

Appendix G: Time to rise scatter plots

Appendix H: Rate of fall scatter plots

Appendix L: Travel time scatter plots

Appendix M

No figures

Appendix N

No figures

Appendix O

Not used

Appendix P

No figures

LIST OF ACRONYMS

AEP	Annual Exceedance Probability
AIDR	Australian Institute for Disaster Resilience
ALS	Airborne Laser Scanning
ANCOLD	Australian National Committee on Large Dams
ARF	Areal Reduction Factor
ARI	Average Recurrence Interval
ARR	Australian Rainfall and Runoff
BoM	Bureau of Meteorology
CL	Continuing Loss
CPU	Central Processing Unit
DECC	NSW Department of Environment and Climate Change (now OEH)
DECCW	NSW Department of Environment, Climate Change and Water (now OEH)
DEM	Digital Elevation Model
DLWC	NSW Department of Land and Water Conservation (now OEH)
DNR	NSW Department of Natural Resources (now OEH)
DTM	Digital Terrain Model
D/S	Downstream
ECL	East Coast Low
ENSO	El Niño Southern Oscillation
EPI	Environmental Planning Instrument
FFA	Flood Frequency Analysis
GEV	Generalized Extreme Value
GIS	Geographic Information System
GPS	Global Positioning System
GPU	Graphics Processing Unit
HDS	Hydrographic Data Station
HEPS	Hydroelectric Power Station
HPC	Heavily Parallelised Compute
IFD	Intensity, Frequency and Duration (Rainfall)
IL	Initial Loss
IPO	Inter decadal Pacific Oscillation
LP3	log Pearson III
LPI	NSW Department of Lands and Property Information
m AHD	metres above Australian Height Datum
NSW SES	NSW State Emergency Service
OEH	NSW Office of Environment and Heritage
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
RCP	Representative Concentration Pathway
RORB	rainfall runoff hydrologic model
RUBICON	quasi two-dimensional hydraulic model
SD	Standard Datum
SMEC	Snowy Mountains Engineering Corporation
TUFLOW	one-dimensional (1D) and two-dimensional (2D) flood and tide simulation software (hydraulic model)
U/S	Upstream
WATHNET	water supply system model package

ADOPTED TERMINOLOGY

Australian rainfall and runoff – A guide to flood estimation (ARR) (Pilgrim, 1987) is a national guideline document, data and software suite that can be used for the estimation of design flood characteristics in Australia. The fourth edition of ARR was published by the Commonwealth of Australia in 2016 (ARR 2016) (Ball et al., 2016). Geoscience Australia supports ARR as part of its role to provide authoritative, independent information and advice to the Australian Government and other stakeholders to support risk mitigation and community resilience.

ARR 2016 recommends flood frequency terminology that is not misleading to the public and stakeholders. Flood events are described in terms of the chance of occurrence in any one year, with this probability normally assigned to a flood based on its peak level. While there is a very high correlation between peak flow and peak level, individual floods show considerable variability in terms of flood volume, rate of rise and duration of inundation. This variability is caused by how wet the catchment is prior to an event and when, where and how much rain falls on the catchment. Floods occur randomly, so one flood event does not change the chance of a subsequent flood occurring. Rare events may occur in clusters: two floods with approximately a one per cent chance per year occurred in Kempsey in 1949 and 1950; the two largest floods in Brisbane occurred two weeks apart in 1893. Therefore, the use of terms such as 'recurrence interval', 'return period', and even 'average recurrence interval' (ARI), are no longer recommended as they imply that a given event magnitude is only exceeded at regular intervals such as every 100 years.

ARR 2016 recommends the use of Annual Exceedance Probability (AEP) to describe flood probabilities or frequency. Annual Exceedance Probability (AEP) is the probability of an event being equalled or exceeded within a year. AEP may be expressed as either a percentage (%) or 1 in X. Floodplain management typically uses the percentage form of terminology. Therefore a 1% AEP event or 1 in 100 AEP has a one per cent chance of being equalled or exceeded in any year. This report adopts the terminology of 1 in 100 AEP.

ARI and AEP are often mistaken as being interchangeable for events equal to or more frequent than 10% AEP. The table below describes how they are subtly different. It categorises flood events according to the ARR 2016 classification.

The probable maximum flood (PMF) is the largest flood that could reasonably be expected to occur for a catchment. For the purposes of floodplain management, and consistent with the NSW Government's *Floodplain Development Manual*, the PMF is estimated using the probable maximum precipitation (PMP) and a single temporal pattern. Due to the conservativeness applied to other factors influencing flooding, a PMP does not translate to a PMF of the same probability. But for the purposes of floodplain management, the probability of the PMP may be assigned to the PMF.

The Bureau of Meteorology (BoM) and NSW State Emergency Service (NSW SES) use the terms 'minor', 'moderate' and 'major' to describe floods. These terms do not relate to a particular probability at any location but are assigned based on local consequences. For this reason, they vary in probability and severity at different locations along the rivers. For example, at Windsor gauge, minor floods are those between 5.8 and 7.0 metres, moderate floods are between 7.0 and 12.2 metres, and major floods exceed 12.2 metres.

Design event quantiles such as a 1 in 100 AEP are used to refer to standard probabilities of events used in design flood estimation for example those listed in the table below.

EY	AEP (%)	AEP (1 in x)	ARI
12			
6	99.75	1.002	0.17
4	98.17	1.02	0.25
3	95.02	1.05	0.33
2	86.47	1.16	0.5
1	63.21	1.58	1
0.69	50	2	1.44
0.5	39.35	2.54	2
0.22	20	5	4.48
0.2	18.13	5.52	5
0.11	10	10	9.49
0.05	5	20	20
0.02	2	50	50
0.01	1	100	100
0.005	0.5	200	200
0.002	0.2	500	500
0.001	0.1	1000	1000
0.0005	0.05	2000	2000
0.0002	0.02	5000	5000
		↓	
		PMP/ PMPDF	

Note: EY = Exceedances per Year; AEP = Annual Exceedance Probability; ARI = Average Recurrence Interval
Source: adapted from ARR 2016 (Ball et al., 2016)

1. INTRODUCTION

1.1. Project overview

This Hawkesbury-Nepean Valley Regional Flood Study (Regional Flood Study) is the technical document describing the existing flood behaviour of the main Hawkesbury-Nepean River from Bents Basin near Wallacia and Warragamba Dam downstream to Brooklyn Bridge, and the backwater flooding associated with this main river flooding. It does not include local catchment flooding or local overland flow inundation.

Additionally, the study includes sensitivity testing for sea level rise and increased rainfall intensity/volume due to climate change.

The current work builds on extensive flood modelling and its review since the 1980s. Updated flood modelling shows that scientific understanding of the probability of flooding on the Hawkesbury-Nepean Valley has not changed significantly; however new techniques allow a better understanding of other characteristics of floods such as rate of rise.

A feature of the current study is its utilisation of a Monte Carlo approach that better replicates the variability observed in real flood events, including the interaction of dam levels and flood behaviour, rate of rise, and evacuation warning. All of these aspects are important for managing flood risk.

This Regional Flood Study has been prepared in the form of a 'traditional' flood study, in accordance with the latest guidelines:

- *Floodplain Development Manual: the management of flood liable land* (NSW Government, 2005) (the manual)
- *Australian Rainfall and Runoff: a guide to flood estimation* (Ball et al., 2016) (ARR 2016)
- *Managing the Floodplain: a guide to best practice in flood risk management in Australia* (AIDR, 2017).

The basis of modelling was established in the *Warragamba Dam Auxiliary Spillway Environmental Impact Study Flood Study* (Webb, McKeown & Associates, 1996, referred to as the 1996 Flood Study). As there have been no major floods since 1990, no additional calibration events were available. Using this earlier work as a basis, the modelling framework was updated to account for all of the factors that influence flooding in the Hawkesbury-Nepean Valley and to apply the latest techniques in design flood estimation.

1.2. Structure of this report

The Hawkesbury-Nepean Valley Regional Flood Study is provided in three volumes, as follows:

- Volume 1 – Main report
- Volume 2 – Appendices
- Volume 3 – Map book
    - Parts A/B – Flood levels, extents and depths
    - Part C – Provisional flood hazard
    - Part D – Hydraulic categorisation.

The following describes the structure of Volumes 1 and 2:

- ***Volume 1 Main report:***
    - *Section 1: Introduction*
    - *Section 2: Modelling approach.* Overview of the modelling approach adopted for the study
    - *Section 3: Background.* Description of the study area, history of flooding of the valley, and previous studies undertaken in the study area
    - *Section 4: Available data.* Data used in the study for the establishment of hydrologic and hydraulic models
    - *Section 5: Flood frequency analysis.* Overview of flood frequency analysis undertaken to a number of gauge records in the catchment
    - *Section 6: Hydrologic modelling.* Development and calibration of a hydrologic model to turn rainfall into flow
    - *Section 7: Hydraulic modelling.* Development and calibration of the RUBICON model to turn flow into flood levels
    - *Section 8: Design flood behaviour.* Development of a Monte Carlo framework to model design flood behaviour in the catchment
    - *Section 9: Results.* Analysis of results from the Monte Carlo model including rate of rise and warning time
    - *Section 10: Climate change and sea level rise.* Assessment of the impact of climate change related rainfall increases and sea level rise on flood behaviour
    - *Section 11: Development of gridded results from a RUBICON model.* Methodology for mapping of RUBICON results
    - *Section 12: Provisional flood hazard and hydraulic categories.* Mapping of hazard and hydraulic categories used for emergency management
    - *Section 13: Evacuation events.* Selection of events for use in the evacuation modelling
    - *Section 14: Limitations and next steps*
    - *Section 15: Conclusions*
    - *Section 16: References*
    - *Glossary*

- ***Volume 2 Appendices:***
    - *Appendix A: Flood records*
    - *Appendix B: Rating curves and hydrologic modelling figures*
    - *Appendix C: Hydraulic model development*
    - *Appendix D: Two-dimensional model development and hazard and hydraulic categories*
    - *Appendix E: Stage frequency curves*
    - *Appendix F: Rate of rise plots*
    - *Appendix G: Time to rise plots*
    - *Appendix H: Rate of fall plots*
    - *Appendix I: (Not used)*
    - *Appendix J: Time to fall plots*
    - *Appendix K: Time above critical level*
    - *Appendix L: Travel time plots*
    - *Appendix M: Evacuation events*
    - *Appendix N: Representative events*
    - *Appendix O: (Not used)*
    - *Appendix P: Gridding software and mapping*

2. ADOPTED APPROACH

The primary objective of this flood study is to define flood behaviour in the study area under historical and existing floodplain conditions, as well as assessing possible future variations in flood behaviour due to climate change.

The first phase involves data collection (refer to Section 4). Information was collected on observed flood levels, rainfall, structures and topography. This information was used to inform the study.

The adopted flood assessment methodology was influenced by the study objectives, best practice and the quality and quantity of available data. There are two basic approaches to determining design flood levels, namely:

- a *flood frequency* approach based upon a statistical analysis of the flood record
- a *rainfall/runoff routing* approach (hydrologic modelling) to obtain flows, which are then input into a hydraulic model of the floodplain.

The flood frequency approach was undertaken for the Warragamba, Penrith and Windsor/Sackville gauges as part of the 1996 Flood Study. A flood frequency analysis (FFA) was updated as part of the current study using the latest techniques and additional data collected since the previous flood study. The results of the FFA are discussed in Section 5. The FFA was used to inform the continuing loss parameter in the hydrologic model and to verify the Monte Carlo results (see Section 8).

A hydrologic (RORB) model was developed to determine inflows from each catchment (Section 6). The hydrologic model was calibrated to a range of historical events. It is largely unchanged from the 1996 Flood Study; however, the design rainfall inputs were updated to reflect current best practice.

A quasi two-dimensional hydrodynamic (RUBICON) model was used to define the flood behaviour using inflows from the hydrologic model and topographic information discussed in Section 4.1. The development of the RUBICON model including calibration and verification to ten historical floods is documented in Section 7. The calibrated hydraulic model was used to assess the flood behaviour. The only modification to the hydraulic model since 1996 is the addition of the M4 culverts.

A Monte Carlo framework was established which aims to mimic observed flood behaviour. The Monte Carlo framework allows thousands of events to be simulated. The development of the framework is discussed in Section 8. Results are presented within the report for the entire ensemble as well as for discrete events: 1 in 5, 1 in 10, 1 in 20, 1 in 50, 1 in 100, 1 in 200, 1 in 500, 1 in 1,000, 1 in 2,000 and 1 in 5,000 AEP design events, and probable maximum flood (PMF) design event, which for this Regional Flood Study was estimated from the probable maximum precipitation (PMP).

Table 2 summarises the timeframe when different components of the project were undertaken.

Table 2: Project components and date undertaken

Project Component	Undertaken	Relevant Section
Data Collection	1996 study: majority of data collected Current study: ALS survey, aerial photography, additional years of rainfall and streamflow data, hydrosurvey, M4 culverts	4
Flood Frequency Analysis	1996 study: majority of data collected Current study: additional years of record; updated to use ARR 2016 techniques	5
Hydrologic Model	1996 study: model calibration Current study: no additional calibration since no large floods since previous study; no changes to model other than making it run faster for Monte Carlo modelling	6
Dam Routing and Dam Water levels	Current study: new techniques applied	4
Hydraulic Model	1996 study: model calibration Current study: no additional calibration since no large floods since previous study; no changes to model other than making it run faster for Monte Carlo modelling and addition of M4 culverts	7
Design Event Modelling	Current study: updated inputs – IFD, spatial patterns, temporal patterns based on extreme storm database; Monte Carlo framework developed	8
Climate Change	Current study: new techniques using latest advice	10
Flood Hazard and Hydraulic Categories	Current study: Mapping of provisional flood hazard and hydraulic categories according to best practice	12
Results: Processing, Outputs and Mapping	Current study: new techniques applied	9, 11, 13 and Appendices E to P

3. BACKGROUND

3.1. Study area

The Hawkesbury-Nepean River drains a catchment of 22,000 square kilometres to the Pacific Ocean at Broken Bay. The catchment is shown in Figure 1 and the study area floodplain is shown in Figure 2.

The Hawkesbury-Nepean Valley consists of a sequence of floodplains interspersed with incised meanders in sandstone gorges. The most upstream floodplain on the Nepean River is near Camden. This is linked by a very narrow gorge upstream of Bents Basin to a small floodplain around Wallacia. Below this, the Nepean River runs for some 15 kilometres through the Nepean Gorge, joining the Warragamba River, to discharge into another floodplain at Penrith and Emu Plains. The floodplain becomes constricted at Castlereagh, although this is not a gorge on the same scale as others in the valley. The sizeable Richmond-Windsor floodplain opens out below Yarramundi. The river then enters the Hawkesbury Gorge (also known as Sackville Gorge) which starts below Wilberforce and extends over 100 kilometres to the ocean at Broken Bay. This gorge is punctuated with several small floodplains.

A consequence of this topography is that in major floods, the river forms a series of large ponds (floodplains) where velocities are low outside the main channels and water slopes flatter. The ponds are connected by steep, fast flowing channels (gorges). Flood levels within the ponds are controlled by the narrow channels downstream that restrict the amount of water which can escape down the river (ERM Mitchell McCotter, 1995).

Warragamba Dam is located on the Warragamba River, 55 kilometres west of the Sydney Central Business District. Warragamba River is a major tributary of the Hawkesbury-Nepean River system. The storage reservoir formed by Warragamba Dam is known as Lake Burragorang.

The catchment that drains to Warragamba Dam extends to the south, to the vicinity of Lake Bathurst where rainfall is comparatively low. This area is drained by the Mulwaree Ponds, which joins the Wollondilly River, flowing from the west at Goulburn. The Wollondilly River then travels in a generally north-easterly direction to eventually enter Lake Burragorang. Of the many tributaries entering the Wollondilly River, the most important is the Wingecarribee River which rises in high rainfall country near Bowral to the east.

Apart from the Wollondilly River, the main inflows to Lake Burragorang are the Coxs and Kowmung Rivers in the west and the Nattai and Little Rivers in the east. The Coxs River rises in the Great Dividing Range west of Lithgow.

Warragamba Dam is situated in a steep, narrow gorge. Before the dam was built the gorge carried the Warragamba River from the junction of the Wollondilly and Coxs Rivers down to the Nepean River below Wallacia. The total length of the Warragamba River was 22 kilometres; now all but the 3.3 kilometres length of river downstream of the dam are submerged below Lake Burragorang.

Although the Nepean River catchment at its junction with the Warragamba River is only 20 per cent of the size of the Warragamba catchment, the Nepean River drains a region of high rainfall along the top of the Illawarra Escarpment, and its contribution to downstream flooding is usually greater than a simple proportion of catchment area might suggest.

Downstream of the junction, the Nepean River continues to flow through a narrow gorge until it enters more open country just upstream of Penrith. The elevation of the floodplain in the vicinity of Penrith, including Emu Plains, is surprisingly high and does not convey floodwaters until floods reach almost the magnitude of a 1 in 100 AEP event (one per cent chance of being equalled or exceeded in any year).

The Grose River is a major tributary that joins the Nepean River at Yarramundi. While it has a catchment of only 650 square kilometres, it drains a high rainfall area and can have a significant effect on flooding at Richmond and Windsor. This catchment responds very quickly to rainfall and can sharply increase river levels at North Richmond before floodwaters arrive from the Nepean River. Flood flows from the Grose River alone can produce flooding of the Hawkesbury River downstream, though not to levels posing the greatest risk to life and property.

The Nepean River is known as the Hawkesbury River below the Grose River junction at Yarramundi.

The Richmond/Windsor lowlands are located below Yarramundi. These are extensive floodplains that are mainly used for agricultural purposes and that are completely inundated in a 1 in 10 AEP event. The main towns in the area, Richmond and Windsor, are for the most part elevated above frequent floods, but are seriously affected by major floods.

South Creek joins the Hawkesbury River at Windsor. Although its catchment area of 640 square kilometres is virtually the same as the Grose River, it has less influence on flooding as it receives less rainfall, responds more slowly, and has extensive flood storage in the lower reaches. The dominant flood mechanism in the lower reaches of South Creek is backwater flooding from the Hawkesbury River.

Below Wilberforce the Hawkesbury River enters another gorge, the Hawkesbury (Sackville) Gorge, which extends for over 100 kilometres to the ocean at Broken Bay. Along this gorge it is joined by the Colo River from the west and the Macdonald River from the north, along with a number of smaller tributaries.

Warragamba Dam controls approximately 40 per cent of the total area of the Hawkesbury-Nepean River catchment (22,000 square kilometres), but more significantly approximately 80 per cent of the catchment to Penrith, and 70 per cent to Windsor. There are five other major dams in the catchment: four on the headwaters of the Nepean River (Avon, Cataract, Cordeaux and Nepean Dams) and the Wingecarribee Reservoir in the headwaters of the Wingecarribee River. The total area controlled by the other dams is a very small proportion of the total catchment and they have minimal impact on even relatively minor floods downstream of Wallacia and Warragamba Dam.

The topography of the Hawkesbury-Nepean catchment varies from rugged, mountainous terrain, which covers nearly half of the area, to flat floodplains. The latter accounts for only a small percentage of the total catchment area but contains the majority of the urban development.

The catchment rises to 600 metres above sea level near the Avon River, 750 metres at the head of the Wollondilly River and over 1,300 metres on the Great Dividing Range at the head of the Kowmung River.

This Regional Flood Study focuses on flood behaviour downstream of Warragamba Dam and Bents Gorge on the Nepean River upstream of Wallacia.

3.2. History overview

Floods have played a major role in shaping the landscape of the Hawkesbury-Nepean Valley. The extensive floodplains of the Hawkesbury-Nepean River – notably around Richmond/Windsor – were formed over millions of years by the deposition of sediment during floods.

Aboriginal people in the Hawkesbury-Nepean Valley experienced loss from flooding, but also learned to adapt to flooding. Governor King learned from the traditional owners of the land that high floods occurred in about 1780 and in March 1788. In 1780, people took refuge in the tallest trees but were still swept away (Governor Phillip King, 1806). In 1799, it was reported that Aboriginal people perceived the threat and warned the new settlers of the coming flood (Governor John Hunter, 1799).

The first European explorers detected signs of significant floods. On their trip up the Hawkesbury River in 1789, at about Yarramundi, Governor Phillip and his party saw in the branches of trees 'vast quantities of large logs which had been hurried down by the force of the waters, and lodged thirty to forty feet above the common level of the river' (John Hunter, 1793).

The fertile floodplain around Windsor was settled by Europeans in 1794, and a good record of flooding is available since about that time, making the Windsor flood record the longest in Australia (see Appendix A). Floods were a frequent occurrence in the early years of the colony. Damage from the major flood of 1809 prompted Governor Macquarie to establish in 1810 five new townships on higher ground: Castlereagh, Richmond, Windsor, Pitt Town and Wilberforce. After further damaging floods in 1816 and 1817, Macquarie issued General Orders calling for settlers to relocate from their low-lying farms to the townships.

The period from 1820 to 1856 had fewer and smaller floods, the largest of these in 1830.

The period from 1857 to 1900 had many floods, including the highest and second-highest floods on record, in 1867 and 1864, respectively. The 1867 flood reached 19.7m AHD at Windsor, causing massive damage and the loss of 12 members of the Eather family at Cornwallis. Although many floods have occurred since then, none have come near the heights reached in 1867.

Nonetheless, some even higher floods occurred prior to the arrival of Europeans in 1788. After the 1867 flood, one observer from the Hawkesbury described that 'There are certainly in this district several indications of much higher inundations, but evidently works of very ancient floods, probably centuries ago' (George Pitt Jr, 1867). Palaeoflood investigations examine and study this ancient evidence. One such investigation examined deposits from floods in Fairlight Gorge near the junction of the Nepean and Warragamba Rivers (Saynor et al., 1993). Analysis of minerals and radiocarbon dating found that, at that location, a flood at least eight metres higher than the 1867 flood had occurred in the Holocene (that is, approximately within the last 10,000 years).

The period from 1901 to 1948 had fewer and smaller floods compared to the 1857–1900 period.

However, the period from 1949 to 1992 had more frequent and larger floods, despite the completion of Warragamba Dam for water supply in 1960.

No moderate or major floods (using NSW SES categories) have been observed at Windsor in the 27 years since 1992.

Since European observations began, the pattern of decades-long periods of either frequent and higher floods or infrequent and smaller floods has led scientists to describe the hydrological regime that evidently characterises the Hawkesbury-Nepean as either flood-dominated or drought-dominated regimes. On top of these underlying regimes are large annual variations in rainfall and runoff, such that floods can still occur in drought-dominated regimes, and droughts in flood-dominated regimes (Warner, 2009).

3.3. Warragamba Dam

Water supply development began in the Hawkesbury-Nepean catchment in the early 1880s with the construction of diversion weirs on the Nepean and Cataract Rivers. The four dams on the upper Nepean were completed between 1907 and 1935.

The Warragamba River was identified as a potential source of water supply to Sydney as early as 1845. Serious consideration was given to its use in the early twentieth century, but no work was undertaken until 1937. Construction of Warragamba Weir, one kilometre downstream of the dam site, began in that year as part of an emergency scheme prompted by the record drought. Pumping from the river started in 1940 and continued until February 1959.

Work on the construction of Warragamba Dam commenced in the late 1940s and was completed in 1960. The dam is of concrete gravity construction, 142 metres high and 351 metres wide. It holds back some 2,027 gigalitres of water.

The dam's primary spillway has four radial gates and a drum gate. Construction of a secondary fuse plug spillway began in 1998 and was completed in 2002.

3.4. Warragamba Dam Auxiliary Spillway Environmental Impact Study – Flood Study

Until recently, flooding in the Hawkesbury-Nepean valley was subject to what was probably the most extensive Flood Study ever carried out in Australia, published in 1996 (Webb, McKeown & Associates, 1996). The 1996 Flood Study included a detailed analysis of primary flood data and used the most up to date technology at the time. Two series of five reports (Parts A to E) document the study, which was subject to an extensive review. Each part documents a separate aspect of the flood study. A version of the reports was published in 1994, which investigated flood mitigation options while the 1996 version investigated fuse plug spillway options. The major difference between the reports is Part E, which investigates options. The review team consisted of most of the prominent experts, both Australian and international, in the field at the time:

- Project steering committee included representatives from the Bureau of Meteorology (BoM), Public Works, Department of Water Resources and Sydney Water
- Internal review team
  - David Pilgrim and David Doran
  - Geoff O'Loughlin
- External review team
  - Eric Laurenson and Tom Fricke
- Domestic flood frequency review
  - George Kuczera
  - Michael Boyd
  - Roger Hadgraft
  - Ray Canterford
  - Jim Irish
  - Frank Harvey
- International review team
  - American consulting experts
  - Canadian consulting experts
  - Adri Verway – hydraulic modelling
  - E Toddini – flood frequency analysis

The earlier modelling work was based on *Australian Rainfall and Runoff – A guide to flood estimation* (ARR) (Pilgrim, 1987) and its recommended approaches and parameters. This included:

- Intensity frequency duration (IFD) data – ARR 1987 rainfall
- Areal reduction factor (ARF) – 0.95
- Hydrologic (rainfall runoff) modelling – RORB
- Losses – initial loss 70 mm, continuing loss 1.9-2.5 mm/hr
- Hydraulic modelling – RUBICON

The hydrologic model (RORB) was calibrated to four gauging stations upstream of Warragamba Dam, Warragamba Dam, and the various stations downstream. The model was calibrated and verified with seven historical floods including two large events that occurred during the study (April/May 1988 and August 1990). The August 1990 event occurred after the models had been established and data collection during the event was targeted at verifying many assumptions. The 1996 Flood Study used an unsmoothed version of the temporal patterns that were developed for the probable maximum precipitation (PMP) work (for catchments of 10,000 square kilometres and 20,000 square kilometres) by the BoM instead of the ARR 1987 temporal patterns, because they produced more consistent results. This study also found the critical duration (duration of rainfall that results in the largest flow for a given AEP) was three days, which is also the limit of the ARR 1987 IFD. Testing of four- and five-day rainfalls using techniques that mimicked the 1987 IFD process confirmed the three-day critical duration assumption. The three-day critical duration is consistent with nearly every large historical event on the catchment, which has been the result of four to seven days of rainfall with the majority of the rainfall occurring over three days.

The 1996 Flood Study conducted flood frequency analyses (FFA) at Warragamba, Penrith and Windsor/Sackville, with the emphasis on Penrith, which has a very good continuous record back to 1892 and is very well gauged. The very long record at Windsor was used with a model derived rating curve based on flow at Sackville and level at Windsor. This record is very reliable back to 1855 because of the detailed observations of astronomer John Tebbutt and can be extended back to 1790 for large floods. At both locations, good records exist for the 1867 flood which is well in excess of the 1 in 100 AEP event. The design flood estimates from the hydrologic model were adjusted to match the FFA estimates.

The hydraulic model (RUBICON) was calibrated and verified using 10 flood events and reproduces the 1867 flood profile down the river with the limited data that were available.

The hydrologic and hydraulic models developed as part of the 1996 Flood Study form the basis of the current body of work.

3.5. Other studies within the Hawkesbury-Nepean catchment

Table 3 presents a list of studies that have been undertaken within the catchment since the 1996 Flood Study. While a number of detailed models of small areas have been established, no valley-wide update to the 1996 Flood Study has been undertaken. Many of these local studies use the 1996 Flood Study as boundary conditions. With the release of this new Regional Flood Study, an update to the studies listed below that used boundary conditions from the 1996 Flood Study may be required.

For areas where a detailed flood study has not been undertaken, the 1996 Flood Study has formed the basis of floodplain management up until the release of the current study. In the areas covered by local studies, if flood levels are higher than in this Regional Flood Study, then the higher of the two should be adopted for planning purposes.

Table 3. Studies undertaken since previous regional flood study

Study Name	Date	Client Organisation	Consultant Organisation
Upper Nepean River Flood Study	Sep-1995	NSW Department of Land and Water Conservation, Wollondilly, Campbelltown, Camden, Liverpool and Penrith Councils	Lyall and Macoun Consulting Engineers
Lower Hawkesbury River Flood Study (final draft)	Apr-1997	NSW Department of Land and Water Conservation	Australian Water and Coastal Studies
Achieving a Hawkesbury-Nepean Floodplain Management Strategy	Nov-1997	NSW Government	Multiple
Upper Nepean River Floodplain Risk Management Study and Plan - Floodplain Management Study	Apr-2001	Camden Council	SMEC
Lower Macdonald River Flood Study	Aug-2004	Hawkesbury City Council	Webb McKeown & Associates (WMAwater)
Hawkesbury-Nepean Floodplain Management Strategy Implementation	Oct-2004	NSW Government	Multiple
South Creek Floodplain Risk Management Study and Plan (Vols 1 and 2)	Dec-2004	Liverpool City Council	Bewsher Consulting
Hawkesbury Floodplain Risk Management Study & Plan	Dec-2012	Hawkesbury City Council	Bewsher Consulting
Torkington Creek, Londonderry, Flood Investigations	Jan-2013	Penrith City Council	Molino Stewart
Brisbane River Foreshore Flood Study	Jul-2013	Gosford Council	Cardno Lawson Treloar
Eastern Creek Hydrologic and Hydraulic Assessment	Dec-2014	Blacktown City Council	WMAwater
Updated South Creek Flood Study (Vols 1 and 2)	Jan-2015	Penrith City Council	Worley Parsons
Nepean River Flood Study	Apr-2015	Camden Council	Worley Parsons
Lapstone, South Glenbrook and South Blaxland Floodplain Risk Management Study and Plan	Jun-2015	Blue Mountains City Council	Jacobs
St Marys Byrnes Creek Overland Flow Flood Study – Final Report	Nov-2015	Penrith City Council	Cardno
Nattai River Floodplain Risk Management Study and Plan	Sep-2016	Wingecarribee Shire Council	WMAwater
Nepean River Flood Study	Nov-2018	Penrith City Council	Advisian

3.6. Changes in flood estimation practice since 1996

The 1996 Flood Study was commenced as the 1987 edition of *Australian Rainfall and Runoff* (ARR 1987) was published. This allowed the most current techniques at the time to be utilised. The inclusion of Professor David Pilgrim (editor of ARR 1987) on the internal review panel also provided a direct link to evolving practice in Australia and overseas.

After the publication of ARR 1987, the evolution of practice slowed for nearly a decade as industry caught up with all the changes in ARR 1987. The first major changes were driven by the dam fraternity as they sought to understand the risk of extreme floods on spillway adequacy and dam failure. In 1998, a revised extreme flood chapter of ARR 1987 (Pilgrim, 1987) was released. This chapter not only updated extreme flood estimation techniques, but provided additional guidance on losses, temporal patterns and spatial patterns. At the same time, the Cooperative Research Centre for Catchment Hydrology was also researching design flood estimation techniques. A major spinoff from the research was the Cooperative Research Centre - FOcussed Rainfall Growth Estimation (CRC-FORGE) technique for estimation of rare rainfalls and the development of areal reduction factors based on Australian data. In most states these techniques became available shortly after. The NSW/ACT CRC-FORGE work was not completed until 2010.

More recently, the completion of ARR 2016 has provided the most up-to-date design rainfall IFDs for use in flood event estimation. The 1996 Flood Study has been updated as part of the current study to include latest best practice and techniques in accordance with ARR 2016.

3.6.1. Flood frequency analysis

ARR 1987 provided a general recommendation that the log Pearson III (or LP3) distribution be used, since it had been shown to generally fit Australian streamflow data well, and provided detailed instructions on fitting, using the widely used log space moments technique. This was consistent with USA practice and allowed use of USA techniques for incorporating historical data and testing for outliers.

Since ARR 1987, more robust techniques have become available and it has become easier for practitioners to trial a range of distributions and techniques. The major changes recommended in the ARR 2016 chapter (Ball et al., 2016) are that log space moments are no longer recommended as a fitting technique as L-moment and Bayesian techniques have been shown to be more reliable. The guidance on distributions has been relaxed to selecting a distribution that best fits the data. Generally, the LP3 and Generalized Extreme Value (GEV) distributions are the best fit to Australian data with the LP3 dominating NSW sites.

3.6.2. Intensity frequency duration

For the 1987 edition of ARR, the BoM produced for the first time IFD maps for the whole of Australia. The analysis was based on BoM data up to 1983. One criticism of the 1987 IFDs was that the extensive rain gauge network owned by WaterNSW's predecessor was not used in their development. This issue was particularly noticeable for much of the Warragamba catchment as the BoM network is biased to flat land, with most gauges located in towns or farms, which resulted in poor coverage of much of the rugged and restricted terrain in the

Hawkesbury-Nepean catchment. The WaterNSW network fills in many of these gaps within the BoM network.

New design rainfalls (IFDs) were released by the BoM (Green et al., 2015) as part of the update to ARR 2016 (Ball et al., 2016). The major improvements over the 1987 IFDs include:

- a larger database of rainfall records was used in the analysis, incorporating data from agencies other than the BoM including the WaterNSW network
- an increased length of rainfall records was used in the analysis
- modern computer data checking techniques were used to quality check and correct the data
- different distributions and fitting methods were investigated and used
- modern regionalisation techniques were used
- modern covariate-based surface gridding techniques were used.

In 2016, the BoM released IFD information for events up to the 1 in 100 AEP. IFDs for 1 in 100 to 1 in 2,000 AEP events were released in February 2017, along with a slight change to the 1 in 50 and 1 in 100 AEP values to tie in with the rare rainfalls. These IFDs were used in conjunction with updated areal reduction factors (ARFs) from ARR 2016. These ARFs are based on Australian data and are used to turn point estimates into areal estimates.

4. AVAILABLE DATA

4.1. Topographic data

A considerable amount of topographic data is available for the study area. However, the accuracy and suitability of these existing datasets for use in the present study varies. This includes contours, hydrosurvey, cross sections and Airborne Laser Scanning.

4.1.1. LiDAR

Airborne Laser Scanning (ALS) or LiDAR ground levels were provided for the study area. The ALS was flown by Lands and Property Information (LPI) and Digital Elevation Model (DEM) tiles were output at 1m resolution.

The 2017 LiDAR (flown between May and June 2017) covers the Hawkesbury-Nepean catchment, extending from four kilometres upstream of Warragamba Dam and approximately 15 kilometres upstream of Wallacia downstream to Wisemans Ferry on the Hawkesbury River and approximately five kilometres up the Colo River. Spatial accuracy of the 2017 LiDAR in the horizontal and vertical directions was reported as 0.8 metres and 0.3 metres, respectively.

Areas such as the Upper Colo and the Hawkesbury River downstream of Wisemans Ferry were supplemented with 2011 LiDAR, which was flown by LPI between February and May 2011, with the same reported accuracy as the 2017 LiDAR data.

Joins between the two datasets were designed to transition in gorge country and along the ridge lines. The join was designed to maximise the use of the 2017 ALS. The ALS was used to inform the two-dimensional model and the mapping of the RUBICON model results.

4.1.2. Cross sections

4.1.2.1. General

The topography along most branches of the RUBICON model was defined by means of surveyed cross sections perpendicular to the direction of flow (exceptions are discussed in Section 7.3.2). The information for each cross section was entered into the model as a series of values giving distance from an arbitrary origin and height in metres above sea level (m AHD).

Extensive data were already available at the beginning of the 1996 Flood Study for the in-bank sections of the main streams, with information provided by the then Sydney Water (now WaterNSW), DLWC (now OEH), Roads and Traffic Authority (now Roads and Maritime Services) and the Penrith Lakes Development Corporation.

The topography of Penrith Lakes has been evolving over several decades, associated with sand mining activities. Given that the final topography was uncertain at the time flood modelling for the current study was being prepared, the Penrith Lakes area was represented in its mid-1990s condition. Detailed studies of Penrith Lakes have shown that the final lake and weir configuration only has a measurable effect on peak flood levels downstream of Penrith Weir and upstream of Yarramundi Bridge.

Little information was available at the beginning of the 1996 Flood Study for overbank areas, so a series of surveys in those areas was specifically commissioned. The survey data were used to model overbank flow paths and floodplain storage areas. No additional cross sections were collected during the current study for the RUBICON modelling.

Following the review by A. Verwey in 1989 (*Hawkesbury-Nepean Hydraulic Model*), in which he suggested that the available data upstream of Penrith Weir were inadequate, further information was obtained for the reach between Regentville Bridge and the weir.

The model upstream of Bents Basin was not designed to give accurate estimates of flood levels, but simply to provide an approximation to the hydraulics of the area sufficient to develop a satisfactory flow hydrograph below the Basin. Accordingly, accurate survey was not required, and information was obtained from topographic maps.

4.1.2.2. Penrith pre-September 1986

A change in the height/flow characteristic at Penrith some time after the 1986 peak presented a particular problem for model calibration and flood frequency analysis, especially as Penrith was the key site in the valley for continuous long-term flow records.

The only relevant pre-September 1986 survey data that could be located were:

- surveyed sections at Victoria Bridge taken in the early 1860s during construction of the bridge. The datum for these surveys is not well established and for this reason they are of little use. (Note: even though the datum is related to the height of the bridge deck, there is a suggestion that the deck level may have been altered between design and construction)
- a cross section at Victoria Bridge taken about 1964 by the then Water Board Gauging Section
- a cross section midway between Victoria Bridge and the weir taken in 1964 and reported by Professor Warner (Warner 1990).

The 1964 cross section from Warner could not be compared directly with more recent survey, but it did indicate that the cross section was larger than in the late 1980s. The 1964 bridge section indicated that the invert then was of the order of two metres lower than in the late 1980s.

There was also some evidence that a considerable amount of fill was placed on the right bank some time between 1964 and 1984.

There were several reports of significant bank collapses in the reach between Victoria Bridge and Regentville during the 1978 flood. It appears that this did not greatly affect Penrith Weir in 1978, probably because the collapses occurred on the falling limb of the flood and the material was not transported very far downstream.

The next large flood occurred in 1986, and as a result of this event large amounts of gravel were deposited at Penrith Weir, particularly on the left abutment where the old fish ladder was left high and dry.

The topographic information is far from conclusive. However, when taken together with the hydraulic evidence, it seems clear that changes occurred during the flood of August 1986, but only after the peak.

4.1.3. Hydrosurvey

Hydrosurvey was available for the area in the vicinity of Windsor and was incorporated in the TUFLOW model (Section 12.1) to represent the river bathymetry. While hydrosurvey exists for downstream portions of the Hawkesbury River, it can be sparse, and uses inconsistent datums. A simplified set of hydrosurvey data were available from Sydney Water and was included in the TUFLOW model to represent bed levels between Cattai and Spencer. The development of a detailed TUFLOW model will require updated bathymetry for the entire Hawkesbury-Nepean system using a consistent datum.

4.1.4. Structures

Major culvert and bridge dimensions were collected as part of the 1996 Flood Study. Culverts under the M4 (refer to Photo 1 to 3) were measured as part of the current study.

Photo 1. Eastern culverts – M4

Photo 2. Western box culvert – M4

Photo 3. Western culverts – M4

4.1.5. Aerial photography

Aerial photography of the catchment was provided by WaterNSW in 2013; however, given the size of the catchment and resolution, the aerial image is generally not shown on catchment mapping presented in this report. The aerial photography was used in the determination of land use types for the two-dimensional modelling discussed in Appendix D.

Historic aerial imagery was available including for 1955, 1956 and 1978 and was used to inform various aspects of the project.

4.2. Historical events

The Hawkesbury River has a significant flood record, dating back to European settlement. This section provides a brief summary of historic flood events. A more detailed flood history as well as the adopted Penrith record can be found in Appendix A.

Table 4 summarises the Windsor flood record for the period of 1799–2012 for all events above 10 metres, including an adjustment for the impact of Warragamba Dam where appropriate (Babister et al., 2016). Until 1964, only limited rainfall and streamflow data were available throughout the Hawkesbury-Nepean catchment. The then Sydney Water began a concerted effort to upgrade the recording network above the dam after a major flood in November 1961. This effort was not completed until after the flood of June 1964, nevertheless the information available for that event was a great improvement on anything collected previously. A number of flood events have sufficient data (flow or stage) available within the catchment for use in the calibration and verification of the hydrologic and hydraulic models (Table 5). Table 5 summarises the number of observed flood levels available for the calibration events and their quality. Since 1992 no large floods that exceed the 1 in 20 AEP level have occurred. The events used for calibration and verification are described in detail in Sections 6.3 and 7.4.

4.3. Rainfall data

Pluviographs are required to define how rain falls in time (temporal pattern) in the hydrologic model. Daily rainfall gauges can be used to derive spatial patterns for hydrologic modelling. Pluviograph coverage within the Hawkesbury-Nepean was sparse during the early years of records. Sydney Water initiated a campaign to install a comprehensive pluviograph network to capture continuous rainfall monitoring in its catchments following the 1961 flood event. This was a gradual process, with pluviograph coverage still poor during the 1964 flood; however, a number of daily rainfall sites were available. The number of gauges covering the catchment has continued to increase, with the most data available for the most recent floods. The number of pluviographs that provide useful information during flood events used for calibration and verification are shown in Table 5 and Table 6. Figure B2 shows the total number of pluviograph gauges available within the catchment.

The recorded depths were plotted on a map of the catchment and lines of equal rainfall (isohyets) were drawn on the basis of the plotted information. Figures B3 to B7 and B16 to B20 show the data used for each storm and the isohyetal map derived from that data.

The pluviograph data were used to obtain temporal patterns at various sites in the catchment. Where sufficient information was available, in the order of 30 separate patterns were derived. The information was obtained in the form of hourly rainfall depths for the duration of the storm.

4.4. Streamflows

Two sets of information are required to provide streamflow data: a record of water levels at a given site over the duration of a flood event (a stage hydrograph) and a relationship between height and flow at the site (a rating curve). Putting these together produces a time varying record of flow (a flow hydrograph, often called simply a hydrograph).

Rating curves are derived from a series of manual spot measurements of flow taken with current meters. At low flows, these measurements are obtained by wading across the stream and recording velocities at several points. During medium to high flows, measurements are usually taken from a boat or a bridge. One set of measurements used to estimate the total flow at a given height is termed a gauging. All available gaugings are plotted as height versus flow and a smooth line through the points adopted as the rating curve. This rating curve is reviewed after each gauging, especially when a flood allows high flow gaugings to be obtained.

Gaugings taken during floods are of vital importance in determining rating curves. However, such gaugings are taken under difficult and dangerous conditions and therefore can be of limited accuracy.

Table 7 summarises the stream gauging stations in the Hawkesbury-Nepean catchment. These are discussed further in Sections 4.4.1 to 4.4.3. Data from these stations were collected as part of the 1996 Flood Study. The additional years of data were collected as part of the current study, however no major floods occurred in this time period. Therefore, this section refers to the findings of the 1996 Flood Study.

Table 4. Windsor flood record

Year	Height
1799	10.5
1806	12.9
1809	14.7
1816	14.1
1817	14.4
1819	12.9
1857	11.91
1860	11.82
1864	15.05
1867	19.68
1869	11.64
1870	14.14
1871	11.67
1873	13.1
1875	12.28
1879	13.62
1889	12.15
1890	12.28
1891	11.24
1894	10.14
1898	10.08
1900	14.5
1904	12.64
1916	10.97
1925	11.5
1943	10.26
1949	12.11
1952	11.8
1956	13.8
1961	15.7*
1963	10.0*
1964	14.8*
1967	11.7*
1969	10.3*
1974	10.5*
1975	11.6*
1978	15.2*
1986	12.6*
1988	12.9*
1990	13.7*
1992	11.0*,#

**Adjusted to remove impact of Warragamba Dam to pre-dam conditions*

 # There is some evidence to suggest the 1992 event reached 11.1m (before adjustment) however 11m was adopted for the flood frequency and is unlikely to impact the resultant flood frequency analysis

Table 5. Historical event data summary

Event	Number of pluviographs	Stage hydrographs	Observations – gauged data (sites)	Observations – other data	Peak at Windsor (m AHD)
Nov-1961	1	6	4	Fair 16 – well spread	14.95
Jun-1964	1	1	3	Poor 14	14.57
Jun-1975	8	1	3	Poor 3	11.20
Mar-1978	8	4	4	Good 25 – well spread	14.46
Aug-1986	55	7	3 (none u/s Penrith)	Poor 14	11.35
Apr/May-1988	60	4	6 (none u/s Penrith)	Fair 18	12.80
Jun/Jul-1988	60	4	0	Poor 17 – none u/s Penrith	10.74
Aug-1990	82	5	6	Poor 11 concentrated around Penrith	13.50

Table 6. Pluviographs used in calibration, and total gauges (daily and pluviograph) used to create spatial pattern

Event	Number of available pluviographs	Total number of rainfall gauges
Jun 1964	1	291
Jun 1975	8	290
Mar 1978	8	286
Aug 1986	55	334
Apr/May 1988	60	344
Aug 1990	82	370
Aug 1998	93	376

Table 7. Stream gauging stations

River	Station	Catchment area (km²)	Period of record	Frequency of recording (2)	Authority (3)
Colo	Upper Colo	4,350	1909-	D to 1971 C since	SW
	Morans Rock	4,640	1971-	C	SW
Coxs	Kelpie Point	1,450	1962-	C	SW
Grose	Burralow	6,50	1945-78	D to 1968 C 1968-78	SW
Hawkesbury	North Richmond	11,600	1964-	D	SW
	Windsor	12,700	1799-	I	BoM
	Sackville Ferry	12,950	1962-79	C	DLWC
Kowmung	Cedar Ford	733	1986-	C	SW
Macdonald	St Albans	1,680	1954-	D to 1972 C since	DLWC
Nattai	Causeway	446	1965-	C	SW
Nepean	Camden	1,380	1860-	I	BoM
	Wallacia	1,760	1925-	D to 1970 C since	SW
	Penrith	11,000	1935-	D to 1968 C since	SW
Warragamba	Warragamba Dam Wall	9,000	1960-77	D	SW
	Warragamba Dam U/S	9,010	1977-	C	SW
	Warragamba Weir	9,010	1940-50 1980-	D C	SW
	Nepean Junction	9,020	1937-83	D 1937-48 C 1953-83	SW
Wingecarribee	Greenstead	583	1954-79	D to 1966 C 1966-79	DLWC
Wollondilly	Golden Valley	1,930	1967-	C	SW
	Jooriland	4,560	1961-	C	SW

(1) All information taken from Australian Water Resources Council (1984)
(2) C – continuous; D – daily; I – infrequent,
(3) Refers to gauge owner in 1996 - SW – Sydney Water; BoM – Bureau of Meteorology; DLWC – Department of Land & Water Conservation.

There are over 100 stream gauging stations in the Hawkesbury-Nepean catchment. Many of these are located on minor streams or creeks, cover a limited time period, or do not have sufficient stream stage to flow rating curves to be useful for modelling purposes. Of the available gauges (refer to Figure B1), eight are particularly useful as calibration locations or for model verification. These gauges had discharge derived from rating curves and were in key locations around the catchment. These gauges and their availability during flood events are shown in Table 8.

Table 8. Streamflow gauges available during flood events

Site	Event month/year						
	Jun 1964	**Jun 1975**	**Mar 1978**	**Aug 1986**	**Apr/May 1988**	**Aug 1990**	**Aug 1998**
Colo River at Upper Colo					Y	Y	Y
Coxs River at Kelpie Point	*P	*P	*P	Y	Y	Y	Y
Wollondilly River at Golden Valley		Y	Y		Y	Y	Y
Wollondilly River at Jooriland	**P	Y	Y	Y	Y	Y	Y
Kowmung River at Cedar Ford		Y	Y	Y	Y	Y	Y
Nattai River at the Causeway		Y	Y	Y	Y	Y	Y
Nepean River at Maldon Weir		Y	Y	Y	Y	Y	Y
Nepean River at Wallacia		Y	Y	Y	Y	Y	Y

Y – Good data

**P – Data only available for part of the event with the peak missing*

***P – Poor data*

4.4.1. Gauging stations upstream of Warragamba Dam

The key stations upstream of Warragamba Dam are on the Coxs River at Kelpie Point, Kowmung River at Cedar Ford, Nattai River at Causeway and Wollondilly River at Jooriland. These four stations monitor the majority of the inflow to the dam storage and are known as the Hydrographic Data Stations (HDS). The rating curves at all four stations were reviewed for the 1996 Flood Study in the light of gaugings taken during the 1988 floods and the ratings were judged to be suitable. However, subsequent work has suggested they may overestimate flows.

Further review of the curves took place after the August 1990 flood. In this event a series of gaugings was taken at Jooriland for flows and levels much higher than any previously obtained. This information led to a major review of the Jooriland rating curve with estimated flows at high stages being reduced by up to 40 per cent.

The station at Jooriland is the most important of the four stations, controlling approximately 50 per cent of the total catchment area to Warragamba Dam. The catchment area is subject to highly variable rainfall during flood events. Good height records are available at this station for all the floods of interest.

Information from the stations at Golden Valley and Greenstead further upstream was utilised to assist in the calibration of the model to the observed data at Jooriland. Data were only readily available at both stations for the floods of June 1975 and March 1978, with a hydrograph at Golden Valley also available for the April/May 1988 flood. Neither station was considered to be of high priority by the operating authorities and the ratings were considered to be of poor quality. Nevertheless, the recorded heights were helpful, particularly for establishing the time at which the river started to rise.

Records at the other HDS are in general very good. The Causeway and Cedar Ford stations were not installed at the time of the 1964 flood. Apart from this, the only height records for the floods of interest which appear to be in error are for March 1978 and August 1990 at Kelpie Point and August 1990 at Causeway. The problems with these records are discussed in Section 6.3.

The other two stations upstream of the dam are those known as Dam Wall and Warragamba Dam U/S (upstream). The former was installed when the dam was completed in 1960 with the intention of recording the reservoir storage level. In the mid-1970s it was realised that the gauge sensor was placed in an area which did not properly represent the dam level when the spillway was in operation. Subsequently, the station was removed and replaced by the upstream station. Further analysis has indicated that any errors in flow estimation associated with the poor placement of the first station should be minimal.

These two stations were important because, together with a record of gate openings, they enabled the flow from the dam during floods to be calculated. This in turn is important because there is an ungauged area of some 1,810 square kilometres between the HDS and the dam wall. This is referred to as the 'Residual Area'. The observed flows at the dam enabled independent calibration of the residual area.

4.4.2. Gauging stations on tributary streams

These stations are located on streams which do not flow into Warragamba Dam, but have a significant impact on flooding downstream of the dam. These tributaries, plus several minor creeks, were used as inflows to the hydraulic model.

The most significant stream in this category is the Nepean River. At first it appeared that the Sydney Water station at Wallacia would provide good data for calibrating the hydrologic model for the Nepean catchment. Unfortunately, the station was found to have two significant deficiencies.

Firstly, the water level is influenced by the level of the Warragamba River in large floods (of the order of the June 1964 and March 1978 events). This means that it is not possible to derive a unique relationship between height and discharge for large events. It might be possible to find a three-way relation between water level and flow at Wallacia and flow in the Warragamba, but the available gaugings do not provide a large enough database to enable this to be done.

Secondly, the height records at the station consistently show a slow rise and long flat peak. It seems that the flow in the Nepean at this point is controlled by the long narrow restriction above Bents Basin, some 10 kilometres upstream of Wallacia. This restriction, together with the extensive Camden floodplain above it, acts as a natural retarding basin. Thus, the Wallacia station is recording the hydraulic characteristics of this basin rather than the runoff characteristics of the Nepean catchment.

The BoM and the then DLWC (now OEH) have stations upstream of Bents Basin, but these are infrequently read and poorly rated. The BoM station at Camden did provide some water level information which was mainly of use in confirming a significant time delay between Camden and Wallacia.

The Colo River is gauged by Sydney Water at two sites. The station at Morans Rock is tidal in non-flood times and affected by the Hawkesbury River backwater during floods. The station is however fitted with a cableway to enable high flow gaugings. The Upper Colo station is some 15 kilometres upstream of Morans Rock. The Upper Colo rating curve was used to produce the observed flows used in model calibration. The recorder at Upper Colo malfunctioned in the 1986 flood and no data is available for that event. Data from Morans Rock was substituted to give an indication of the fit, but this record shows clear evidence of backwater effects from the Hawkesbury River. The 1988 flood on the Colo was only a fresh and was considered too small to provide meaningful calibration data.

4.4.3. Gauging stations downstream of Warragamba Dam

There are a number of important gauging stations on the main river downstream of the dam. These lie within the area covered by the hydraulic model (refer Figure 9) and their usefulness to this study was in calibration and verification of that model. They were not used to directly calibrate or check the hydrologic model but are listed in Table 7 for completeness. Of the stations listed in Table 7, North Richmond, Windsor, Sackville Ferry, Penrith, Warragamba Weir and Nepean Junction fall into this category.

4.5. Design rainfalls

The latest IFDs developed in ARR 2016 (Green et al., 2015) were used for available frequencies up to the 1 in 2,000 AEP event sourced from BoM. IFDs for events rarer than the 1 in 2,000 AEP to the probable maximum precipitation (PMP) were extrapolated in accordance with ARR 2016.

In addition to the design IFDs and ARFs, the daily read rainfall gauge network was used to construct a catchment average rainfall series. Sufficient daily rainfall data were available from 1871 to 2011.

4.6. Stage storage relationship

The stage storage relationship defines how much storage is available in the dam at a particular water level. Following a detailed survey of the dam, an updated stage storage relationship for Warragamba Dam was made available from WaterNSW in 2017. Updated stage storage relationships were also available for the Upper Nepean dams. The hydrologic model was updated to represent the new data. This resulted in only minor changes in dam storage and downstream flood levels.

4.7. Synthetic sequences of dam water levels

The starting water level in Warragamba dam prior to an event influences the amount of the flood captured by the dam and therefore downstream flooding. This section documents the data and approach used to estimate pre-event dam water levels.

WATHNET is a water supply system model package that is used by many urban water supply authorities to model the reliability and security of water supply systems. The package was developed by Professor George Kuczera of the University of Newcastle (Kuczera, 1997). The model uses historical inflow sequences or stochastically generated inflows. The package includes routines to generate stochastic annual flow sequences using a multi-site lag-one model which are then disaggregated into monthly values using the method of fragments.

WaterNSW uses the WATHNET package to examine different operational strategies and their effect on water supply reliability. The dataset that was made available during the current study includes 2,000 sets of monthly 100-year stream flow replicates and associated Warragamba Dam water levels. While the focus of the WATHNET modelling is on system reliability, which is generally focused on dry sequences, the dataset also provides insight into the relationship between high monthly inflows and the preceding month's dam levels. The stochastic data replicates the ENSO (El Niño Southern Oscillation) and IPO (Inter decadal Pacific Oscillation) phases that are extremely important to understanding flood sequences. Given these sequences are also important for droughts it was assumed that they also accounted for persistent wet periods.

Flooding on the east coast of Australia is strongly influenced by ENSO and IPO. While a wet or dry ENSO state will typically persist for up to 4-7 years, an IPO state typically persists for a period of over 20 years. In addition to producing very distinctive wet and dry phases that affect dam water levels, nearly all major flood events occur in the wet phases when dam levels are much higher than normal. This also causes the clustering of floods where multiple floods can occur in one year and several major events within a couple of years often with large gaps between these clusters.

For each replicate of 100 year inflows provided from WATHNET, the monthly inflows were ranked and the dam level in the preceding month was extracted. These were combined to produce drawdown probability graphs for differently ranked inflows. Figure 3 shows the ranked inflows for the 1 in 100, 1 in 50, 1 in 20, 1 in 10, and 1 in 5 AEP inflows. The cumulative ranked inflows are shown in Figure 4 for the 1 in 100 inflow which is likely to correspond to the largest flood event in the 100 year sequence. From these results it can inferred that there is approximately an 80 per cent probability that the dam will be near full prior to a flood event. It is not possible to compare these curves to the historical data as only one data point could be extracted for each curve, so the ranks were grouped together. Figure 4 presents the grouped ranks drawdown curves and the dam level for the largest 10 events since the dam first filled in 1961 (which is equal to the 1-20 rank for each 100-year sequence). For a small sample, the 10 largest events in 50 years compare well to the equivalent rank 1-20 curve. These figures also show that for frequent events, the pre-flood event drawdown is approaching the average drawdown curve.

This simple analysis assumes that large monthly inflows will correspond to floods and neglects the impact of inflows for the period immediately preceding an event. This approach allows the modelling to more accurately reproduce the correlation between dam levels and when floods occur.

The sequences supplied for the current study reflect the latest stage storage relationship for the dam, removal of the hydroelectric power station (HEPS), the change of the triggers for pumping from the Shoalhaven system, the operation of the Sydney desalination plant, and restrictions. These are based on absolute volume, rather than a proportion. They also reflect some changes to what reductions WaterNSW expect in demand due to restrictions.

5. FLOOD FREQUENCY ANALYSIS

Where long flood records exist, flood frequency analysis (FFA) is the most robust method of estimating the probability of flooding. It is a direct approach where a statistical distribution is fitted to the largest flood in each year. FFA is the foundation of nearly every design flood estimation technique used in Australia. Nearly every method is directly derived from FFA results or is verified and calibrated to FFA results. It is generally necessary to use rainfall runoff modelling techniques in conjunction with FFA to produce a full hydrograph and to model catchment changes and dam options.

The majority of the data discussed within this section was collected in 1996 and was updated to include the period since 1996. The FFA documented within this section uses the most up to date techniques recommended in ARR 2016 (Ball et al., 2016).

5.1. Pre-1990 data

As part of the 1996 Flood Study, an extensive data collection project was carried out by WMAwater (then Webb McKeown and Associates) and WaterNSW (then Sydney Water). While the study used data collated by many earlier parties, where possible original sources were found and verified. Where flood marks or the structures that were referred to still existed, these were surveyed and documented. This data collection allowed long term streamflow records to be established at Warragamba, Penrith and Windsor. An incomplete record was also established at North Richmond, but this was mainly used to confirm Windsor levels as there is a near perfect correlation between flood levels at the two sites for large events.

This allowed the following datasets to be compiled for use in the FFA.

Table 9. Flood record lengths as used in earlier study

Location	Period of continuous record	Prior events
Warragamba	1909–1990	Estimates of 1864,1867, 1900 events
Penrith	1893–1990	Reliable measurement of 1867 event and some information on other large events in the 1860s
Windsor	1857–1990 for events > 8m AHD	Information on prior events back to 1790

The record length of the Penrith and Windsor datasets are remarkable by Australian standards. The Penrith record is continuous back to 1892 and includes a very reliable reading of the 1867 event that can still be measured today, and good information on some earlier events. The Windsor data extends back to 1855 because of the extraordinary work of the astronomer John Tebbutt. His expertise in recording and measuring flood events was ahead of his time. He recorded all flood events at Windsor above about six metres with extraordinary accuracy during his adult life. He also established some earlier flood levels through discussions with relatives and other sources. Using Tebbutt's work and other resources it is possible to establish approximate flood levels back past 1806 and the relative magnitude of all events from 1790. The earlier records have compounding measurement uncertainty as they are measured relative to the subsequent event. It is also worth noting that flood records at Windsor are more reliable during Tebbutt's phase from 1855 to 1915 than in the period from 1915 to 1960.

Flood records at Penrith were converted to flows using the high rating curve developed from hydraulic modelling and gaugings. The August 1990 event provided the opportunity to verify this rating. At Windsor, a rating relationship was established between flood levels at Windsor and flow at Sackville using the hydraulic models and some limited gaugings. This approach was used as Windsor is in the middle of a very large flood storage zone and Sackville can be used to represent the discharge.

5.2. Post-1990 data

The data since 1990 were updated using WaterNSW records at Warragamba, Penrith and Windsor. During this period, no large events occurred, but several minor events occurred in 1992, 1998, 2011 and 2012. Similar length periods without major flooding have occurred in the historical record. The post-1990 dataset was combined with the dataset from the 1996 Flood Study for the current FFA (Table 10).

Table 10. Flood record lengths used in current study

Location	Period of continuous record	Prior events
Warragamba	1909–2012	Estimates of 1864,1867, 1900 events
Penrith	1893–2012	Reliable measurement of 1867 event and some information on other large events in the 1860s
Windsor	1857–2012 for events > 8m AHD	Information on prior events back to 1790

5.3. Pre-Dam Conversion

The formation of Lake Burragorang caused major changes to the hydraulic characteristics of the lower Wollondilly and Coxs Rivers and the Warragamba River itself. This had an impact on the size and timing of the outflow from the Warragamba catchment, and this in turn affected the flow regime at Penrith and further downstream. As a result, the observed flows of the post-dam floods do not form a homogeneous dataset with the observed flows of pre-dam floods.

In order to establish the magnitude of the dam's impact on flooding in the Hawkesbury-Nepean Valley, a hydraulic model of the area upstream of the dam site was established (refer to 1996 Flood Study Part C). The model was used to estimate flows that would have emerged from the Warragamba River in recent floods if the dam had not been built. These flows were input to the model of the downstream valley to determine the impact of the dam on a particular flood at any point in the valley. ‘Pre-dam’ levels were estimated in this way at Penrith and Windsor for the major floods which occurred in November 1961, June 1964, June 1975, March 1978, August 1986 and April/May 1988.

While it would have been possible to carry out similar modelling for all the floods in the partial and annual series since 1960, this could not be justified in terms of the time and cost involved. Instead, a simplified approach was adopted as detailed in Appendix D.B of Part D of the 1996

Flood Study. While this approach may not provide a precise correction for each individual event, the overall effect would be to remove most of the bias caused by the impact of the dam on the smaller floods. The estimated pre-dam flows are presented in Appendix A.

5.4. Flood frequency analysis

The earlier FFA was carried out at Warragamba, Penrith and Windsor/Sackville with the emphasis on Penrith, which has a very good continuous record back to 1892 and is very well gauged. Older techniques did not work as well with incomplete records. The very long record at Windsor was used with a model-derived rating curve based on flow at Sackville and level at Windsor. This record is very reliable back to 1855 because of the good work of John Tebbutt and can be extended back to 1790 for large floods using the newly available techniques. At both locations, good records exist of the 1867 flood which is well in excess of the 1 in 100 AEP event. High flow gauging at Penrith is shown on Figure C26 (reproduced from the 1996 Flood Study) and suggests that the adopted rating is slightly underestimating flow. All flows were converted to pre-dam in order to undertake the FFA on a comparable basis.

The earlier studies at Penrith and Windsor investigated a range of distributions, but concluded that the log Pearson III (LP3) distribution fitted by log space moments provided the best fit. At Warragamba, a more complex five parameter mixed distribution produced a marginally better fit than the LP3. The log space moments fitting method is no longer recommended for the LP3 distribution as L-Moment and Bayesian techniques have been shown to be more efficient. Bayesian techniques also have the advantage of allowing 'prior' information about earlier flooding to be included even when there is some uncertainty about this information. Table 11 presents the adopted FFA from the earlier studies.

Table 11. Adopted FFA from 1996 Flood Study

Probability	Flow (m^3/s)		
1 in x AEP	**Warragamba**	**Penrith**	**Windsor**
10	4,660	5,870	4,850
20	6,610	8,740	6,260
50	9,660	13,500	8,530
100	12,400	17,800	10,600

ARR 2016 recommends the use of Bayesian techniques for flood frequency analysis (refer to Kuczera and Franks 2016). The use of Bayesian techniques is made easier by the use of FLIKE (a software package developed by Professor Kuczera for extreme value analysis of flood records, University of Newcastle 2013). The updated datasets were tested for LP3 and GEV distributions using FLIKE with and without the prior flood information. The Grubbs and Beck multiple low flow outlier test was also applied. This resulted in a significant number of low flow outliers being removed to provide a better fit to the upper part of the curve (rarer events) and smaller confidence intervals. A number of trial fits were tested (refer to Table 13). The adopted fits are included as Figure 5 to Figure 7 and Table 12 below.

Table 12. Adopted FFA current study

Probability	Flow (m^3/s)		
1 in x AEP	**Warragamba**	**Penrith**	**Windsor**
10	5,260	5,830	4,650
20	7,510	8,500	6,320
50	10,400	12,400	8,880
100	12,400	15,600	11,100

The updating of the FFA to currently recommended techniques and the inclusion of 22 years of additional data has resulted in only minor changes at Warragamba and Windsor in the 1 in 100 AEP estimate and 12 per cent reduction in 1 in 100 AEP flows at Penrith where the adopted flow reduces from 17,800 m^3/s to 15,600 m^3/s.

Table 13. Trial fits for flood frequency analysis

Location	Option	Fitting method	Description
Penrith	A	LP3	1893–1990
	B		1791–1990 with 1867 event included at 20,000 m^3/s and 102 years below 20,000 m^3/s
	C		1791–2012 with 1867 event included at 20,000 m^3/s and 102 years below 20,000 m^3/s
	D*		1791–2012 – Low flows censored applied with 1867 event included at 20,000 m^3/s and 102 years below 20,000 m^3/s
	E	GEV	1791–2012 with 1867 event included at 20,000 m^3/s and 102 years below 20,000 m^3/s
Warragamba	F	LP3	1909–1990
	G		1791–1990 with 1864, 1867 and 1900 events included and 115 years below 14600 m^3/s
	H		1791–2012 with 1864, 1867 and 1900 events included and 115 years below 14600 m^3/s
	I*		1791–2012 with 1864, 1867 and 1900 events included and 115 years below 14,600 m^3/s, 43 years below 500 m^3/s
	J	GEV	1791–2012 with 1864, 1867 and 1900 events included and 115 years below 14,600 m^3/s, 43 years below 500 m^3/s
Windsor	K	LP3	1855–1990 with 76 years below 2020 m^3/s
	L		1791–1990 with 64 years below 14,700 m^3/s and 76 years below 2020 m^3/s
	M*		1791–2012 with 64 years below 14,700 m^3/s, 97 years below 2020 m^3/s
	N	GEV	1791–2012 with 64 years below 14,700 m^3/s, 97 years below 2020 m^3/s

** Adopted methods*

6. HYDROLOGIC MODELLING

6.1. Overview

In Australia, there are several commonly used rainfall runoff routing hydrologic models. These models tend to use an identical rainfall excess model with slightly different routing approaches. The models produce similar results when calibrated. This study adopted the RORB runoff routing model (Laurenson & Mein, 1992), which is the most commonly used model and is nearly exclusively used for catchments with large dams.

The standard package was modified to allow large sub-catchments to be independently modelled and to simulate the impact of Warragamba Dam and the implemented procedure for operating its gates during flood events, known as the 'H14 protocol'. The final model layout consists of 121 sub-areas.

The model was calibrated to available streamflow and rainfall data, mainly at stations upstream of the dam, and the calibration parameters used to estimate suitable parameters in uncalibrated catchments in the downstream valley.

Within the overall Flood Study, the hydrologic model was used for two critical functions:

- to estimate historical hydrographs from observed rainfalls at locations without flow records. These were used as described in Part C of the 1996 Flood Study (and are repeated here for completeness) to assist in calibration of the hydraulic model.
- to generate design hydrographs from design rainfalls of known probability.

The hydrologic model adopted for the current study is based on that developed for the 1996 Flood Study. This section of the report is largely directly transferred from the 1996 report. The only change is a minor modification to the dam routing as discussed in Section 6.2.2.

6.2. Description of model

The RORB runoff routing model divides the catchment into a number of sub-areas which can be allocated a specific total rainfall depth and temporal pattern. The stream network is divided into reaches which the model treats as storages through which flows are routed.

At the end of each storage reach is a node which can represent the input point for flow generated from rainfall falling over a sub-area, the confluence of streams, the point of inflow to a storage reservoir, or a gauging station or other point at which streamflow information is required.

Topographic data required by the model includes the area of each individual sub-area, the length of stream represented by each reach and the interconnection of reaches to form the river network. A node is inserted at the location of each gauging station so that flows can be extracted.

The model results can be adjusted by varying one or both of the routing parameters (k_c and m) within the model and/or two loss rates – the initial loss (IL) and the continuing loss (CL) – which regulate how much of the rainfall enters the river system as runoff (this is called the 'rainfall excess').

Where:

k_c:	a storage coefficient used in routing flows through the model. It takes account of the volume of water contained within the channel and immediate floodplain.
m:	an exponent which can allow flows to vary non-linearly with respect to rainfall excess.
Initial Loss:	a volume or average depth of water subtracted from the beginning of rainfall to simulate the initial loss of water into dry soil and for filling depressions, etc. None of this water appears as runoff during the flood.
Continuing Loss:	an average rate of loss subtracted from rainfall throughout the remainder of the storm after initial loss is satisfied to simulate the continuing loss of some water into the ground.

Experience in other catchment studies, and published results derived for gauged catchments by researchers and practitioners, define the ‘normal’ ranges of acceptable parameter and loss rate values. However, there remains enough variation within these ‘normal’ ranges to significantly affect flow estimates. Recommended practice is to adjust the parameter and loss rates in a calibration process, which involves entering observed rainfalls and adjusting values of the parameters and loss rates until an observed flow hydrograph is reproduced. Due to the spatial variability and sampling problems with recorded hydrological data, and the simplifications involved in all models, the fit is never exact, but close approximations are usually obtained.

The calibration process for the hydrologic model is described in detail in Section 6.3.

6.2.1. RORB model layout

6.2.1.1. Sub-areas

The RORB model layout adopted for this study consists of 121 sub-areas grouped into 12 sub-catchments as shown on Figure 8.

The catchment sub-division upstream of the dam is substantially the same as that used in the Snowy Mountains Engineering Corporation report *Warragamba Dam – Review of spillway design flood* (SMEC, 1982) and for all RORB models since. Some minor changes were made during the 1996 Flood Study to the program input and all stream lengths were checked with a number of amendments being made to these. The areas of the sub-areas were also checked and amended as discussed below.

The layout downstream of the dam to Wisemans Ferry was developed by Sydney Water (now WaterNSW) in the course of its studies in the late 1980s. This layout was critically examined and found to be satisfactory during the 1990s.

The five sub-areas downstream of Wisemans Ferry (B117-B121) were added as part of the 1996 Flood Study to provide input to the hydraulic model from tributaries in the lower valley.

The adopted total catchment areas to key gauges are listed in Table 14.

Table 14. Adopted catchment areas to key gauges

River	Station	Area (km^2)
Wollondilly	Jooriland	4,560
Nattai	Causeway	446
Kowmung	Cedar Ford	733
Coxs	Kelpie Point	1,450
Warragamba	Dam Wall	9,000
Nepean	Wallacia	1,760
Grose	Burralow	650
South Creek	Windsor	640
Hawkesbury	Lower Portland	13,450
Colo	Morans Rock	4,640
Macdonald	St Albans	1,680
Hawkesbury	Brooklyn	21,600

6.2.1.2. Sub-catchment areas

The 121 sub-areas are grouped into 12 sub-catchments to provide flow estimates on the major tributaries. The sub-catchments are listed in Table 15 together with the number of sub-areas above each station.

Table 15. RORB sub-areas

Downstream station	Number of sub-areas above station
Jooriland	16
Causeway	3
Cedar Ford	4
Kelpie Point	7
Warragamba Dam	39
Wallacia	17
Penrith	62
Burralow	5
South Creek	4
Colo	21
St Albans	11
Brooklyn	121

The RORB manual recommended that a minimum of five sub-areas should be incorporated in any model and this implies that a similar minimum should be used upstream of any gauging station at which calibration is attempted. In this regard the sub-catchments above Causeway, Cedar Ford and South Creek lack the required number, while the Grose at Burralow has the minimum number. As these catchments are relatively small contributors to the overall flow, the previously adopted layout was retained.

For the two stations upstream of the dam (Causeway and Cedar Ford), the adopted layout has the advantage that all the sub-areas are in series thus representing reasonably distributed storages. Both Causeway and Cedar Ford were involved in the calibration process, and good results were obtained using parameters comparable to those used elsewhere.

Similarly, it was considered that the limited number of sub-areas on the Grose River above Burralow and on South Creek would give an adequate indication of their contribution to flood flows through the Windsor/Richmond area.

A separate project has been undertaken to update the model with more sub-areas which will be beneficial for the development of a detailed two-dimensional model in the future.

6.2.2. Routing through Warragamba Dam

A special sub-routine, DAMROU, was added to the RORB program to model flow through the Lake Burragorang Reservoir taking account of the gate operations at the dam. The subroutine was modified as part of the current study to also include simulation of the fuse plug operation on the auxiliary spillway as built.

Using the standard RORB procedure for storages, DAMROU assumed that the surface of the lake is horizontal (level) during the passage of a flood. During the 1996 Flood Study, there was considerable discussion as to whether such an assumption was applicable to a long narrow reservoir such as Lake Burragorang, especially in very large floods. Following the 1978 flood, the then Sydney Water installed a series of recorders along the reservoir in an attempt to detect flood slope in a future large event. No slope was detected in the flood of 1986 or during either of the 1988 events; however, none of these qualify as a major event.

Warragamba reservoir dynamics and pre-dam event analysis (Sydney Water Board et al., 1990) reports on a mathematical hydraulic model of the reservoir which was set up by the then Sydney Water to examine this question. The model showed very little slope along the lake even in a probable maximum flood. This result was confirmed by modelling carried out as part of the 1996 Flood Study. As a result, it was concluded that the RORB assumption of a level pool routing should give reasonable results for the Warragamba outflows. This conclusion was supported by the calibration runs, although these were only for relatively small events.

Another issue that was considered was the time the flood wave took to travel through the reservoir. The standard RORB routing procedure for a reservoir reach simply translates the hydrograph from the top of the reach to the bottom. This assumes that the influence of the inflow travels through the reservoir instantaneously. While this is a reasonable approximation for small reservoirs, the question arose as to whether it was reasonable in this case where the length of the lake from the dam to the head of the Burragorang Valley is 50 kilometres. While this issue would not greatly affect the calculation of flood magnitudes at the dam, it clearly has an impact on the relative timing of flows from Warragamba and the downstream tributaries, especially the Nepean River.

In the 1990s, Sydney Water conducted a literature search on the issue but did not find any substantive information. Theoretical calculations showed that the influence of the flood wave could travel the length of the reservoir in less than one hour (given large parts of the reservoir are greater than 50 metres deep the wave speed would be in excess of 20 m/s), in which case the assumption of instantaneous translation would be reasonable. The hydraulic model of the reservoir also indicated that the adopted approach was acceptable. The standard RORB assumption of zero travel time was therefore retained.

6.3. Calibration

The natural processes by which rainfall is converted to streamflow are extremely complex and not fully understood. It is therefore necessary to calibrate an appropriate model against observed inputs and outputs (rainfall and streamflows) in order to achieve the best possible results.

In order to establish and properly calibrate a hydrologic model, a number of flood events are required which provide good data for both rainfall and streamflow and cover a range of flood sizes.

Up until approximately 1964, only limited rainfall and streamflow data were available throughout the Hawkesbury-Nepean catchment. The then Sydney Water began a concerted effort to upgrade the recording network above the dam after a major flood in November 1961. This effort was not completed until after the flood of June 1964, nevertheless the information available for that event was a great improvement on anything before. In some other areas, such as the Grose River and to a lesser extent the Colo River, the data collection network remained inadequate until the late 1980s.

As a result of data limitations, only floods from 1964 and later were considered suitable for use in calibrating the hydrologic model. Table 16 lists the 10 highest events at Windsor since 1964.

Table 16. Floods above 10m AHD at Windsor since 1961 and how they were used

Date*	Height (m AHD)	Used
Nov-61	14.95	No, hydraulic model inflows only
Jun-64	14.57	Calibration
Nov-69	10.21	No
May-74	10.43	No
Jun-75	11.2	Calibration
Mar-78	14.46	Calibration
Aug-86	11.35	Calibration
Apr/May-88	12.80	Calibration
Jul-88	10.89	Verification
Aug-90	13.50	Verification
Feb-92	10.86[#]	No

** Smaller events in October 1987 and April 1989 were also modelled to produce hydraulic model inflows.*

[#] There is some evidence to suggest that the level of the 1992 event was higher.

A decision was taken to concentrate on the larger floods and consequently the smaller events of November 1969, May 1974 and July 1988 were not considered further. The five largest events before 1990 (June 1964, June 1975, March 1978, August 1986 and April/May 1988) at Windsor were all utilised in the calibration procedure. The flood of August 1990 occurred after the original calibration was completed and was therefore used as an independent check of the model together with the July 1988 flood.

6.3.1. Approach

The only stations suitable for use in calibrating the RORB model were the four Hydrographic Data Stations (HDS) upstream of Warragamba Dam, the dam itself and a gauge on the Colo River. The Colo River was independent of the other five stations and was therefore calibrated separately. The four HDS stations at Jooriland, Causeway, Kelpie Point and Cedar Ford were also calibrated independently, and the adopted parameters then used to calibrate the Residual Area against observed flows at the dam.

Four parameters can be adjusted to assist in model calibration (k_c, m, Initial Loss and Continuing Loss).

The availability of data for each flood at the various stations is shown in Table 17. The following procedure was adopted at each station:

i) m was set at 0.8 which is the recommended default value in the RORB manual. Testing of sensitivity of the results to this assumption is described in the 1996 Flood Study, where it is shown that m = 0.8 gives results which are at least as good as those derived using higher or lower m values.
ii) Each flood was fitted individually with complete freedom in choice of the remaining parameters, i.e. k_c, initial loss and continuing loss.
iii) Based on these results, a single value of k_c was derived which gave the best fit across the observed floods.
iv) Using this $k_{c,}$ a value of initial loss was obtained for each event to produce a suitable time of rise of hydrograph.
v) Finally, continuing loss was adjusted for each event to give the best possible fit of shape, peak and volume. Efforts were concentrated on fitting the rising limb and peak of the various hydrographs as these were of critical importance in establishing maximum flood levels, the prime aim of the overall study.

Table 17. Hydrographs used in calibration and verification

Location	June 1964	June 1975	March 1978	August 1986	April/ May 1988	August 1990
Jooriland	Y	Y	Y	Y	Y	Y
Causeway	N	Y	Y	Y	Y	Y
Cedar Ford	N	Y	Y	Y	Y	Y
Kelpie Point	Y	Y	P	Y	Y	N
Warragamba Dam	Y	Y	Y	Y	Y	Y
Upper Colo	P	Y	Y	N*	N	N

Y – Yes
N - Not available
P - Peak only
** used Morans Rock*

Once the four upstream stations had been calibrated, the adopted values were retained while the residual area was calibrated using the same procedure.

The results for each station are presented and discussed below. In order to put the adopted k_c values into context, the results from two recognised formulae used to derive k_c for ungauged catchments are presented in each table. The equations relate k_c to area as follows:

Boyd: $k_c = 1.17A^{0.56}$
Kleemola: $k_c = 1.22A^{0.46}$

The latter is recommended in ARR 1987 (Pilgrim 1987), although there is some evidence from the 1996 Flood Study that it gives k_c values which are too low. Although the Boyd formula was originally based on only five catchments, the results of numerous subsequent calibrations have confirmed its applicability.

6.3.2. Wollondilly River at Jooriland

The Jooriland catchment presents the most diverse terrain and variable rainfall of any of those used in the calibration. The parameters which produce the best fit hydrographs are listed in Table 18 and the hydrographs reproduced in Figures B8 to B12.

Table 18. Jooriland –fit parameters for RORB modelling

Flood	k_c	IL (mm)	CL (mm/hr)	Peak (m^3/s)		Diff (%)	Volume ($m^3 \times 10^6$)		Diff (%)
				Observed	Modelled		Observed	Modelled	
Jun-64	80	95	0.1	3380	3260	-4	277	264	-5
Jun-75	80	90	0.2	3950	3920	-1	330	317	-4
Mar-78	80	120	0.2	3970	3820	-4	454	498	10
Aug-86	80	135	0.1	1270	1220	-4	168	124	-26
Apr/May-88	80	110	0.2	2620	2560	-2	250	208	-17

Notes: Estimated k_c; Boyd 131; Kleemola 59
$A = 4{,}560\ km^2$

All peaks fit very well with regard to peak flow and timing. The volume for the 1978 flood is slightly overestimated by 10 per cent principally because it is necessary to model additional flow at the beginning of the flood in order to match the peak. The modelled 1978 hydrograph shape proved to be very sensitive to the selection of representative pluviographs for the various sub-areas. The adopted hydrograph gives the best shape for a reasonable allocation of pluviographs. It would be possible to establish a better fit to the rising limb of the hydrograph, but it would then not be possible to obtain a reasonable match to the peak, even with zero continuing loss.

The model underestimates the volume of the two smaller floods of 1986 and 1988 due to the model's inability to match the recession flows. As this part of the hydrograph is of less importance in this study, the fits are considered acceptable.

The initial loss, which varies from 90 mm to 135 mm, is very high by normal standards. The initial loss of 110mm in April 1988 might seem particularly surprising in view of the well documented very wet conditions in Sydney for that month. However, the antecedent rainfall in the catchment area was somewhat different to that experienced in the Metropolitan area. While there were substantial falls up to 12 April, there was no significant rain between then and the beginning of the flood producing storm on 29 April. Although there is evidence that initial loss accrues at a high early rate, an initial loss of 110mm after only two dry weeks is unusual.

While it is difficult to explain these losses, the supporting evidence is strong. The initial losses are determined by adjusting the value until the time of rise of the observed hydrograph is matched. This is an objective part of the record, independent of any errors in rating curves, etc. The initial losses are consistently high for all floods on this catchment and for fits on adjoining catchments (refer to the following sections). The available data from Golden Valley and Greenstead also confirm that the adopted losses are of the correct order.

The high initial losses also imply that in all the historical storms, parts of the catchment area do not contribute to runoff at Jooriland. This means that the k_c value derived from calibration only applies to the contributing area. Nevertheless, there are two arguments in favour of accepting the derived k_c value as representative of the whole sub-catchment. The first is that the RORB procedure of proportioning storage by the ratio of sub-area stream length to total stream length provides a rational basis for extrapolating into the non-contributing area. The second is that the adopted k_c value of 80 gives a good fit for all the calibration floods despite a considerable variation in the proportion of non-contributing area between floods.

In contrast to the high initial losses, the fitted continuing losses are very low. It may be that this reflects in part a re-emergence of the initial loss as interflow. On the other hand, it may be that the continuing loss is being underestimated because of an overestimation of flow by the rating curve. Data from the smaller 1998 event (which occurred after model calibration) which did not cause any significant spill from Warragamba dam strongly suggests that the Jooriland rating is overestimating flows by 10-15 per cent and this is the most likely cause of the high continuing losses.

Sensitivity testing of parameter values was undertaken in the 1996 Flood Study and the original parameter values in Table 18 were adopted.

6.3.3. Nattai River at Causeway

The RORB model of the Nattai River at Causeway only incorporates three sub-areas and hence difficulties might be expected in calibration. However, a reasonable fit is achieved.

The adopted parameters are listed in Table 19 and the resulting hydrographs presented on Figures B8 to B12. The volume is high for the June 1975 flood due to a high modelled recession. In March 1978 the peak is low and the volume high, however Figure B10 shows that both the observed and modelled hydrographs are irregular for that flood and the general agreement of the two is considered fair when the high recession is discounted. The problem of determining suitable representative pluviographs, which was experienced at Jooriland, is again evident at Causeway.

In a similar manner to Jooriland, the initial losses are high, but in this case the continuing losses fall into a more typical range. The adopted k_c value agrees with that predicted by Boyd's equation.

Table 19. Causeway – fit parameters for RORB modelling

Flood	k_c	IL (mm)	CL (mm/hr)	Peak (m^3/s)		Diff (%)	Volume ($m^3 \times 10^6$)		Diff (%)
				Observed	Modelled		Observed	Modelled	
Jun-64	No observed hydrograph								
Jun-75	35	90	2.5	443	416	-6	30.9	35.4	+14
Mar-78	35	160	1	596	533	-11	56.5	66	+17
Aug-86	35	140	2.5	247	263	+6	22.4	20.3	-9
Apr-88	35	100	2	498	544	+9	39.3	37.2	-5

Notes: Estimated k_c; Boyd 36; Kleemola 20
$A = 446\ km^2$

6.3.4. Kowmung River at Cedar Ford

Four floods are available for calibration at Cedar Ford. The gauge at Cedar Ford was not installed until 1968. It is not possible to obtain a good fit to all events using a single k_c value. The two larger floods (June 1975, March 1978) fit well with a k_c of 35, while the smaller floods (August 1986, April/May 1988) give very good fits with a k_c of about 50.

It was decided that the larger events must take precedence in determining the calibration parameters as these are more likely to reflect conditions during the larger design floods. The values listed in Table 20 were derived accordingly. The hydrographs are presented on Figures B8 to B12.

Table 20. Cedar Ford – fit parameters for RORB modelling

Flood	k_c	IL (mm)	CL (mm/hr)	Peak (m^3/s)		Diff (%)	Volume ($m^3 \times 10^6$)		Diff (%)
				Observed	Modelled		Observed	Modelled	
Jun-64	No observed hydrograph								
Jun-75	35	70	3.5	1,350	1,270	-6	80.6	86.9	+8
Mar-78	35	120	1.0	2,000	1,830	-9	126	132	+4
Aug-86	35	140	1.0	808	909	+13	101	82.6	-18
Apr-88	35	140	1.3	650	695	+7	54.0	42.1	-22

Notes: Estimated k_c; Boyd 47; Kleemola 25 A = 733 km²

Both the June 1975 and March 1978 events have very sharp peaks, especially March 1978. At first glance these seem to indicate a recorder error, but close examination of the original record gives no indication of any malfunction. The peak at the adjacent Kelpie Point station is also very sharp in March 1978. Therefore, the records were accepted for use in calibration. To fit such peaks would require a very low k_c value, and it could be argued that 35 is still too high. However, taking a lower k_c would also advance the hydrograph thus requiring still higher initial losses and hence lower continuing losses. It would also mean even less satisfactory fits for the August 1986 and April/May 1988 floods. The parameters adopted are considered to give the best compromise between the various constraints. The fits are considered acceptable, especially with regard to peak flows.

6.3.5. Coxs River at Kelpie Point

The Coxs River catchment to Kelpie Point is represented in the RORB model by seven sub-areas. The catchment is reasonably uniform. The fits to observed data at Kelpie Point are the best of the four HDS with the exception of March 1978 where the recorder malfunctioned after the peak and the 'observed' flows are incorrect. The March 1978 flood also produced a very sharp rise to the peak, similar to that observed at Cedar Ford. The RORB model is calibrated to match the rising limb in March 1978, but could not quite reach the peak. Nonetheless, a five per cent discrepancy is considered acceptable.

The adopted parameters are listed in Table 21 and the results shown on Figures B8 to B12.

Table 21. Kelpie Point – fit parameters for RORB modelling

Flood	k_c	IL (mm)	CL (mm/hr)	Peak (m^3/s)		Diff (%)	Volume ($m^3 \times 10^6$)		Diff (%)
				Observed	Modelled		Observed	Modelled	
Jun-64	70	60	2.1	1,370	1,380	+1	204	172	-16
Jun-75	70	60	4.0	1,380	1,370	0	99.0	97.3	-2
Mar-78	70	90	2.0	2,330	2,220	-5	No obs. volume		
Aug-86	70	130	1.0	2,280	2,170	-5	249	254	+2
Apr-88	70	60	1.9	851	848	0	97.0	95.1	-2

Notes: Estimated k_c; Boyd 69; Kleemola 35 A = 1,450 km²

The initial losses (with the exception of August 1986) are lower than at the other stations while the continuing losses show a greater variability, although still within a normally accepted range. As was the case for the Causeway station, the k_c value is as predicted by the Boyd equation.

6.3.6. Warragamba Dam residual area

The residual area presents some unique challenges for hydrologic calibration. The quality of the modelled fit is subject not only to unknowns within the area, but also to discrepancies between modelled and observed hydrographs at the four upstream stations. Also, the observed data are dam outflows which are sensitive to volume and gate operations rather than changes in k_c. A further complication is the fact that historically, the gate operation has rarely followed exactly the H14 gate operation regime which was incorporated into RORB.

Figures B13 and B14 compare observed outflows from the dam with RORB results with the dam routing module inserted (note that the pre-auxiliary spillway dam routing module was used for calibration). The results are considered acceptable, and where discrepancies occur, it is usually due to the fact that the H14 gate operation was not followed exactly during the flood.

Table 22 shows the results for the residual area for both the theoretical pre-dam case and the observed results with the dam in place.

Table 22. Residual area – fit parameters for RORB modelling

Flood	Pre- or post-dam	IL (mm)	CL (mm/hr)	Peak (m^3/s)		Diff (%)	Volume (m^3 x 10^6)		Diff (%)
				Observed(1)	Modelled		Observed	Modelled	
Jun-64	Pre			7,800	7,850	+0.6			
	Post	50	1.0	7,050	7,100	+0.7	753	765	1.5
Jun-75	Pre			6,400	6,430	+0.5			
	Post	120	4.0	4,630	4,990	+7.7	513	511	-0.4
Mar-78	Pre			8,120	7,850	-3.3			
	Post	80	4.0	6,260	6,360	+1.6	778	853	9.6
Aug-86	Pre			5,600	5,350	-4.4			
	Post	120	4.0	2,760	2,840(2)	+2.6	367	380	3.6
Apr-88	Pre			5,950	5,700	-4.2			
	Post	50	3.0	4,630	4,940	+6.6	538	511	-5.1

(1) *In the pre-dam case, the 'observed' peak is obtained by hydraulic modelling*

(2) *The model result was obtained by modifying the gate operation procedure to match that used in 1986.*

6.3.7. Colo River at Upper Colo and Morans Rock

The Colo River catchment is largely undeveloped and includes an expanse of declared wilderness area. In contrast to the Sydney Water catchment areas above Warragamba Dam, the Colo had historically been poorly covered by both daily read and, in particular, pluviograph stations.

The streamflow data are also less comprehensive. Records at Upper Colo were used as the primary data, as Morans Rock is affected by tailwater from the Hawkesbury River in large floods. Only scattered observations were made at Upper Colo in June 1964 and the recorder malfunctioned in August 1986, the second largest of the calibration floods on the Colo. The Morans Rock record was used for this event, but the rating was not checked, and the chart shows clear evidence of backwater effects from the Hawkesbury River. The parameters adopted are shown in Table 23 and the fits shown on Figure B15.

The results are considered to be good in light of the sparsity of rainfall data. The March 1978 flood is by far the largest observed on the Colo and may be larger than a 1 in 100 AEP event. The fit to the flood is good up to the peak with both of the double peaks being well modelled. However, the model overestimates the recession flows and hence the total volume.

The high initial losses observed throughout the rest of the Hawkesbury-Nepean Valley are again in evidence for most of the floods modelled.

Table 23. Upper Colo/Morans Rock – fit parameters for RORB modelling

Flood	k_c	IL (mm)	CL (mm/hr)	Peak (m^3/s)		Diff (%)	Volume ($m^3 \times 10^6$)		Diff (%)
				Observed	Modelled		Observed	Modelled	
Jun-64	70	105	1.0	1640	1590	-3	No obs. volume		
Jun-75	70	115	1.0	963	932	-3	89.8	88.6	-1
Mar-78	70	60	1.0	5680	5420	-5	641	724	+13
Aug-86	70	165	2.5	2550*	2480	-3	No obs. volume		

** observed at Morans Rock*
Estimated k_c; Boyd 132; Kleemola 59 A = 4,640 km^2

6.3.8. Uncalibrated catchments

6.3.8.1. General

Establishment of calibration parameters for the catchments above Warragamba Dam and on the Colo River are described in the sections above. This section focusses on estimating flows from other catchments within the valley. It was not possible to directly calibrate the RORB model for these catchments, either because there was no gauging station, or if there was then there was no rating curve to produce flows, or the station records were influenced by backwater from the main river. In order to estimate flows from these catchments to provide input to the hydraulic model, the problem was approached in two stages:

i) the storage coefficient k_c was estimated from the catchment area

ii) rainfall losses were estimated on a flood by flood basis interactively with the hydraulic model.

The following sections detail these procedures.

6.3.8.2. Storage coefficient k_c

Table 24 summarises the storage coefficients determined from the calibration runs described above.

Figure B24 shows a plot of the results on log-log scale. Also shown on the plot are the Boyd and Kleemola fits and three fits related to the data in Table 17. The three fits show:

A the line of best fit (least squares) to all six data points

B the line of best fit if the residual area is deleted

C the line of best fit if the residual area and Kelpie Point are deleted.

Table 24. Storage coefficients for the gauged catchments

Station	Area (km^2)	k_c	No. of events
Jooriland	4,560	80	5
Causeway	446	35	4
Cedar Ford	733	35	4
Kelpie Point	1,450	70	5#
Residual Area	1,811	23	5
Colo	4,640	70	4*

 # one with no observed volume

** two with no observed volume*

The k_c value of 23 for the residual area plots poorly on Figure B24, but this can be explained by recognising that the residual area is not a cohesive sub-catchment, but is rather a collection of smaller areas which make contributions to the main rivers at various points. The value of 23 thus corresponds with a much smaller area, which would be consistent with the other derived values of k_c. The residual area has therefore been deleted in Lines B and C.

Kelpie Point plots in an atypical position (hence Line C), but it is difficult to justify ignoring the data point. An attempt was made to fit the data with a k_c (50), which would match the other catchments, but this was not satisfactory. It was considered that Kelpie Point should be retained in the fit and curve B adopted as the best representation of the available data. The k_c values of ungauged catchments were based on this curve. The equation of curve B is:

$$k_c = 4.23A^{0.344}$$

While this relationship is based on fewer catchments than used by Kleemola, all the catchments used are in the region under consideration. Also, the Kleemola data included analysis of the four Sydney Water HDS, using old rating curves which have subsequently been superseded. Thus, the curve B relationship is preferred for estimation of k_c values in ungauged catchments considered in this study. It should be noted that no validity is claimed for the relationship outside the range of catchment areas considered (that is, 400 to 5,000 square kilometres), nor outside the Hawkesbury-Nepean catchment. Curve B gives k_c values for the ungauged catchments as shown in Table 25.

Table 25. Storage coefficients for the ungauged catchments

Stream	Station	Area (km^2)	k_c
Nepean	Wallacia	1,760	(55)[1], 70
Nepean	Penrith	489[2]	35
Grose	Burralow	650	40
South Creek	Windsor	640	40
Macdonald	St Albans	1,680	55
Hawkesbury	Brooklyn	2,732[3]	65

(1) The storage coefficient value of 55 at Wallacia was calculated from Curve B. When the hydraulic model was run, it was found that the value of 55 produced stage hydrographs which were consistently too peaky when compared with observed levels. Subsequently, the Wallacia k_c was increased to 70 which gave satisfactory results.

(2) downstream of Wallacia and Warragamba Dam

(3) This is the area downstream of Penrith, Burralow (Grose River), Windsor (South Creek), Colo and St Albans.

6.3.8.3. Loss rates

Loss rates for the ungauged catchments were determined interactively with the hydraulic model. The 1996 Flood Study Part C gives details of this procedure. For completeness, the loss rates used throughout the catchment are shown in Table 26, which also includes the additional floods used for low flow calibration (discussed in detail in the 1996 Flood Study).

Table 26. RORB parameters for historical storms

Station	k_c	Nov-61		Jun-64		Jun-75		Mar-78		Aug-86		Oct-87		Apr/May-88		Jul-88		Apr-89		Aug-90	
		IL	CL	IL	CL	IL	CL	IL	CL	IL	CL	IL	CL	IL	CL	IL	CL	IL	CL	IL	CL
Jooriland	80	120	2.0	95	0.1	90	0.2	120	0.2	135	0.1	70	0	110	0.2	40	0.1	45	0.5	70	0
Causeway	35	120	2.5	100	2.5	90	2.5	160	1.0	140	2.5	50	2.0	100	2.0	100	0	30	1.5	70	1.5
Cedar Ford	35	120	2.0	120	1.0	70	3.5	120	1.0	140	1.0	50	1.0	140	1.3	120	0	30	1.2	50	1.5
Kelpie Pt	70	120	2.0	60	2.1	60	4.0	90	2.0	130	1.0	50	1.0	60	1.9	110	1.0	30	1.1	80	0
Residual Area	23	120	2.0	50	1.0	120	4.0	80	4.0	120	4.0	50	4.0	50	3.0	120	4.0	30	2.0	50	1.5
Wallacia	70	80	2.5	120	0.5	90	1.5	130	1.5	90	4.0	25	3.5	50	1.0	20	4.0	50	2.0	50	2.0
Penrith	35	80	2.0	100	2.0	80	2.0	60	1.5	80	3.0	50	2.0	60	2.0	50	2.0	30	1.0	50	1.5
Burralow	40	100	2.0	80	1.0	115	2.5	60	1.5	80	3.0	50	1.5	60	2.0	50	2.0	30	1.0	50	1.5
South Creek	40	100	2.0	80	1.0	115	2.5	60	1.5	80	3.0	50	1.5	60	2.0	50	2.0	30	1.0	50	1.5
Colo	70	100	2.0	105	1.0	115	1.0	60	1.0	165	2.5	50	1.5	60	2.0	50	2.0	30	1.0	50	1.5
Macdonald	55	100	2.0	80	1.0	115	2.5	60	1.0	100	2.0	50	1.5	60	2.0	50	2.0	30	1.0	50	1.5
Brooklyn	65	100	2.0	80	1.0	115	2.5	60	1.0	100	2.0	50	1.5	60	2.0	50	1.5	30	1.0	50	1.5

Note: The body of the table shows losses for each station and storm in the form of IL and CL, whereby:

- *IL is Initial Loss in mm*
- *CL is Continuing Loss rate in mm/hr.*

7. HYDRAULIC MODELLING

7.1. Adopted model

The distance from Warragamba Dam to the ocean is approximately 200 kilometres and includes:

- narrow incised valleys (from Warragamba to Penrith)
- deep river channels that can convey a 1 in 50 AEP flood (Penrith)
- wide floodplains with a large flood range (Windsor)
- a choked river valley that transitions into a drowned river valley (downstream of Windsor to the ocean).

These diverse hydraulic features mean that, until the recent invention of high capacity Graphics Processing Unit (GPU) and GPU-based hydrodynamic models such as TUFLOW HPC (Heavily Parallelised Compute), two-dimensional modelling of the entire valley was not possible. Even with current GPUs, it is necessary to represent the gorge upstream of Penrith in a relatively simplistic representation. While this floodplain is challenging for two-dimensional models, the quasi two-dimensional model developed in the earlier studies (RUBICON) can be run fast enough (5,000 times faster than the two-dimensional model) that it can be used in a Monte Carlo environment (refer to Section 8). The remainder of this chapter describes the fast quasi two-dimensional model while the quasi calibrated two-dimensional model (TUFLOW HPC) is discussed in Appendix D.

Open channel flow is governed by two complex equations known as the Saint Venant equations. The first equation relates to continuity or the total mass of water, while the second considers motion, how the water moves under the applied forces, including friction. The flow along a branch is calculated from water levels and topographic data which is represented in the model in the form of cross sections at selected grid points. The model also uses these cross sections to calculate storage within the branches.

The RUBICON model solves the Saint Venant equations using a variation of the four-point Preissmann scheme. The model allows complex, looped one-dimensional networks, as well as storages and complex structures.

The model simulates the river channels and floodplains as a series of ‘branches’ joining together at ‘nodes’. Each branch in turn contains at least two ‘grid points’. Branches can simulate open channel flow and/or weir type flow such as occurs over levees or road embankments.

Weir type flow is used to describe flow over levees, road embankments or weirs operating as control structures. Under these conditions the second Saint Venant equation of motion is replaced by an equation specifically representing flow over the structure. Flow is calculated as critical, free overflow or as submerged flow controlled by the downstream water level.

Branches join together at nodes. Any given node can have any number of branches joining to it. In addition to joining branches together, nodes can act as input points for flow or stage hydrographs or can represent floodplain storage for simulating backwater areas.

The adopted RUBICON model is as developed for the 1996 Flood Study with the addition of the M4 culverts and a variable tidal boundary for design event modelling. Therefore, the following sections are largely a reproduction of the 1996 Flood Study report.

7.2. Model development

Ten historic flood events were used in the development of the hydraulic model. These ranged in size from the November 1961 flood, which was the second largest in the valley in the past 200 years, to a small fresh in October 1987, which produced no outflow from Warragamba Dam. The events used are shown in Table 27 in Section 7.4 along with further details of the model calibration and the approximate probability of the event occurring under current conditions. The calibration process was undertaken for the 1996 Flood Study and was not revised during the current study as no new floods suitable for calibration had occurred. The steps involved in the calibration were:

- initial calibration to obtain model stability and reasonable fits to observed data
- review by Mr A Verwey, co-author of the RUBICON program (with outcomes presented in *Hawkesbury-Nepean Hydraulic Model*, 1989)
- fine tuning of the model using flood events of March 1978, August 1986 and April/May 1988 (particularly around Windsor and Penrith)
- comparison to the flood of August 1990 (which occurred during the model's development).

7.3. Model description

7.3.1. Model layout

RUBICON is termed a 'quasi two-dimensional model'. This means that the actual mathematics of the program only models flow in one dimension and the two-dimensional component is imposed by the practitioner with suitable choice of branch and node locations. It was therefore important that the adopted model layout accurately represent all potential flow paths.

The adopted layout is shown on Figure 9. The layout covers a total river length of 360 kilometres and incorporates the following elements:

- 75 nodes
- 106 branches
- 454 grid points
- 11 point inflows
- 14 distributed inflows
- 362 cross sections
- 35 weirs
- 7 storage nodes.

On the Nepean River, the model was extended upstream to Camden in order to provide an approximate representation of the large floodplain area around the town. Topographic data in this area were relatively sparse, and accurate modelling begins below Bents Basin.

The Warragamba River is modelled from the dam, which is represented by a point inflow. Below the Warragamba Junction, the model is one-dimensional through the Nepean Gorge until breakouts are encountered downstream of the M4 (formerly F4) bridge at Regentville. Flow paths are modelled along Emu Plains to the west and Peachtree Creek to the east. These are represented largely by cross sections with a weir inserted at the embankment of the Main Western Railway Line. Further flow paths are modelled across the Penrith Lakes area before all the flow is brought back together at Castlereagh. The model was developed with an approximate representation of Penrith Lakes in its 1990 format. This representation was adopted for the calibration events.

The major changes in the river system have all occurred in the area around and below Penrith. The model was established to represent the key changes and these are outlined in the following sections.

A breakout into Richmond Lowlands occurs just below Yarramundi Bridge and this is linked to the main river by several weir branches up to the river bend at Cordners Corner at the end of Cordners Lane. Here another breakout is modelled, which bypasses Windsor and re-joins the river near Wilberforce. The model is very complicated in the Richmond/Windsor area with numerous cross links to represent the flow paths and storage distribution which occurs in larger floods.

South Creek and Eastern Creek are both modelled by a series of branches which extend upstream as far as the M4 Motorway and Richmond Road respectively. This provides a good representation of the backwater storage volumes available on the creek floodplains.

Below Windsor flow is modelled through Pitt Town Bottoms and across Halls Point and Gronos Point. The model becomes largely one-dimensional downstream with small branches used to model the floodplains of various tributaries such as Cattai, Little Cattai and Currency Creeks. An overflow path operates between Wilberforce and Currency Creek in the 1 in 500 AEP and rarer events. A branch linking Freemans Reach to Currency Creek is included in the model for the probable maximum flood (PMF) and dam break events.

The Colo River is modelled up to the gauging station at Morans Rock, but because of the gorge, this does not involve any substantive storage.

In the estuarine area some of the larger tributaries such as Berowra, Cowan and Mooney Mooney Creeks are modelled as storages, since flows from these creeks are negligible in terms of major catchment wide flooding.

The model terminates between Barrenjoey and Box Heads at the entrance to Broken Bay. A tidal boundary is applied at this location to represent the ocean level and is used as the downstream boundary condition.

7.3.2. Weirs

Weir relationships were used to define levee banks, both natural and man-made, and road and rail embankments. Information was obtained mainly from survey carried out specifically for the 1996 Flood Study. Once the model layout had been determined, natural flow barriers were identified and surveyed.

In most cases weirs were used to model the flow of water out of the river over natural or man-made levees into floodplain areas. Penrith Weir was specifically modelled as was the high embankment of the railway line on both sides of the river at Penrith.

Weirs were defined in the model by a series of horizontal crests. For each crest a height (m AHD) and length were provided together with weir flow coefficients.

7.3.3. Structures

Due to the significant flood depths experienced in the valley many of the bridges and culvert structures would be submerged in most flood events. For the most part the culvert capacity compared to the flow in the main channel is insignificant. Therefore, only the major bridges and culverts are included in the hydraulic model. This approach is appropriate for a regional scale model. Culverts and small bridges are represented as a relationship between flow and upstream and downstream water level. The following culverts and bridges are specifically modelled:

- 3 culverts under the M4 on Peach Tree Creek
- Bridge under railway line on Peach Tree Creek

Many of the bridge decks in the valley are low and are below frequent flood levels. These bridges are not represented in the model. For larger bridges such as Victoria Bridge the piers of the bridge are incorporated into the relevant cross section.

The culverts under the M4 Motorway were also added to the model to improve the mapped outputs of flood extent at Jamisontown. This had minimal effect on flood levels given the culverts convey 66.9 m^3/s flow in a 1 in 100 AEP.

7.3.4. Flood storage areas

The model contains six flood storage areas at Londonderry, Oakville, Currency Creek, Berowra Creek, Cowan Creek and Mooney Mooney Creek.

These represent areas of the floodplain which do not convey main river flow but simply store floodwaters as the level rises. The storages were represented as height versus surface area. The surface areas were calculated from contour maps with assistance from limited ground survey. For the storages in the lower estuary the stage storage relationship is based on the estuarine surface area at 0 m AHD.

7.3.5. Inflows

Inflow hydrographs to the model were defined at 25 different points as shown on Figure 9.

7.3.5.1. Warragamba Dam

Observed outflow hydrographs from Warragamba Dam were used for all historical floods. In October 1987 there was no outflow from the dam available.

7.3.5.2. Nepean River at Wallacia

While a record of water level over time (a stage hydrograph) was available for all of the calibration events at this location, it was not possible to obtain a direct interpretation of flows due to backwater effects from the Warragamba River. The inflows at this point were therefore derived interactively with the RORB model.

There is significant flow attenuation in the area upstream of Bents Gorge that was not reproduced by the RORB model inflows. To address this issue, flow from the RORB model at Wallacia was input upstream of Bents Gorge (approx. 10 km upstream). Adjustments were then made to both the RORB and RUBICON parameters (i.e. k_c, Manning's 'n') in an effort to match the observed stage hydrograph at Wallacia. While this procedure represented a degree of double routing, this additional routing is effectively representing the extra storage upstream of Bents Gorge (see Figures C12 to C25).

7.3.5.3. Other inflows

All other inflows were derived from the RORB model. The major tributaries of the Grose, Colo and Macdonald Rivers were specifically represented as were several smaller tributaries such as Erskine, South, Eastern and Cattai Creeks. A series of 'distributed inflows' were also included to represent either a collection of minor tributaries or rain falling directly onto large storage areas such as the Richmond Lowlands.

7.3.6. Ocean levels

Where possible, ocean level information was obtained from DLWC (now OEH) gauges near the river entrance. Where DLWC (now OEH) could not provide gauging information, tidal records were obtained from Sydney and Newcastle.

7.4. Model calibration

7.4.1. Available data

Two types of data were needed to calibrate the hydraulic model. The first were inputs, such as streamflows and ocean conditions. These are discussed in Sections 7.3.5 and 7.3.6. The second type of data were observed heights and, if available, flows within the modelled area that could be compared with the model output. These are discussed below.

Table 27 shows the floods used and their role in the calibration/verification process. After the model was calibrated and verified over the full length of the river, the model was further refined in the Penrith area, as the calibration/verification process showed that channel changes probably occurred between the March 1978 event and 1986. Once this was confirmed and additional data were collected, a recalibration and verification were undertaken in the Penrith area. The peak flood levels at Penrith are included in the table to give an indication of the relative sizes of the events.

Table 27. Calibration and verification floods

Flood	Modelled peak at Penrith (m AHD)	AEP range under current conditions	Initial calibration*	Fine tuning#	Verification	Penrith Refinement	
						Recalibration	Verification
Nov 1961	23.89	5-2% AEP	X		X		X
Jun 1964	23.74	5-2% AEP	X		X		X
Jun 1975	21.49	10-5% AEP	X		X		X
Mar 1978	23.35	5-2% AEP	X	X		X	
Aug 1986	19.95	20-10% AEP	X	X			X
Oct 1987	17.58	<20% AEP			X		X
Apr/May 1988	22.62	10-5% AEP	X	X			X
Jul 1988	20.32	20-10% AEP			X		X
Apr 1989	18.50	<20% AEP			X		X
Aug 1990	23.44	5-2% AEP		X		X	
Jun 1867	27.49	>1% AEP			X		

**These events were used as an initial calibration and model consistency check prior to the fine tuning and calibration. Refer to 1996 Flood Study.*

 # discussed as fine tuning in 1996 Flood Study and calibration in this report. Followed expert review.

Figures referred to in the following sections are reproduced from the 1996 Flood Study in Appendix C.

7.4.1.1. Peak heights

The most readily available information on historical floods was peak heights recorded at various sites throughout the floodplain. This information came from a variety of sources: automatic recorders, regularly read official gauges, private gauges, private observations, and debris and silt lines located after the floods.

The quality of the data varied from very good to very unreliable, and it was often difficult to identify the quality of some information, especially when only one report was available for a particular area. The hydraulic model assisted in assessing data quality by identifying points which were inconsistent with information from other areas in the same flood event or with information from other floods in the same location.

Even well observed data were subject to some error in reading, and discrepancies of ±0.1 m were considered acceptable. The reasons for this include:

- the physical difficulties of observing flood levels near the flood peak
- surface waves and surges, which made an assessment of the 'still water' level difficult. This was particularly true at sites where water velocities were high.
- interference with the recorder or measuring equipment due to flood borne debris
- In the case of debris marks observed after a flood, the marks may not represent the peak height. They may have been deposited below the peak, or they may have been driven above the average water level by high velocities and waves.

All of these factors lead to a large degree of scatter in observed levels as can be seen from the data points plotted in Figures C1 to C11. Data points are listed in Appendix C.

7.4.1.2. Stage hydrographs

At certain points along the river, various government agencies have established gauging stations which provided continuous records of stage height over time (stage hydrographs). This information was collected where available for the calibration and verification floods.

Table 28 lists the stations on the main stream within the model extent.

Figures C12 to C25 reproduce available stage hydrographs at key stations for various floods and compare these with hydrographs produced by the model.

Table 28. Gauging stations within the modelled area

Gauging station	Authority
Wallacia	Sydney Water/WaterNSW
Nepean Junction	Sydney Water/WaterNSW
Penrith	Sydney Water/WaterNSW
Castlereagh	Sydney Water/WaterNSW
North Richmond	Sydney Water/WaterNSW
Freemans Reach	Sydney Water/WaterNSW
Windsor	Sydney Water/WaterNSW
Port Erringhi	DLWC/OEH
Sackville	DLWC/OEH
Sackville Ferry	DLWC/OEH
Merrit Farm	DLWC/OEH
Dargle	DLWC/OEH
Lower Portland	DLWC/OEH
Clifton Lodge	DLWC/OEH
Little Patonga	DLWC/OEH

7.4.1.3. Flows

The measurement of flows during flood events is subject to even greater errors than the measurement of levels. Measurements are taken from boats or bridges. Spot measurements of velocity are taken at various distances across the river and depths below the surface. These measurements are then used in conjunction with a surveyed cross section of the site to estimate the average volume of water passing through the section per unit time. One set of such measurements is called a gauging. Gaugings taken over a range of flows are then used to derive a rating curve of flow against water level or stage.

The difficulties of taking these measurements at the height of a flood are considerable. Under the most favourable of conditions, an accuracy of ±10% is the best that can be achieved, and these errors are carried over into the rating curve. ARR 1987 (Section 1.4) (Pilgrim, 1987) suggests that a more realistic error band for flow records might be ±25% under very extreme flow conditions.

The key source of flood flow information within the modelled area is Penrith, which has been regularly visited by Sydney Water personnel during floods. Flood gaugings taken at Penrith over the period 1972–1990 are listed in Table C1.

Gaugings have been taken at other sites in the Hawkesbury-Nepean Valley by both Sydney Water (now WaterNSW) and DLWC (now OEH). However, there are not enough gaugings available at any other site to provide a good assessment of the model performance.

7.4.2. Calibration floods

Table 29 summarises the catchment wide difference in level between modelled flood levels and gauge or observed level data. The performance of the model in each event is further described in the following sections.

Table 29: Calibration of model to available flood levels

Event	Gauge data difference (m)		Other data difference (m)		Overall difference (m)	
	Mean	Median	Mean	Median	Mean	Median
March 1978	-0.01	0.01	0.13	0.07	0.06	0.04
August 1986	-0.12	-0.12	0.01	-0.03	-0.05	-0.08
April / May1988	0.14	0.12	0.03	-0.02	0.09	0.05
August 1990	-0.25	-0.23	-0.37	-0.28	-0.31	-0.26

7.4.2.1. March 1978

The March 1978 flood is considered first because it formed the basis of the model calibration throughout the valley with the exception of Penrith. At Penrith, it was used as the main tool to determine pre-September 1986 conditions.

Figure C1 shows the profile of peak heights for the 1978 flood along the main river from Wallacia to Broken Bay. The fit of the model to various observed hydrographs is shown on Figure C12. These figures also show the scatter of observed data points which indicates some of the difficulties associated with obtaining accurate flood data (see Sections 7.4.1.1 and 7.4.1.3).

The peak height profile shows a good fit to the data up to river chainage 90 kilometres (between Portland and Sackville). Between 90 and 110 kilometres (Port Erringhi), the model is low (up to -0.52m) although the fit to the official gauge reading at Sackville is acceptable (-0.18m). A more exact fit could not be obtained without making unacceptable adjustments to Manning’s ‘n’. The adopted profile also presents a good compromise with the fit of the April/May 1988 flood in this reach (Section 7.4.2.3). It is important to note that the difficulties in obtaining consistently accurate fits in this reach do not affect the fits at Windsor (where the level matches very well) and further upstream.

Through the Windsor flood storage area, the fit is good, but there is a discrepancy at Yarramundi between the model and the official reading. Comparison with other floods indicates that the Yarramundi level should be of the order of a metre higher than the North Richmond level (as shown by the model) and it thus appears that the reported level for Yarramundi for 1978 is in error.

The reported reading at Jacksons Lane (159.3 kilometres) is also low compared with the model; however, it was subsequently found to be unreliable by other studies.

The section upstream of Jacksons Lane was modified to reflect apparent changes in topography prior to 1986 (refer to Section 7.5.1). The number and extent of changes made were tempered by the need to maintain consistency of roughness values and to be able to explain qualitatively the changes that were assumed. The overall fit is quite good, although marginally high at the Penrith gauging station. Figure C12 shows a good overall fit to the observed hydrograph.

Comparisons of observed hydrographs with modelled levels are available at four locations (Figure C12) and indicate generally good agreement, particularly in the vicinity of the peaks. At Penrith, the model replicates the major fluctuations of the hydrograph very accurately. At both North Richmond and Windsor there is an early rise, likely due to local runoff from the Grose River, which the model does not reflect (because of insufficient hydrologic data). Over the last four or five metres of the rising limb, the modelled hydrograph is slightly early at Windsor.

While the modelled shape at North Richmond matches that observed reasonably well over the peak, the modelled levels are high. Given the constraint of the need to fit the Windsor information, the North Richmond levels could not be reduced without making Manning's 'n' (roughness value) unreasonably low. The modelled levels are low at North Richmond in the August 1986 flood and good for April/May 1988. The adopted values represented a good compromise between the conflicting data for the different floods.

Throughout the catchment, the model is on average 0.06m higher than the calibration data. This is a minor difference and is considered acceptable (Table 29).

7.4.2.2. August 1986

The peak height profile is shown on Figure C2, and a comparison of observed and modelled hydrographs is shown on Figures C13 and C14.

The peak height fit (Figure C2) is generally good, with the model being low (0.38 m) upstream of Colo Junction (84 kilometres) and around Castlereagh (145–155 kilometres, where the model was 0.18 m below the observed level). The slightly low fit near the Colo is likely due to the fact that the RORB-generated Colo flow occurred much earlier than the observed flow from the Colo.

The modelled hydrograph at Wallacia matches the observed rising and falling limbs, but fits the peak section very poorly. The observed hydrograph is an unusual shape and is unlike any other observed event at Wallacia. Although there is no obvious fault with the record, it was necessary to disregard it on this occasion. This decision was assisted by the fact that the modelled and observed hydrographs at Penrith agree well.

At Penrith the modelled hydrograph is slightly low, but the shape is good. One observed point early in the flood seems incorrect. At North Richmond and Windsor, the modelled hydrographs fail to match an early peak, which is likely caused by inflows from the Grose River or local runoff and could not be modelled properly due to lack of data. Both hydrographs fit the peaks well but fall below the recession limbs of the observed levels.

Further down the river at Port Erringhi (Figure C14), the model produces an initial rise which is higher and earlier than that observed. This is due to the early rise of the modelled Colo flows compared to the observed. This also happens at Sackville Ferry and Merritt Farm. The significant aspect of these hydrographs is that despite the timing difficulties with the Colo, the peak section of the Port Erringhi hydrograph gives a good fit. This is a further indication that the Colo River does not exert a significant influence on peak flood levels around Windsor and Richmond. An underestimation of the limited observed data occurs at Merritt Farm and Sackville with differences between modelled and observed in the order of 0.38 m. While a poor calibration occurs for this event at these locations a better fit occurs on a number of other events at these locations.

Throughout the catchment, the model is on average 0.05 m lower than the calibration data. This is a minor difference and is considered acceptable (Table 29).

7.4.2.3. April/May 1988

The peak height profile for the April/May 1988 flood is shown on Figure C3 and observed and modelled hydrographs on Figure C15.

The modelled peak height profile produces a good fit generally, except between 90 kilometres and Gronos Point (chainage 116.5 km). Throughout this section the modelled profile is slightly high (on average, a 0.27 m difference). But this result is balanced by low levels in this section for both the 1978 and 1986 floods.

The hydrographs give generally good fits. Wallacia matches very well over the peak period. The shapes at Penrith are good with the model being about three hours behind the observed. As with the 1986 event, there is a higher observed flow on the rising limbs of both the North Richmond and Windsor hydrographs. Again, this points to limitations of the hydrologic data rather than issues with the RUBICON model. The peaks at both sites show good agreement.

Throughout the catchment, the model is on average 0.1 m higher than the calibration data. This is a minor difference and is considered acceptable (Table 29).

7.4.2.4. August 1990

The fit to the August 1990 flood data is shown on Figures C4, C16 and C17. The modelled profile fits well at the lower end (below 50 kilometres) and at Windsor where the model is 0.02 m lower on average than the observed data points. However, it is significantly low between Wisemans Ferry and Sackville where on average the model is 0.54 m below observed gauge levels. As noted for other floods, the problem seems to have more to do with the hydrologic input, especially the relative timing of the Hawkesbury and Colo flows, rather than with hydraulic considerations.

This flood was used as the primary tool for calibration in the vicinity of Penrith as it occurred after the original calibration process, highlighted the apparent changes to the channel at Penrith and showed consistency with the events post 1986. For this reason, the modelled hydrograph at Penrith is very good. The hydrograph at Windsor matches well in the 18 hours leading to the peak, but then the modelled results fall too quickly. The fit at Sackville shows the same tendency.

Overall, the results from this event give good confirmation of the model's accuracy throughout most of the valley.

Throughout the catchment, the model is on average 0.3 m lower than the calibration data. This moderate difference is considered acceptable (Table 29).

7.4.3. Verification floods

As shown in Table 27, six floods were used for verification of the model. Three of these (November 1961, June 1964 and June 1975) were used in the initial coarse calibration phase, while three more floods (October 1987, July 1988 and April 1989) were introduced in the latter stages of verification as a review recommendation. An assessment of the fit of the record 1867 flood was also made. Table 30 presents the catchment wide differences in level between modelled levels and gauge or observed level data for the verification events.

Table 30: Verification of model to available flood levels

Event	Gauge data difference (m)		Other data difference (m)		Overall difference (m)	
	Mean	Median	Mean	Median	Mean	Median
November 1961	0.05	0.05	0.08	0.08	0.07	0.07
June 1964	-0.03	-0.06	-0.30	-0.22	-0.16	-0.14
June 1975	0.21	0.30	0.54	0.62	0.37	0.46
October 1987	NA	NA	-0.02	0.09	-0.02	0.09
July 1988	NA	NA	-0.11	-0.10	-0.11	-0.10
April 1989	NA	NA	-0.14	-0.13	-0.14	-0.13
June 1867	NA	NA	0.76	0.12	0.76	0.12

7.4.3.1. November 1961

The November 1961 flood was the largest flood of the 10 events used in calibration/verification. While a reasonable amount of height data were available, rainfall and streamflow data were limited. Therefore, input hydrographs for the RUBICON model were not of the same quality as for most of the other events. Nevertheless, they were adequate to provide a useful indication of the model's performance for a large flood.

The modelled peak height profile is shown on Figure C5 with stage hydrographs compared on Figures C18 and C19.

Given the limitations in the available hydrologic information, the fits are very good. On the peak height profile, the limited data between 90 and 112 kilometres indicates that the modelled levels are on average 0.4 m higher than observed data. At Windsor, the model fits the official reading well (0.05m higher than the gauge reading) and falls in the middle of the other observed points.

The model is low at North Richmond as reflected in the hydrograph fit (Figure C18). The hydrograph fit at Windsor is good at the peak although lacking in volume. The hydrographs at Penrith and Wallacia show general agreement with observations though varying somewhat in detail. This again can be explained by the lack of hydrologic data.

The modelled hydrographs are high at Lower Portland and Sackville for this event (Figure C19). There is sparse observed data and limited hydrologic data for this event. Throughout the catchment, the model is on average 0.07 m higher than the observed data. This is a minor difference and is considered acceptable (Table 30).

7.4.3.2. June 1964

The modelled peak height profile (Figure C6) is generally satisfactory at and above Windsor. Below 100 kilometres the fit is low (approximately 1 metre), but again this can probably be attributed to lack of hydrologic data in the downstream catchments, particularly for the Colo River. At Penrith, the modelled peak level is slightly high by about 0.2 metres, but it is a similar magnitude low at Regentville.

The comparison between the modelled and observed hydrographs at Wallacia (Figure C20) is fair with the model low at the peak. At Penrith, the modelled hydrograph shows a steep drop immediately after the peak. This was caused by a rapid reduction in outflow at Warragamba Dam. This rapid reduction has two effects on peak levels around Penrith. Firstly, they are particularly sensitive to the relative timing of the Warragamba and Nepean Rivers. Secondly, infrequently observed levels could be in error by more than usual because of the quick change in river level. In these circumstances the overall match of peaks through the Penrith reach is considered to be satisfactory.

The profile between North Richmond and Gronos Point is generally good although the model is low at Windsor. Throughout the catchment, the model is on average 0.16 m lower than the observed data. This is a minor difference and is considered acceptable (Table 30).

7.4.3.3. June 1975

The modelled peak height profile for the June 1975 flood (Figure C7) fits the observed level at Penrith with the profile only 0.01 m below observed data. However, the modelled levels are significantly high at Windsor (0.3m higher than the gauge data and 0.6m higher than other observed points) and Gronos Point (1.1m). As there are no observed hydrographs at Penrith, North Richmond and Windsor, it is difficult to determine the cause. The problem cannot be attributed to hydrologic data as the hydrology indicates no inflow from South Creek and the other areas around Windsor. The nature of hydrologic/hydraulic modelling is such that departures such as these must be expected in the verification process. The results from this flood should be seen in the light of the largely satisfactory results from other events.

Figure C21 shows a fair fit to the Wallacia hydrograph, the only available stage hydrograph data for this flood.

Throughout the catchment, the model is on average 0.37 m higher than the observed data. This is a considerable difference; however, given the fit at Penrith and lack of data in other areas, it is considered acceptable (Table 30).

7.4.3.4. October 1987

The October 1987 flood was a small fresh with no outflow to the valley from Warragamba Dam. The flood was included at the request of Mr Verwey (following his review of the 1996 Flood Study) to test the low flow calibration of the model. The peak height profile is shown on Figure C8 and the stage hydrographs on Figure C22.

This is the smallest of the ten flood events and is considerably below the calibration range of the model. Accordingly, the very good fit obtained throughout most of the valley indicates that the adopted calibration is generally satisfactory.

Throughout the catchment, the model is on average 0.02 m lower than the observed data. This is a minor difference and is considered acceptable (Table 30).

7.4.3.5. July 1988

The peak height profile for the July 1988 flood is shown on Figure C9 and the hydrographs on Figure C23. Figure C9 shows a large scatter of observed levels in the vicinity of Windsor. These are generally within the typical variability and uncertainty associated with observed flood levels.

The model fits well through the middle of the observed Windsor levels with an average difference at Windsor of -0.1 m, and fits well elsewhere in the valley.

The hydrographs show a poor fit at Wallacia. At Penrith, the modelled hydrograph shape is good although a little high. At both North Richmond and Windsor, the modelled hydrograph rises later than the observed event. While the delayed rise is quite pronounced in this event, similar behaviour is observed in other events around 10 m at Windsor. This delayed rise is probably caused by a combination of the Richmond Lowlands filling too quickly and the difficulties of estimating inflows from the Grose River and/or local runoff. At both sites the model fits well in the vicinity of the peak.

Throughout the catchment, the model is on average 0.11 m lower than the observed data. This is a minor difference and is considered acceptable (Table 30).

7.4.3.6. April 1989

The modelled peak height profile (Figure C10) shows a generally good fit throughout the valley, but is on average 0.4 m lower between Windsor and Castlereagh. The hydrographs (Figures C24 and C25) show generally good fits, especially at Penrith. At North Richmond, the model is a little low but gives the best shape of any of the floods.

Throughout the catchment, the model is on average 0.14 m lower than the observed data. This is a minor difference and is considered acceptable (Table 30).

7.4.3.7. June 1867

There is no flow data and virtually no rainfall data available for this flood. Inflows were generated by factoring probable maximum precipitation (PMP) rainfalls until the RUBICON model produced the observed level (19.7 m AHD) at Windsor. The point of the exercise was to see how well the model would match observed levels at other sites in the valley based on the limited information known about the 1867 flood.

The results are shown on Figure C11 and are very good between Sackville and Penrith. The only observed level downstream of Sackville is at Wisemans Ferry where the modelled peak is several metres high. Possible explanations are a low observed level given the magnitude of the event upstream and/or the factored PMF has too much flow input for the Colo and Macdonald Rivers.

7.4.4. Key locations

This section provides a summary of the model's fit to observed peak heights at Penrith and Windsor.

7.4.4.1. Penrith

Table 31 compares the modelled and observed levels at Penrith for the calibration and verification floods. Overall the average difference at Penrith is 0.056m.

Table 31. Comparison of peak heights – Penrith

Date	Use in model	Observed (m AHD)	Modelled (m AHD)	Modelled difference (m)
Nov-61	D	23.89	23.94	0.05
Jun-64	D	23.74	23.98	0.24
Jun-75	D	21.49	21.43	-0.06
Mar-78	C	23.35	23.47	0.12
Aug-86	D	19.95	19.83	-0.12
Oct-87	B	17.58	17.61	0.03
Apr/May-88	B	22.62	22.74	0.12
Jul-88	B	20.32	20.56	0.24
Apr-89	B	18.50	18.45	-0.05
Aug-90	A	23.44	23.43	-0.01

Notes: A: calibration post-August 1986
B: verification post-August 1986
C: calibration pre-September 1986
D: verification pre-September 1986

The Penrith results need to be considered in two groups – pre-September 1986 and post September 1986 – to reflect the apparent change in local topography which occurred during the flood of August 1986.

Pre-September 1986

The difficulties associated with modelling this period at Penrith (particularly the lack of topographic data) are discussed in Section 4.1.2.2.

The March 1978 flood was the main calibration tool for the Penrith area for this period, and was used to determine reasonable approximations to the topography which would have existed at the time. The calibration process considered all the flood levels in the vicinity of Penrith and the observed hydrograph at the site (Figure C12). Care was also taken to use consistent values of Manning's 'n' throughout the reach and to apply consistent reasonable adjustments to the channel topography.

The adopted compromise to all these factors meant that the model was 0.12 metres high at the peak of the 1978 flood at the official Penrith gauge. This is a satisfactory result and is balanced by the fit to the August 1986 event which is 0.12 metres low. Two of the verification floods, November 1961 and June 1975, fall within 0.1 metre of the observed levels and this was taken as confirmation of a good fit.

The one exception to the good fits is June 1964. In this flood the discharge from Warragamba Dam was rapidly reduced at the peak of the event. The effect of this at Penrith can be seen on Figure C20. Because of the rapid reduction, the 1964 flood was particularly sensitive to timing variations. In these circumstances the discrepancy between the modelled and observed peaks is considered to be reasonable.

Post-September 1986

August 1990 was used to calibrate the Penrith area for post September 1986 conditions and gives a very good match to peak heights. Three of the verification floods: October 1987, April/May 1988 and April 1989, give good matches. These floods cover the range of verification events from the smallest to the largest.

The only flood which does not fit well is July 1988, for which the modelled peak is 0.24 metres high. This result is still considered acceptable given all the unknowns of flood observation and model simulation.

7.4.4.2. Windsor

Table 32 compares the observed levels at Windsor with those produced by the model. The observed levels listed are those observed at Windsor Bridge gauge or, more recently, at the automatic recorder upstream of the bridge. As can be seen from Figures C1 to C11 there can be a large scatter of observed data, even within the relatively level flood pool around Windsor.

For the main calibration flood of March 1978, the model is 0.05 metres high at Windsor Bridge, while for the other calibration events, August 1986 and April/May 1988, the profiles are 0.03 metres and 0.01 metres low respectively. This represents very good agreement between the events.

Of the verification floods, the modelled peak for November 1961, July 1988 and August 1990 all fall within 0.1 metres of the observed level. Of the remaining three floods, June 1975 is acknowledged as giving a poor fit around Windsor (Section 7.4.3.3). The June 1964 modelled flood level is low at Windsor but high at both Gronos Point and North Richmond, thus the model presents a reasonable fit to the data in the general vicinity, suggesting that some of the observations may be inaccurate. The April 1989 modelled flood level is very low however this is a much smaller event than the other events and it is not known if the peak water level was recorded at the gauging location as this would affect the fit. No stage hydrograph was recorded at Windsor for this event.

In summary, six of the nine events being compared give good agreement between the observed and modelled values. Another two events are considered acceptable. Given the variability of flood data, this is a strong endorsement for the general applicability of the model at this location.

Table 32. Comparison of peak heights – Windsor

Date	Observed (m AHD)	Modelled (m AHD)	Modelled difference (m)
Nov-61	14.95	14.98	0.03
Jun-64	14.57	14.3	-0.27
Jun-75	11.2	11.52	0.32
Mar-78	14.46	14.51	0.05
Aug-86	11.35	11.32	-0.03
Oct-87	N/A	5.35	N/A*
Apr/May-88	12.80	12.79	-0.01
Jul-88	10.74	10.68	-0.06
Apr-89	9.22	8.57	-0.65
Aug-90	13.50	13.44	-0.06

** N/A – Not available*

7.5. Rating curves

Section 5 gives details of the flood frequency analyses carried out to determine design flood levels throughout the valley. Flood frequency analysis uses long term records of flood levels at Penrith and Windsor. Before these records could be effectively used for flood frequency analysis, the flood levels had to be transformed into flows. This was achieved by means of rating curves which relate levels and flows. The following sections describe how model results were combined with gauging data to assist in the preparation of suitable rating curves.

7.5.1. Penrith gauging station

The gauging station at Penrith is operated by WaterNSW and a series of rating curves were already available. These curves were applicable for various periods corresponding to stable hydraulic conditions at the site. A new curve was produced when alterations were made to the weir, or when a change in topography became apparent, such as occurred in August 1986.

7.5.1.1. Current rating curve

Section 7.4.1.3 discusses the derivation of high flow rating curves and Table C1 shows high flow gaugings taken at Penrith since 1972. The highest gauging available during much of the RUBICON calibration process was at 22 m AHD, with a flow of 5940 m^3/s. This was one of seven surface velocity gaugings taken in May 1988. A series of gaugings was taken on the 2nd, 3rd and 4th August, 1990 with five of them (taken on the 2nd) being the highest ever obtained at Penrith. The heights of these gaugings ranged from 22.94m AHD to 23.42m AHD. All available high flow gaugings since 1986 are plotted on Figure C26, together with the rating curve used by Sydney Water's Hydrographic Branch.

Also shown on Figure C26 is a curve derived from the hydraulic model. This curve was obtained by running a series of various sized floods and plotting height versus discharge at the gauging station. These plots always showed a 'loop' effect with more flow occurring on the rising stage than on the falling stage at the same height. This effect is a well-known phenomenon and is the reason why gaugings are classified as being taken on the rising or falling limb of the hydrograph. Despite the loop effect, discharge curves are traditionally drawn as single valued functions taking more or less an average position between the rising and falling gaugings. This is particularly appropriate when the peak discharge is the primary concern. The 'modelled curve' on Figure C26 was thus obtained by drawing a smooth line through the peaks of the various plots. The adopted curve is tabulated in Table B5 in Appendix B.

Both the Sydney Water and modelled curves lie below the set of surface velocity gaugings taken in 1988 (between 21 and 22m AHD). As water tends to flow faster at the surface than at depth, it is usual practice to multiply surface gaugings by a factor to compensate. In the absence of any site-specific information, a factor of 0.85 is normally applied, as it was to these gaugings. Subsequent information available from the August 1990 gaugings, indicates that in the straight, uniform channel of the Nepean River at Penrith a higher factor, between 0.9 and 0.95, might be more appropriate. If this is applied, it brings the series of gaugings very close to the rating curves.

In general, the Sydney Water curve and the modelled curve agree quite well. However, there are two regions where they trend apart. The first is around 18 m to 19 m AHD where the Sydney Water curve dips down to fit a series of gaugings taken in April 1989. The model curve shows no such dip, and provides a better fit to four gaugings taken on the tail of the August 1990 flood.

The more serious discrepancy is at the highest gaugings, around 23 m AHD. The discrepancy is a particular problem because of its implications for extending the rating curve beyond the limit of the gaugings. The highest gauging is equivalent to a probability of about 1 in 20 AEP.

The Sydney Water curve was made to fit precisely through the gaugings. The modelled curve passes above the top four gaugings giving a flow value 12.5 per cent lower than the highest gauged discharge. While it would be possible to adjust the model to provide a better fit to the gaugings, this was not done for the following reasons:

- The known peak outflow from Warragamba Dam was 6850 m^3/s for the August 1990 flood. The peak height at Wallacia was 39.31m AHD which corresponds to a flow of no more than 2,000 m^3/s. Based on the rainfall to the south of Penrith, the peak flow from the intermediate area was approximately 500 m^3/s. Adding these together (which will overestimate the peak) gave a total flow at Penrith of 9,350 m^3/s. The top three gaugings produced flows higher than this figure and thus were not compatible with the other available flow information.

- The gaugings were based on only four or five full depth velocity measurements across the river and were computed using a cross section taken after the event. While these are reasonable methods to adopt in high flow gaugings, they cannot produce results with scientific accuracy. In these circumstances the maximum discharge discrepancy of 12.5 per cent between the gaugings and the model rating curve was more than acceptable.
- It seems fairly common that a group of gaugings taken at the same time will display a systematic trend. Thus, the trend evidenced in the set of gaugings was not unexpected. If, for example, the cross section had changed, or the current meter was in error, a systematic error would occur.
- The extended modelled rating curve (adjusted as discussed in Section 7.5.1.2) produced an estimated flow of 20,000 m^3/s for the 1867 flood. This agreed almost exactly with the estimate derived by the Hydrographic Branch (MWS&DB, 1985).
- Since the high gaugings were taken on the rising stage of the hydrograph, they could be expected to fit slightly below the curve.

In view of these considerations, it was decided that there was no reason to force the rating curve exactly through the gaugings. Indeed, given all the potential sources of error and approximation, the gaugings provided a reasonable confirmation of the model rating curve that was adopted.

7.5.1.2. Previous rating curves

Physical conditions at Penrith have changed several times since systematic recording began. Each change affected the relationship between water level and flow rate and hence the rating curve. Some of the changes, particularly since 1960, are reasonably well defined, but before 1960 the lack of information on the topography of the river bed and floodplain, and how it changed with time, decrease the certainty of the relationships.

The current rating curve, which is discussed in Section 7.5.1.1, dates from after the peak of the August 1986 flood, when there were apparent changes in the bed level of the weir pool. Four other rating curves were produced from RUBICON for the period preceding September 1986. The periods covered by each curve and the physical changes involved are listed in Table 33, together with the changes made to RUBICON to simulate the different conditions.

Aerial photographs were also available for both 1955 and 1956, but because of the rapid changes taking place due to excavation for the dam, these would only have been applicable for a very limited period. Consequently, these photographs were not used. Peak flows for the period 1950–1960 were determined by interpolating between the estimated flows for the periods 1910–1949 and 1960–August 1970 (refer Table 33). The rating tables are presented as Table B1 to Table B5 (Appendix B), and are plotted on Figure C28.

Table 33: Periods covered by each rating curve

Date	Comments
a) Pre-1910	• Before Penrith Weir was built in late 1909. • In the absence of other information, the river topography in the vicinity of the gauge was assumed to be the same as for the period 1910-1949. • The function in RUBICON which simulates the weir was removed from the model.
b) 1910–1949	• After the weir was built, but before significant excavation occurred at McCanns Island to provide aggregate for the building of Warragamba Dam. • The then DLWC was commissioned as part of the 1996 Flood Study to obtain cross sections downstream of the weir using photogrammetry from aerial photos taken in 1949 and these were then used in the model.
c)1960–August 1970	• After construction of Warragamba Dam, but before minor raising of the Penrith weir in September 1970. • Apart from an adjustment to the Penrith weir, the river was defined as for the September 1970 to August 1986 period.
d) September 1970	• After raising of the weir, but prior to the build-up of gravel in the weir pool in August 1986 (see Section 4.1.2.2). • Survey data for this period were very limited. • Cross sections and roughness values in the weir pool and downstream were essentially based on the 1990s survey, with adjustments made to take account of the limited topographic information available for the period and the fit to the available flood data. • The curve is plotted on Figure C27 together with the limited high flow gaugings available for the period.

7.5.2. Windsor-Sackville

The floodplain at Windsor is wide and complex. For this reason, measurement of flows in overbank floods is not practical and accurate simulation of such flows using the hydraulic model would be difficult, if not impossible.

The flood level at Windsor for a particular flow is largely determined by the hydraulic constriction caused by the gorge downstream of Wilberforce. This restriction cannot be isolated to a single point, but is rather a function of the entire downstream gorge. The DLWC (now OEH) has taken some flood gaugings in the gorge at Sackville, but there is no recorded relationship between height at Windsor and flow in the gorge.

The RUBICON model was therefore used to determine a relationship between height at Windsor and flow in the gorge for a series of historical and design floods. Sackville was chosen as the representative location of the gorge flow because it is downstream of a possible high flood breakout from the Windsor floodplain through Currency Creek and all flow is concentrated to a single flow path.

RUBICON was run for a series of floods of varying sizes and the height at Windsor was plotted against the flow at Sackville. A curve was then drawn through the peaks of the floods and smoothed to provide a consistent correlation of Windsor height and Sackville flow. The end product is reproduced on Figure C29 and Table B6 (Appendix B). It was assumed that this relationship had remained unchanged throughout the period of record. While there have been significant topographical changes on the Windsor floodplain, principally clearing of trees and sand mining downstream of Windsor Bridge, these changes do not control the flood level at Windsor except in very small events. The level is controlled by the restriction of the gorge and this has remained basically unchanged. The few pockets of clearing along the gorge have minimal impacts on flood behaviour.

8. DESIGN FLOOD BEHAVIOUR

8.1. Methodology

8.1.1. Overview

Real flood events exhibit an enormous degree of variability, most of which is determined by exactly when and where rainfall falls. Flood events are also influenced by how wet the catchment is and, in the case of the Hawkesbury-Nepean Valley, the levels in Warragamba Dam prior to an event. To better capture this variability, design flood estimation in Australia is moving from a single event per quantile (such as the 1 in 100 AEP) to Monte Carlo modelling where thousands of events need to be run. For the current study, the variability in key input variables was estimated from observed events and a Monte Carlo framework established (refer to Sections 8.1.3 to 8.1.11 and Diagram 1).

The adopted modelling framework is consistent with emerging best practice in flood estimation. There has been a strong move to Monte Carlo approaches to flood estimation over the last 10 years. The recently revised ARR (Ball et al., 2016) recommends the use of Monte Carlo approaches or ensemble modelling for most flood estimation problems. Ensemble modelling is a simplification of Monte Carlo modelling where a large number of events are run, but without using a Monte Carlo sampling strategy. These approaches recognise that temporal patterns have a large impact on flood levels and a single temporal pattern cannot represent the effects of this variability.

It has been common practice to use Monte Carlo approaches when assessing the impact of rare to extreme floods on dams. It is also not an unusual approach to sample losses and pre-event dam levels. The major advancement in this study was to extend the sampling to pre-burst rainfall and tributary timing. Sampling pre-burst rainfall is recommended in ARR 2016 (Ball et al., 2016) while the tributary timing is novel, but does largely address the problems with assuming a single uniform spatial rainfall pattern and allows the model to reproduce the observed tributary timing differences. While Monte Carlo frameworks are often used to improve the estimation of flood levels by capturing much of the variability of actual floods, the adopted approach has been demonstrated to also reproduce a range of other flood characteristics that are important for evaluating mitigation options and evacuation strategies.

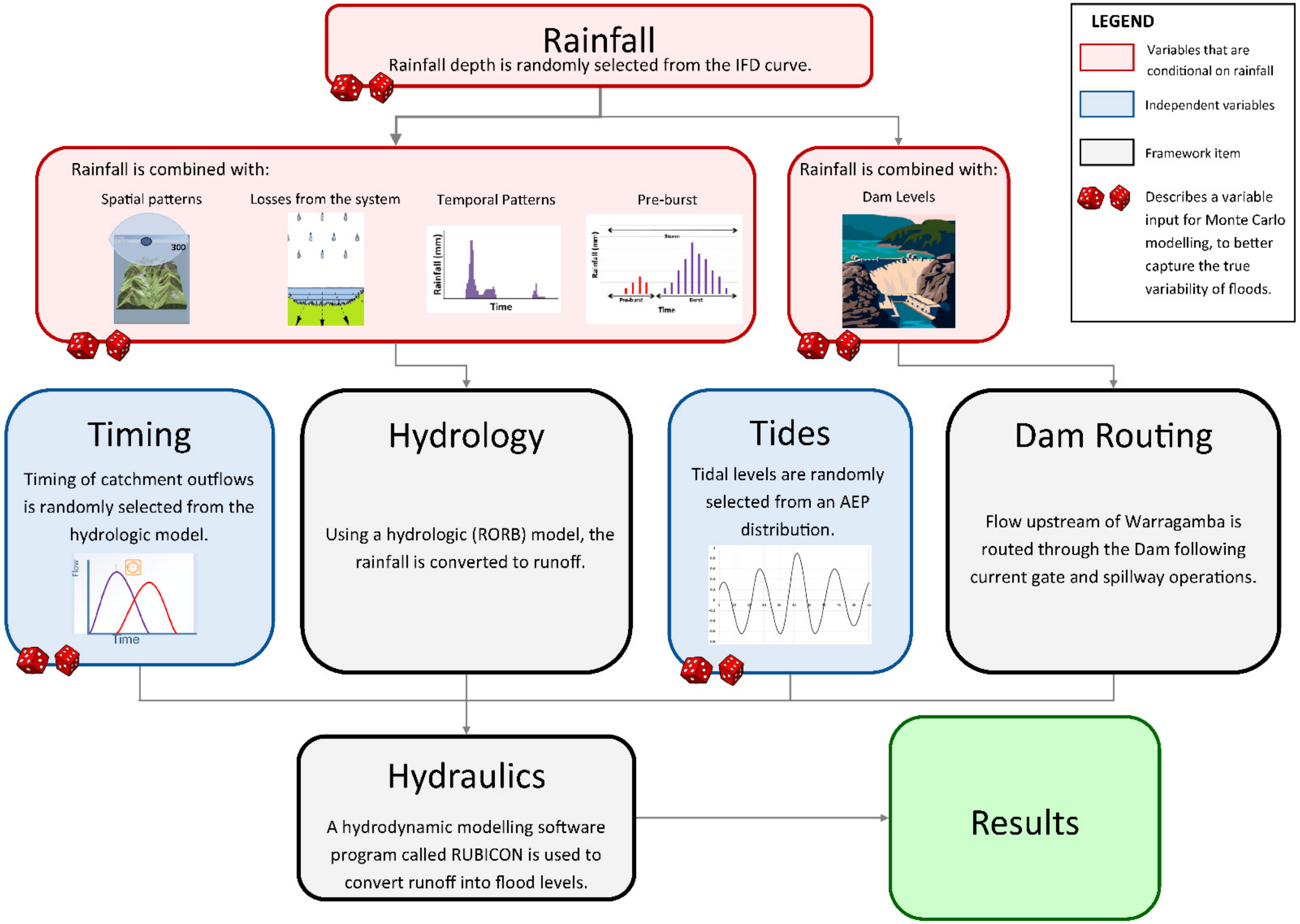

Diagram 1. Monte Carlo framework flowchart

8.1.2. Similar approaches

A similar approach was adopted in the *Brisbane River Catchment Flood Study* (BMT WBM et al., 2017) where a large sample of events was generated using a space-time model. The space-time model approach is conceptually more appealing and significantly more expensive and proved to be considerably more complex than separately sampling from spatial patterns, temporal patterns and tributary timing. Independent sampling produces a large number of events that do have some similarities but are reasonably different while the space-time generation on the Brisbane River produced only 700 events. This sample size was limiting and many of the events were relatively similar. The *Brisbane River Catchment Flood Study* space-time approach also caused problems in assigning sampling probabilities to individual events. Space-time generation was considered a more appropriate approach for the Brisbane River because a range of storm mechanisms can cause extreme flooding and events can occur either upstream or downstream of the dam. The Hawkesbury-Nepean system is much simpler as most events are generated by east coast lows which cover nearly the entire catchment.

The *Brisbane River Catchment Flood Study* considered the pre-event water levels in a much simpler way than the Hawkesbury-Nepean assessment documented in this report, but did consider the correlation between pre-event dam levels and antecedent conditions.

8.1.3. Inputs

The following sections describe how the inputs to the Monte Carlo analysis were determined. This study considered variability in the following key design flood inputs:

- rainfall intensity and frequency
- spatial pattern of rainfall
- temporal pattern of rainfall
- initial loss
- pre-burst rainfall
- dam drawdown
- relative timings of tributary inflows
- tides.

Continuing loss was used as a calibration parameter. A distribution of values must be developed for each variable, for sampling in the Monte Carlo model. These distributions are discussed below.

8.1.4. Rainfall

New design rainfalls (IFDs) were released by the Bureau of Meteorology (Green et al., 2015) as part of the update to *Australian Rainfall and Runoff* (ARR 2016). The major improvements over the 1987 IFDs include:

- A larger database of rainfall records was used in the analysis, incorporating data from agencies other than BoM.
- An increased length of rainfall records was used in the analysis.

- Modern computer data checking techniques were used to quality check and correct the data.
- Different distributions and fitting methods were investigated and used.
- Modern regionalization techniques were used.
- Modern covariate-based surface gridding techniques were used.

The latest IFDs developed for ARR 2016 were adopted for the current study and were available for frequencies up to the 1 in 2,000 AEP event. Extrapolation to the probable maximum precipitation (PMP) from the 1 in 2,000 AEP rainfall was undertaken using the Weinmann method described in ARR 2016 Book 8.

While use of many other organisations' data networks helped in filling spatial gaps in the BoM network, thereby improving the ARR 2016 in comparison to the previous ARR 1987 IFDs, there is still uncertainty about how much orographically enhanced rainfall occurs on some of the more rugged parts of the Hawkesbury-Nepean catchment.

Table 34 shows the ARR 2016 IFDs for a range of AEPs and 72-hour duration at Penrith, sampled by the Monte Carlo framework. Figure 10 shows the ratio of the rainfall that is applied to key sub areas within the model compared to the entire catchment rainfall. This is depicted as a box plot showing the range applied in all Monte Carlo events.

Table 34. Design rainfall depths sampled by the Monte Carlo framework (72 hours, Penrith)

AEP (1 in x)	Rainfall (mm)
1.582	92.0
2	105.9
5	150.9
10	182.9
20	215.3
50	253.8
100	282.7
200	312.6
500	351.5
1,000	381.1
2,000	410.9
5,000	459.0
10,000	506.7
20,000	566.7
50,000	670.3
87,719	754.2

8.1.5. Temporal patterns

The design temporal patterns were based on the BoM extreme storm database (Meighen & Kennedy, 1995/1997). The number of temporal patterns available in the BoM extreme storm database is dependent on the storm duration. Testing during and subsequent to the 1996 Flood

Study showed that a three-day duration is critical for downstream flood levels. For the three-day duration, 17 temporal patterns were available in the BoM extreme storm database. Temporal patterns were selected randomly from the available patterns. This means that each pattern is chosen multiple times. However, this approach is considered superior to the design event approach where only one temporal pattern is chosen. Temporal patterns are presented in Figure 11.

8.1.6. Areal reduction factors and spatial patterns of rainfall

Areal reduction factors convert point design rainfall estimates (IFD) into spatial estimates. As part of ARR 2016, long duration areal reduction factors (ARFs) were derived for use with the new IFDs using the same Australian dataset as the new IFDs. Short duration ARFs were also derived using gridded rainfall surfaces in areas with dense pluviometer networks. Both short and long duration ARFs use Australian rainfall data as input and are therefore more representative of local conditions than the ARFs recommended in ARR 1987 (Pilgrim, 1987), which were based on USA data. The ARR 2016 ARFs were used for the current modelling (refer to Figure 12).

A database of 125 observed spatial patterns of rainfall across the catchment was generated as part of the catchment average rainfall analysis. Figure 10 shows the ratio of the rainfall that was applied to key sub areas within the model compared to the entire catchment rainfall for each of the 125 patterns as a box plot.

For each event a spatial pattern was selected from the closest 20 ranked patterns by catchment average depths. The adopted ranked approach minimises the scaling of frequent event patterns.

Due to the nature of adopting Monte Carlo spatial patterns there is a slight bias in design rainfalls across the catchment. While the rainfall at Penrith is always equal to the design rainfalls, the design rainfalls at other locations vary. For example, there is less than a 1 per cent bias in rainfall depth at Windsor and there is a slight bias towards the Warragamba catchment as opposed to the Upper Nepean catchment.

8.1.7. Losses

8.1.7.1. Initial loss

Initial loss is the amount of rainfall which is considered to infiltrate the soil or be lost in depression storage. Based on Sydney Water Board et al. (1990), a median initial loss of 30 mm was considered representative for this catchment. A standardised loss curve was developed (Figure 13A) which ranged from 4.5 to 98 mm based on the methodology described in Hill et al. (2014).

No correlation was enforced between the loss value and the dam level. This means that if the dam level was low (possibly during a drought) then it is likely that the soil would have low moisture content and that the losses to the soil would be high. Conversely when the dam level is high it is likely to be a wet period and the soil may be saturated meaning there would be low losses. However, the model is not constrained by this and therefore it is possible to have a high initial loss when the dam is full.

8.1.7.2. Continuing loss

Continuing loss is the ongoing infiltration loss. Continuing loss was used as a calibration factor to the pre-dam flood frequency curve. The calibrated losses were then applied to all other scenarios. Continuing loss values were varied with AEP (as per Table 35) with a linear interpolation between defined values. Initial loss was sampled from the distribution and the sampled value was used across the entire catchment, while continuing loss values were spatially varied as per Table 35.

Table 35. Continuing loss value

Rainfall AEP (1 in x)	Continuing loss (mm/hr)	
	Upstream of Warragamba	Upper Nepean and downstream of Warragamba
10,000,000	0.10	0.10
110,000	0.26	0.26
2,000	0.61	0.61
1,000	0.71	0.71
500	0.82	0.82
200	1.00	1.00
100	1.20	1.50
50	1.40	1.50
20	2.00	1.50
10	2.50	1.00
5	2.70	1.00
2	2.70	1.00

8.1.8. Pre-burst rainfall

The traditional design event approach uses a peak rainfall burst with no accounting for rainfall that occurs prior to the most intense burst of the storm, rather than considering a complete storm event. For this study a burst approach was used with pre-burst rainfall added to the start of the event.

The distribution of possible pre-burst rainfall was determined by calculating the ratio of the pre-burst rainfall to each three-day rainfall burst using a spatial catchment rainfall analysis undertaken of the historical rainfall record.

If the selected pre-burst was greater than the selected initial loss value, then the initial loss was set to zero. This occurred for approximately 12 per cent of cases.

The pre-burst to burst ratio was calculated for the three-day burst (Figure 13B). Testing showed that only a small portion of additional rainfall occurred between three to seven days prior to the event.

8.1.9. Dam drawdown

The level of Warragamba Dam before an event was sampled from a series of probability drawdown graphs that relate the largest monthly inflows to the water level in the preceding month (discussed in Section 4.7). For rare events (approaching the 1% AEP), dam drawdown can be neglected and the dam assumed to be at full supply level.

8.1.10. Timing of tributaries

The coincident timing of tributary inflows can exacerbate flooding. This is of particular importance when designing a dam operation strategy to ensure that the timing of dam outflows and rain falling downstream of the dam do not coincide.

The timing of tributary inflows was calculated for the following catchments compared to the Warragamba River timing:

- Nepean River
- Grose River
- Colo River.

This could be extended to include the timing of the catchments upstream of Warragamba Dam (Wollondilly, Coxs/Kowmung system and the direct catchment area of the dam). This was not undertaken as part of this study.

The timing of the tributary flows is important for evacuation planning, particularly in the Richmond/Windsor area where interactions of local flows can significantly affect rate of rise and reduce evacuation times.

Catchment average rainfalls for three-day storm events were calculated for the catchments listed above. A total of 125 observed events were used. The time at which 50 per cent of the rainfall mass occurs was calculated. For each catchment the difference between the time at which 50 per cent of the rainfall mass occurred and the time for it to occur on the Warragamba catchment was calculated. Negative values occur when 50 per cent of the rainfall mass on the other catchment occurs before it does on the Warragamba catchment. The distributions are shown on Figure 14. In the majority of cases there is no or very little timing difference. This curve is randomly sampled by the Monte Carlo model. The timing differences are applied to the inflow hydrographs from the hydrologic model before the hydraulic model is run.

8.1.11. Tides

As part of the current study, flood levels as a result of tidal inundation and the impacts of sea level rise were required. Historically the RUBICON model had adopted a constant mean tide level of 0 m AHD.

Australian Rainfall and Runoff's *Project 18: Coincidence of Fluvial Flooding Events and Coastal Water Levels in Estuarine Areas* (Zheng et al., 2014) identified a weak correlation between rainfall and elevated ocean levels. Given the response time of the catchment and the fact that the flood peaks will reach the lower reaches of the Hawkesbury-Nepean after several days have passed, it is considered that elevated ocean levels are not likely to coincide with the flood peak.

The *Fort Denison Sea Level Rise Vulnerability Study* (Watson & Lord, 2008) provides guidance on water levels in Sydney Harbour. Table 36 and Table 37 present data from that study to provide guidance on the likelihood of extreme tidal levels. The largest recorded tide in Sydney Harbour is 1.475 m AHD.

Table 36. Record water level events at Fort Denison

Date	Level (m AHD)
25 May 1974	1.475
27 April 1990	1.425
10 June 1956	1.395
30 June 1984	1.345

Note: Adapted from Watson & Lord, 2008

Table 37. Sydney Harbour design still water levels

AEP (1 in x)	AEP (%)	Level (m AHD)
5.52	18.13 (5 year ARI)	1.315
10	10	1.345
20	5	1.375
50	2	1.415
100	1	1.435
200	0.5	1.455

Note: Adapted from Watson & Lord 2008

A generalised extreme value distribution was fitted to the values in Table 37 to derive tide levels at AEPs from 1 in 5 to 1 in 100,000 AEP as shown in Table 38. Levels from this probability distribution were randomly sampled from the uniform distribution and taken as the peak of the tidal range for the event. When a value outside of the AEP range was sampled the level for the closest AEP was used. Ocean levels below 0 m AHD were set to 0 to avoid model instability. To ensure no bias was introduced due to the timing of tidal peaks, the starting period from the tidal range was randomly selected for each event.

The inclusion of a variable tide results in a 4mm increase in flood levels in a 1 in 100 AEP event at Windsor. Tidal levels have most impact downstream of Spencer. The inclusion of variable tides has a more significant effect on peak flood levels for the 1 in 5 and 1 in 10 AEP events, increasing flood levels by 18 mm and 10 mm respectively at Windsor bridge. The effect for more frequent AEPs is more pronounced because the additional volume of water from the tide is larger relative to the volume of the frequent flood events than it is for rarer events. This increase in level is conservative as it assumes the annual maximum tide always coincides with the annual maximum rainfall event on the Hawkesbury.

Table 38. Tidal peak distribution used

AEP (1 in x)	Peak tide level (m AHD)
5	1.228
5.52	1.315
10	1.345
20	1.375
50	1.415
100	1.435
200	1.455
500	1.473
1,000	1.484
2,000	1.494
5,000	1.504
10,000	1.51
20,000	1.515
50,000	1.521
100,000	1.525

8.2. Sampling strategy

A sampling strategy was selected which properly explores flood events where key evacuation timing becomes crucial and major floodplain damages occur.

For these reasons a strategy was adopted that focuses on the critical 1 in 20 to 1 in 500 AEP range. While smaller floods cause significant community disruption, they do not pose a significant threat to life or property.

Normal practice is to adopt either importance or stratified sampling where more emphasis is placed on events at the rarer scale. This type of approach is much more efficient than crude sampling and results in each regular quantile (1 in 100 AEP, 1 in 200 AEP) having a relatively similar sampling density. For this study a sampling approach focusing on events in the key range for flood damages and risk to life was adopted.

Two sets of 10,000 events were run, one which contained 10,000 randomly selected events between a no flood event and the PMF (which is equal to a 10,000-year historical sample) and one which contains 10,000 events greater than a 1 in 20 AEP (which represents a 200,000-year historical sample of events of rarer than the 1 in 20 AEP rainfall). The two sets were merged on the basis of the underlying rainfall probabilities where a sample with the equivalent length of 200,000 years was produced by combining 10,000 events above the 1 in 20 AEP rainfall and 9,500 events from the crude sampling case below 1 in 20 AEP rainfall. This process essentially assumes that over a 200,000-year period each of the events above 1 in 20 AEP rainfall is unique and each of the more frequent events occurs 20 times. This results in the majority of events being between the 1 in 20 and 1 in 2,000 AEP which is critical for managing flood risk in the Hawkesbury-Nepean Valley.

8.3. Modifications to the hydrologic and hydraulic models for Monte Carlo modelling

The hydrologic RORB model (refer to Section 6) was modified so that it could run in a Monte Carlo environment. The randomly selected rainfall, spatial patterns, temporal patterns, pre-burst and losses were applied to the hydrologic model to determine flows for the design events.

The hydraulic RUBICON model (refer to Section 7) was recompiled in a 64-bit Windows environment with modifications to allow it to run in a Monte Carlo framework with concurrent simulations.

8.3.1. Baseflow

8.3.1.1. Approach

Examination of the flood event hydrographs at the calibration sites shows that baseflow generally does not make a significant contribution to flows during flood events within the Hawkesbury-Nepean catchment. However, it was found that adding an allowance for baseflow improved the fit of the model to the event recessions at four sites: Coxs River at Kelpie Point, Wollondilly River at Jooriland, Kowmung River at Cedar Ford, and Nepean River at Maldon Weir.

The selection of approach for quantifying baseflow followed the procedure outlined in *Australian Rainfall and Runoff – A guide to flood estimation* (Hill et al., 2016). A preliminary assessment using Figure 5.4.3 of ARR 2016 indicates a baseflow peak factor of 0 to 0.3 over the Hawkesbury-Nepean catchment. Following the decision tree shown in Figure 5.4.2 of ARR 2016, a direct analysis procedure was used to estimate baseflow from the recorded streamflow data.

The method of Chapman and Maxwell (1996) was used. This requires an estimate of the filter parameter given by the recession constant of the hydrograph. Baseflow is estimated using Equation 1.

$$q_b = \frac{k}{2-k} q_b(i-1) + \frac{1-k}{2-k} q_i \qquad \text{Equation 1}$$

where: $q_b(i)$ = filtered baseflow response for the i^{th} sampling instant
q_i = original streamflow for the i^{th} sampling instant
k = filter parameter

The filter parameter was calibrated to give the best fit to the hydrograph recession over a range of events at each site.

8.3.1.2. Results

The filter parameter that provided the best fit at each site is shown in Table 39.

Table 39. Baseflow filter parameter

Site	k
Coxs River at Kelpie Point	0.9993
Wollondilly River at Jooriland	0.9993
Kowmung River at Cedar Ford	0.9985
Nepean River at Maldon Weir	0.9993

The events used to calibrate the k parameter covered a range of AEPs from 1 in 5 AEP to 1 in 100 AEP. Therefore, it was considered that the method is appropriate for use for events up to 1 in 100 AEP. In the Monte Carlo model, the baseflow was added to the dam inflows and Nepean River inflow to the RUBICON model.

8.4. Validation of methodology

8.4.1. Flood frequency analysis

To verify the Monte Carlo framework, a comparison to flood frequency analysis and a comparison to the long-term flood records was undertaken. All flows were converted to pre-dam flows (flows prior to the construction of Warragamba Dam) in order to undertake the flood frequency analysis on a comparable basis (discussed in detail in Section 5).

Diagram 2 compares the adopted flood frequency analysis (Section 5), pre-dam peak discharges and the results from the Monte Carlo method at Windsor. A very good match is achieved between the 1 in 10 AEP and 1 in 100 AEP events. The results vary significantly at the rare end where the Monte Carlo method has less data points. However, the Monte Carlo analysis is largely within the confidence limits of the flood frequency analysis. Similar plots are provided for Penrith and Warragamba in Diagram 3 and Diagram 4 respectively.

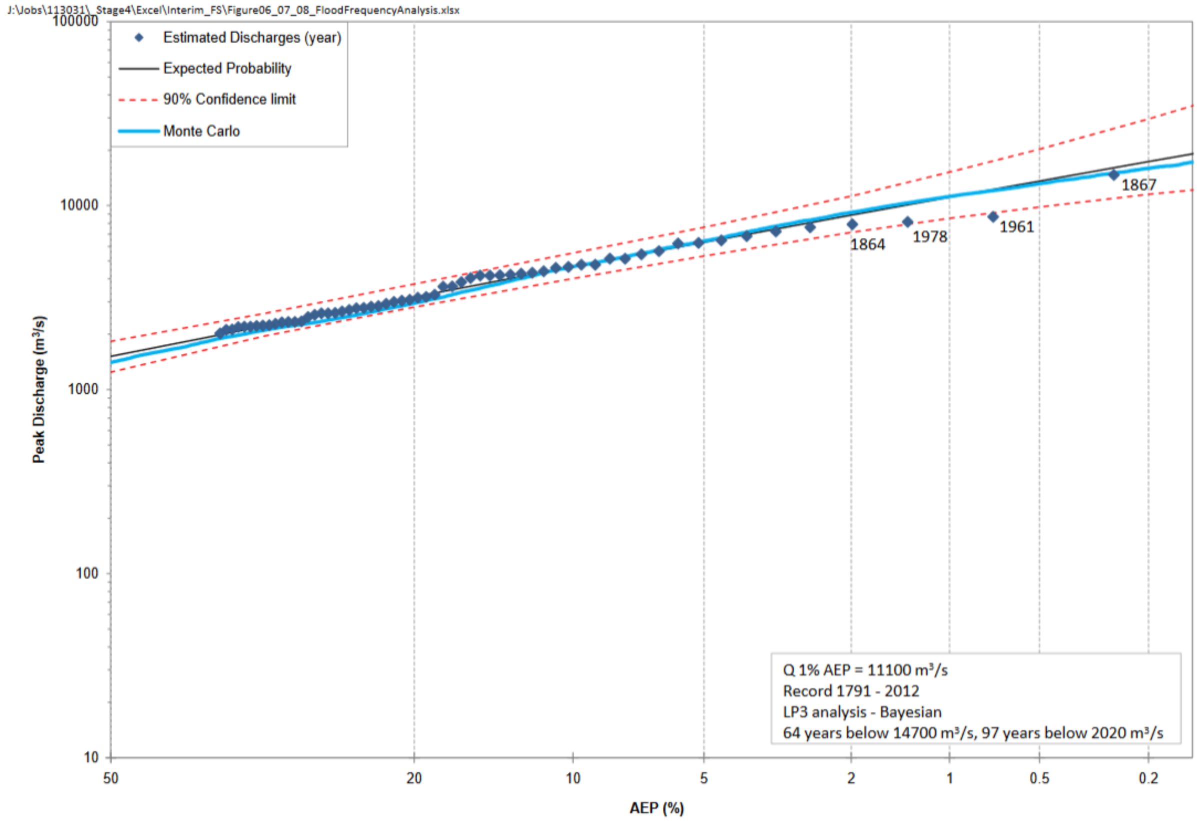

Diagram 2. Pre-dam flood frequency analysis compared to Monte Carlo results – Windsor

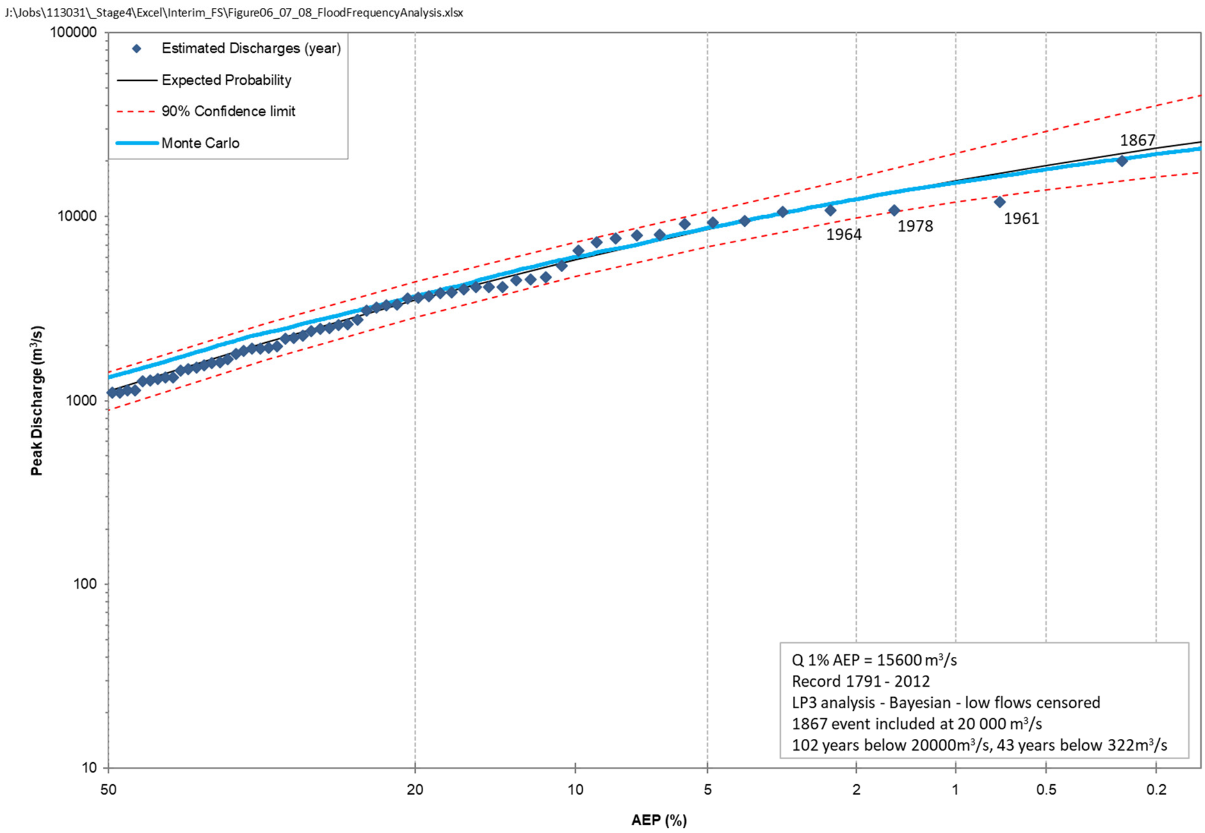

Diagram 3. Pre-dam flood frequency analysis compared to Monte Carlo results – Penrith

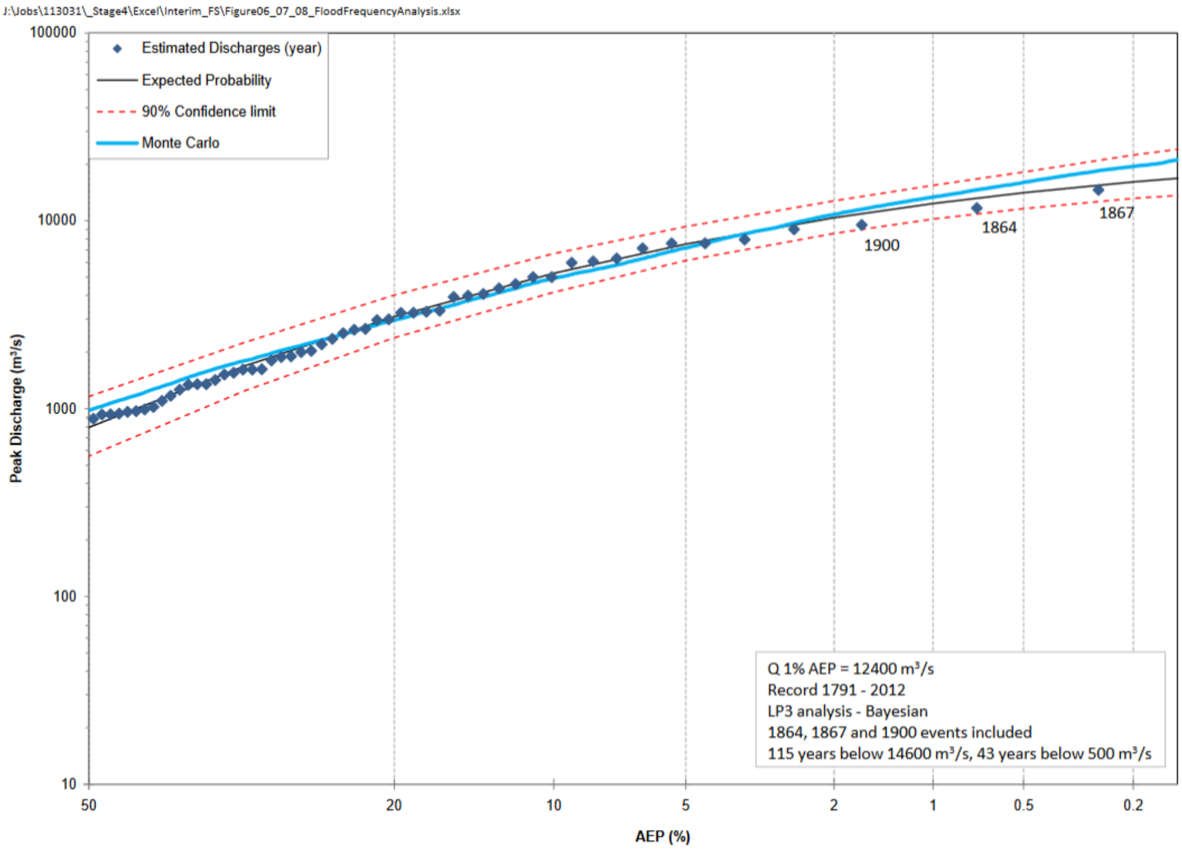

Diagram 4. Pre-dam flood frequency analysis compared to Monte Carlo results – Warragamba

Table 40 provides a comparison of the quantile estimates for the at-site FFA and the Monte Carlo pre-dam run. A good match is achieved for most quantiles. The Monte Carlo model is high compared to the pre-dam FFA at Warragamba and Penrith for the 1 in 2 AEP quantile. However, a good match is achieved for the 1 in 2 AEP quantile at Windsor.

Table 40: Comparison of pre-Dam FFA and Monte Carlo quantile results

AEP (1 in x)	Peak Discharge (m³/s)								
	Warragamba			Penrith			Windsor		
	Monte Carlo	At-Site FFA	Difference (%)	Monte Carlo	At-Site FFA	Difference (%)	Monte Carlo	At-Site FFA	Difference (%)
2	980	790	24%	1340	1120	20%	1410	1440	-2%
5	2920	3090	-6%	3690	3510	5%	2940	3250	-10%
10	4940	5260	-6%	6020	5830	3%	4660	4660	0%
50	10800	10380	4%	12390	12440	0%	9190	8500	8%
100	13380	12350	8%	15240	15630	-2%	11210	10490	7%
200	16020	14110	14%	18070	18960	-5%	13150	12720	3%
500	19450	16110	21%	21850	23480	-7%	15940	16110	-1%

8.4.2. Comparison of secondary flood characteristics

To confirm the Monte Carlo framework was accurately replicating observed flood behaviour, a number of flood characteristics of the modelled events were compared to the observed events. Flood characteristics for the traditional design event method are also plotted. Flow above a threshold, volume above a certain level, rate of rise between key heights, and time of inundation above key evacuation bridge levels were compared for modelled and observed events. Detailed time series hydrographs consistent with the existing dam were only available for events post 1960 and even some of them are incomplete. Six of the top 10 events in the continuous period of record (1893–present) occur in the last 57 years. While 1867 is the highest ranking event, 1961 is the highest in the continuous record. This skews the sample for a volume frequency curve and hence this has not been presented.

The heights which the river is above for 12, 24, 48, 72 and 96-hour periods were extracted from the modelled and observed events at Penrith. Diagram 5 shows the 24-hour results. The historical events are reasonably distributed within the range of modelled events.

Using a series of historical rating curves for conditions at Penrith, the volume of flow the river is above a certain height, for a given period, was calculated. Diagram 6 presents the volume of flow in the river for a 48-hour period at Penrith. Given the limited comparison data a reasonable representation of observed data is produced by the modelled events.

The rate of rise between 4 and 10 metres at Windsor was extracted (Diagram 7). The limited time series data available for some historic events (either through gauge fault or only three-hour data being available) mean that some events plot at the edges of the modelled event range. However, a good representation of observed rate of rise is achieved by the Monte Carlo modelling.

8.5. Probable maximum flood estimation

8.5.1. Probable maximum precipitation

The probable maximum precipitation (PMP) is the 'greatest depth of precipitation for a given duration meteorologically possible over a given size storm area at a particular location at a particular time of the year' (DECCW, 2009).

Early work on the development of the Generalised Southeast Australian Method (GSAM) Probable Maximum Precipitation method was carried out on the Warragamba catchment and used in the 1996 Flood Study. The initial GSAM work was location specific with PMP estimates to Warragamba Dam and the catchment to Wisemans Ferry. After the 1996 Flood Study was completed, the GSAM method was updated so that PMP estimates could be obtained anywhere in the South Eastern Australia zone.

For the current study the PMP estimates were obtained from the updated GSAM method (2008) from WaterNSW (SCA, 2008). Table 41 lists the average depth of precipitation over the total catchment area to Warragamba Dam.

Table 41. Probable maximum precipitation depths (mm) - Average to Warragamba Dam

Duration (hours)	Warragamba Dam
24	500
48	660
72	770
96	860
120	910

8.5.2. Calculating a probable maximum flood in the Hawkesbury-Nepean

Estimating probable maximum flood (PMF) levels is a complex and uncertain task as there is limited physical evidence or data on such extreme floods. Therefore, nearly all design steps rely on either significant extrapolation from observed events or physical reasoning. The two main uses for the PMF are also very different:

- for dam design, the PMF is important for defining the upper design case for spillways and dam failure;
- for floodplain management, the PMF represents a reasonable upper bound to flooding. This upper bound is used to define the point beyond which the likelihood of flooding is negligible.

To ensure consistency in PMF estimation, the steps for derivation have been standardised in Book 8 of ARR 2016 and the Australian National Committee on Large Dams' Acceptable Flood Capacity Guidelines (ANCOLD, 2000).

The PMF, for the purposes of this study and for floodplain management, is estimated using the PMP, a single temporal pattern, low catchment losses (initial loss 0 mm and continuing loss 1 mm/hr), and dam storages within the catchment assumed to be full. This simplified approach is used in floodplain management as the majority of effort in floodplain management is focussed on events that have a realistic probability of occurring and that are practical to manage, while a PMF is generally used to quantify the worst-case outcome and to inform emergency planning.

Conceptually, the PMF cannot be assigned a probability, as the probability of the combination of the inputs (antecedent conditions, temporal patterns, spatial patterns) that result in the upper limiting value is not known. However, for the purposes of floodplain management, the probability of the PMP may be assigned to the PMF. PMP/PMF calculations are strictly to a point or location based on upstream or influencing catchment area. In addition, it is common practice on large catchments to use a single reference probability to avoid the slight changes in theoretical PMP probability as you move down the catchment. This is a very reasonable assumption given the wide uncertainty bounds around the nominal probability of the PMP.

For the *Brisbane River Catchment Flood Study* (BMT WBM et al., 2017), which has a similar catchment size to the Hawkesbury-Nepean catchment to Windsor, a 1 in 100,000 AEP was adopted as a reference probability. A probability of 1 in 100,000 AEP is also a good reference probability for the Hawkesbury-Nepean Valley (refer to Table 42). All the critical cases are very close to this nominal probability, particularly when the recommended uncertainly bounds from ARR are considered.

For Wallacia, three possible PMF cases need to be considered so that flood levels are not underestimated, as Wallacia is influenced by a number of flood mechanisms: the upstream catchment on the Nepean River to Wallacia PMF, the backwater PMF from the Warragamba catchment, and the combined case. Modelling has shown however that for floods rarer than 1 in 50 AEP the dominant flood mechanism is backwater from the Warragamba River. More discussion on flooding mechanisms at Wallacia can be found in Section 9.4.1.1.

For the Hawkesbury-Nepean catchment, the following PMP rainfall probabilities apply.

Table 42. PMP probability to specific locations

Location	Effective catchment area (km^2)	Probability (AEP)
Warragamba	9,000	1 in 110,000
Wallacia (upstream- Nepean River)	1,760	1 in 570,000
Wallacia (based on backwater from Warragamba)	9,000	1 in 110,000
Wallacia (based on total catchment to Warragamba confluence)	10,790	1 in 93,000
Penrith	11,250	1 in 89,000
Windsor	12,880	1 in 78,000
Sackville	13,240	1 in 76,000

For this study, a PMF to Penrith was adopted as this provides the worst case at Penrith and is within 150mm of the flood levels resulting from the PMF to Windsor in the lower valley. It also represents the worst case at Wallacia. The levels are also consistent with 1 in 100,000 AEP (PMF) estimates from the Monte Carlo framework.

8.6. Other documentation of the adopted approach

Monte Carlo approaches in flood estimation have typically focused on capturing the variability in input conditions and how this variability affects peak flood levels and flow. The approach adopted here was the first time that a Monte Carlo approach has been used for assessing warning time and evacuation strategies and one of the first times it has been adopted for a flood study in Australia.

Aspects of this and other studies of the Hawkesbury-Nepean Valley undertaken using the adopted approach were presented in several peer reviewed conference papers and a journal article. The following list of peer reviewed papers and journal articles were prepared covering the approach adopted in these studies:

- Australian and New Zealand Disaster and Emergency Management Conference, Broadbeach, Gold Coast (QLD), 3–5 May 2015, *Use of a Monte Carlo framework for Emergency Management* – focus on emergency management

- Floodplain Management Association National Conference, May 2015, *A New Way of Examining Emergency Response Time and the Benefits Gained from Management Measures* – focus on overall approach and emergency management
- Hydrology and Water Resources Symposium, Dec 2015, *Monte Carlo Modelling in Decision Making* – focus on verification
- Australian Journal of Water Resources, 2016, *A Monte Carlo Framework for assessment of how mitigation options affect flood hydrograph characteristics* – focus on verification.

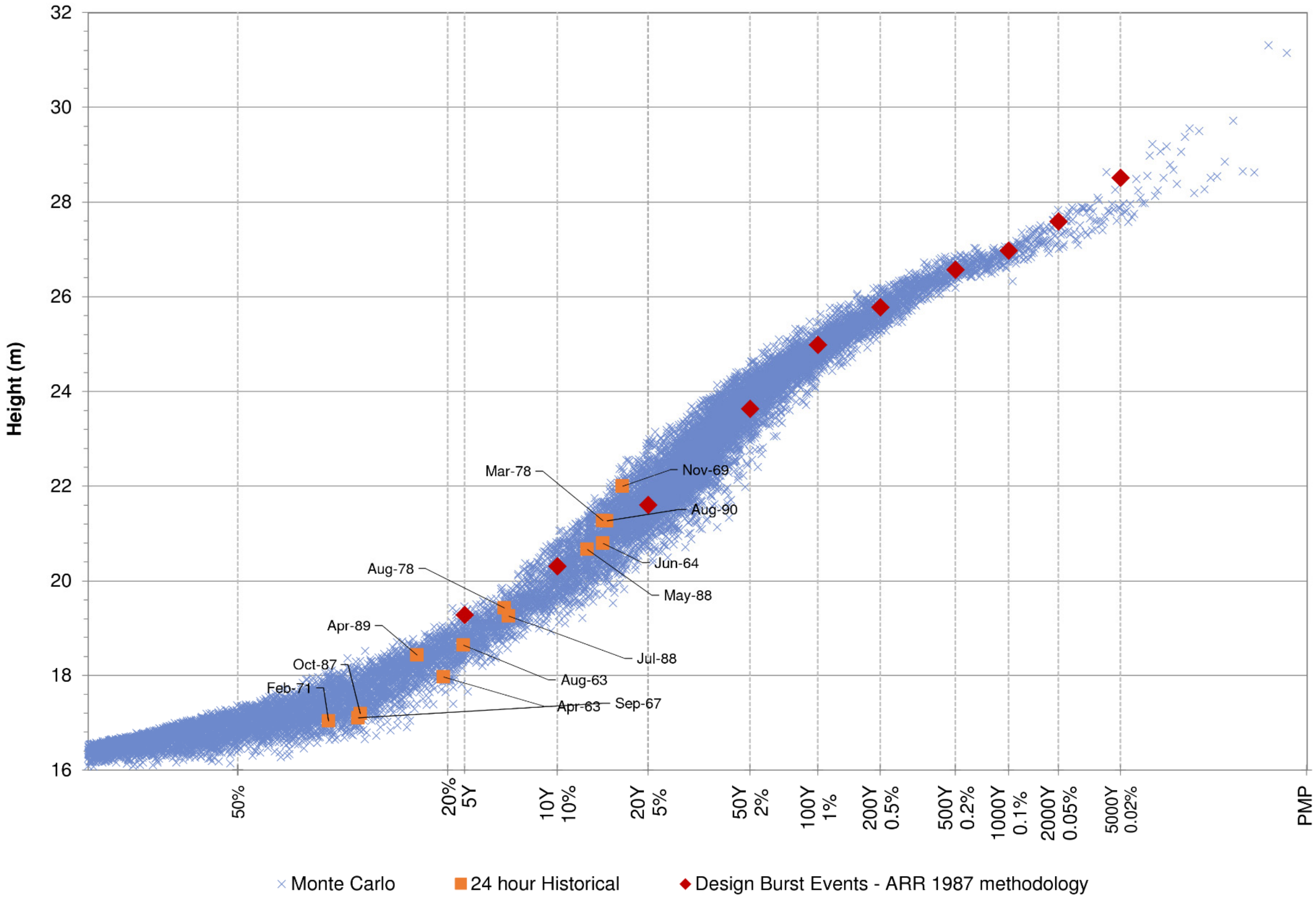

Diagram 5. Height the river is above for 24 hours versus frequency at Penrith

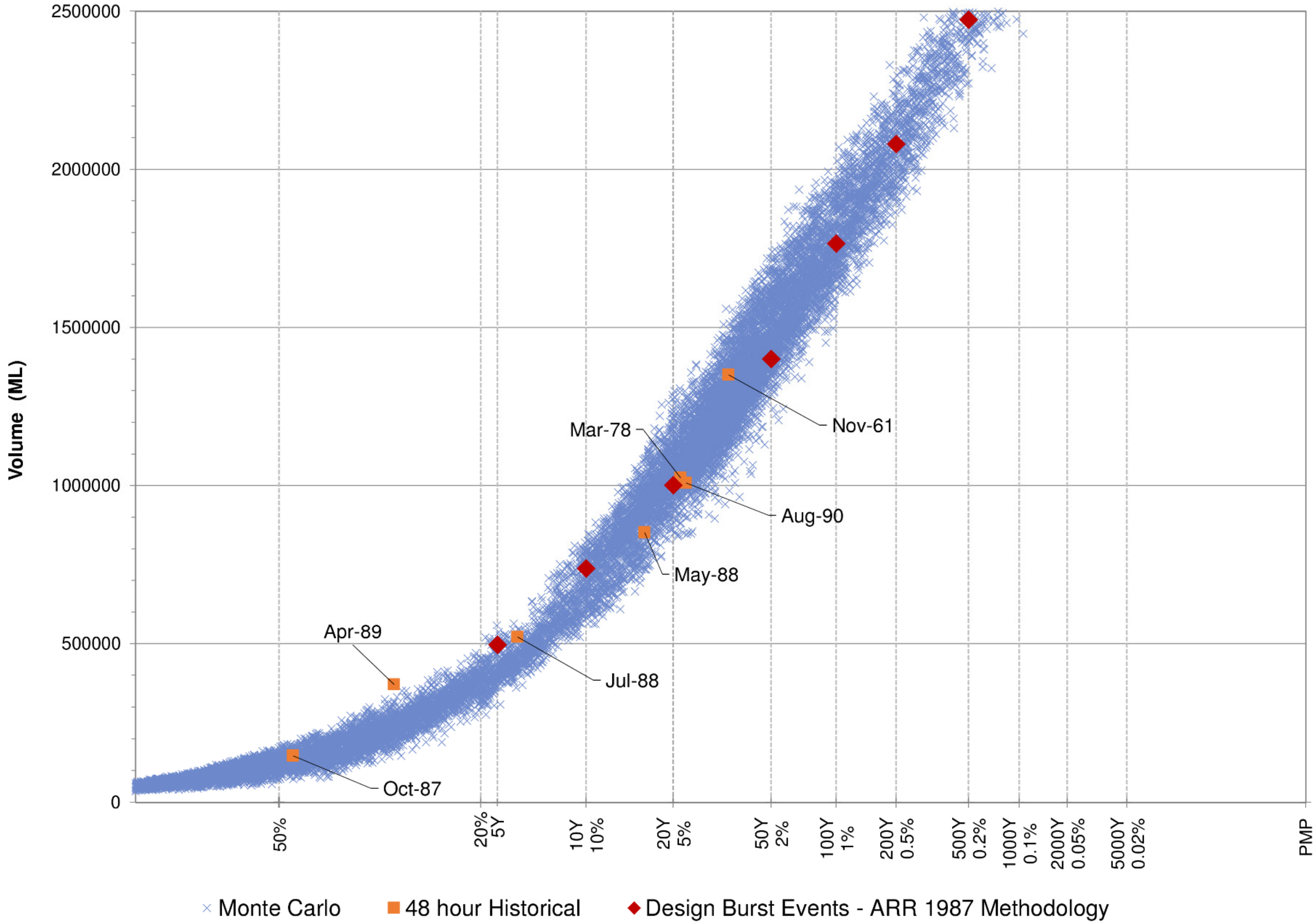

Diagram 6. 48-hour volume of flow in the river versus frequency at Penrith

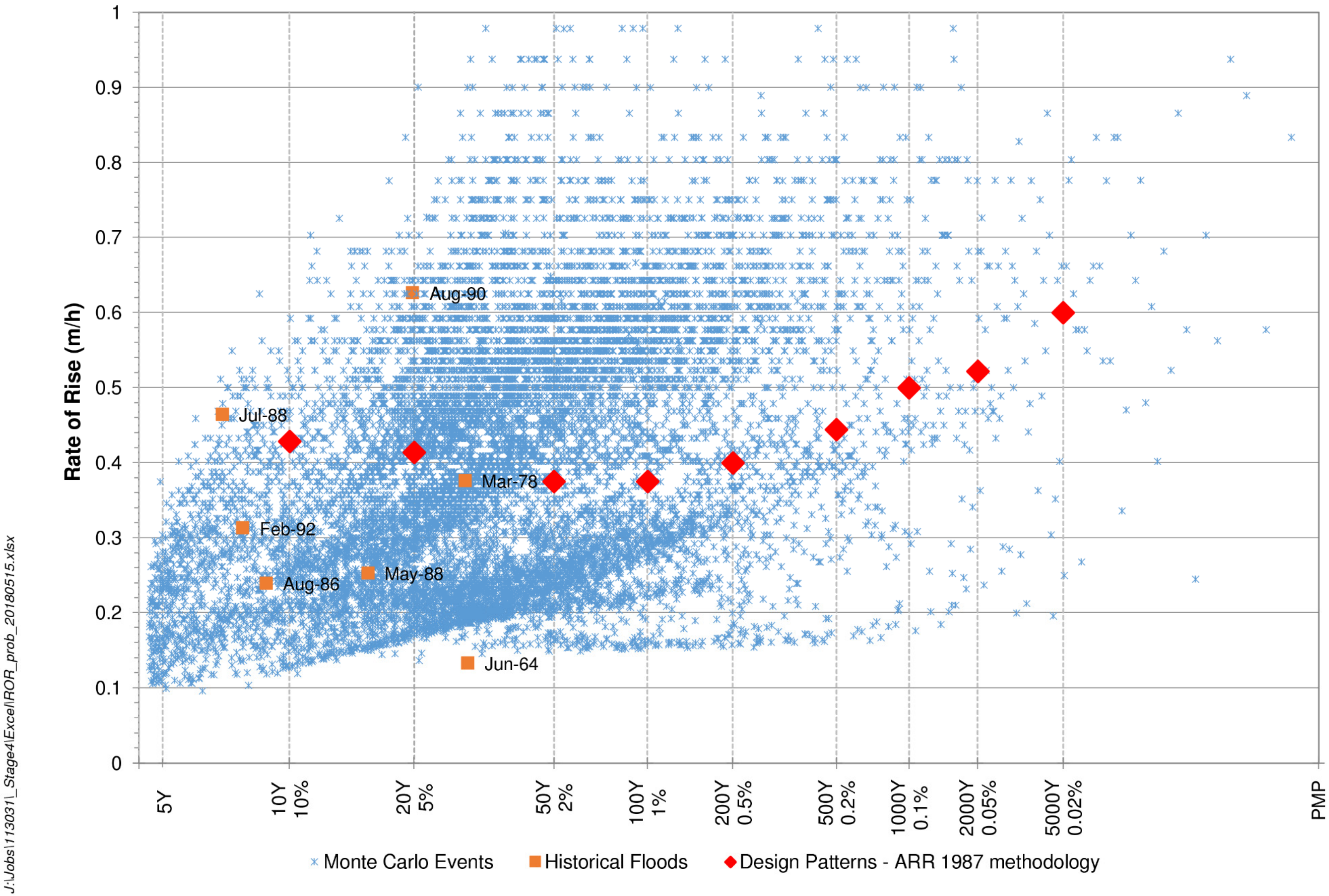

Diagram 7. Rate of rise between four and 10 metres versus frequency at Windsor

9. RESULTS

This section presents the results of the study and some of the key outputs of the study. Commentary is provided on the differences between previous studies and the current study.

9.1. Key reporting locations

A number of key reporting locations were identified for this study based on their significance as key gauging locations, evacuation routes, roads and bridges, etc. As well as reporting locations on the main river branch, locations were also chosen on South Creek and Rickabys Creek. These locations are listed in Table 43. For each of these key reporting locations a number of graphs, figures and model results were extracted and produced.

Table 43. Key reporting locations

No.	Name	River / Creek	Description
1	Brooklyn Bridge (M1)	Hawkesbury River	Pacific Motorway (M1) crossing of Hawkesbury River
2	Spencer	Hawkesbury River	Approx. 2.5 kilometres upstream of Mangrove Creek confluence
3	Gunderman - Singletons Mill	Hawkesbury River	Between villages of Gunderman and Singletons Mill
4	Wisemans Ferry	Hawkesbury River	Wisemans Ferry crossing (Old Northern Road) - approx. 300 metres downstream Macdonald River confluence, approx. 1,300 metres downstream of Webb's Creek Ferry (St Albans Road), approx. 1,800 metres downstream of Webb's Creek confluence
5	Leets Vale	Hawkesbury River	Approx. 2.5 kilometres upstream of Leets Vale Caravan Park
6	Lower Portland	Hawkesbury River	Lower Portland Ferry crossing (West Portland Road) at Colo River confluence
7	Sackville	Hawkesbury River	Sackville Ferry crossing (Sackville Road) - approx. 450 metres downstream of Currency Creek confluence
8	Ebenezer	Hawkesbury River	At Riverside Oaks Golf Resort
9	Cattai Creek/Gronos Point	Hawkesbury River	Just upstream of Cattai Creek confluence (opposite Gronos Point)
10	South Creek at Richmond Road	South Creek	South Creek at Richmond Road
11	Windsor	Hawkesbury River	Hawkesbury River Bridge crossing at Windsor
12	Rickabys Creek at Blacktown Road	Rickabys Creek	Rickabys Creek at Blacktown Road
13	North Richmond	Hawkesbury River	Hawkesbury River Bridge crossing at North Richmond
14	Yarramundi Bridge	Nepean River	Between Hawkesbury/Springwood Road Bridge and Grose River confluence
15	Penrith	Nepean River	Victoria Bridge - approx. 600 metres upstream of Penrith Weir
16	Blaxlands Crossing	Nepean River	Blaxlands Crossing Bridge in Wallacia at Silverdale Road
17	Bents Basin	Nepean River	Bents Basin Campground downstream of gorge

9.2. Assigning Annual Exceedance Probabilities

For each variable of interest, each of the Monte Carlo events were combined, ranked and AEPs assigned to each event. The design event quantiles (for example, 1 in 100 AEP) for the variable of interest (flood level, rate of rise, inundation time etc) were extracted. Therefore, the event that results in a 1 in 100 AEP flood level at a particular location is not necessarily the event that results in a 1 in 100 AEP flow at the location.

The design event flood surfaces are an envelope of the 1 in 100 AEP levels at each calculation point in the model. They therefore do not represent a single 1 in 100 AEP event in the traditional sense.

9.3. Overview of outputs

The following outputs were extracted from the results:

- peak flood levels as surfaces and at key locations
- flood level profiles
- flood depths and extents
- provisional flood hazard (Section 12 and Appendix D)
- hydraulic categories (Section 12 and Appendix D)
- stage frequency curves (Appendix E)
- rate of rise versus flood probability between a number of key levels (Appendix F)
- time to rise versus flood probability between a number of key levels (Appendix G)
- rate of fall versus flood probability between a number of key levels (Appendix H)
- time to fall versus flood probability between a number of key levels (Appendix J)
- time above a critical level versus flood probability between a number of key levels (Appendix K)
- flood peak travel times (Appendix L)
- representative subset of events for use in evacuation modelling (Appendix M)
- representative subset of events for use in detailed flood modelling (Section 9.6 and Appendix N)
- time series of flood level surfaces.

Flood maps are presented as an overall study area map followed by zoomed in maps of the following areas:

- Richmond-Windsor
- Penrith
- Wallacia.

The method used to map the results to provide spatial results is contained in Appendix P. Detailed flood mapping prepared by Infrastructure NSW is presented in Volume 3: Map Book.

9.4. Flood levels, depths and extents

9.4.1. Peak flood levels and extents

A unique feature of the Hawkesbury-Nepean Valley is the range of flood levels it experiences. For example, Windsor, which is tidal, and some 125 kilometres from the ocean, has a 1 in 100 AEP level of 17.3 m AHD. The probable maximum flood (PMF) level, which in Australia is typically only up to two to three metres higher than the 1 in 100 AEP event, is over nine metres higher at 26.7 m AHD at Windsor. A much smaller range of flooding occurs in the lower reaches of the estuary. Table 46 presents the peak flood levels for various design event quantiles at the key reporting locations described in Table 43.

Figure 15 to Figure 18 present the flood extents for a range of events (1 in 5 AEP, 1 in 100 AEP and PMF). Generally, while the flood level changes dramatically the overall flood extent often does not, particularly in the gorge areas. However, some infilling of low flood islands occurs, and flood extents at Penrith change dramatically when flow is out of bank in events of 1 in 100 AEP or rarer.

9.4.1.1. Wallacia

Flood levels at Wallacia vary largely due to the constrictive effects of the Fairlight Gorge between Warragamba Dam and Penrith. The 1 in 10 AEP event has a level of 37.21 m AHD at Blaxlands Crossing, while the 1 in 100 AEP event has a level of 44.65 m AHD. In flood events up to a 1 in 10 AEP event, the flood extent remains restricted to the low-lying overbank floodplain areas of the Nepean River. In a 1 in 100 AEP event, significant areas of the suburb are inundated by floodwaters. The PMF event reaches a level of 66.34 m AHD, some 21.7 metres above the 1 in 100 AEP event – the largest increase in flood level between the 1 in 100 AEP and PMF events across the Hawkesbury-Nepean catchment. Wallacia is completely inundated in a PMF event.

9.4.1.2. Penrith

At Penrith, the 1 in 10 AEP event flood level is 21.34 m AHD and the 1 in 100 AEP event has a flood level of 25.78 m AHD. Up to the 1 in 100 AEP event, the flood extent at Penrith remains mostly confined within the banks of the Nepean River, although some flooding occurs in Emu Plains on the southern side of the railway embankment. In the PMF event, flood levels reach 32.76 m AHD, inundating large areas of Penrith, with flood extents extending as far as 2.5 kilometres along the Great Western Highway to the east of Victoria Bridge. The flood extent also includes Emu Heights to the west of the Nepean River.

9.4.1.3. Windsor

The floodplain at Windsor is the most severely affected by flooding on the Hawkesbury-Nepean River. In a 1 in 10 AEP event, flood levels at Windsor Bridge are 11.93 m AHD. By this level, a number of properties are isolated on low flood islands after access roads are cut.

In the 1 in 100 AEP event, the flood level at Windsor Bridge is 17.32 m AHD and the flood extent increases substantially from the 1 in 10 AEP event. In the 1 in 100 AEP event, the suburb of McGraths Hill is completely submerged, and while some areas of Windsor, South Windsor and Pitt Town are above the 1 in 100 AEP extent, they are isolated as flood islands. Windsor Road is inundated as far as Vineyard Railway Station (about six kilometres from Windsor Bridge). Macquarie Street is overtopped near Windsor Railway Station and again at the low point near Bligh Park.

In the PMF event, flood levels reach 26.72 m AHD at Windsor Bridge, inundating virtually all of the flood islands including Windsor and Richmond. Backwater flooding up South and Eastern Creeks inundates part of suburbs as far south as St Marys, including Marsden Park, Shanes Park, Llandilo, Vineyard, Riverstone and Schofields.

9.4.1.4. Downstream of Sackville

Downstream of Sackville, the river meanders away from the floodplain and into the gorge country of the Lower Hawkesbury River. The 1 in 10 AEP event has a level of 3.62 m AHD at Wisemans Ferry, while the 1 in 100 AEP event has a level of 7.05 m AHD. At Wisemans Ferry, overbank flow occurs in events as frequent as the 1 in 5 AEP event. In the 1 in 10 AEP event, overbank depths of up to 1.4 metres occur, and in the 1 in 100 AEP event, overbank depths at Wisemans Ferry reach six metres. The PMF event reaches a level of 14.41 m AHD, 7.36 metres above the 1 in 100 AEP event. Due in part to the topography and the smaller difference in flood levels between frequent and rare events, the change in flood extent from the 1 in 100 AEP to the PMF event is relatively small. However, roads are often cut in frequent events and evacuation of isolated communities can be an issue.

9.4.2. Peak flood depths

Figure 19 to Figure 38 present the peak flood depths for the 1 in 5, 1 in 100, 1 in 500 and 1 in 2,000 AEP events, and the PMF. Depths were calculated as the difference between the level mapped from the RUBICON quasi two-dimensional hydraulic model results and the best available DEM. Information within the banks of the Hawkesbury-Nepean River or its tributaries should not be used for any assessment (other than flood extents) without further detailed investigation.

Depths around the Richmond Lowlands floodplain and through Freemans Reach in a 1 in 100 AEP generally exceed eight metres at the peak of the flood. Depths along Rickabys Creek and South Creek on the Windsor floodplain exceed 10 metres (noting this is from backwater flooding). In general, on the Windsor floodplain, the depths in the 1 in 100 AEP event exceed two metres. On the Penrith floodplain, depths greater than one metre occur in a 1% AEP event, particularly on the west bank of the Nepean River around Emu Plains.

9.5. Comparison of peak flood levels with previous studies

9.5.1. Hawkesbury-Nepean Flood Study 1996

Table 47 compares design event levels from the current study with previous regional studies including the 1996 Flood Study (Webb, McKeown & Associates, 1996). In general, flood levels for more frequent events have reduced. This is unsurprising because the 1996 Flood Study assumed the dam to be full before an event while the current study incorporates more realistic dam levels prior to a flood. Flood events more frequent than a 1 in 5 AEP event are not presented as they can result from a number of mechanisms.

Minor changes have occurred to the flood levels between the 1 in 20 AEP and the 1 in 100 AEP event as a result of the recalibration to the flood frequency analysis. The 1 in 100 AEP level has changed by less than 0.3 metres at the locations with long term gauges: Penrith, North Richmond and Windsor. As a result of the recalibration and improved flood frequency estimation techniques, the 1 in 100 AEP flood level at North Richmond has increased by 0.1 metres and the level at Windsor has not changed.

Peak flood levels in the 1 in 200 to 1 in 1,000 AEP range have reduced due to a change in design flood estimation methodology and design inputs. The 1 in 2,000 and 1 in 5,000 AEP events were not estimated in the previous study. PMF levels have increased following an increase in PMP rainfall depths due to a change in the calculation method used to estimate these. This work was undertaken by WaterNSW between the 1996 Flood Study and the current study.

9.5.2. Upper Nepean River Flood Study 1995

The *Upper Nepean River Flood Study* (DLWC, 1995) considered the impact of Nepean River flows and Warragamba River flows on flood levels at Wallacia. The Upper Nepean Study demonstrates that even in small floods, Warragamba River can have an impact on flood levels at Wallacia. This study adopted a simplistic approach, considering investigation of frequency combinations beyond its scope.

Most small events are dominated by Nepean River flows; however, the impact of the dam and Warragamba flows is such that there are three cases that must be considered when determining design flood levels. These are:

- Warragamba dominated events
- Nepean dominated events, and
- Combination of Warragamba and Nepean flows.

Large flood events can also be influenced by these three mechanisms; however, the influence of the dam is more important and a typical large flood will be dominated by the Warragamba as it has a significantly larger catchment area. Any large rainfall on the Warragamba system will have a corresponding large rainfall on the Nepean. The backwater effect from Warragamba means that there is not a one to one relationship between height and flow at Wallacia. For these reasons it is not possible to undertake a traditional flood frequency analysis based on flow at Wallacia.

Neither the *Upper Nepean River Flood Study* nor this Regional Flood Study has directly addressed the complex joint probability problem at Wallacia. The Upper Nepean Study produces higher flood levels between the 1 in 5 and 1 in 200 AEP events than the current study. The current study produces higher levels for rarer events (greater than 1 in 500 AEP) due to the impacts of backwater from the Warragamba River.

While joint probability techniques were known, they were not practical when the 1995 Upper Nepean Study was undertaken. These techniques are now possible and all the necessary information to undertake the analysis for Wallacia is now available. In order to undertake a joint probability analysis a detailed study is required with:

- a detailed hydraulic model so that the rating curve and backwater effects can be modelled and extrapolated properly. Flow behaviour at Wallacia is complex and requires a detailed 2D model.
- a fast model that can be used as a Monte Carlo model and that reproduces observed flood behaviour
- stage frequency curve of historical information pre and post Warragamba Dam construction
- hydrologic model that models the routing effect upstream of Theresa Park Weir. This would need to consider a full range of durations and how the peak flows align.

The *Upper Nepean River Flood Study* produced a range of 1 in 100 AEP estimates (varying by 2 m) and adopted a mid-range estimate. It is possible that for floods in the order of a 1 in 100 AEP and rarer events, both studies have underestimated the slope of the stage frequency curve. This is confirmed when examining historical floods. While Warragamba Dam mitigates floods, it does increase the likelihood of high discharges from the dam aligning with the peak of the Nepean River flows. This can occur in two ways:

- extended duration of high flow though at a lower level than natural flows
- the storage allows the flood wave to travel through Lake Burragorang's 50 kilometres in less than an hour.

Given the need for further investigation of joint probability, as an interim measure it is recommended that the higher flood level from either the 1995 *Upper Nepean River Flood Study* or this Regional Flood Study be adopted at Wallacia. Table 44 summarises the differences in levels between the 1995 Upper Nepean Study and the current study. The PMF level from the 1995 Upper Nepean Study corresponds to between a 1 in 2000 AEP and 1 in 5000 AEP in this Regional Flood Study. The substantially higher level for the Regional Flood Study PMF reflects the very large backwater flows from the Warragamba River to the Nepean River, which backup to inundate Wallacia.

Table 44: Comparison of 1995 Upper Nepean River Flood Study and current study

AEP (1 in x)	Blaxlands Crossing			Bents Basin		
	1995 Upper Nepean Study (m AHD)	Current Study (m AHD)	Difference (m)	1995 Upper Nepean Study (m AHD)	Current Study (m AHD)	Difference (m)
5	**36.8**	35.1	-1.7	**39.6**	38.3	-1.3
10	N/A	37.2	N/A	N/A	40.3	N/A
20	**42.5**	39.4	-3.1	**43.6**	41.9	-1.7
50	N/A	42.6	N/A	N/A	43.9	N/A
100	**45.8**	44.7	-1.2	**46.5**	45.5	-1.0
200	**47.3**	46.5	-0.8	**47.9**	47.0	-0.9
500	N/A	48.9	N/A	N/A	49.2	N/A
1000	N/A	50.8	N/A	N/A	51.0	N/A
2000	N/A	54.2	N/A	N/A	54.3	N/A
5000	N/A	58.3	N/A	N/A	58.4	N/A
PMF	56.9	**66.3**	9.4	57.1	**66.4**	9.3

Note: Higher flood levels per quantile are shown in bold

9.5.3. Lower Hawkesbury River Flood Study 1997

The 1997 *Lower Hawkesbury River Flood Study* (AWACS study) investigated flow below Sackville and from the Colo River. The study adopted practical approaches available in the 1990s to investigate the joint probability of Colo River and Hawkesbury River flooding. The key drivers for flooding in this reach are:

- the magnitude of Colo and Hawkesbury flows
- the timing difference between the Colo and Hawkesbury flows.

The AWACS study assumed a fixed timing difference of 35 hours between the Colo and Hawkesbury flows. The current study allows for variable timing and uses slightly higher Hawkesbury flows that are consistent with upstream flows (noting that the AWACS study reduced flows from the 1996 Flood Study). The current study has produced flood levels that are 0.4-0.6 metres higher at key locations than the AWACS study (refer to Table 47).

The results from the current study are possibly slightly high (in the order of 100-200mm) as the true design flood levels are very dependent on timing of inflows from the Colo. The challenge is the variation in the timing difference. As the timing difference has an asymmetrical response, an average assumption may not be valid. Flood levels reduce in the order of 200mm when no inflows occur on the Colo or with long timing differences. Short timing differences will result in increased flood levels. The estimate of the current study could be improved by focusing the model on the Colo/Hawkesbury junction and undertaking a recalibration with better bathymetry.

9.5.4. Changes in probable maximum flood estimates

Changes in rainfall estimates, spatial patterns and calculation methods have resulted in minor changes to extreme event estimates. The major cause of changed flood levels is due to changes in the configuration of Warragamba Dam's auxiliary spillway, particularly the height of the fuse plugs and the dimensions of the side spillway.

The 1996 Flood Study presented two PMF results being:

1. assuming no side spillway on Warragamba Dam and that the dam doesn't fail during a PMF
2. assuming no side spillway on Warragamba Dam and that the dam fails during a PMF.

The second case results in higher levels downstream as it allows water to flow over the dam crest. The difference in water level downstream between these cases is in the order of 3.4–4.7 metres.

The 1997 Hawkesbury Nepean Flood Management Strategy incorporated designs for the auxiliary spillway, subsequently completed in 2002. Due to fuse plug failure and flow down the side spillway to ensure dam stability, the PMF level is higher downstream than Case 1 from the 1996 Flood Study but lower than Case 2 from the 1996 Flood Study.

Table 45 below shows how PMF levels have changed with spillway configuration and assumptions at Warragamba Dam since the 1996 Flood Study. The calculation of the PMF is discussed in Sections 8.5.1 and 8.5.2.

The PMF for the current study is 0.2 metres higher than the 1997 PMF at Windsor and North Richmond and 0.5 metres higher at Penrith.

Table 45. Probable maximum flood levels

Location	Case Flood Level (m AHD)			
	1996 Dam conditions (no side spillway and no failure) (1)	1996 Dam conditions (no side spillway and dam failure) (2)	1997 Concept design side spillway (3)	Current dam with side spillway as built (4)
Wallacia/Blaxlands Crossing	NA	NA	NA	66.34
Penrith	30.9	35.6	32.1	32.76
North Richmond	25.6	29.0	26.5	26.81
Windsor	25.5	28.9	26.4	26.72

Notes:

(1) prior to the side spillway being constructed, with the dam modelled not to fail in PMF.
(2) prior to the side spillway being constructed, with the dam expected to fail in PMF.
(3) the concept design assumed the fuse plugs operated at lower level than as built.
(4) PMF estimated using PMP - see Sections 8.5.1 and 8.5.2

Table 46. Peak flood levels at key reporting locations

No.*	Name	Peak Flood Levels (m AHD)										
		1 in 5 AEP	1 in 10 AEP	1 in 20 AEP	1 in 50 AEP	1 in 100 AEP	1 in 200 AEP	1 in 500 AEP	1 in 1000 AEP	1 in 2000 AEP	1 in 5000 AEP	PMF
1	Brooklyn Bridge (M1)	1.55	1.59	1.62	1.66	1.69	1.72	1.85	1.89	1.95	2.14	3.02
2	Spencer	1.81	1.90	2.04	2.39	2.74	3.18	3.83	4.40	4.93	5.72	6.83
3	Gunderman-Singletons Mill	2.21	2.77	3.55	4.66	5.52	6.47	7.55	8.47	9.29	10.49	11.98
4	Wisemans Ferry	2.74	3.62	4.71	6.07	7.05	8.12	9.35	10.37	11.31	12.70	14.41
5	Leets Vale	3.67	5.07	6.48	8.08	9.15	10.14	11.54	12.62	13.56	14.94	17.27
6	Lower Portland	4.81	6.54	8.16	9.91	11.09	12.09	13.57	14.82	15.82	17.27	20.15
7	Sackville	6.27	8.44	10.12	12.14	13.24	14.22	15.57	16.72	18.01	19.20	23.58
8	Ebenezer	8.27	10.69	12.65	14.97	16.25	17.32	18.67	19.64	20.81	21.81	25.98
9	Cattai Creek/Gronos Point	8.86	11.27	13.24	15.61	16.93	18.00	19.31	20.28	21.42	22.38	26.51
10	South Creek at Richmond Road	9.82	11.88	13.71	16.04	17.31	18.34	19.62	20.58	21.69	22.64	26.70
11	Windsor	9.85	11.93	13.74	16.05	17.32	18.35	19.63	20.58	21.70	22.64	26.72
12	Rickabys Creek at Blacktown Road	10.05	12.04	13.80	16.09	17.35	18.37	19.64	20.60	21.71	22.66	26.73
13	North Richmond	11.39	13.67	15.35	16.53	17.55	18.55	19.80	20.74	21.85	22.79	26.81
14	Yarramundi Bridge	12.03	14.46	16.37	17.43	18.19	19.11	20.28	21.17	22.29	23.23	27.14
15	Penrith	19.57	21.34	23.30	24.79	25.78	26.47	27.10	27.50	28.37	29.44	32.76
16	Blaxlands Crossing	35.06	37.21	39.42	42.57	44.65	46.49	48.93	50.75	54.22	58.33	66.34
17	Bents Basin	38.29	40.33	41.89	43.87	45.46	47.02	49.24	50.99	54.25	58.36	66.37

**Locations as per Figure 1*

Table 47. Comparison of peak flood levels for design quantiles compared with previous flood studies

1 in X chance per year flood	Penrith (Victoria Bridge)			North Richmond Bridge			Windsor Bridge			Wisemans Ferry (Webbs Creek Ferry site)		
	1997 study[1] (1996 study[2])	Current study	Change	1997 study[1] (1996 study[2])	Current study	Change	1997 study[1] (1996 study[2])	Current study	Change	1997 study[3]	Current study	Change
	m AHD	m AHD	m	m AHD	m AHD	m	m AHD	m AHD	m	m AHD	m AHD	m
5	20.1	19.6	-0.5	12.5	11.4	-1.1	11.1	9.9	-1.2	3.2	2.8	-0.4
10	21.6	21.3	-0.3	14	13.7	-0.3	12.3	11.9	-0.4	NA	3.7	NA
20	23.4	23.3	-0.1	15.3	15.4	0.1	13.7	13.7	0	4.4	4.8	0.4
50	24.9	24.8	-0.1	16.4	16.5	0.1	15.7	16.1	0.4	5.6	6.2	0.6
100	26.1	25.8	-0.3	17.5	17.6	0.1	17.3	17.3	0	6.7	7.2	0.5
200	26.9	26.5	-0.4	18.9	18.6	-0.3	18.7	18.4	-0.3	7.5	8.2	0.7
500	27.5	27.1	-0.4	20.4	19.8	-0.6	20.2	19.6	-0.6	NA	9.5	NA
1000	28.6 (28.0)	27.5	-1.1	22.1 (21.5)	20.7	-1.4	21.9 (21.3)	20.6	-1.3	NA	10.5	NA
2000	NA	28.4	NA	NA	21.9	NA	NA	21.7	NA	NA	11.4	NA
5000	NA	29.4	NA	NA	22.8	NA	NA	22.6	NA	NA	12.8	NA
PMF	32.1 (30.9)	32.8	0.7	26.5 (25.6)	26.8	0.3	26.4 (25.5)	26.7	0.3	16.3	14.5	-1.8

1. Webb, McKeown & Associates (1997). Note, these design flood levels allowed for Warragamba Dam’s auxiliary spillway, which was completed in 2002.
2. Webb, McKeown & Associates (1996). Note, these older design flood levels did not allow for Warragamba Dam’s auxiliary spillway, which was completed in 2002, and assumed that the dam does not fail in the PMF event. Should the dam fail, the PMF levels at Penrith, North Richmond and Windsor were modelled to peak at 35.6m AHD, 29.0m AHD and 28.9m AHD, respectively.
3. Australian Water and Coastal Studies Pty Ltd (AWACS) (1997), Tables 10.1 and 10.3.

9.6. Representative design flood events

A Monte Carlo approach and a fast quasi-two-dimensional model was used for this Regional Flood Study. A fast quasi-two-dimensional model is required to run the thousands of events in a realistic timeframe. Running a more detailed two-dimensional model of the entire catchment for all Monte Carlo events would take an extremely long time. It is likely in the future that detailed two-dimensional models may be required for smaller areas.

The 1996 Flood Study has historically provided boundary conditions for detailed two-dimensional models established in the valley since 1996. As this study adopted a Monte Carlo sampling approach rather than a standard design event approach, most events will have a range of exceedance probabilities at the various locations of interest within the model, rather than a single exceedance probability assumed at all locations with the traditional approach. This is due to the likelihood of the rank of peak flood levels within the Monte Carlo events changing as the flood progresses downstream.

To provide design events that can be used as boundary conditions for future detailed modelling and that can be visualised using WaterRIDE, a set of representative events were chosen. These are different to the evacuation events. As much as possible the events were selected to maintain AEP as the flood progressed downstream. Further detail on how these events were chosen is provided in Appendix N. The representative events for a given AEP should be enveloped to form a design flood surface.

9.7. Other outputs

Figure 39 (upstream of Lynchs Creek confluence) and Figure 40 (downstream of Mahons Creek confluence) present flood profiles (flood level versus chainage from the ocean). The profiles are sampled from the envelope of the design quantiles at each model calculation point. Flood profiles are presented for a range of flood events from 1 in 5 AEP to PMF.

For the figures discussed in this section, critical levels were identified for each particular key reporting location based on NSW SES evacuation triggers. The purpose of the following graphs is largely to inform emergency response planning. This information will assist emergency response agencies in understanding the variability of floods that may be experienced, rather than the single design event and therefore single number previously available to them.

Stage versus frequency curves are presented for key locations in Figure Set E. Critical levels are shown on the plots, such as when roads are cut.

Rate of rise versus flood probability plots are presented for key locations in Figure Set F. Rate of rise is the change in flood height per hour for a particular event to increase from a chosen key level to another chosen key level.

Time to rise versus flood probability plots are presented for key locations in Figure Set G. The time to rise is the time it takes a particular event to increase from a chosen key level to another chosen key level.

Rate of fall versus flood probability plots are presented for key locations in Figure Set H. Similar to rate of rise, rate of fall is the change in height per hour from when the upper level is reached to when the receding limb falls below the lower level for an event hydrograph.

Time to fall versus flood probability plots are presented for key locations in Figure Set J. The time to fall is the total time from when the upper level is reached to when the receding limb falls below the lower level.

Time above a critical level versus flood probability between a number of key levels is presented in Figure Set K. This is the total time the stage hydrograph for each Monte Carlo event exceeds a particular level.

The travel time between a number of points is presented in Figure Set L. To assess travel times, floods were categorised depending on whether the peak of the flood was dominated by flows from the Nepean system, flows from the Warragamba system or a combination. Levels are peak flood level to peak flood level and therefore can be misleading, particularly for long drawn out peak events. Negative numbers indicate that the downstream location peaks first. Diagram 8 provides an example for Penrith to North Richmond and highlights the complexity of flow within the valley.

More details on how to interpret these plots and key findings can be found in the relevant appendix.

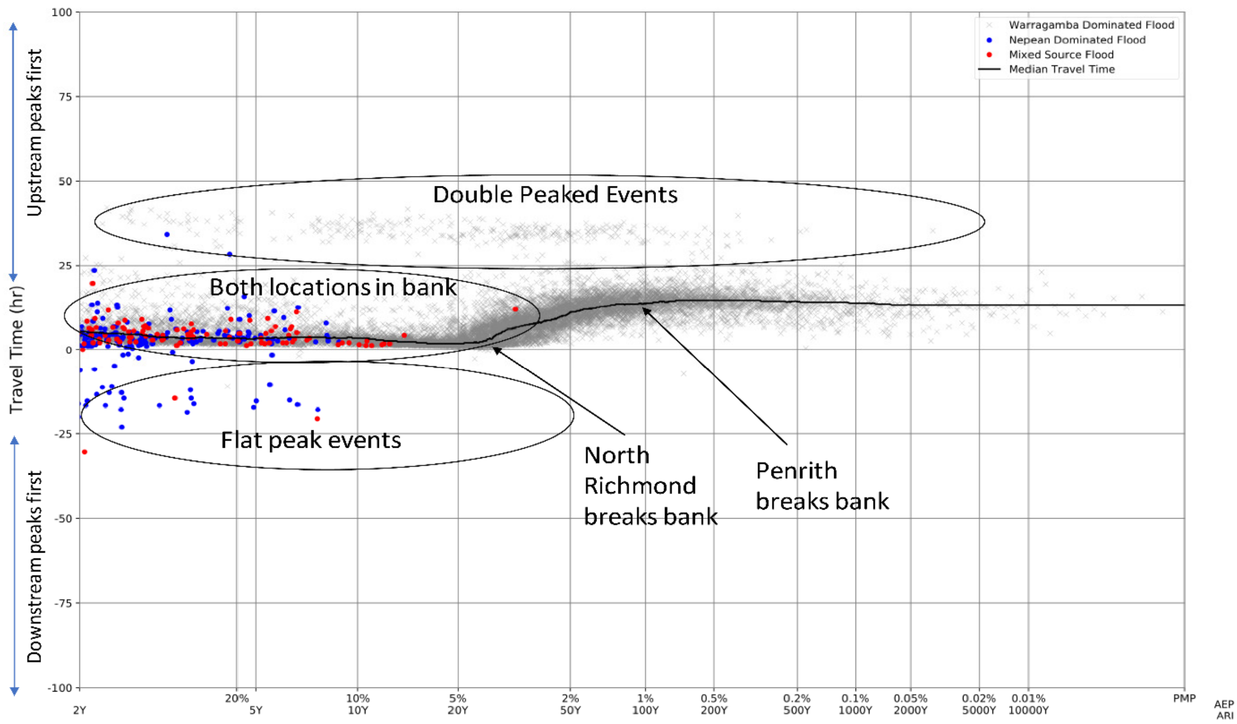

Diagram 8. Example time between peaks graph – Penrith to North Richmond

10. CLIMATE CHANGE AND SEA LEVEL RISE

10.1. Background

There is strong evidence that increases in global temperatures will lead to an increase in the intensity of rare rainfall, and that extreme flooding globally has increased over the twentieth century (CSIRO & BoM, 2015). Global warming has been observed for several decades and has been linked to changes in key parts of the hydrologic cycle including changes in rainfall behaviour, rainfall intensity, soil moisture and runoff (Bates et al., 2008). Climate change can alter flood behaviour in the Hawkesbury-Nepean by changing:

- probability of long duration rainfall intensities
- storm type and frequency
- rainfall spatial and temporal patterns
- antecedent conditions
- dam levels prior to flood producing rainfall.

The interaction of these characteristics makes predicting the impact of climate change on flood behaviour complex.

10.1.1. Rainfall depth and frequency

The interaction of a warming climate and rainfall is complex. A warmer climate leads to an increase in the potential moisture-holding capacity of the atmosphere which is one of the key factors in the depth of precipitation in rarer rainfall events; however, on large catchments like the Hawkesbury-Nepean, long duration rainfall events are also dependant on sources and transport of moist air. Statistically significant increases in rainfall intensity have been detected in Australia for short duration rainfall events and are likely to become more evident towards the end of the twenty-first century (Westra et al., 2013). Changes in long duration events are expected to be smaller and harder to detect, but projections analysed by CSIRO (2007) showed that an increase in daily precipitation intensity is likely under climate change. It is worth noting that a warming climate can lead to decreases in annual rainfall along with increases in flood producing rainfall.

10.1.2. Storm type and frequency

Nearly all of the large flood-producing events on the Hawkesbury-Nepean have been either caused by east coast lows (ECLs) or the interaction of ECLs and other rain-producing systems. ECLs are the major flood producing mechanism on large catchments on the east coast of New South Wales and are being very actively studied. The historical flood record on the New South Wales and Victorian east coast shows that floods produced by ECLs are less prevalent further south. If climate change pushes ECL events further south, then it is plausible in the Hawkesbury-Nepean Valley that the frequency of ECL events will increase.

10.1.3. Spatial and temporal rainfall behaviour

The influence of warmer climate on the spatial and temporal aspects of rainfall is less understood than changes in rainfall intensity. Work by Abbs and Rafter (2009) suggests that increases will be more pronounced in areas with strong orographic enhancement, which could lead to larger increases in the Nepean catchment than the Warragamba catchment. An analysis of historical storms found that, regardless of the climate region or season, temperature increases are associated with patterns becoming less uniform (Wasco & Sharma 2015). Therefore, the parts of the storm with the largest rainfalls increase in rainfall intensity and the parts of the storm with lower rainfalls decrease in rainfall intensity.

10.1.4. Antecedent conditions

Changes to rainfall and evaporation as a result of climate change will result in a change in the antecedent conditions prior to an event. It is likely that evaporation will increase (Bates et al., 2008) by 2030 and 2070 by approximately two per cent. Increased evaporation in combination with decreased rainfall could result in decreases in annual runoff, but the impact on flood events is likely to be less.

10.1.5. Dam levels prior to flood producing rainfall

Along with antecedent conditions, a warming climate could change pre-event levels in Warragamba Dam. This aspect is complex as it not only changes runoff into the dam (including the large events that often fill the dam), but also the operational response to such changes. For example, if climate change results in a decrease in inflows to the dam, this could result in an operational change to more frequent pumping from the Shoalhaven, so the dam level over time may be higher than expected from what the inflows alone suggest. WATHNET model results for future climate conditions were not available from WaterNSW at the time of the study, and were assumed to be the same as current conditions as operational changes could be made to maintain current levels.

10.2. Assessment methodologies

The NSW Government's *Floodplain Development Manual* requires that flood studies and floodplain risk management studies consider the impact of climate change (rainfall increase and sea level rise) on flood behaviour. The then Department of Environment and Climate Change's *Floodplain Risk Management Guideline* (DECC, 2007) recommended the following climate change scenarios (rainfall by the year 2070) be considered:

Increase in peak rainfall and storm volume:

- low level rainfall increase = 10 per cent
- medium level rainfall increase = 20 per cent
- high level rainfall increase = 30 per cent.

A high level rainfall increase of up to 30 per cent was recommended for consideration due to the uncertainties associated with this aspect of climate change.

Engineers Australia, CSIRO and the Bureau of Meteorology recently released a guide to the assessment of climate change impact on flood behaviour as part of the revision of *Australian Rainfall and Runoff*. This work recommends an interim approach based on simple temperature scaling using temperature projections from the CSIRO Climate Futures Tool (www.climatechangeinaustralia.gov.au). Scaling based on temperature is recommended as climate models are much more reliable at producing temperature estimates than rainfall, and an ensemble of climate models can be used to estimate annual mean surface temperature. Given the sensitivity of the catchment to climate change, the ARR procedures were adopted.

Detailed investigations are underway to assess the potential impact of climate change on flood behaviour in the Hawkesbury-Nepean catchment as part of the ongoing work under the Flood Strategy.

10.2.1. Sensitivity of increased rainfall intensity/volumes

Climate change predictions are made based on modelling changes to temperature and rainfall for various Representative Concentration Pathways (RCPs), which consider projected increases in greenhouse gas concentrations. Examples relative to temperature for a baseline period of 1975–2004 are shown in Table 48 for 2050 and 2090. ARR recommends that RCP 4.5 and 8.5 be used for impact assessment (Ball et al., 2016). RCP 4.5 is recommended as a low emissions pathway as RCP 2.6 is considered too optimistic. RCP 8.5 is recommended for consideration where the expense of considering it can be justified on socioeconomic and environmental grounds (for example major infrastructure projects). This first pass estimate does not account for the fact that increases are expected to be higher for shorter duration events, and smaller for large catchments like the Hawkesbury-Nepean. For Sydney, a rainfall increase of between 10 and 20 per cent is projected. This is in broad agreement with the DECC guideline.

Table 48. Projected increases in temperature and rainfall for Sydney (adapted from CSIRO Climate Futures Tool)

Year	**Low (based on RCP 4.5)**		**Medium* (average of RCP 4.5 and 8.5)**	**High (based on RCP 8.5)**	
	Temperature (°c)	Rainfall (%)	Rainfall (%)	Temperature (°c)	Rainfall (%)
2030	0.892	4.5	4.7	0.979	4.9
2040	1.121	5.6	6.2	1.351	6.8
2050	1.334	6.7	7.8	1.765	8.8
2060	1.522	7.6	9.4	2.230	11.2
2070	1.659	8.3	11.0	2.741	13.7
2080	1.780	8.9	12.6	3.249	16.2
2090	1.825	9.1	13.9	3.727	18.6

Source: CSIRO and BoM 2015

** Approximation based on the average of RCP 4.5 and 8.5.*

Another RCP considered by climate scientists is RCP 6. However, between 2030 and 2060 the RCP 6 results are inconsistent with the RCP 4.5 and 8.5 results. To remove this inconsistency, a medium emissions scenario was created by averaging the RCP 4.5 and 8.5 scenarios (Table 48).

To reduce the number of hydrologic model runs and cover the full range of possible climate change scenarios, the following rainfall increases were chosen for the assessment:

- 4.9 % (high 2030)
- 9.1% (low 2090)
- 13.9% (medium 2090)
- 18.6% (high 2090)

If a modelled flood event had 100 mm of rainfall in a catchment under ‘2016’ or historic average climate conditions, under a 4.9 per cent rainfall increase scenario that same modelled event would have 104.9 mm of rainfall in the same catchment over the same time period.

These scenarios can be directly used to represent 2030 conditions and low, medium and high emissions 2090 conditions, but more importantly they allow for interpolation at decadal time scales. These rainfall increases can be represented as the approximate time scales when they may occur in Table 49. It is noted that the last decade has seen emissions tracking towards the upper end of the RCPs (Sanderson et al., 2016).

Table 49. Climate change pathways, rainfall increases and approximate time scales

Modelled climate change rainfall increase	**Expected year when rainfall increase realised under different climate change projections**		
	Low emissions	Medium emissions	High emissions
	RCP 4.5		RCP 8.5
4.9%	2034	2032^	2030
9.1%	2090	2071^	2051
13.9%	2330*	2200*^	2071
18.6%	2565*	2328*^	2090

** Extrapolation based on low emissions 2090 increase*

^ Average of low and high emissions

10.2.2. Sensitivity of sea level rise

The NSW Government’s *Sea Level Rise Policy Statement* (DECCW, 2009) included benchmarks for assessing the impact of sea level rise on flood risk. Projected rises in sea level (relative to 1990 mean sea level) of 0.4 metres by 2050 and 0.9 metres by 2100 were recommended for assessment. The 2012 *Stage 1 Coastal Management Reforms* (OEH, 2012) no longer recommend state-wide sea level rise benchmarks. Local councils are now able to consider local conditions and determine appropriate levels.

Consistent with advice from the NSW Chief Scientist, the previously recommended 0.4 metres by 2050 and 0.9 metres by 2100 were adopted for testing sensitivity to sea level rise.

10.3. Impact of climate change rainfall increases on flood behaviour

The rainfall increases discussed in Section 10.2.1 were applied to the rainfalls of each Monte Carlo event. Figure 49 and Figure 50 show the flood stage frequency curves for Windsor and Penrith for the existing climate and the climate change scenarios.

The change in probability of the current 1 in 100 AEP event by 2090 is presented in Table 50. A 1 in 100 AEP event at Windsor under existing climate would become a 1 in 80 AEP event with a 4.9 per cent rainfall increase and a 1 in 65 AEP event with a 9.1 per cent rainfall increase. A 1 in 100 AEP event at Penrith under existing climate would become a 1 in 78 AEP event with 4.9 per cent rainfall increase and a 1 in 65 AEP event with a 9.1 per cent rainfall increase.

Table 51 describes the impact of climate change rainfall increases on key reporting locations for the 1 in 100 AEP event. Blaxlands Crossing is the worst affected key reporting location with a 9.1 per cent rainfall increase causing a 1.09 metre increase in the 1 in 100 AEP flood level.

Table 50. Probability of the current 1 in 100 AEP event by climate change scenario

Location	Current 2016 average climate (1 in X AEP)	Climate change scenario (% rainfall increase) probability of the current 1 in 100 AEP event (1 in X AEP)			
		4.9%	**9.1%**	**13.9%**	**18.6%**
Windsor	100	80	65	54	44
Penrith	100	78	65	54	46

Diagram 9 and Diagram 10 present the change in flood level from the existing 1 in 100 AEP flood level for a high emissions scenario for Windsor and Penrith respectively. By 2051 (9.1 per cent increase in rainfall intensity), 1 in 100 AEP flood levels at Windsor would increase by 0.71 metres.

Figure 51 and Figure 52 highlight the differences in time to reach and fall below critical levels at Windsor. These highlight that the increased flood peaks resulting from increased rainfall due to climate change would cause less evacuation time on average (as the time to rise between 4 and 14 m AHD decreases) and increased inundation times in flood affected areas. As the rainfall associated with climate change increases, so too do inundation times.

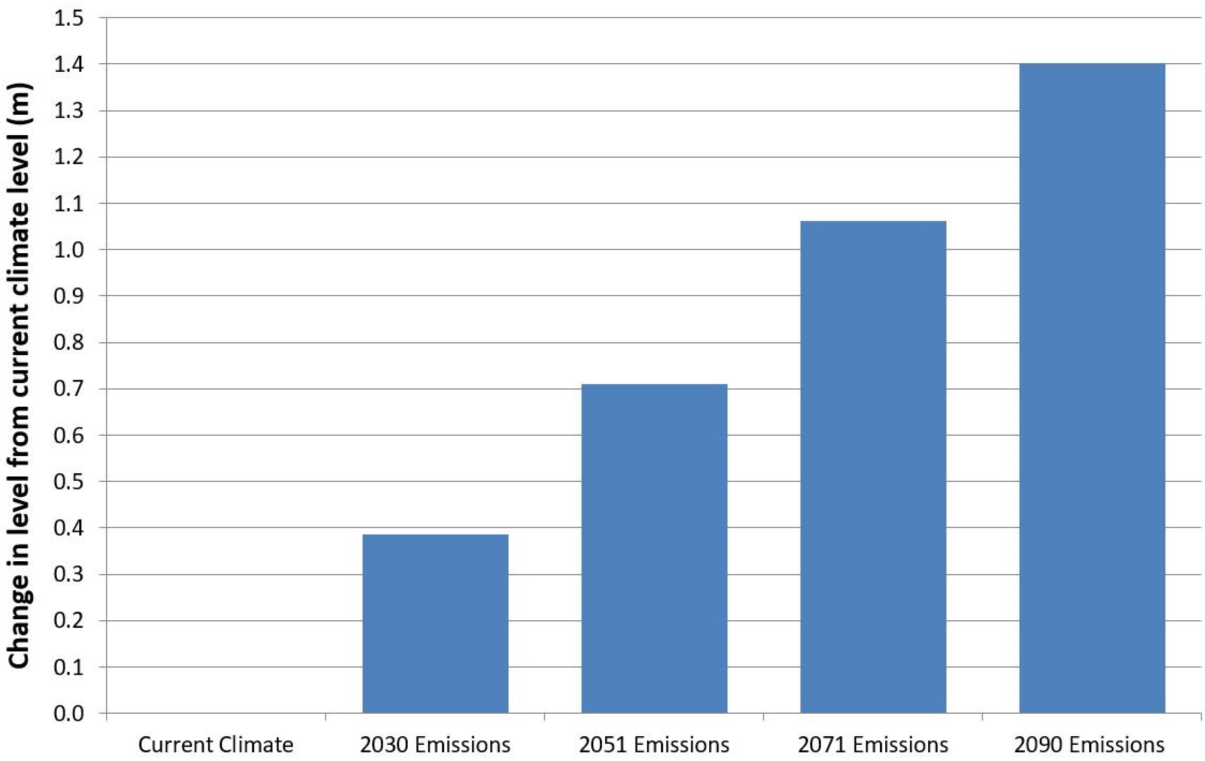

Diagram 9. Impact of climate change rainfall intensity increases on 1 in 100 AEP peak flood levels at Windsor (high emissions)

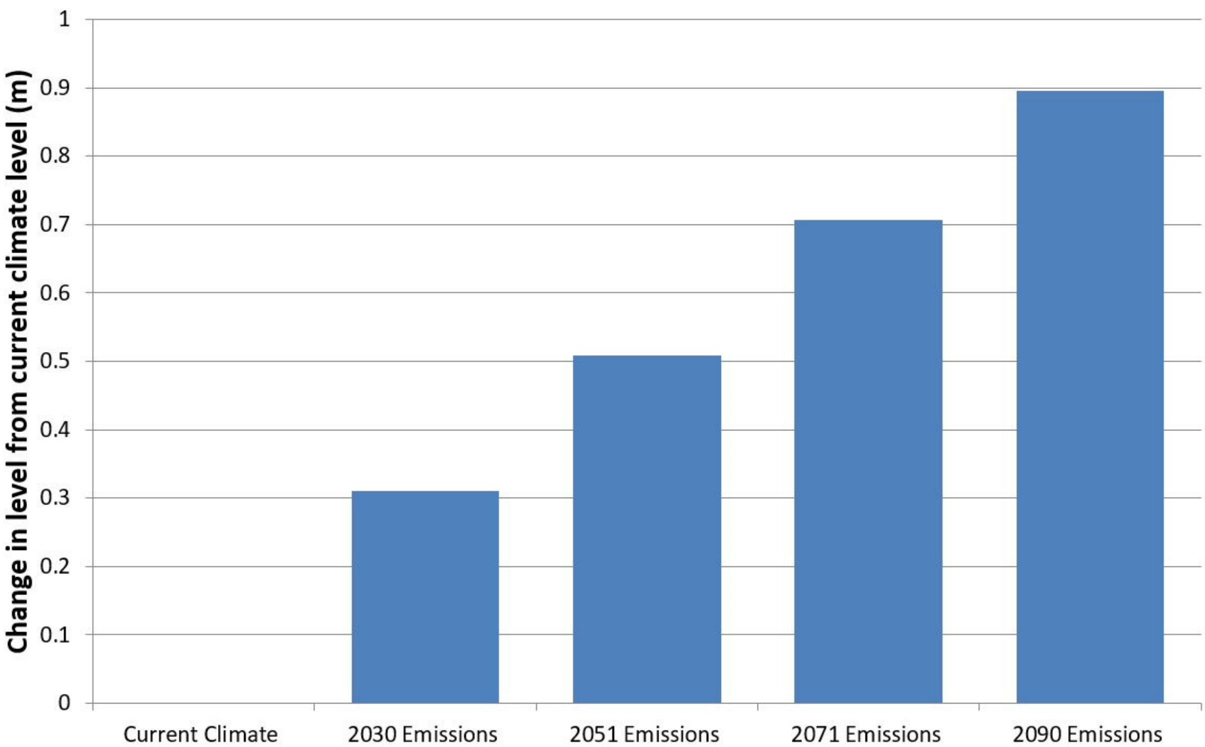

Diagram 10. Impact of climate change rainfall intensity increases on 1 in 100 AEP peak flood levels at Penrith (high emissions)

10.4. Impacts of sea level rise

The Monte Carlo events were rerun using the sea level rise boundary conditions and the results analysed. The impacts of sea level rise are generally contained to the estuary downstream of Lower Portland. The larger the flood event, the less distance upstream the sea level rise influence stretches. Under very frequent events sea level rise will affect areas further upstream. Table 51 summarises the impact of sea level rise on key reporting locations for the 1 in 100 AEP event. A 0.9 metre sea level rise would increase the 1 in 100 AEP flood level at Windsor by 0.01 metres. At Spencer, a 0.9 metre sea level rise would increase the 1 in 100 AEP flood level by 0.56 metres.

Figure 53 shows how an increase in sea level of 0.4 metres and 0.9 metres would affect flood timing at Windsor. The time taken to reach a flood level of both four metres and 10 metres in each scenario at Windsor was assessed. A 1:1 line shows where there is parity in the time to reach certain levels between climate change scenarios and the existing case. As each of the climate change scenarios had scattered results lying on both sides of the 1:1 line, a regression of the results for each scenario is presented on the graph. Figure 53 illustrates that the time taken to reach 4 m AHD in the sea level rise scenario of 0.4 metres is less than in the existing case. This trend is exacerbated in the 0.9 metres sea level rise scenario, in which the events reach 4 m AHD at Windsor in a shorter time than the 0.4 metres sea level rise scenario. Despite this, neither sea level rise scenario significantly affects the time to reach 10 metres at Windsor (as illustrated where both trends closely align with the 1:1 line on the graph). It is possible that the effects on timing occur further downstream.

Figure 54 compares flood profiles for existing conditions to the sea level rise scenarios, for three flood events.

Table 51. Impact of climate change rainfall increase and sea level rise at key reporting locations for the 1 in 100 AEP event

Name	1 in 100 AEP Levels (m AHD)							Difference (m)					
	Current sea level & climate	Sea level rise 0.4 m	Sea level rise 0.9 m	Rainfall increase 4.9%	Rainfall increase 9.1%	Rainfall increase 13.9%	Rainfall increase 18.6%	Sea level rise 0.4 m	Sea level rise 0.9 m	Rainfall increase 4.9%	Rainfall increase 9.1%	Rainfall increase 13.9%	Rainfall increase 18.6%
Brooklyn Bridge (M1)	1.69	2.10	2.59	1.71	1.72	1.72	1.73	0.41	0.90	0.02	0.03	0.03	0.04
Spencer	2.74	2.97	3.30	2.90	2.99	3.17	3.32	0.23	0.56	0.16	0.25	0.43	0.58
Gunderman - Singletons Mill	5.52	5.59	5.67	5.80	6.07	6.34	6.63	0.07	0.15	0.28	0.55	0.82	1.11
Wisemans Ferry	7.05	7.09	7.15	7.39	7.68	7.99	8.30	0.04	0.10	0.34	0.63	0.94	1.25
Leets Vale	9.15	9.17	9.20	9.52	9.82	10.15	10.49	0.02	0.05	0.37	0.67	1.00	1.34
Lower Portland	11.09	11.09	11.11	11.46	11.78	12.14	12.50	0	0.02	0.37	0.69	1.05	1.41
Sackville	13.24	13.24	13.26	13.62	13.91	14.26	14.59	0	0.02	0.38	0.67	1.02	1.35
Ebenezer	16.25	16.25	16.26	16.65	16.98	17.36	17.71	0	0.01	0.40	0.73	1.11	1.46
Cattai Creek/Gronos Point	16.93	16.93	16.93	17.33	17.66	18.03	18.38	0	0	0.40	0.73	1.10	1.45
South Creek at Richmond Road	17.31	17.31	17.32	17.70	18.02	18.37	18.71	0	0.01	0.39	0.71	1.06	1.40
Windsor	17.32	17.33	17.33	17.71	18.03	18.39	18.73	0.01	0.01	0.39	0.71	1.07	1.41
Rickabys Creek at Blacktown Road	17.35	17.35	17.36	17.74	18.06	18.41	18.74	0	0.01	0.39	0.71	1.06	1.39
North Richmond	17.55	17.55	17.55	17.92	18.24	18.58	18.91	0	0	0.37	0.69	1.03	1.36
Yarramundi Bridge	18.19	18.19	18.19	18.54	18.82	19.14	19.45	0	0	0.35	0.63	0.95	1.26
Penrith	25.78	25.78	25.78	26.09	26.29	26.49	26.68	0	0	0.31	0.51	0.71	0.90
Blaxlands Crossing	44.65	44.65	44.65	45.17	45.74	46.40	47.03	0	0	0.53	1.09	1.76	2.39
Bents Basin	45.46	45.46	45.46	45.91	46.39	46.95	47.50	0	0	0.45	0.93	1.49	2.04

11. DEVELOPMENT OF GRIDDED RESULTS FROM RUBICON MODEL

Flood mapping capabilities for the RUBICON model were developed as part of the 1996 Flood Study. This mapping was revised and developed into software for the NSW SES as part of a flood prediction project. The development of the digital elevation model (DEM) included careful consideration of break lines, overflow paths and backwater areas of the Hawkesbury-Nepean floodplain. This software allows flood surfaces to be readily developed from the RUBICON results which would not normally be produced by this style of model.

All gridded results presented (other than those in Section 12) are based on this approach. An overview of the mapping technique is provided in Appendix P.

Flood extents and levels are suitable for use over the entire study area, for Hawkesbury-Nepean mainstream regional flooding from Bents Basin near Wallacia to Brooklyn Bridge.

Flood surface mapping has been trimmed on several tributaries, including Colo River, South Creek and Eastern Creek, to reflect the expected upstream limit of backwater flooding from main river flooding. The extent to which the tributary has been trimmed varies with the flood size. Upstream of the extent of mapping of these tributaries, flooding from the local catchment is expected to dominate and hence higher flood levels would be expected.

The flood surface mapping has also been trimmed at the downstream extent of the study area at Brooklyn Bridge (M1). Additional smoothing processes were necessary to produce flood surfaces and hence flood contours. At the downstream extent of mapping, this additional smoothing process has resulted in small variations in flood levels around Brooklyn Bridge between the raw peak flood levels listed in Table 46 and the peak flood surfaces (and hence flood contours) shown in Volume 3. These variations of around 200mm relate to only the very largest flood events.

Mapped flood depths reflect the accuracy of the underlying LiDAR used to develop the digital elevation model. To more precisely determine the depth of flooding at a particular location, the flood level should be compared to a surveyed ground level.

Mapped provisional flood hazard categories incorporate depth and so are subject to the same caveat as above.

Due to the nature of the bathymetric data, any flooding information within the banks of rivers or streams should not be used for any assessment (other than flood extents) without detailed investigation.

Baseflow has been incorporated in the peak flood surfaces, including the probable maximum flood.

Flood mapping over Penrith Lakes is indicative only as the flood modelling does not incorporate the latest infrastructure of the Penrith Lakes Scheme.

Penrith City Council's *Nepean River Flood Study* (Advisian, 2018) may provide more detailed analysis of flood behaviour within its study area (Nepean River flooding between the Glenbrook Creek confluence – upstream of the M4 Motorway Bridge crossing – and Yarramundi Bridge).

12. PROVISIONAL FLOOD HAZARD AND HYDRAULIC CATEGORIES

For the purposes of floodplain risk management in New South Wales, floodplains are divided into flood hazard categories and hydraulic categories (floodway, flood storage and flood fringe). Further details of this process are provided in the *Floodplain Development Manual: the management of flood liable land* (NSW Government, 2005, Appendix L). Flood hazard and hydraulic categories define the most dangerous parts of the floodplain, and are used to inform land use and emergency planning.

An essential input for defining flood hazard and hydraulic categories is the velocity of floodwaters. The RUBICON model used for the other elements of the current study characterises velocity as cross-sectional average velocity. Depending on the width of the floodplain this can be one value covering several hundred metres. In order to undertake an assessment of flood hazard and hydraulic categories, a distribution of velocity across the floodplain is required. A two-dimensional hydraulic model (TUFLOW) of the floodplain was developed for this purpose. The model was quasi calibrated and is suitable only to provide an indication of the flow distribution.

12.1. Two-dimensional model development

The development of a two-dimensional hydraulic model for quantifying the velocity distribution is detailed in Appendix D.

12.2. Provisional hazard

Managing the Floodplain: a guide to best practice in flood risk management in Australia (AIDR 2017) provides a revised flood hazard classification, relating combinations of flood depths and velocities to risks to vehicles, people and buildings. The classification is divided into six categories (Diagram 11):

- H1 - Generally safe for people, vehicles and buildings
- H2 - Unsafe for small vehicles
- H3 - Unsafe for vehicles, children and the elderly
- H4 - Unsafe for people and vehicles
- H5 - Unsafe for people and vehicles. All buildings vulnerable to structural damage. Some less robust building types vulnerable to failure
- H6 - Unsafe for people and vehicles. All buildings types considered vulnerable to failure.

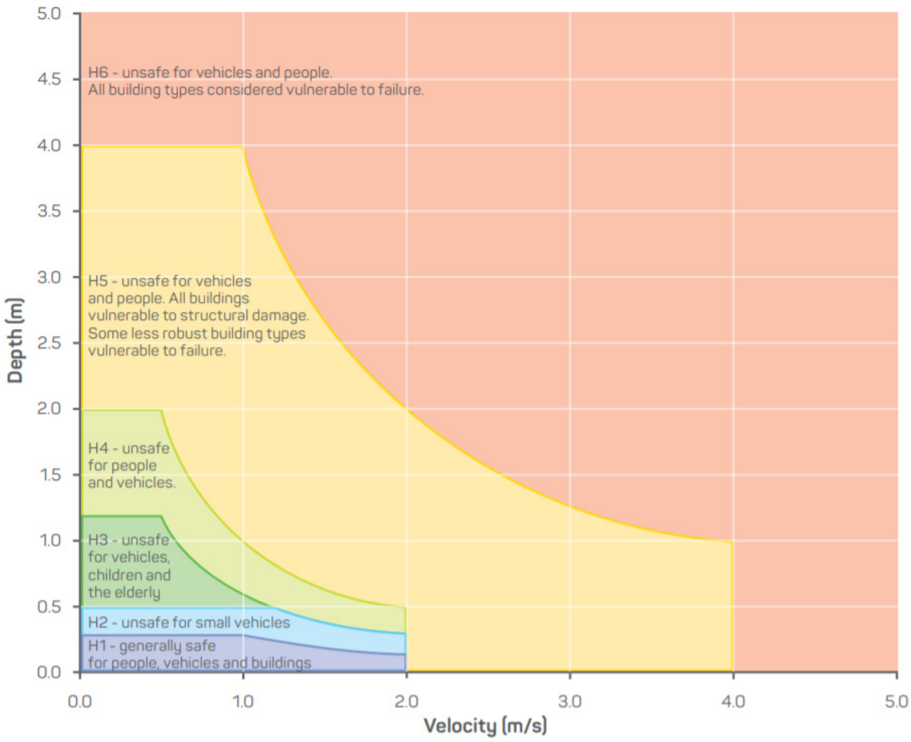

Diagram 11. Flood hazard vulnerability curves (AIDR 2017)

The *Floodplain Development Manual* (NSW Government, 2005) requires that other factors be considered in determining the 'true' hazard including: size of flood, effective warning time, flood readiness, rate of rise of floodwaters, depth and velocity of floodwaters, duration of flooding, evacuation problems, effective flood access, type of development within the floodplain, complexity of the stream network and the inter-relationship between flows. However, to assess the full flood hazard all adverse effects of flooding have to be considered. As well as considering the provisional (hydraulic) hazard it also incorporates threat to life, danger and difficulty in evacuating people and possessions and the potential for damage, social disruption and loss of production.

The conversion from 'provisional' hazard to 'true' hazard requires subjective decisions on how these aspects interact with the population at risk. To overcome this problem the practice has evolved to map provisional hazard and to separately identify evacuation risk over the full range of flood events. For this reason, a true hazard conversion has not been carried out.

Hazard classification was carried out on the 1 in 100 AEP event adopting gridded depth and velocity results output from the TUFLOW 2D hydraulic model. However, the RUBICON hydraulic model produced a more accurate representation of other events, compared to the TUFLOW 2D model, and was used for the hazard classification of events other than the 1 in 100 AEP event. The RUBICON model does not produce varying velocities across the floodplain, but testing of velocities with the 2D model suggests that the hazard is largely depth driven due to the significant flood depths within the Hawkesbury-Nepean floodplain. Other than on the very edge of the floodplain, depths exceed five metres in a 1 in 100 AEP event. Thus, the adoption of a universal peak velocity of 1 m/s across the floodplain for the hazard classification of events other than the 1 in 100 AEP event is considered acceptable.

Figure 41 to Figure 44 present the provisional flood hazard classifications for the 1 in 100 AEP event. Under this classification for a 1 in 100 AEP event, the majority of the floodplain is considered unsafe for vehicles and people, with buildings requiring special engineering design and construction or buildings being vulnerable to failure. Some less hazardous areas occur west of Bligh Park, south of the Airforce base.

Provisional flood hazard classifications for other events are provided in Figure Set D. In a probable maximum flood (PMF), the majority of the floodplain is considered unsafe for vehicles or people with all building types considered vulnerable to failure (H6 flood hazard classification).

12.3. Hydraulic categories

Hydraulic categories describe the flood behaviour by categorising areas depending on their function during the flood event, specifically, whether they convey large quantities of water (floodway), store a significant volume of water (flood storage), or do not play a significant role in either storing or conveying water (flood fringe). As with categories of flood hazard, hydraulic categories play an important role in informing floodplain risk management in an area. Although the three categories of hydraulic function are described in the *Floodplain Development Manual* (NSW Government, 2005), their definitions are largely qualitative, and the manual does not prescribe a method to determine each area.

The manual gives an indication of criteria for the quantification of flood storage areas. The manual defines flood storage areas as areas outside of the floodway which if completely filled with solid material, would increase peak flood levels by ‘more than 0.1 metres and/or would cause the peak discharge anywhere downstream to increase by more than 10 per cent’.

A range of methods have been developed that aim to define these areas such as Howells et al. (2003), encroachment and conveyance methods. In order to define hydraulic categories for the Hawkesbury-Nepean, a conveyance and encroachment assessment was undertaken with detailed testing in the hydraulic model. The conveyance assessment defines the areas of the channel that convey the majority of the flood flow and are characterised by high velocities and depths. The floodplain outside this area was then tested in the hydraulic model through encroachment assessment by increasing the roughness and checking that flood levels are not increased by more than 0.1m. An iterative encroachment assessment is then used to refine the floodway extent and reduce impacts (increased flood levels) if necessary.

The encroachment analysis found that while through Penrith the floodway in a 1 in 100 AEP event is largely confined to the river and immediate adjoining low-lying floodplain, at Windsor large areas of floodplain are important for conveying floodwater. It was therefore prudent to undertake further analysis of the floodway, which led to it being subdivided into a primary floodway and a secondary floodway.

Flood storage and flood fringe areas make up the remaining area of the floodplain. These were combined given the negligible flood fringe areas.

12.3.1. Primary floodway

12.3.1.1. Method

The primary floodway was identified as any area that is subject to high relative velocities. The flow width was modified in accordance with Albert et al. (2018) to remove areas of low velocity or flow that do not contribute to the floodway.

The following definitions were adopted to define the total flow width:

- the depth x velocity product (VxD) along the crossline must be greater than 0.2
- the VxD at a point along the crossline proportional to the VxD at the centre must be greater than 0.1.

Specifically, the primary floodway was defined as the area that conveys 80 per cent of the flow width defined above *and* where velocities are greater than 0.5 m/s.

12.3.1.2. Results

The primary floodway was calculated for the 1 in 100 and 1 in 500 AEP events.

In the 1 in 100 AEP event, the primary floodway is generally located within the main river channel. At Wallacia and Penrith, the primary floodway in both the 1 in 100 AEP and 1 in 500 AEP events does not extend beyond the low-lying overbank areas.

On the Windsor floodplain, the primary floodway for the 1 in 100 AEP event extends beyond the banks of Rickabys Creek and South Creek in small areas, but does not intersect existing residential or industrial development. The primary floodway of the 1 in 500 AEP event changes little from the 1 in 100 AEP event primary floodway, however it does extend further into the Windsor floodplain, particularly around the suburb of Mulgrave. The calculation and assessment of a primary floodway for the 1 in 500 AEP event did not result in any additional primary floodways developing.

12.3.2. Secondary floodway

12.3.2.1. Method

While the majority of the flow is contained in the primary floodway, large areas in the Richmond Lowlands and Windsor area convey a significant amount of the flow, and still meet the strict floodway definition, but are characterised by lower velocities. These areas are typified by deep wide areas of the floodplain that when considered collectively convey a significant amount of the flow. These areas are also very important for flood storage and have many of the characteristics of a traditional flood storage area and could be considered as the transition zone from floodway to flood storage. Standard encroachment analysis confirmed that blockage of these areas results in unacceptable increases in flood levels. Analysis also indicated that because of the relatively low velocities, the impact of small isolated obstructions such as individual buildings and farm sheds is small and very localised.

Velocity and depth results were adopted to indicate areas of higher flow conveyance. At each grid cell, the peak velocity (v), peak depth (d) and their product (VxD) was considered, and the cell was categorised based on the following criteria:

1. if VxD > 0.5 or
2. if both v and VxD > 0.2

The result of the above criteria was modified using engineering knowledge of the catchment characteristics to produce a continuous secondary floodway. This floodway was tested by increasing the Manning's 'n' roughness on all areas of the catchment outside of the secondary floodway to a value of 0.2. The increased roughness on all areas outside of the secondary floodway resulted in a flood level impact of less than 0.1 metres, confirming the adopted secondary floodway.

12.3.2.2. Results

Defining a floodway is difficult in areas where a large proportion of the flood flow is conveyed as deep, low velocity floodwaters. Small localised obstructions will only have a minor impact, but it is essential that the ability of these areas to convey significant flow is not reduced. These areas are also performing an important flood storage function. The analysis shows that any significant filling of the flood storage areas in the Richmond and Windsor areas will have a very broad impact on multiple suburbs.

At Penrith, nearly all the flow is contained in the river while in the Windsor area a significant amount of the flow is conveyed down the Richmond lowlands as relatively deep, low velocity flow. In a 1 in 100 AEP event at Penrith, approximately 99 per cent of the flow is contained within the river at a level just below the level required for major breaks outs at Penrith and Emu Plains to occur. The area north of Windsor which contains a large area of farmland conveys a significant amount of the flow. Downstream of Gronos Point, the floodway is once again largely confined to the river and adjoining low lying floodplain. In a 1 in 100 AEP the overbank in the Wallacia area is a floodway.

In the 1 in 500 AEP event, an additional floodway breaks out along Nepean Street, Emu Plains before joining Lapstone Creek and returning to the Nepean River floodway. The secondary floodway also widens at Wilberforce, north of Windsor, in the 1 in 500 AEP event, allowing for an additional floodway to develop, connecting to Currency Creek. Downstream of Sackville, there are only very minor differences between the floodways of the 1 in 100 AEP and 1 in 500 AEP events.

13. EVACUATION EVENTS

Figure 55 and Figure 56 depict scatter plots of time to rise versus probability for the complete ensemble of Monte Carlo events. It was not practical for the complete ensemble to be run by the evacuation model, so it was necessary to select a representative subset.

The traditional design modelling approach using ARR 1987 design inputs and methodology is shown. With this approach, only one event would be modelled for each quantile (for example, a 1 in 100 AEP event). Each event would share relative temporal and spatial rainfall characteristics, producing design events that appear as simple scaled versions of a single set of design inputs.

Optimising an evacuation strategy and transport network upgrades based on a set of similar, traditional design events could produce a strategy that is sub-optimal for all real events. The large Monte Carlo ensemble allows a robust optimisation. Rate of rise and peak flood level were identified as the best variables for selecting events as peak flood level determines locations requiring evacuation and rate of rise determines the size of the evacuation window. It was also recognised that the cutting of evacuation routes by local runoff would have a significant effect on evacuation and that this aspect could not be described by a flood characteristic at a single location.

The first step in selecting a representative sample was to investigate the probability of evacuation times for events with different peak levels. Each of the events were placed in a series of quantile bins and each bin was ranked with a probability distribution fitted to each bin. The 10 per cent, 50 per cent and 90 per cent evacuation times can be derived from fitting a log normal distribution to the quantile bins. The evacuation times are well described by a simple log normal fit. This allowed 72 sample events to be selected that could be characterised by their level probability and by their evacuation probability. The selected events are shown on Figure 55 and Figure 56.

Appendix M lists the sample of events used for evacuation modelling. Additional events can be selected for evacuation modelling to test the variability in evacuation risk across the floodplain.

14. LIMITATIONS AND NEXT STEPS

14.1. Limitations

This study has adopted best practice in flood estimation techniques. While every effort was made to ensure the model is an accurate representation of the catchment, there are limitations associated with the following aspects:

- the accuracy of the aerial LiDAR survey used for mapping flood depths
- limited in-bank bathymetry for developing maps of flood depths and provisional flood hazard mapping. Results presented within channel should be confirmed using detailed survey and modelling.
- the complex nature of the catchments upstream and downstream of the study area
- the accuracy of bridge, weir and other structure data
- the effect on flood behaviour of development and existing structures outside the project boundary were modelled with the best available information
- the assessment of climate change was based upon the best available information at the time of writing
- environmental flows are not included in the model
- the accuracy of the resultant flood levels is generally considered to be +/- 200mm. However, levels at Wallacia are likely to be more uncertain than this.

14.2. Hierarchy of results

The current study uses best practice to provide valley-wide flood levels and other flood characteristics that replace earlier flood estimates from the 1996 Flood Study (Webb, McKeown and Associates, 1996) and 1997 *Lower Hawkesbury River Flood Study* (AWACS, 1997). The changes in design flood levels from the earlier studies are generally small as they are based on the long flood record at Penrith and Windsor. The major change is that it is now possible to explicitly account for the probability of Warragamba Dam being drawn down prior to an event.

The current study provides boundary conditions that can be used in the development of more detailed flood models. These more detailed models may be two-dimensional models with fine grids that will better represent the local flood behaviour. The representative design events described within this report should be used as design events for these studies. Where more than one representative design event exists for a given AEP, depending on the location of interest, all events should be run through the detailed model and the results enveloped to create a 'design event' flood surface. Where other flood characteristics other than flood level are of interest, an alternate set of representative events may be required.

Locations where it is anticipated a local model will be developed include Penrith. Penrith is unusual as a 1 in 50 AEP flood event is contained in-bank with complex breakouts occurring at the 1 in 100 AEP event. A detailed two-dimensional model is required to accurately describe this behaviour.

The current study provides flood levels from Hawkesbury-Nepean River dominated flood events. Therefore, local catchments are modelled to have the same duration rainfall event as that which causes the highest flood levels in the main river. Shorter duration events are likely to result in higher flood levels within tributaries such as South Creek. Local flood studies will also be required in these locations.

14.3. Recommendations for the next phase of flood modelling

A roughly calibrated two-dimensional model was established for the purposes of providing velocity distributions for flood hazard and hydraulic categorisation of the floodplain. The development of GPU style models means that it is now possible to model the 200 kilometres stretch of river covered by the RUBICON model in two dimensions. However, given the size of the floodplain it is still only possible to develop a model with a grid cell size in the order of 20 metres with reasonable run times that will not inhibit a study program. The use of too fine a grid cell size, while providing more detailed mapping will mean that run times will be in the order of weeks and calibration will rely heavily on the modeller’s skill and educated guesses. A consistent set of detailed bathymetry of the river is required to inform the two-dimensional model.

15. CONCLUSIONS

This report provides an update to the publicly available flood information for the Hawkesbury-Nepean Valley which was previously undertaken in 1996. It provides detailed analysis of flood behaviour to assist floodplain managers.

As part of the 1996 Flood Study, a detailed hydrologic model (RORB) of the catchment was developed to convert rainfall into runoff. Extensive collection of flood information and river cross sections was undertaken. A hydraulic model (RUBICON) was developed to convert the runoff into flood levels. The models were extensively calibrated to historic flooding events including June 1964, June 1975, March 1978, August 1986, April/May 1988, July 1988, and August 1990. The model underwent a thorough peer review by a number of Australian and international experts. The models developed for the 1996 Flood Study form the basis for the current study.

The study uses best practice and the latest techniques in flood estimation to define flood behaviour in the Hawkesbury-Nepean Valley. A Monte Carlo framework was developed to run thousands of events that reproduce the variability observed in real floods. The framework allows for the assessment of rate of rise and other factors important for emergency management. Flood levels for the 1 in 100 AEP event have changed a minor amount due to the change in methodology and increased knowledge of flood behaviour since the 1990s. The levels of more frequent flood events have lowered because the modelling now accounts for dam drawdown.

An assessment of the impacts of rainfall increases and sea level rise due to climate change was undertaken. Increases in rainfall intensity due to climate change would result in a significant increase in flood levels, with a 9.1 per cent climate change rainfall increase causing the 1 in 100 AEP flood levels at Windsor to increase by 0.71 metres. A 1 in 100 AEP event at Windsor under existing climate would become a 1 in 80 AEP event with a 4.9 per cent rainfall increase and a 1 in 65 AEP event with a 9.1 per cent rainfall increase. The impacts of sea level rise on the 1 in 100 AEP event would be largely confined to the lower reaches of the catchment.

Provisional flood hazard and hydraulic categories were defined for a range of events. Nationally-accepted hazard categories for the 1 in 100 AEP event indicate that the majority of the floodplain is considered unsafe for vehicles and people, with buildings requiring special engineering design and construction, or being vulnerable to failure.

The study provides spatial maps of flood levels as a result of main river flooding. The study will be used by a range of stakeholders including councils within the valley and the NSW Government to inform flood planning and emergency management. The outputs of this Regional Flood Study will provide contemporary information on flood risk important for increasing community awareness of their flood risk and for building resilience. The study provides a range of representative events that can be used to derive detailed local models.

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GLOSSARY

Adapted from the *Floodplain Development Manual* (NSW Government 2005).

acid sulfate soils	Are sediments which contain sulfidic mineral pyrite which may become extremely acid following disturbance or drainage as sulfur compounds react when exposed to oxygen to form sulfuric acid. More detailed explanation and definition can be found in the NSW Government Acid Sulfate Soil Manual published by Acid Sulfate Soil Management Advisory Committee.
Annual Exceedance Probability (AEP)	The chance of a flood of a given or larger size occurring in any one year, usually expressed as a percentage. For example, if a peak flood discharge of 500 m^3/s has an AEP of five per cent, it means that there is a five per cent chance (that is one-in-20 chance) of a 500 m^3/s or larger event occurring in any one year (see ARI).
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level.
Average Annual Damage (AAD)	Depending on its size (or severity), each flood will cause a different amount of flood damage to a flood prone area. AAD is the average damage per year that would occur in a nominated development situation from flooding over a very long period of time.
Average Recurrence Interval (ARI)	The long-term average number of years between the occurrence of a flood as big as, or larger than, the selected event. For example, floods with a discharge as great as, or greater than, the 20 year ARI flood event will occur on average once every 20 years. ARI is another way of expressing the likelihood of occurrence of a flood event.
caravan and moveable home parks	Caravans and moveable dwellings are being increasingly used for long-term and permanent accommodation purposes. Standards relating to their siting, design, construction and management can be found in the Regulations under the LG Act.
catchment	The land area draining through the main stream, as well as tributary streams, to a particular site. It always relates to an area above a specific location.
consent authority	The Council, government agency or person having the function to determine a development application for land use under the EP&A Act. The consent authority is most often the Council, however legislation or an EPI may specify a Minister or public authority (other than a Council), or the Director General of DIPNR, as having the function to determine an application.
development	Is defined in Part 4 of the Environmental Planning and Assessment Act (EP&A Act). **infill development:** refers to the development of vacant blocks of land that are generally surrounded by developed properties and is permissible under the current zoning of the land. Conditions such as minimum floor levels may be imposed on infill development.

	new development: refers to development of a completely different nature to that associated with the former land use. For example, the urban subdivision of an area previously used for rural purposes. New developments involve rezoning and typically require major extensions of existing urban services, such as roads, water supply, sewerage and electric power. **redevelopment:** refers to rebuilding in an area. For example, as urban areas age, it may become necessary to demolish and reconstruct buildings on a relatively large scale. Redevelopment generally does not require either rezoning or major extensions to urban services.
disaster plan (DISPLAN)	A step by step sequence of previously agreed roles, responsibilities, functions, actions and management arrangements for the conduct of a single or series of connected emergency operations, with the object of ensuring the coordinated response by all agencies having responsibilities and functions in emergencies.
discharge	The rate of flow of water measured in terms of volume per unit time, for example, cubic metres per second (m^3/s). Discharge is different from the speed or velocity of flow, which is a measure of how fast the water is moving for example, metres per second (m/s).
ecologically sustainable development (ESD)	Using, conserving and enhancing natural resources so that ecological processes, on which life depends, are maintained, and the total quality of life, now and in the future, can be maintained or increased. A more detailed definition is included in the Local Government Act 1993. The use of sustainability and sustainable in this manual relate to ESD.
effective warning time	The time available after receiving advice of an impending flood and before the floodwaters prevent appropriate flood response actions being undertaken. The effective warning time is typically used to move farm equipment, move stock, raise furniture, evacuate people and transport their possessions.
emergency management	A range of measures to manage risks to communities and the environment. In the flood context, it may include measures to prevent, prepare for, respond to and recover from flooding.
flash flooding	Flooding which is sudden and unexpected. It is often caused by sudden local or nearby heavy rainfall. Often defined as flooding which peaks within six hours of the causative rain.
flood	Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or local overland flooding associated with major drainage before entering a watercourse, and/or coastal inundation resulting from super-elevated sea levels and/or waves overtopping coastline defences excluding tsunami.
flood awareness	Flood awareness is an appreciation of the likely effects of flooding and a knowledge of the relevant flood warning, response and evacuation procedures.
flood education	Flood education seeks to provide information to raise awareness of the flood problem so as to enable individuals to understand how to manage themselves and their property in response to flood warnings and in a flood event. It invokes a state of flood readiness.
flood fringe areas	The remaining area of flood prone land after floodway and flood storage areas have been defined.

flood liable land	Is synonymous with flood prone land (that is, land susceptible to flooding by the probable maximum flood (PMF) event). Note that the term flood liable land covers the whole of the floodplain, not just that part below the flood planning level (see flood planning area).
flood mitigation standard	The average recurrence interval of the flood, selected as part of the floodplain risk management process that forms the basis for physical works to modify the impacts of flooding.
floodplain	Area of land which is subject to inundation by floods up to and including the probable maximum flood event, that is, flood prone land.
floodplain risk management options	The measures that might be feasible for the management of a particular area of the floodplain. Preparation of a floodplain risk management plan requires a detailed evaluation of floodplain risk management options.
floodplain risk management plan	A management plan developed in accordance with the principles and guidelines in this manual. Usually includes both written and diagrammatic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives.
flood plan (local)	A sub-plan of a disaster plan that deals specifically with flooding. They can exist at State, Division and local levels. Local flood plans are prepared under the leadership of the State Emergency Service.
flood planning area	The area of land below the flood planning level and thus subject to flood related development controls. The concept of flood planning area generally supersedes the flood liable land concept in the 1986 Manual.
Flood Planning Levels (FPLs)	FPL's are the combinations of flood levels (derived from significant historical flood events or floods of specific AEPs) and freeboards selected for floodplain risk management purposes, as determined in management studies and incorporated in management plans. FPLs supersede the Astandard flood event@ in the 1986 manual.
flood proofing	A combination of measures incorporated in the design, construction and alteration of individual buildings or structures subject to flooding, to reduce or eliminate flood damages.
flood prone land	Is land susceptible to flooding by the probable maximum flood (PMF) event. Flood prone land is synonymous with flood liable land.
flood readiness	Flood readiness is an ability to react within the effective warning time.
flood risk	Potential danger to personal safety and potential damage to property resulting from flooding. The degree of risk varies with circumstances across the full range of floods. Flood risk in this manual is divided into three types: existing, future and continuing risks. They are described below. **existing flood risk:** the risk a community is exposed to as a result of its location on the floodplain. **future flood risk:** the risk a community may be exposed to as a result of new development on the floodplain.

continuing flood risk: the risk a community is exposed to after floodplain risk management measures have been implemented. For a town protected by levees, the continuing flood risk is the consequences of the levees being overtopped. For an area without any floodplain risk management measures, the continuing flood risk is simply the existence of its flood exposure.

flood storage areas Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. The extent and behaviour of flood storage areas may change with flood severity, and loss of flood storage can increase the severity of flood impacts by reducing natural flood attenuation. Hence, it is necessary to investigate a range of flood sizes before defining flood storage areas.

floodway areas Those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flows, or a significant increase in flood levels.

freeboard Freeboard provides reasonable certainty that the risk exposure selected in deciding on a particular flood chosen as the basis for the FPL is actually provided. It is a factor of safety typically used in relation to the setting of floor levels, levee crest levels, etc. Freeboard is included in the flood planning level.

habitable room **in a residential situation:** a living or working area, such as a lounge room, dining room, rumpus room, kitchen, bedroom or workroom.

in an industrial or commercial situation: an area used for offices or to store valuable possessions susceptible to flood damage in the event of a flood.

hazard A source of potential harm or a situation with a potential to cause loss. In relation to this manual the hazard is flooding which has the potential to cause damage to the community. Definitions of high and low hazard categories are provided in the Manual.

hydraulics Term given to the study of water flow in waterways; in particular, the evaluation of flow parameters such as water level and velocity.

hydrograph A graph which shows how the discharge or stage/flood level at any particular location varies with time during a flood.

hydrology Term given to the study of the rainfall and runoff process; in particular, the evaluation of peak flows, flow volumes and the derivation of hydrographs for a range of floods.

local overland flooding Inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.

local drainage Are smaller scale problems in urban areas. They are outside the definition of major drainage in this glossary.

mainstream flooding Inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary, lake or dam.

major drainage Councils have discretion in determining whether urban drainage problems are associated with major or local drainage. For the purpose of this manual major drainage involves:

- the floodplains of original watercourses (which may now be piped, channelised or diverted), or sloping areas where overland flows develop along alternative paths once system capacity is exceeded; and/or

- water depths generally in excess of 0.3 metres (in the major system design storm as defined in the current version of Australian Rainfall and Runoff). These conditions may result in danger to personal safety and property damage to both premises and vehicles; and/or

- major overland flow paths through developed areas outside of defined drainage reserves; and/or

- the potential to affect a number of buildings along the major flow path.

mathematical/computer models The mathematical representation of the physical processes involved in runoff generation and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.

merit approach The merit approach weighs social, economic, ecological and cultural impacts of land use options for different flood prone areas together with flood damage, hazard and behaviour implications, and environmental protection and well-being of the State's rivers and floodplains.

The merit approach operates at two levels. At the strategic level, it allows for the consideration of social, economic, ecological, cultural and flooding issues to determine strategies for the management of future flood risk which are formulated into Council plans, policy and EPIs. At a site specific level, it involves consideration of the best way of conditioning development allowable under the floodplain risk management plan, local floodplain risk management policy and EPIs.

minor, moderate and major flooding Both the State Emergency Service and the Bureau of Meteorology use the following definitions in flood warnings to give a general indication of the types of problems expected with a flood:

minor flooding: causes inconvenience such as closing of minor roads and the submergence of low level bridges. The lower limit of this class of flooding on the reference gauge is the initial flood level at which landholders and townspeople begin to be flooded.

moderate flooding: low-lying areas are inundated requiring removal of stock and/or evacuation of some houses. Main traffic routes may be covered.

major flooding: appreciable urban areas are flooded and/or extensive rural areas are flooded. Properties, villages and towns can be isolated.

modification measures Measures that modify either the flood, the property or the response to flooding. Examples are indicated in Table 2.1 with further discussion in the Manual.

peak discharge The maximum discharge occurring during a flood event.

probable maximum flood (PMF) The PMF is the largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation, and where applicable, snow melt, coupled with the worst flood producing catchment conditions. Generally, it is not physically or economically possible to provide complete protection against

	this event. The PMF defines the extent of flood prone land, that is, the floodplain. The extent, nature and potential consequences of flooding associated with a range of events rarer than the flood used for designing mitigation works and controlling development, up to and including the PMF event should be addressed in a floodplain risk management study.
probable maximum precipitation (PMP)	The PMP is the greatest depth of precipitation for a given duration meteorologically possible over a given size storm area at a particular location at a particular time of the year, with no allowance made for long-term climatic trends (World Meteorological Organisation, 1986). It is the primary input to PMF estimation.
probability	A statistical measure of the expected chance of flooding (see AEP).
quantile	A defined probability.
risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. In the context of the manual it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
runoff	The amount of rainfall which actually ends up as streamflow, also known as rainfall excess.
stage	Equivalent to Awater level@. Both are measured with reference to a specified datum.
stage hydrograph	A graph that shows how the water level at a particular location changes with time during a flood. It must be referenced to a particular datum.
survey plan	A plan prepared by a registered surveyor.
water surface profile	A graph showing the flood stage at any given location along a watercourse at a particular time.
wind fetch	The horizontal distance in the direction of wind over which wind waves are generated.