Swan and Helena Rivers Flood Study: Hydrology

Final report

Version 3

May 5, 2016

Department of Water, the Cities of Bayswater, Belmont and Swan are acknowledged as significant partners on this project.

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Contents

Glossary

Anabranching	Splitting or bifurcation of a river channel into a number of smaller, generally parallel channels.
Annual exceedance probability (AEP)	Estimate of the probability of a rainfall or flood event occurring in any given year. Typically measured as a percentage (e.g. X %), but other terminology such as 1 in X AEP or X year ARI have been used in the past.
Attenuation	The decrease in peak flow and increase in flood duration associated with the movement of a flood hydrograph along a river or through a storage such as a lake or a dam.
Australian Height Datum (AHD)	Vertical datum for elevation measurements – approximately equal to mean sea level.
Australian Rainfall and Runoff (ARR)	The national flood estimation guidelines for Australia, published by Engineers Australia. The previous version was released in 1987, and this project coincided with the partial release of the new version in December 2014.
AWAP	Gridded daily rainfall data over all of Australia covering the period 1900 to current. Provided by the Bureau of Meteorology and formally known as Australian Water Availability Product.
Baseflow	The volume of water which enters a stream after travelling through the soil profile. Not included as surface runoff.
CRC-FORGE	A statistical technique used to estimate rainfall intensity-frequency-duration up to an AEP of 0.05% (1 in 2,000).
Design flood	Hypothetical flood which has a defined annual exceedance probability.
Design rainfall	Hypothetical rainfall event which has a defined annual exceedance probability.
Flood frequency analysis	Statistical analysis of gauged flood data to estimate annual exceedance probabilities at a gauge location.
Flood hazard	A measure of the force associated with flood water at a point – typically defined as flood depth times flood velocity.

Flood routing	Process of transferring a flood hydrograph along a river channel, which results in a degree of attenuation based on the characteristics of the channel.
Flood study	Technical input to the floodplain management process which includes estimation of design flood flows, water levels, flood extents, flood hazard and flood damages.
Flood mapping	The process of determining the extent and depth of flood waters based on a defined flow rate.
Flow	The volume of water passing a defined point, typically measured in cubic metres per second (m^3/s).
Full supply level	The water level of a dam above which outflows commence.
Gauging station	Site where water level is measured and converted into streamflow using a rating curve.
GIS	Geographic Information Systems. Software and computer analysis techniques used to store, display and analyse geographic data.
GSDM	Generalised Short Duration Method. The procedure for estimating the short duration probable maximum precipitation depth.
GTSMR	Generalised Tropical Storm Method Revision. The procedure for estimating the long duration probable maximum precipitation depth in tropical regions.
GEV	Generalised Extreme Value distribution. A type of statistical probability distribution.
Initial loss	The volume of rainfall lost at the start of a storm event – typically represents interception by vegetation and wetting of the upper soil layers.
LiDAR	Light Detection and Ranging. A digital elevation model captured from aerial survey, typically by an aircraft or satellite.
LPIII	Log Pearson III distribution. A type of statistical probability distribution.

Monte Carlo simulation	Method of hydrological modelling where the model inputs (e.g. rainfall depth, loss rate, reservoir starting level) are sampled from probability distributions.
Pluviometer	Rainfall station which records rainfall at a sub-daily timestep (typically 6 minute timestep).
Pre-burst rainfall	Rainfall which occurs prior to the main rainfall burst in a design storm.
Probable maximum flood (PMF)	Estimate of the largest flood which can be reasonably expected to be generated from a particular catchment given the maximum rate of rainfall for that duration and season.
Probable maximum precipitation (PMP)	The hydro-meteorological estimate of the most intense rainfall that could theoretically be considered to occur over a catchment.
Proportional loss	An ongoing loss of rainfall throughout a storm event, measured as a percentage of the rainfall depth in each timestep.
Rainfall intensity-frequency-duration (IFD)	Estimates of the intensity of design rainfall over a range of storm durations and for a range of annual exceedance probabilities
Rating curve	Gauge-specific relationship which is used to convert gauged water level into streamflow.
RORB	Hydrological model used to estimate streamflow based on historic or design rainfall inputs.
Runoff	The volume of rainfall which falls on a catchment and runs off into the stream network – excludes losses from soil absorption, evaporation and other processes.
Runoff coefficient	Measure of the proportion of rainfall which is converted into runoff throughout the duration of a storm event.
SRTM	Shuttle Radar Topography Mission. A global scale digital elevation model with a resolution of 9 arc seconds (approximately 30 m).
SWMOD	A type of rainfall loss model developed and applied primarily to catchments in south-west Western Australia.

Executive summary

As part of a wider flood study for the Swan and Helena Rivers, the Eastern Metropolitan Regional Council (EMRC) in partnership with the Department of Water (DoW), City of Swan, City of Bayswater and City of Belmont commissioned this flood hydrology study. Stage One of the ‘Understanding and Managing Flood Risk in Perth’s Eastern Region’ project has received financial support from the Natural Disaster Resilience Program managed by the State Emergency Management Committee (Western Australia) under the Commonwealth Government’s National Partnership Agreement on Natural Disaster Resilience. The aim of the study was to provide design flood hydrographs for a range of annual exceedance probabilities which could be used in a subsequent hydraulic modelling study in order to determine flood levels, velocities and hazards.

The Swan Avon River catchment has a total area of 124,000 km^2, and stretches from the river mouth at Fremantle to the Goldfields region just west of Kalgoorlie. The catchment is hydrologically complex, and exhibits significant differences in land uses, soil types and runoff characteristics. Additionally, the catchment features a large number of lake systems and highly anabranched river channels, which have a significant influence on floods.

DoW maintains a network of streamflow and rainfall gauges across the catchment. These were typically commissioned from the 1970s onwards, and whilst they provide good spatial coverage of the catchment, it is clear from anecdotal evidence that there were a number of large floods in the years prior to the start of the gauge network. This means that streamflow gauge data alone is insufficient for the purposes of estimating design floods.

The primary focus of this project was the development, calibration and verification of a hydrological (RORB) model of the Swan Avon River catchment. The model was calibrated using three historic flood events, selected from the gauge data record. Modelling results demonstrated that these events could be reproduced well using reasonable model parameter values. A fourth historic event was used to validate the calibration results.

The model was then verified to the gauged record of flood frequency estimates at two gauges; the Swan River at Walyunga and the Helena River at Craignish. At Walyunga, there was considerable uncertainty surrounding the estimation of the gauged flood frequency, especially when estimates of historic floods prior to the gauged record were included. Nevertheless, it was found that the model results were consistent with the gauged information as well as anecdotal evidence on flood behaviour within the catchment.

The model was then used in design mode to estimate flood peaks and flood hydrographs for a range of annual exceedance probabilities, including 10% (1 in 10), 5% (1 in 20), 2% (1 in 50), 1% (1 in 100), 0.2% (1 in 500) and 0.05% (1 in 2,000) AEPs. An estimate of the probable maximum flood (PMF), which is the largest flood which could conceivably be generated from this catchment, was also made. The design flood frequency curve at Walyunga is shown as Figure ES-1.

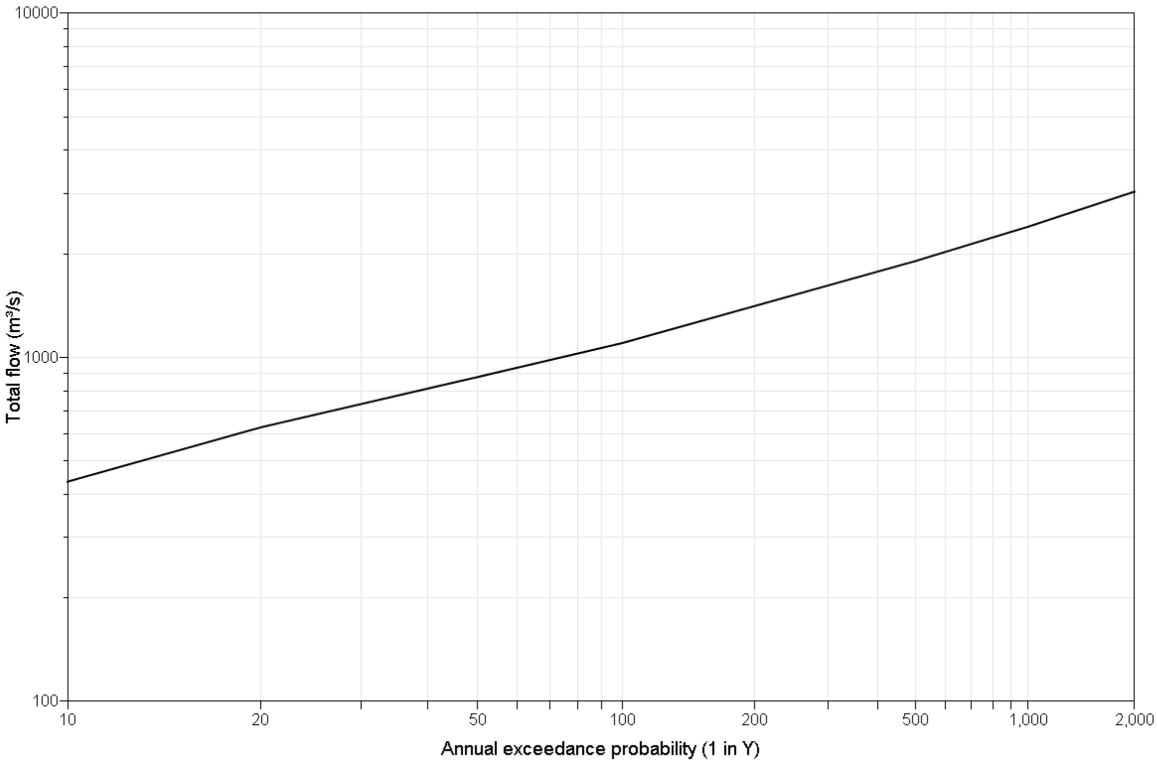


Figure ES-1: Design flood frequency curve at Walyunga

As noted previously, the results of the model calibration and verification process provide a considerable degree of confidence that the model is able to reproduce and extrapolate the historic flood behaviour of the catchment. As such, the results produced as part of this project are considered suitable for use in future hydraulic modelling studies. There are, however, a number of areas of additional work that could be considered as part of a future update to this project. These include:

- Incorporation of the latest guidance on design rainfalls and rainfall patterns which will shortly be available as a result of the revision of Australian Rainfall and Runoff;
- Further detailed modelling of the large lake systems in the upper reaches of the Lockhart and Yilgarn Rivers;
- Adoption of a more sophisticated modelling framework to better sample the space-time variability of design rainfalls over these upper catchments.

1. Introduction

This report summarises the analysis, modelling and results of the Swan and Helena Rivers Flood Study, which was commissioned by the Eastern Metropolitan Regional Council (EMRC) in August 2015 and completed in April 2016. The report summarises and explains the wider flood study process, the input spatial and hydrologic data used to undertake the investigation, analysis of that data and development, calibration and verification of the hydrological model. The modelling results, and modelled flood hydrographs for a range of design floods are also documented.

The Swan River is Perth's major waterway. Although it is a number of years since flood conditions were experienced on the Swan, the current 1% annual exceedance probability (AEP) flood extents used for planning and emergency preparedness and management are in need of review. The focus of this study is on the design flood hydrology for the Swan River, and the intention is use to latest available techniques and data sets to derive updated design flow estimates for a range of floods from the 10% AEP up to the probable maximum flood (PMF). These design flow estimates will then be used in a subsequent project focusing on the hydraulic modelling of the Swan River.

The current set of data and techniques used for design flood hydrology in Australia is in a state of flux. The national flood guidelines, Australian Rainfall and Runoff (ARR), is nearing the end of a multi-year update project funded by the Commonwealth Government. This is the first major update of ARR since the mid-eighties, and the revised version will encourage practitioners to use a number of modelling techniques developed since that time. In addition to this, hydrologic data sets such as rainfall intensity-frequency-duration data and design rainfall temporal patterns have been updated to account for the almost 30 years of additional recorded data since the original data sets were derived.

This report was prepared by Hydrology and Risk Consulting Pty Ltd (HARC). The results and conclusions expressed in this report have been based on data provided by EMRC, the Western Australian Department of Water (DoW) and some material accessed from the public domain. Should this data prove to be incomplete or inaccurate, the conclusions documented herein may change. HARC has undertaken reasonable checks on the quality and accuracy of the data provided for this project, but does not ultimately warrant its suitability for use.

Additionally, the complex hydrological issues at play in this catchment mean that this report should be read in full. All hydrological analysis and modelling is subject to a degree of uncertainty; some effort has been made throughout this report to document and quantify these uncertainties and it is important for the end users of the hydrological estimates made here to be aware of this. HARC accepts no liability for improper or imprudent use of the provided design flood estimates or any of the analysis or modelling undertaken to derive those estimates.

2. Overview of flood study

2.1 The flood study process and scope of this project

The flood study is a key component of the wider floodplain management process, which in Australia is defined and explained by *Managing the Floodplain: A Guide to Best Practice in Flood Risk Management in Australia* (Australian Attorney-General's Department, 2013). This document defines the components of a flood study as:

- Determining hydrologic aspects and varying flow over time
- Determining hydraulic aspects, including water levels and velocities as they vary with time
- Understanding varying flood (or hydraulic) function within the floodplain
- Understanding varying flood hazard within the floodplain
- Assessing the scale of potential impacts of floods on the existing community
- Assessing the potential impacts of floods on areas of the floodplain that may be considered for future development
- Understanding the potential impacts of climate change on flooding and the community.

Importantly, *Managing the Floodplain* also stresses the need to consider the full range of floods. Historically, the 1% annual exceedance probability (AEP) flood has been the focus of floodplain management in Australia. It is now recognised that floodplain managers need to consider floods which are both smaller and significantly larger than this event to fully understand the behaviour and implications of floods.

This study can be seen as the first phase in a wider flood study of the Swan River, as it deals primarily with the first component only, "determining hydrologic aspects and varying flow over time". The results from this project are intended to be used as inputs into future studies which will consider the remaining components such as flood mapping and flood hazard and damages assessment.

The scope of this project was therefore to use analytical tools and techniques to estimate the flood flows which would enter the study area under a range of design conditions. Design conditions in this case refer to theoretical floods whose probability of occurring can be estimated from reference to the historic streamflow gauging record. This project focused on estimating flow hydrographs with AEPs of 10%, 5%, 2%, 1%, 0.2%, 0.05% and the probable maximum flood (PMF). Consideration has also been given to estimating the possible increases in flood magnitude as a result of climate change.

2.2 Study area

In any flood hydrology study, it is key to understand the points at which the design flows are being derived, as this affects the subsequent estimation of design rainfall intensities and other inputs. For this project, the study area is defined as the reach of the Swan River from the Walyunga gauging station to the city of Perth. The study area also includes inflows from Ellen Brook at the Railway Parade gauging station and the Helena River at the Craignish gauging station. The study area is shown in Figure 2-1.

Riverine flooding, caused by heavy or prolonged catchment rainfall events, is the dominant mechanism of flooding within the study area. A recent study prepared for the Department of Water has shown that estuarine flooding, a form of major storm/tidal surges in the ocean produced from low pressure systems and increased wind velocities acting on the water, is the dominant flooding mechanism downstream of Perth city.

2.3 Key stakeholders

This project was commissioned by the EMRC in partnership with DoW, City of Swan, City of Bayswater and City of Belmont. The 'Understanding and Managing Flood Risk in Perth's Eastern Region' project has received financial support from the Natural Disaster Resilience Program managed by the State Emergency Management Committee (Western Australia) under the Commonwealth Government's National Partnership Agreement on Natural Disaster Resilience.

DoW are the floodplain management authority for Western Australia. DoW also own and operate the state's network of streamflow and rainfall gauges. For this project, DoW provided background information, previous studies, rainfall and streamflow data and technical oversight of the project.

Independent technical review of this project was undertaken by Associate Professor Rory Nathan of the University of Melbourne. Associate Professor Nathan was engaged directly by EMRC as the independent reviewer and undertook periodic reviews of the work at a number of key milestones throughout the project.

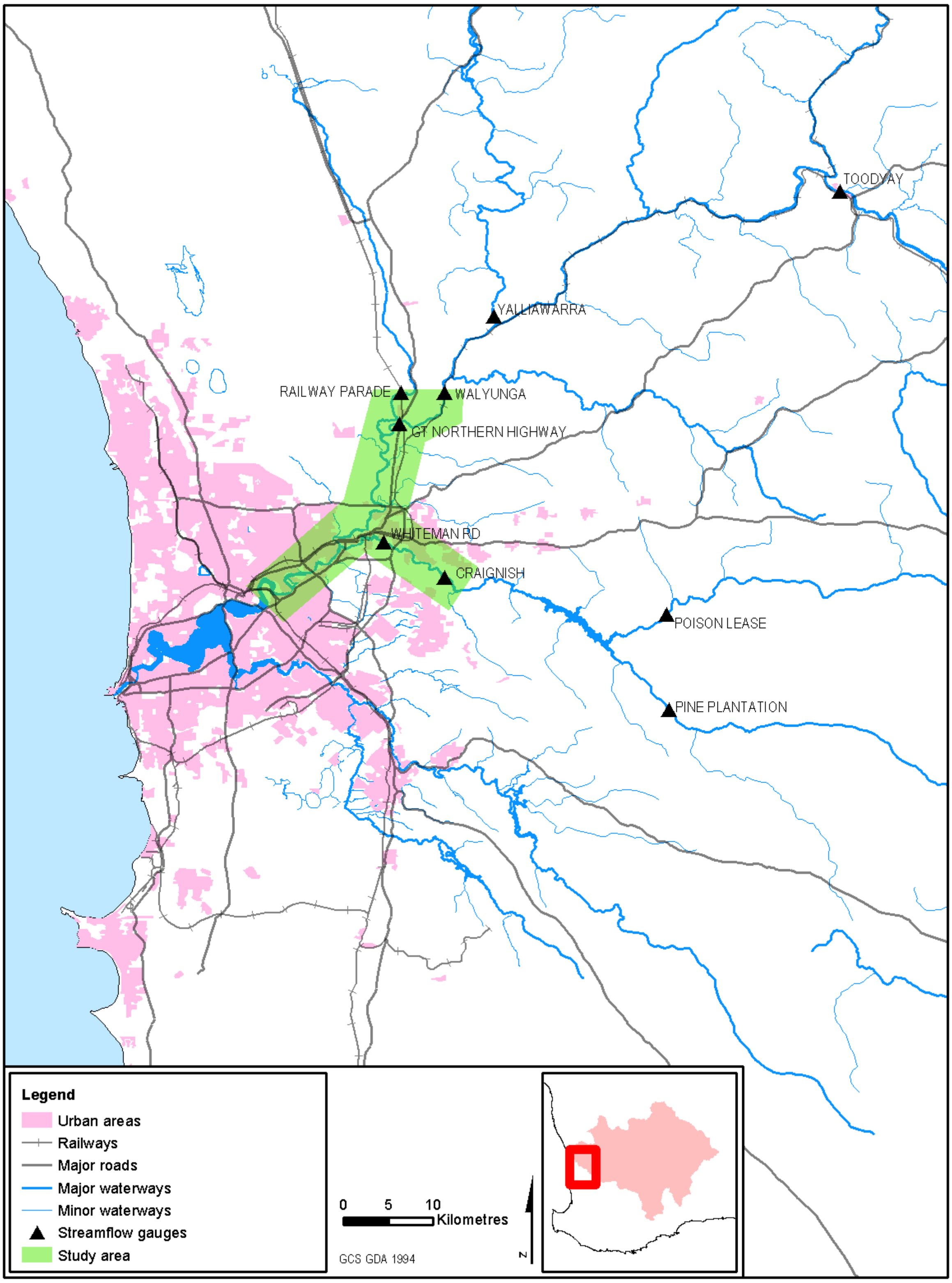


Figure 2-1: Swan and Helena Rivers flood study area

3. Catchment and data review

3.1 Catchment overview

The total catchment area of the Swan Avon River system is approximately 124,000 km^2, making it one of the largest river basins in Western Australia. The catchment covers a large proportion of the south-western region of Western Australia, and runs from close to the town of Coolgardie, some 500 km east of Perth, to its outlet to the Indian Ocean at Fremantle. The catchment covers a wide range of hydrological regimes and land uses, including the relatively wetter, forested areas of the Darling Scarp, the Wheatbelt region and the semi-arid Goldfields region.

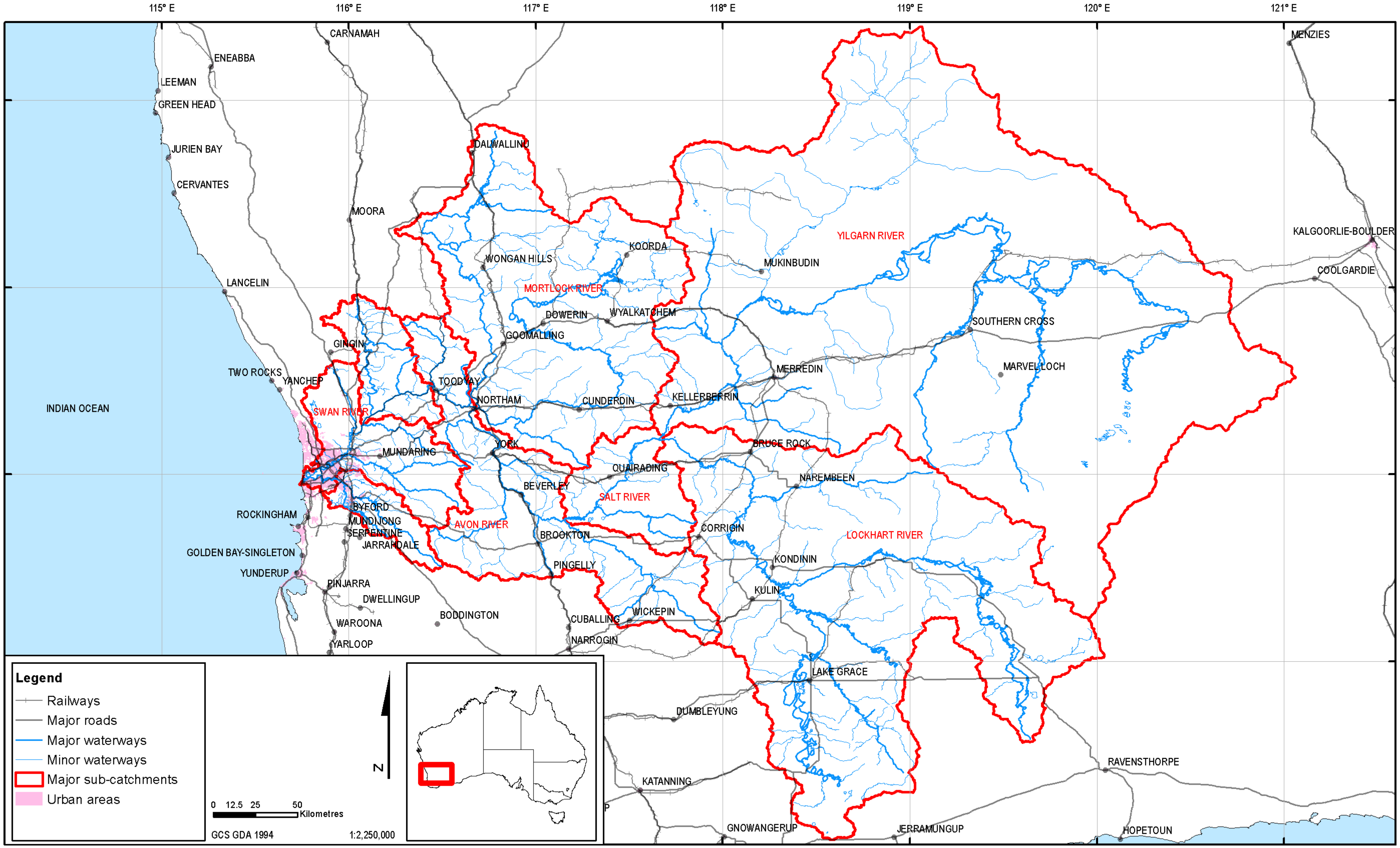
The most eastern upstream sub-catchments of the Swan River are the Yilgarn River (58,000 km^2) and Lockhart River (28,000 km^2). The confluence of these river systems near the town of Quairading forms the Salt River (3,000 km^2), which passes through the Yenyenning Lakes system before entering the Avon River. The Yilgarn, Lockhart and Salt River catchments are characterised by semi-arid climate and highly braided river systems. There are numerous lake complexes throughout this region, which act to attenuate (and often capture entirely) flood events moving through the system.

A significant tributary of the Avon River is the Mortlock River (17,000 km^2), which joins the Avon some 100 km downstream of the Salt River. Whilst not as highly braided as the Yilgarn and Lockhart Rivers, it still exhibits a degree of anabranching and features the presence of a number of lake systems. These include Lake Hinds, Lake Ninan and a large series of lakes around the Cowcowing region.

The Avon River sub-catchment itself (12,000 km^2) is a relatively steeper and more channelized stream. It passes through the upstream sections of the Darling Scarp, and as such is characterised by significantly higher average rainfalls than the upper sub-catchments. The river traverses through a steep gorge-like section which starts near Northam and runs down the Darling Scarp into Perth. Downstream of the Walyunga National Park and the confluence with Wooroloo Brook, the river becomes the Swan River.

From Walyunga, the Swan River passes through the outer eastern suburbs of Perth, becoming wider and adopting the characteristics of a tidal estuary. The river has a number of tributaries through this reach, including Ellen Brook, Susannah Brook and Jane Brook. Downstream of Guildford, a major tributary called the Helena River joins the Swan River. The Helena River has a catchment area of 2,000 km^2, predominately covering the forested areas of the Darling Scarp. Mundaring Dam is located on the Helena River and regulates a large proportion of flow from the catchment.

A locality map showing the entire Swan and Avon River catchment is included as Figure 3-1.



■ **Figure 3-1: Swan and Avon River catchment**

3.2 Recorded flood history and streamflow gauging

Perth has a long history of significant flooding dating back to mid-19th century. The flood of record in Perth occurred in 1872, with other large events occurring in 1862, 1945, 1955, 1963 and 1974 (Binnie, 1985). The most recent significant recent flood event in Perth occurred in 1983. Whilst these floods were all generated from heavy rainfall events over the Avon River catchment, the extent of the rainfall (and hence flooding) varies dramatically from event to event. Some events result in discharge through the various lake systems in the upper reaches of the Avon River catchment, whereas some do not.

The earliest streamflow gauging within the Swan River catchment was on the Helena River upstream of Mundaring Dam, which commenced in 1966. However, it was not until the mid-seventies that the first streamflow gauge commenced on the Avon River at Walyunga. Throughout the 1970s and 1980s a number of additional gauges were located throughout the catchment, both on the Avon River and a number of its key tributaries. There are pluviometers co-located with a number of these streamflow gauges, as well as a small number of stand-alone pluviometers.

It is important to note in the context of this project that although there is a reasonable (30 – 40 years) of gauged streamflow data on the Avon River, many of the larger historic floods occurred prior to the start of the gauged record. The largest event recorded at the Walyunga gauge was in July 1974, but historic and anecdotal evidence suggests this event was significantly smaller than the floods in the 19th century, 1945, 1955 and 1963. This has significant implications for how gauged data was used to estimate historic flood frequencies as part of this project.

DoW has been active in undertaking streamflow gaugings at their gauges. In general, well-defined rating curves exist at most of the gauges, and some have gaugings close to the flood of record.

The streamflow gauges used throughout this project are summarised in Table 3-1, and their locations are shown on the map in Figure 3-2. Note that this is not a comprehensive list of the available gauge locations, rather a summary of the key sites used in this project.

Table 3-1: Summary of selected streamflow gauges in the Swan Avon River catchment

Gauge name	Number	Stream	Total catchment area to gauge (km^2)	Period of record	Flood of record (m^3/s)	Highest gauged flow (m^3/s)
Gairdners Xing	615015	Yilgarn River	58,378	Feb 1976 – current	64.7 (Jun 1989)	22.0
Kwolyn Hill	615012	Lockhart River	28,398	Feb 1976 - current	83.8 (Jan 2000)	56.4
Qualandary Crossing	615022	Salt River	90,046	Apr 1982 – Sep 1998	33.3 (Jul 1989)	17.7
Boyagarra Rd	615063	Avon River	3,159	Apr 2007 – current	26.3 (Jul 2008)	7.6
Yenyenning Confluence	615029	Avon River	93,205	Mar 1997 - current	191.4 (Jan 2000)	181.1
Bells Farm	615047	Avon River	93,210	Nov 2008 - current	11.4 (Jul 2009)	4.4
Beverley Bridge	615025	Avon River	94,377	Jun 1995 – current	188.5 (Jan 2000)	180.1
Waterhatch Bridge	615027	Dale River	2,004	Jun 1995 – current	149.9 (Jul 1996)	16.5
Brouns Farm	615014	Avon River	96,488	Nov 1975 – Nov 2001	324.6 (Jul 1983)	224.9
Balladong St York	615024	Avon River	97,330	Jun 1995 - current	180.4 (Jan 2000)	102.2
Northam Weir	615062	Avon River	98,115	Jan 1977 - current	362.1 (Jul 1983)	354.9
O'Driscolls Farm	615020	Mortlock River East Branch	9,662	Apr 1975 - current	140.0 (Jul 1983)	40.8
Frenches	615013	Mortlock River North Branch	6,827	Jun 1975 - current	70.4 (Jul 1983)	58.3
Stirling Tce Toodyay	615026	Avon River	115,392	Jun 1995 - current	326.9 (Jan 2000)	266.6
Yalliawirra	616019	Brockman River	1,543	Apr 1975 – current	32.4 (Jul 1995)	31.1
Walyunga	616011	Swan River	118,761	May 1970 – current	652.6 (Jul 1983)	635.7
Gt Northern Highway	616076	Swan River	118,811	Jul 1996 - current	336.7 (Jan 2000)	276.2
Railway Parade	616189	Ellen Brook	795	Apr 1965 - current	58.8 (Jul 1987)	33.2
Pine Plantation	616002	Darkin River	672	Jun 1968 – current	36.3 (Aug 1974)	31.3
Poison Lease GS	616216	Helena River	593	May 1966 - current	26.1 (Jul 1974)	15.0
Craignish GS	616018	Helena River	1,602	Apr 1974 – current	107.3 (Aug 1974)	N/A
Whiteman Road	616086	Helena River	1,646	Jan 1988 – current	23.3 (Jul 2008)	21.6

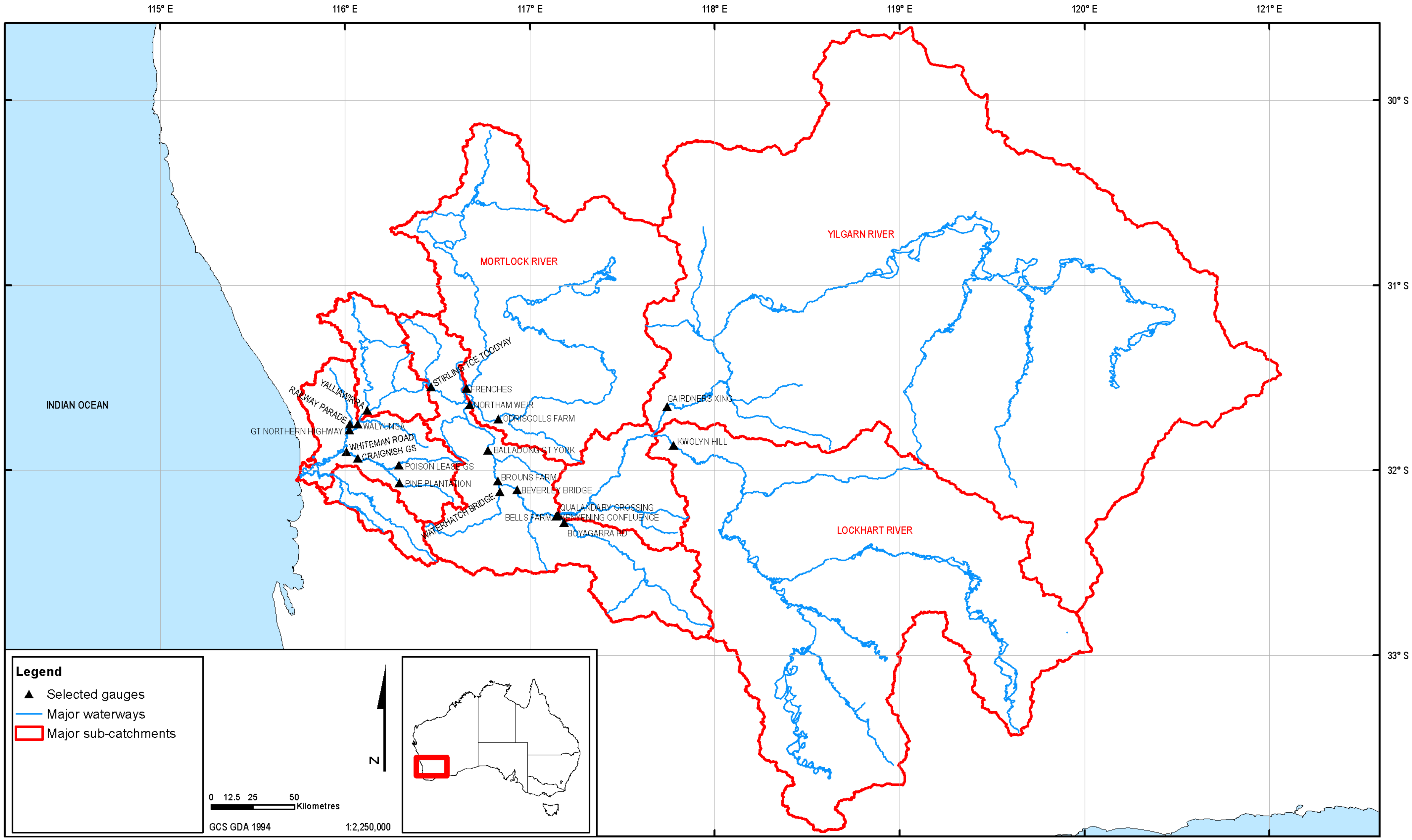


Figure 3-2: Selected Swan Avon River catchment streamflow gauges

3.3 Previous flood studies and investigations

A range of previous hydrological investigations and flood studies have been completed for the Swan Avon River and its tributaries. These studies have been reviewed to determine their use as part of the current project.

- Swan Avon Rivers Flood Study (PWD, 1978). Completed by the Public Works Department in 1978, the study is the earliest available estimate of design peak flood flows for the Swan River. The study undertook flood frequency analysis at a number of gauge locations as well as developing a daily Sacramento rainfall-runoff model to extend and infill the gauged data.
- Avon River Flood Study (Binnie, 1985). This study is the most comprehensive review of design flood hydrology for the entire Swan Avon River catchment. It made extensive use of gauged data and hydrological modelling to investigate design flow rates on the Avon River, and the key results are estimates of these peak flow rates. The report includes a comprehensive review of historic floods in the catchment, and some care has been taken to account for the relative contribution of the catchments upstream of the major lake systems.
- Avon River Flood Study 1985: Flood Hydrology (PWD, 1985). This study was conducted by the Public Works Department in conjunction with Binnie (1985), and features the development and calibration of a continuous simulation Sacramento model of the Avon River catchment. The study concentrated primarily on the catchment downstream of the major lake systems. It provides some useful reference points for design flood peak estimates along the Avon River.
- Ellen Brook Flood Study: Hydrology (PWD, 1987). Ellen Brook is a tributary of the Swan River, joining the river near Aveley. Although the catchment is a significant size in its own right (640 km^2), it is likely to only provide a very minor contribution to Swan River floods due to differences in critical storm duration. This study developed two hydrological models of the Ellen Brook catchment and then used them to determine design peak flow estimates.
- Mundaring Weir Extreme Flood Study (DoE, 2004a) and Lower Helena Pumpback Dam Extreme Flood Study (DoE, 2004b). These two studies are important references on the design flood hydrology of the Helena River. They were completed with the intention of providing extreme flood estimates for the purposes of dam safety, however included development of a RORB model for the Helena River to the dams and derivation of complete design rainfall frequency curves. An SWMOD loss model for the Helena River catchment was also developed as part of this project.

3.4 Topographic and spatial data

A range of topographic data covering the Swan Avon River catchment was provided by DoW for use in this project. The data includes aerial survey (LiDAR) data covering the Perth region, as well as lower resolution grids covering much of the catchment. The average horizontal resolution of the data is in the order of 10 m, and the data was provided as a number of tiles which were 'stitched' together to form a complete data set. As coverage of the catchment was not complete, the data was supplemented using publicly available Shuttle Radar Topography Mission (SRTM) data, which is available on a horizontal resolution of approximately 30 m.

Other spatial data provided for the project includes:

- Major and minor streams;
- Dams and lakes;
- Catchment and sub-catchments;
- Roads and railways;
- Streamflow and rainfall gauge locations;
- Catchment soil types.

Extensive aerial photography covering the catchment was also provided. Similarly to the topographic data, this data was provided in tiles with varying resolutions (most has a horizontal resolution of 50 cm) and acquisition dates.

4. Hydrologic data review

The Swan Avon River catchment presents a number of challenges in the application of hydrological analysis and modelling tools, primarily as a result of its size, but also due to the presence of highly anabranched channels and in-stream lake systems. There is a significant amount of recorded rainfall and streamflow data available for the catchment, and this data provides a valuable record of how the catchment has responded to historic storm events. Prior to attempting to model the catchment, this data was analysed and reviewed.

4.1 Rainfall data

Spatial variability of design rainfalls across the Swan Avon River catchment is a useful means to investigate the historic likelihood of flood inducing rainfalls. Gridded information on the spatial distribution of the new (2013) 168 duration, 1% AEP rainfalls was obtained from the Bureau of Meteorology and analysed. This is shown in the map in Figure 4-1, and it can be seen that the highest design rainfalls occur along the Darling Scarp, with a significant decrease in rainfall intensity moving from west to east.

In order to investigate the spatial extent of historic storms over the catchment, the Bureau of Meteorology's Australian Water Availability Product (AWAP) data was analysed. This data consists of daily rainfall grids covering the entire continent from 1 January 1900 onwards. The data was used to investigate the largest 7-day rainfall events occurring over the entire catchment within this period of record. The top ten largest catchment rainfall events were then selected and mapped, which demonstrated that in over 100 years of recorded data, heavy rainfalls over this catchment have almost always demonstrated partial area coverage of the catchment. It is also noteworthy that the majority of these events occurred in summer.

The map in Figure 4-2 shows the spatial distribution of rainfall over the catchment for these top 10 events. Note that the catchment boundary is shown in red, and the rainfall gradient is from light (low, less than 5 mm) to dark (high, greater than 100 mm) blue, with a constant scale used for all events. Each event in Figure 4-2 is referenced with its rank and the dates on which it occurred. It is clear that within these top ten storms, no single storm has achieved complete coverage of the catchment. Even those events which come closest (i.e. the rank 5 and 6 events) still leave considerable portions of the catchment with zero or negligible rainfall depths.

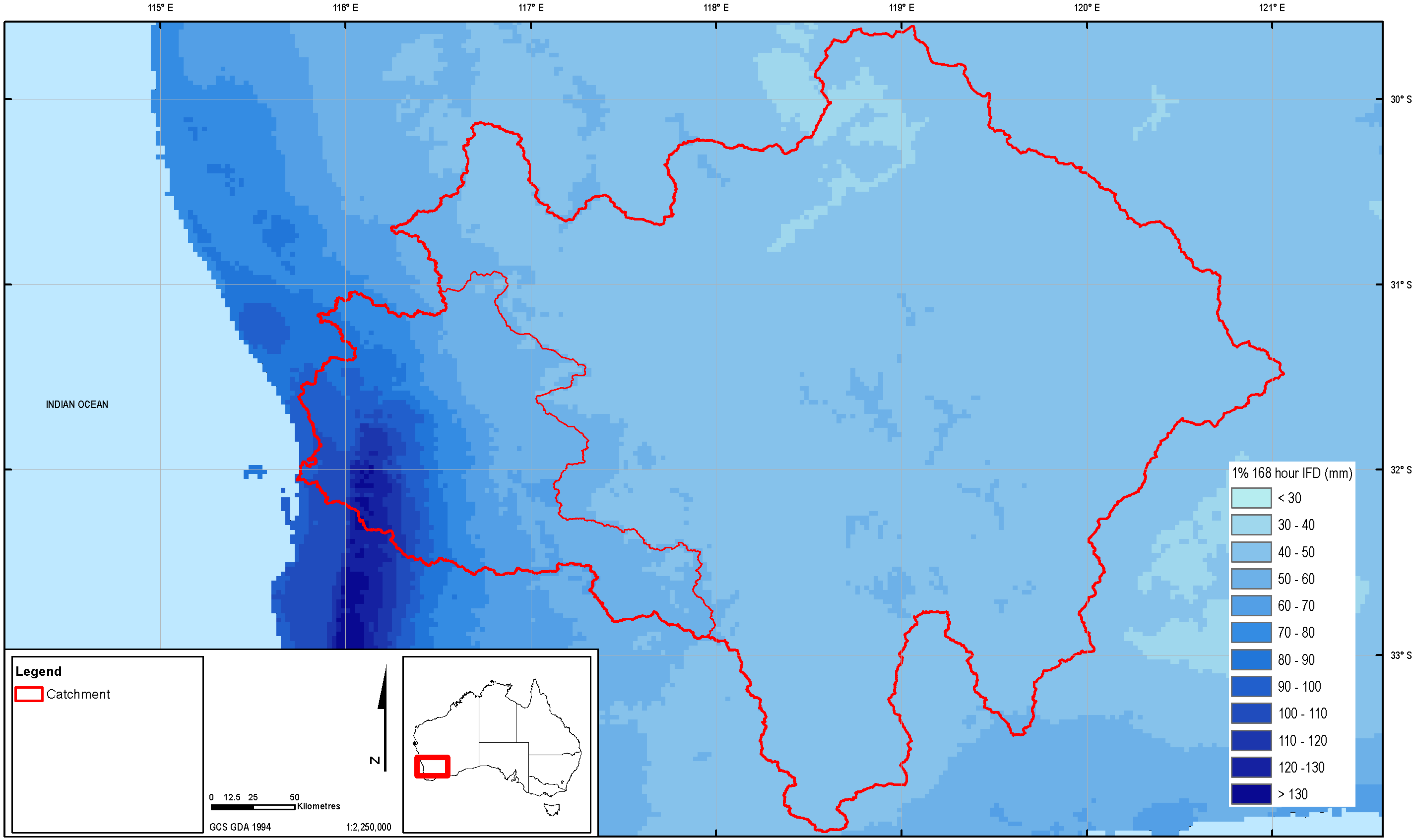
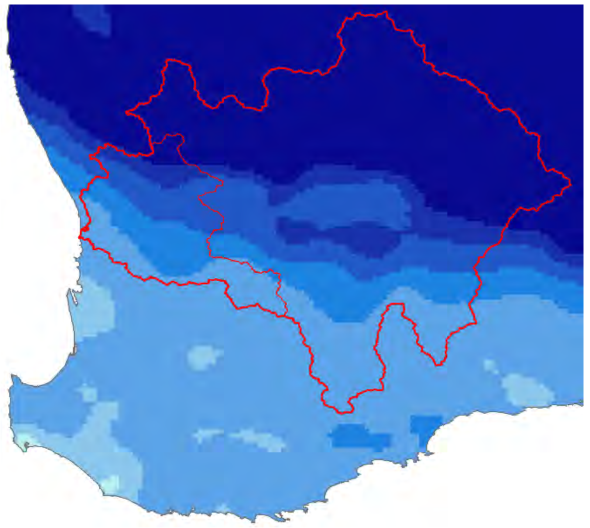
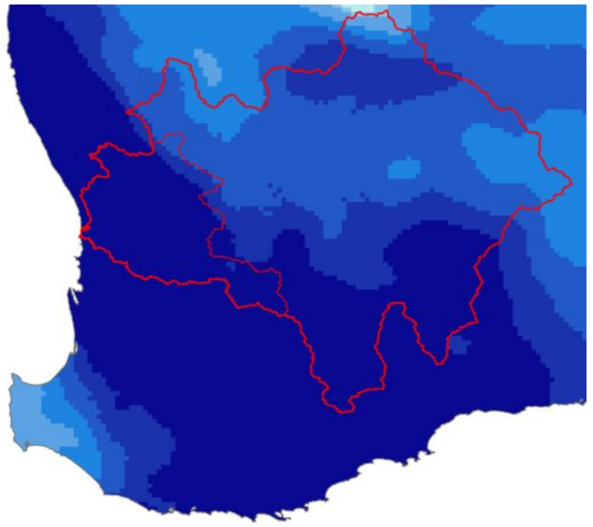


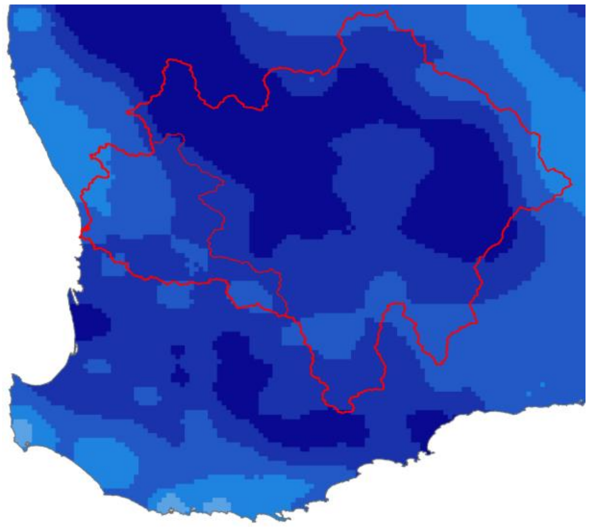
Figure 4-1: Spatial variability of design rainfall (1% AEP, 168 hours)



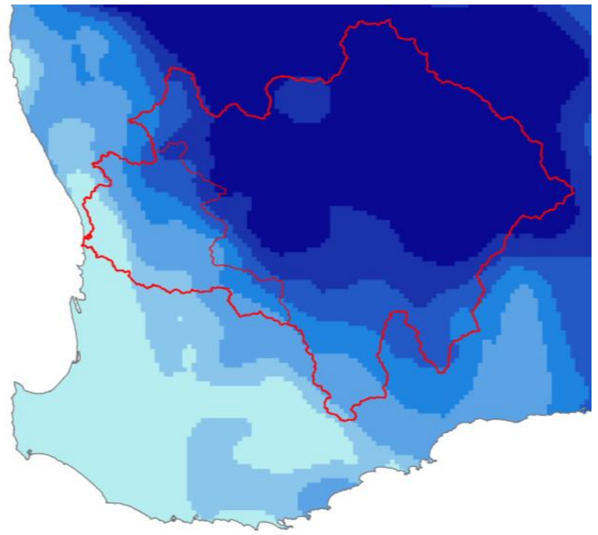
Rank 1: 18/03/1999 - 24/03/1999



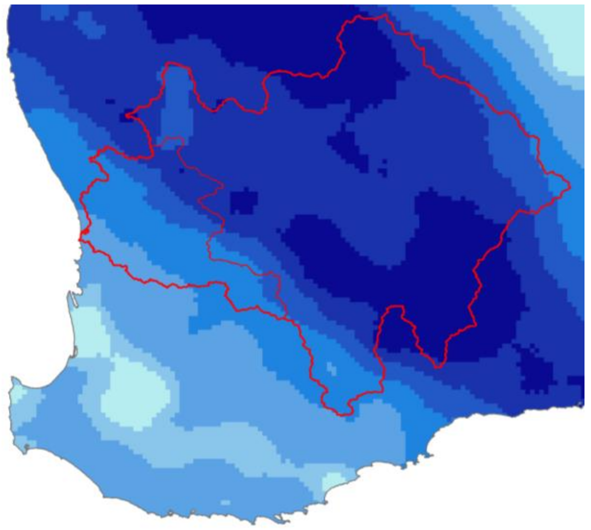
Rank 2: 12/02/1955 - 18/02/1955



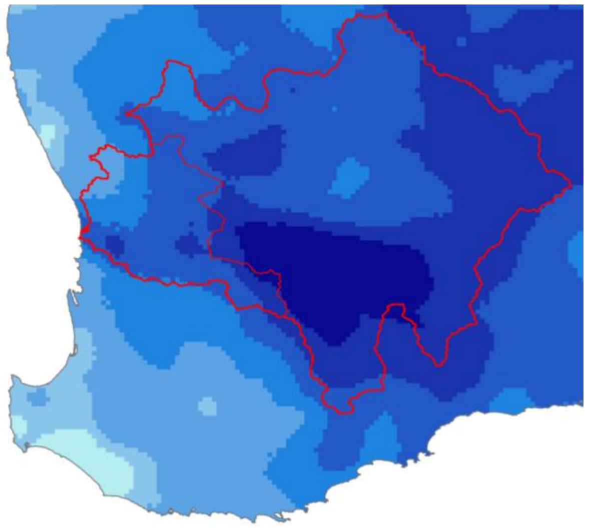
Rank 3: 23/03/1971 - 29/03/1971



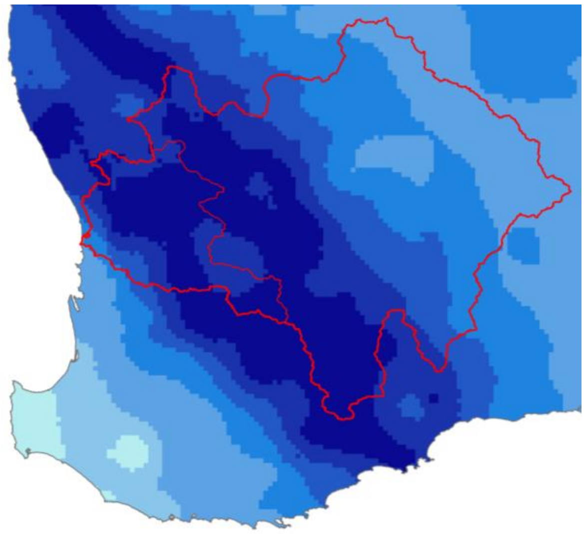
Rank 4: 22/03/1953 - 28/03/1953



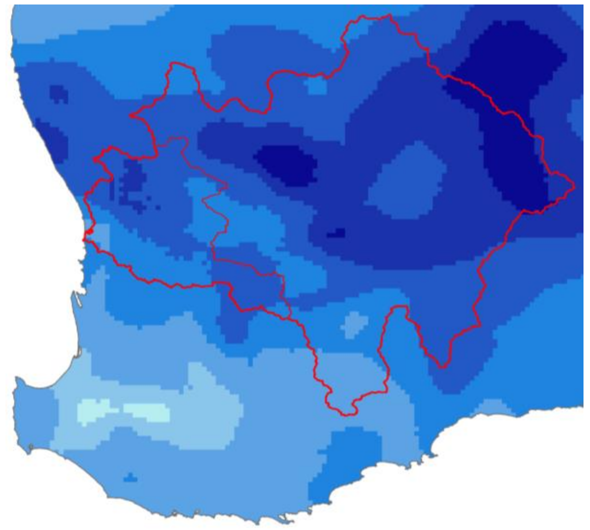
Rank 5: 12/02/1978 - 18/02/1978



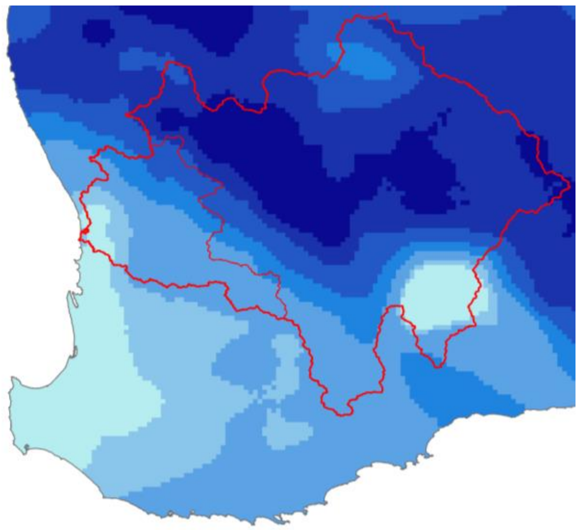
Rank 6: 21/01/2000 - 27/01/2000



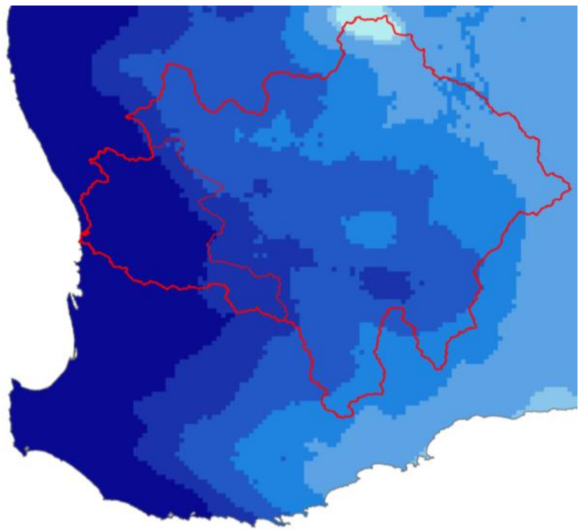
Rank 7: 23/01/1990 - 29/01/1990



Rank 8: 30/03/1974 - 05/04/1974



Rank 9: 16/03/1917 - 22/03/1917



Rank 10: 16/06/1945 - 22/06/1945

Figure 4-2: Top ten 7-day storms from AWAP data

4.2 Streamflow data

A wealth of streamflow data was provided and analysed for this project, representing gauged flood events recorded in the Swan Avon River catchment since 1970. Much of this data was subsequently used for calibration and verification of the hydrological model, however a high level overview of selected data is provided here.

4.2.1 Baseflow separation

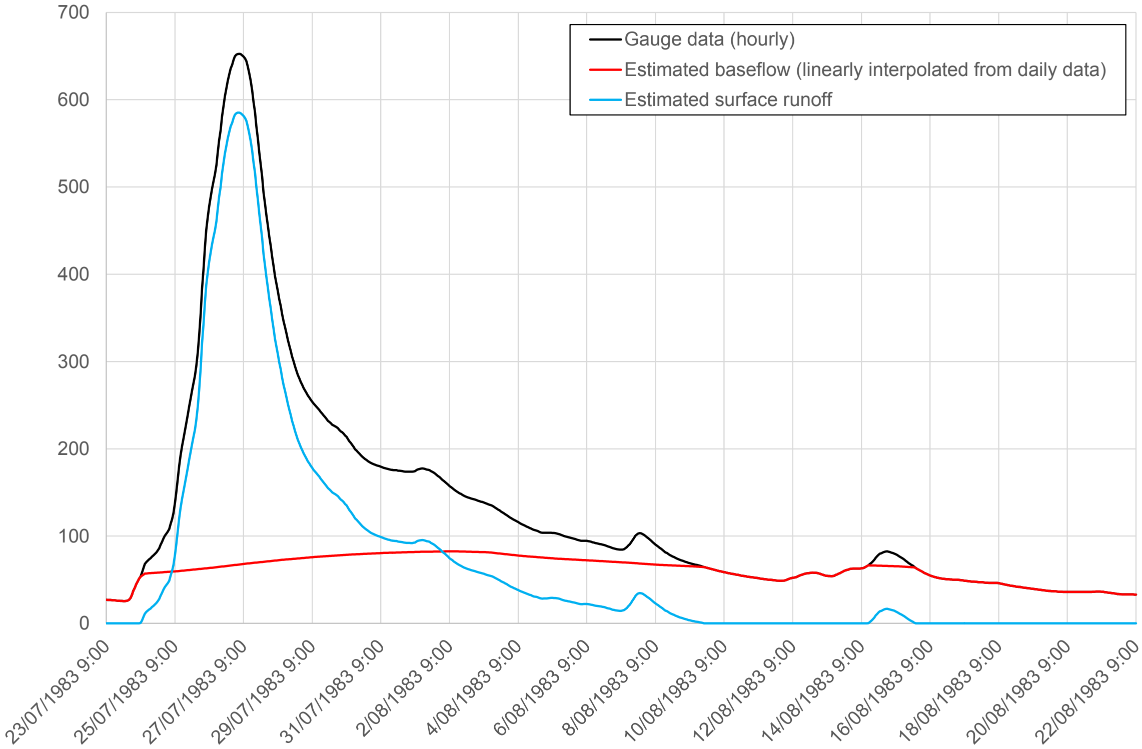
Prior to using the gauged streamflow data for modelling and analysis, it was necessary to quantify and separate out baseflow. Baseflow is the proportion of streamflow which results from the interaction of river channels with unsaturated and saturated groundwater zones. Baseflow is generated from a proportion of storm rainfall which does not convert to direct surface runoff, and tends to peak on the falling limb of the surface runoff hydrograph. As the RORB rainfall-runoff model used for this project does not simulate baseflow, the calibration and verification of the model must be undertaken by comparison with surface runoff data only.

There are a range of techniques which can be used to separate baseflow, including manual estimation based on a flood event hydrograph and application of a digital filter to either an event hydrograph or longer term gauge record. The suitability and applicability of these techniques is discussed at some length in the hydrological literature, particularly in Nathan and McMahon (1990) and Australian Rainfall and Runoff Revision Project 7: Baseflow for Catchment Simulation Stage 1 Report (SKM, 2009).

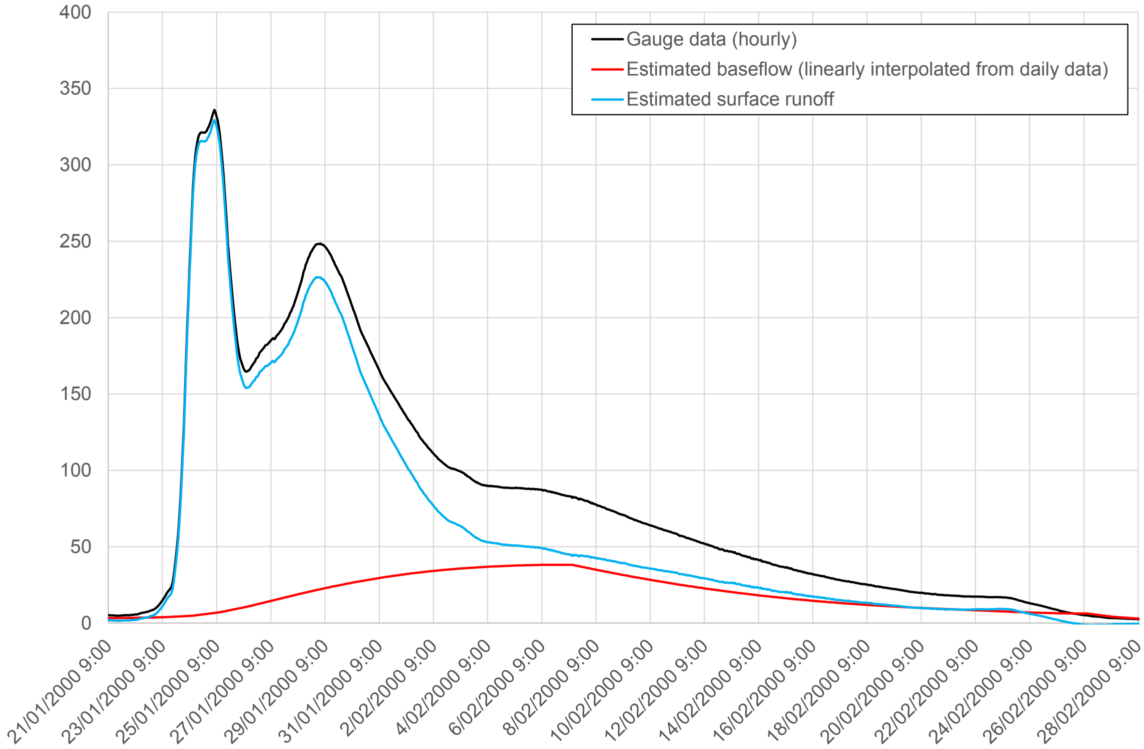
The issue of baseflow separation for key streamflow gauges in the Swan Avon River catchment is complicated by the large size of the catchment and consequent presence of partial area hydrographs for much of the gauging record. At the Walyunga gauge, many of the hydrographs for the largest flood events demonstrate numerous peaks in streamflow clustered around a single dominant peak. These smaller peaks may be the result of either local runoff from the area immediately upstream of the gauge, or runoff which occurred a number of days ago in the upper reaches of the catchment. This makes the identification of the point on the hydrograph where surface runoff ceases and baseflow commences highly subjective. Additionally, for the few large events which have occurred in summer, there is minimal baseflow prior to the rising limb of the hydrograph.

Trials of the application of a range of baseflow separation techniques at different gauges within the catchment on both continuous daily data and specific flood event hourly data were inconclusive. It was found that baseflow estimation on the continuous daily record appeared to produce more reasonable results than attempts to separate baseflow on hourly data for specific flood event hydrographs. As such, the approach adopted for all gauges for this project was to apply the Nathan and McMahon (1990) digital filter to the daily data. The computed baseflow hydrographs were then linearly disaggregated to an hourly timestep and used to separate baseflow for the selected flood events adopted for calibration. Examples of baseflow separation at Walyunga for the July 1983 event and Great Northern Highway for the January 2000 event are shown in Figure

4-3 and Figure 4-4. It can be seen that for the July 1983 event, the digital filter appears to overestimate baseflow, particularly on the rising limb of the hydrograph. Part of the reason for this is that the hydrograph plotted in Figure 4-3 is actually the last in a series of flood peaks. Alternatively, the estimate of baseflow for the January 2000 event appears more reasonable.



■ **Figure 4-3: Baseflow separation at Walyunga, July 1983**



- **Figure 4-4: Baseflow separation at Great Northern Highway, January 2000**

4.2.2 Flood frequency analysis

Flood frequency analysis was a key component of this project, and was used to determine the recorded flood characteristics at a number of streamflow gauge locations. The flood frequency estimates were then used to ensure that the hydrological models developed during the course of this project were providing a reasonable representation of the recorded flood history at the gauging stations.

In general, the flood frequency analyses completed for this project were undertaken on annual maxima, and so were suitable for estimating flood quantiles rarer than the 10% AEP. Annual maxima were extracted from the gauging records at each site of interest, using the water year definition for Western Australia (April to March) from CRC-FORGE (Durrant and Bowman, 2004). Where there was missing data during the water year, the extracted annual maxima were checked to determine whether it was likely that the peak of the largest flood had been missed – this was typically not found to be the case, although some smaller peaks were censored from the analysis for this reason.

Fitting of a probability distribution to the annual maxima was undertaken using the Bayesian approach provided in the program TUFLOW FLIKE. This method is consistent with the approach recommended in the 2015 revision of Australian Rainfall and Runoff, and allows the advantage of being able to consider historic floods prior to the gauged record (where such information exists).

Although there are a large number of gauges within the catchment where this analysis could have been undertaken, the results presented here are limited to the gauges at Qualandary Crossing (615022), Walyunga (616011), Whiteman Rd (616086) and Craignish (616018). These locations were selected for analysis due to their location in the catchment and relative length and quality of gauged record. Further details on the flood frequency analyses at each site are described in the following sections.

4.2.2.1 Qualandary Crossing

The Qualandary Crossing gauge is located at the outlet of the Yenyenning Lakes on the Salt River, and is immediately upstream of the confluence of the Salt and Avon Rivers. The location of this gauge provides valuable information on the relative contribution of flow from the Salt, Yilgarn and Lockhart River catchments, particularly when compared with flood frequency analyses at gauges further downstream. The total catchment area upstream of this gauge is just over 90,000 km^2.

The gauged flow record provided by DoW commences in April 1982 and ceases in September 1998. There is a period of missing data, followed by water level gaugings only between May 2000 and the current day. To derive a complete record of gauged flow, the site rating curve was obtained from DoW and applied to the water level data from May 2000 onwards. This indicated that the lakes have not spilled over that time.

Unfortunately, other gauge records (primarily the gauge at Yenyenning confluence, which captures both the Salt and Avon River flows) indicate that the period of missing data between September 1988 and May 2000 included a significant outflow from the lakes, most likely the flood of record at this location. The Yenyenning Confluence gauge record for this event demonstrates two peaks – one presumably resulting from the upper Avon River and a second more attenuated peak which is likely to represent outflows from the Yenyenning Lakes (see Figure 4-5). This second peak is in the order of 74 m^3/s (after baseflow separation).

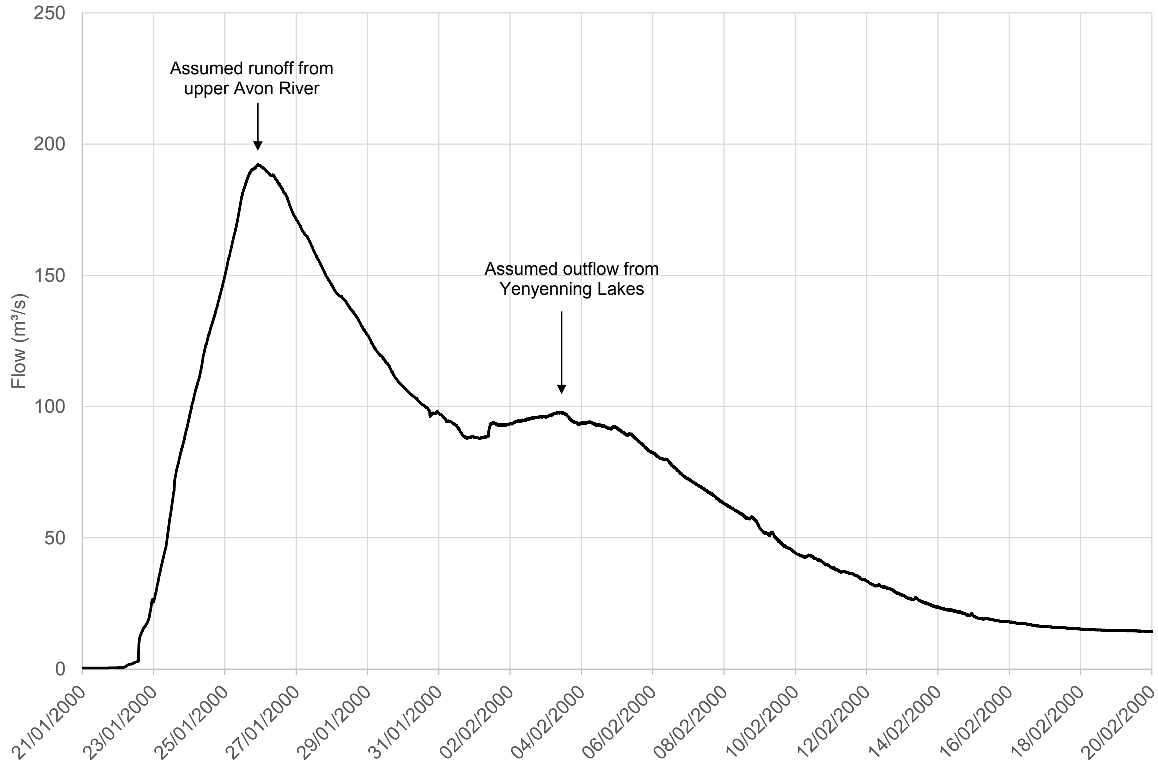


Figure 4-5: January 2000 event at Yenyenning Confluence gauge

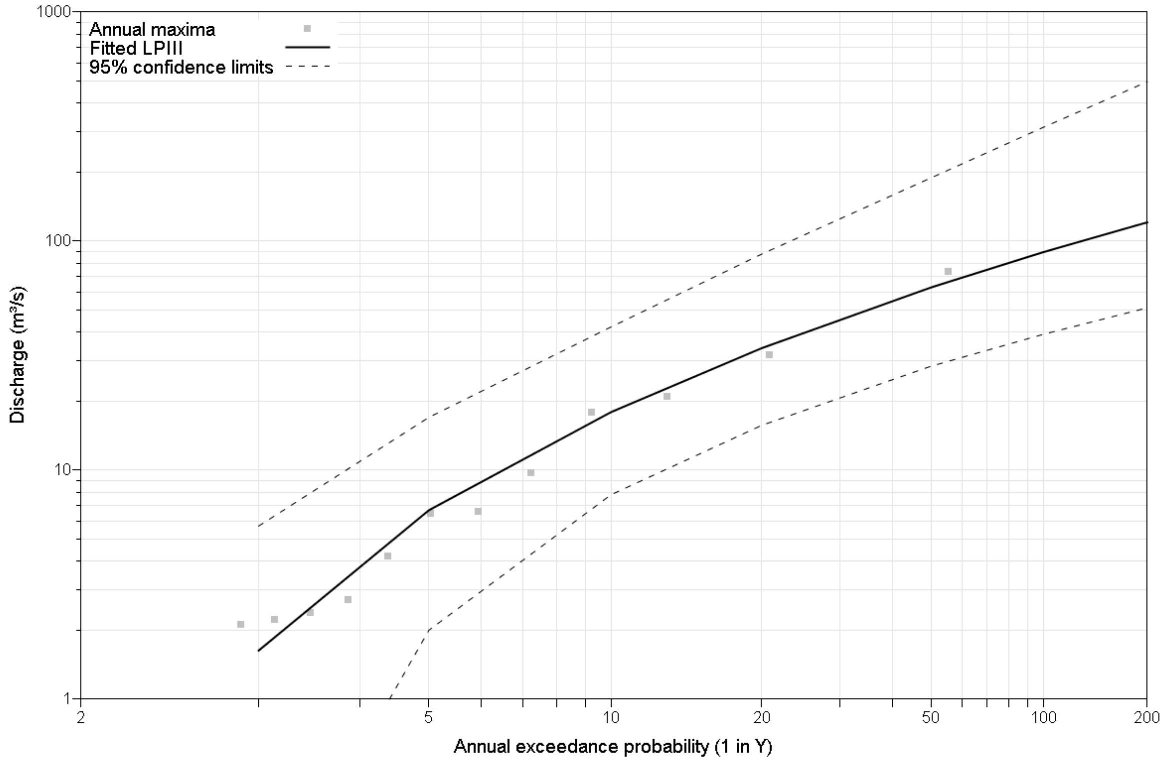
Over the period where gauged flows were available at Qualandary Crossing, annual and seasonal maxima were extracted. The estimated 74 m^3/s peak outflow was then added to this gauge record for January 2000, creating a composite, infilled gauge record of 33 years. The 10 largest floods at the gauge are shown in Table 4-1.

Table 4-1: Largest gauged floods at Qualandary Crossing

Rank (gauged floods only)	Event	Season	Gauged peak flow (m^3/s)	Estimated peak surface flow (m^3/s)
1	January 2000	Summer	98	74
2	July 1989	Winter	33	32
3	July 1983	Winter	22	21
4	August 1992	Winter	21	18
5	September 1998	Winter	10	10
6	August 1996	Winter	6.7	6.6
7	November 1995	Summer	6.6	6.5
8	July 1993	Winter	4.7	4.2
9	August 1986	Winter	3.5	2.7
10	July 1994	Winter	3.6	2.4

An LP3 probability distribution was fitted to the annual maxima at Qualandary Crossing using TUFLOW FLIKE. As noted previously, a Bayesian fitting technique was adopted, and the multiple

Grubbs-Beck test within FLIKE was used to identify and censor maxima. This test resulted in all non-zero maxima being included in the fit. The results are shown in Figure 4-6 and Table 4-2.



- **Figure 4-6: Annual flood frequency estimates at Qualandary Crossing**
- **Table 4-2: Summary of estimated flood quantiles**

AEP	Flood quantile estimate (m^3/s)
10%	18
5%	34
2%	63
1%	90

No attempt was made to undertake this analysis on a seasonal (summer/winter) basis, given the relatively short period of record and lack of definition of the annual maxima. However, it is clear from inspection of the largest peaks in Table 4-1 that the available record of the gauge is dominated by winter events, save for the January 2000 event.

The fitted frequency relationship at the Qualandary Crossing gauge is worth some consideration. It is highly unusual to have such low flood peaks generated from a catchment of over 90,000 km^2. Although it is arguable that the gauged record here has not captured a number of larger, historic floods, it is still remarkable that over a 30 year period the largest flow at this gauge is only estimated to be 74 m^3/s.

4.2.2.2 Walyunga

The Walyunga gauge is located at the bottom end of the Avon River, where it becomes the Swan River. It defines the upper end of the flood study project area, and as such is an important point of truth for the modelling completed in this project. The gauge has a total upstream catchment area of close to 119,000 km^2, and has 45 years of record with relatively little missing data.

At the Walyunga gauge, there is also historic evidence of flood peaks which have occurred prior to the start of the gauged record. These floods are an important input to any flood frequency analysis, as they are larger than any flood recorded at the gauge since records began in 1970. The Bayesian probability distribution fitting technique incorporated in TUFLOW FLIKE allows for these historic floods to be included in the frequency analysis, and as will be shown below, they significantly increase the estimates of design flood quantiles at this gauge.

These historic flood peak estimates were sourced from information provided by DoW (pers. comm. Damon Grace, DoW, 2/3/16) and are the result of previous studies which have attempted to estimate flows associated with historic floods using a hydraulic modelling approach. Whilst these historic records provide a useful additional database for flood frequency analysis, it must be acknowledged that there is considerable numerical uncertainty associated with these estimates and as such they should be treated with some caution. No attempt was made to estimate baseflow associated with these peak flows, as there was no information available on antecedent conditions or the shape and duration of the flood hydrographs. It was felt that any attempt to estimate baseflow would only add further numerical uncertainty to the estimates. It should also be noted that there is anecdotal evidence of even larger flood events in the Swan River from the 19th century – whilst there are no quantified estimates of flow associated with these floods, they would most likely further increase the estimated flood quantiles at this location.

The gauged and historic annual maxima are shown in Table 4-3 and Table 4-4.

Table 4-3: Largest gauged floods at Walyunga

Rank (gauged floods only)	Event	Season	Gauged peak flow (m^3/s)	Estimated peak surface flow (m^3/s)
1	July 1983	Winter	653	584
2	June 1981	Winter	364	362[1]
3	August 1974	Winter	455	327
4	July 1978	Winter	308	286
5	January 2000	Summer	285	279
6	January 1990	Summer	259	257
7	July 1975	Winter	266	247
8	July 1996	Winter	259	228
9	July 1995	Winter	256	215
10	August 1973	Winter	163	162[1]

[1] Missing data prior to the rising limb of the hydrograph prevented a reliable estimate of baseflow for these events

Table 4-4: Historic peak flow estimates at Walyunga

Rank (gauged and historic floods)	Event	Season	Estimated peak flow (m^3/s)
1	July 1926	Winter	1,100
2	July 1917	Winter	1,000
2	June 1945	Winter	1,000
3	June 1946	Winter	950[1]
4	August 1963	Winter	930
5	July 1958	Winter	800[2]
6	June 1964	Winter	750
7	February 1955	Summer	700
9	July 1967	Winter	500

[1] These values are considered approximate (pers. comm. Damon Grace, DoW, 2/3/16)

Prior to attempting to fit a distribution to these flood peaks, the stationarity of the data was investigated. Stationarity refers to the tendency for a statistical distribution to hold the same properties over time, and is a key assumption in flood frequency analysis that the input data is stationary. There are a range of factors which could void this assumption, including the effect of land clearing in a catchment over time, changing rainfall patterns and increasing urbanisation. To investigate this issue, the plotting position (effectively the AEP) of each gauged and pre-gauge flood peak was estimated assuming a length of record of 99 years (1917 to 2015). The flood peaks were then plotted on a log-Normal scale as shown in Figure 4-7.

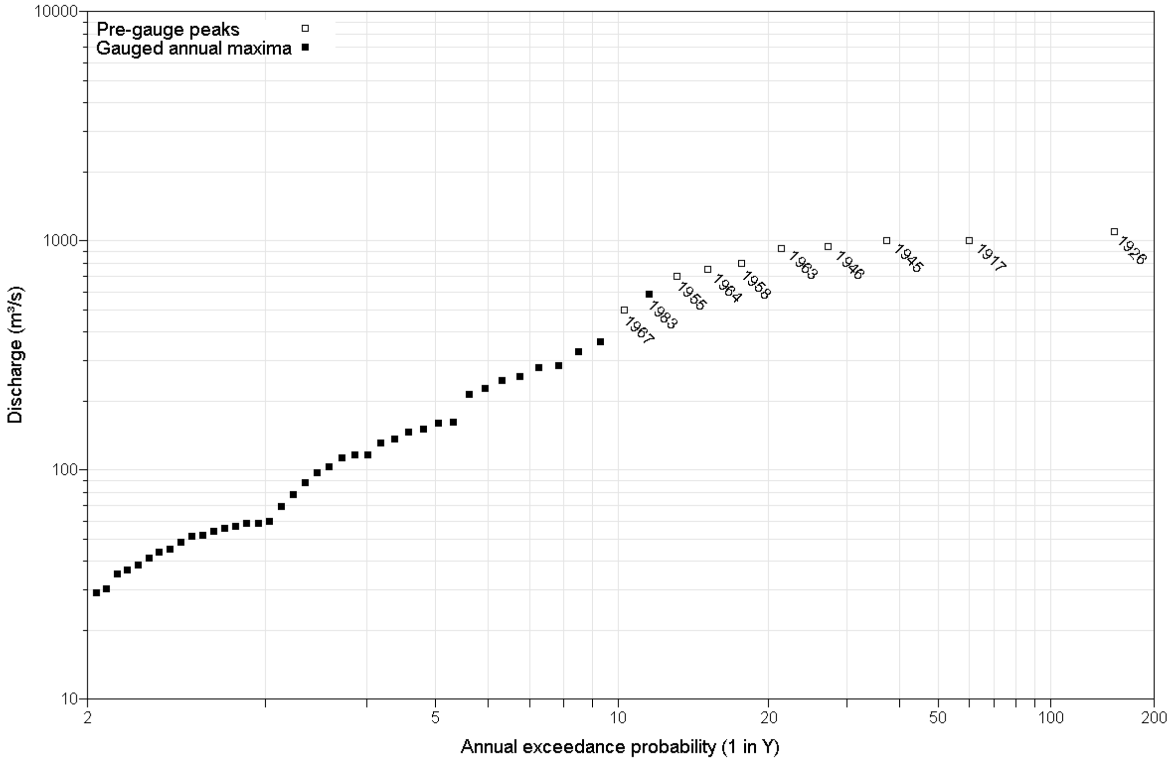


Figure 4-7: Walyunga gauged and pre-gauge flood peaks

It can be seen from the plotted peaks that there is a significant change in slope at around the 5% AEP. Prior to this, the flood peak magnitude increases with log-Normal AEP in a linear fashion, but beyond this there is virtually no increase in peak flow with AEP. This is a strong indication of non-stationarity in the data, which therefore makes it unlikely that a homogenous statistical distribution can be fitted. The causes of this non-stationarity are difficult to determine, but may be attributable to factors such as:

- the general decrease in rainfall observed in south-west WA since the mid-1970s;
- changing land use;
- systemic errors in the estimation of the pre-gauge flows.

Despite this, the data corresponds with anecdotal evidence that a number of very large floods occurred at Walyunga prior to the commencement of gauged records in 1970. It was therefore considered necessary to account for this pre-gauge data in some form, as it is likely that a flood frequency analysis fitted only to the gauge maxima would significantly underestimate the actual design flows. The key issue then became how to best make use of the pre-gauge data.

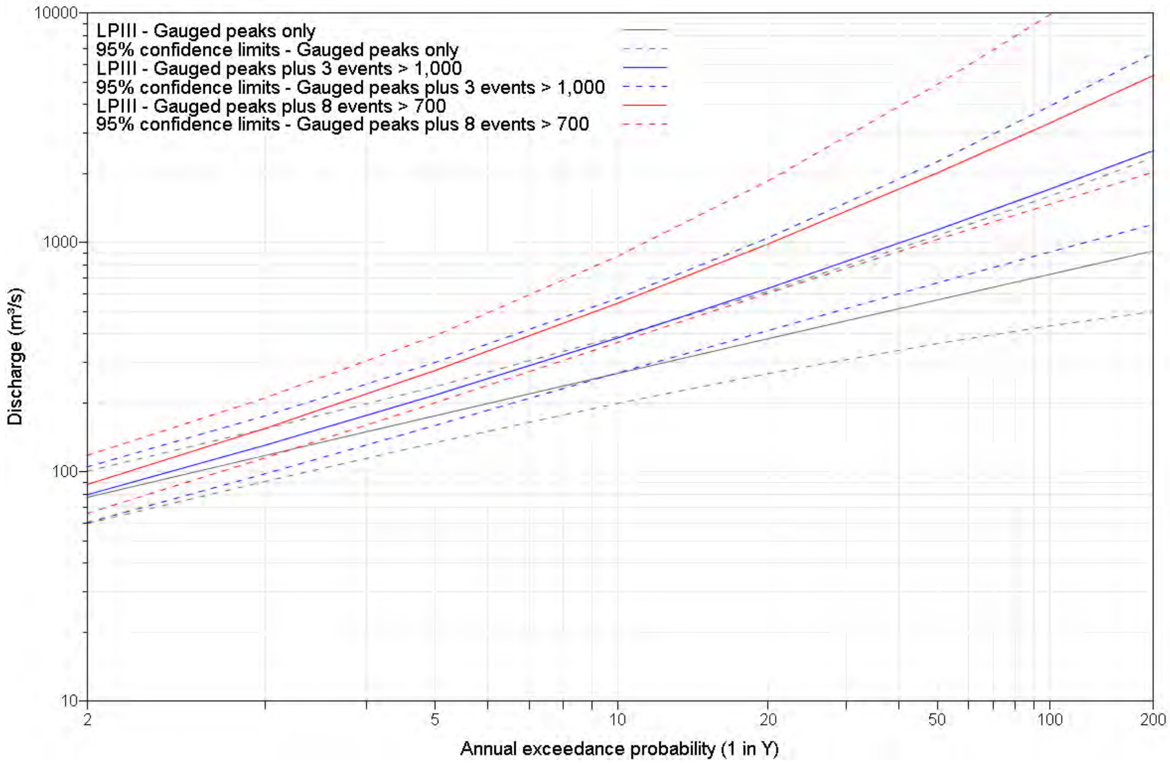
The flood frequency analysis program TUFLOW FLIKE allows for pre-gauge information to be input as a number of 'censored' flows. The user must input the numbers of floods larger than a certain threshold which have occurred over the pre-gauge period. All censored pre-gauge flows must be larger than the largest gauge maxima. For Walyunga, it would therefore be possible to specify the pre-gauge flows at (for example) three floods larger than 1,000 m^3/s between 1917 and 1969, or eight floods larger than 700 m^3/s between 1917 and 1969, or any combination thereof.

It was found that the inherent non-stationarity in this data set leads to significant variability in the fitted flood frequency curves depending on the method employed to include the pre-gauge events.

To highlight this issue, three flood frequency analyses were prepared using TUFLOW FLIKE. These three analyses all used the gauged annual maxima, but with different combinations of pre-gauged flows. All the analyses were undertaken using an LPIII distribution fitted using the Bayesian approach. The presence of low flows outliers was investigated using the multiple Grubbs-Beck test included in FLIKE, which revealed there were no outliers, so no low flow censoring was undertaken. The three analyses were undertaken on the following combinations of gauge and pre-gauge peaks:

- Gauged annual maxima only (ie no pre-gauge floods), shown in grey on Figure 4-8
- Gauged annual maxima plus three events larger than 1,000 m^3/s between 1917 and 1969, shown as blue on Figure 4-8
- Gauged annual maxima plus eight events larger than 700 m^3/s between 1917 and 1969, shown as red on Figure 4-8.

The results of this analysis are presented in Figure 4-8.



- **Figure 4-8: Walyunga flood frequency analyses**

Figure 4-8 demonstrates the very large degree of uncertainty generated from the inclusion of the pre-gauge peaks within the FLIKE flood frequency analysis. The estimate of the 1% AEP peak flow assuming no pre-gauge information is 730 m^3/s, however this increases to 1,700 m^3/s when three events larger than 1,000 m^3/s are included and 3,300 m^3/s when eight events larger than 700 m^3/s are included. This large variability is a direct result of using non-stationary data within FLIKE.

The overall implication of this analysis is that there is considerable uncertainty in the flood frequency estimates for the Walyunga gauge which limits its utility for determining the existing flood risk and verifying catchment hydrological models. Nevertheless, the verification of the Swan Avon River RORB model presented in Section 9.2 attempts to make best use of this data as a means of confirming the adopted RORB loss model parameter values as well as anecdotal evidence of the behaviour of the catchment.

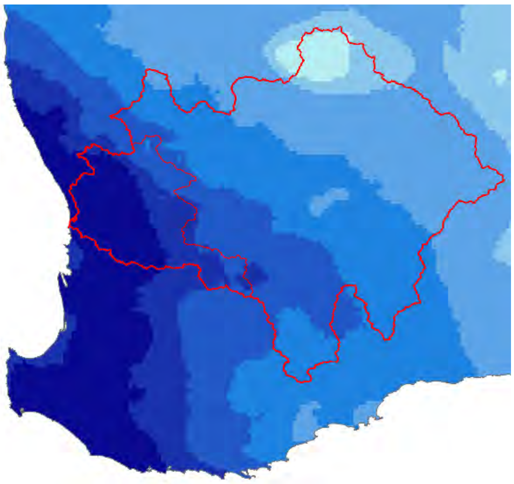
Further consideration was given to the spatial patterns associated with the rainfall events which generated the large floods prior to and within the gauge data record. AWAP daily rainfall data was extracted for the six historic flood events prior to 1970, and mapped to show the distribution of rainfall over the catchment. Similarly, maps were produced for the rainfall associated with the six largest flood peaks which have been recorded at the gauge from 1970 onwards. Both sets of maps are shown in Figure 4-9 and Figure 4-10 – note that a consistent scale has been used, making the coloured depth regions comparable across all maps. It can be seen that there is substantial variability in the spatial pattern and extent of these 12 storms; generally speaking the majority of the maps demonstrate the largest rainfall depths concentrated over the lower reaches of the catchment, however it is noteworthy that the largest events (such as the 1946, 1963 and 1926 events) all indicate significant rainfall depths over the upper reaches of the catchment as well.

The flood frequency analysis was also completed for summer only and winter only flood events, excluding the pre-gauge historic flood peaks given the uncertainties associated with them. The procedure for this was generally the same as for the annual analysis, save that seasonal maxima were extracted from the gauged record for summer (October to March) and winter (April to September). Some adjustment was required to these seasonal analyses to ensure that they were consistent with the overall annual analysis, particularly for the larger floods. The adopted annual and seasonal flood frequency curves are shown in Figure 4-11. It can be seen that winter floods dominate the majority of the frequency curve, with summer floods starting to become more influential for less frequent floods. Only one of nine recorded pre-gauge floods occurred in winter, so it is expected that this trend would also be observed if the pre-gauge floods were included in the analysis.

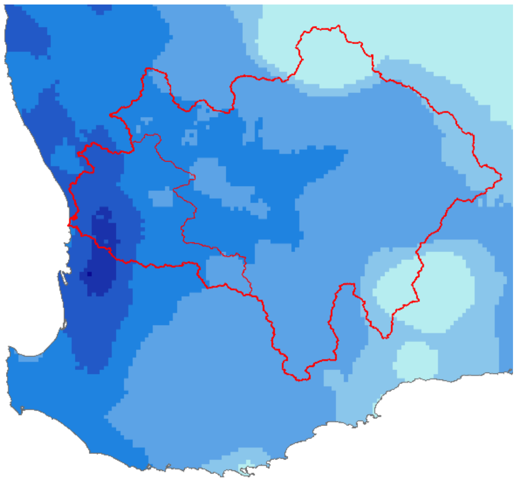
A comparison was also undertaken between the gauged frequency curves at Walyunga and Qualandary Crossing. This was done for two cases, a comparison of the curves using all available gauged data (i.e. pre-gauged floods were not included), and restricted to the period of common gauged record (1983 to current). This comparison was undertaken on an annual basis, and is shown in Figure 4-12. It can be seen from this plot that there is a significant difference between the flood frequency curves at Qualandary Crossing and Walyunga, regardless of whether the concurrent period is considered or not. This implies that for floods within the range of this analysis (10% to 0.5%) are predominantly generated from that portion of the catchment between these two gauges; conversely, the 90,000 km^2 catchment upstream of the gauge at Qualandary Crossing has minimal influence on floods at Walyunga.

The relatively low contribution of the catchment upstream of the Yenyenning Lakes presumably reflects a number of factors, in particular:

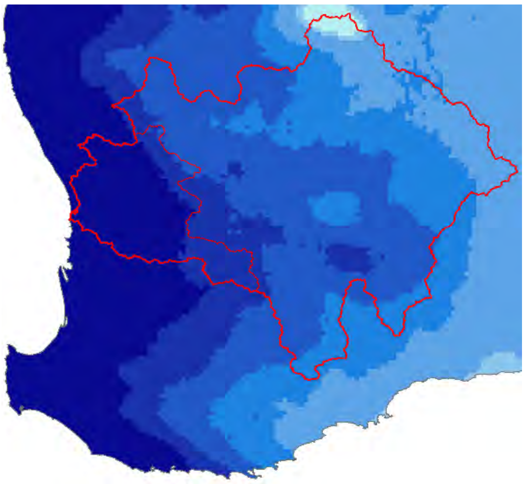
- The lack of flow gauging at Qualandary Crossing prior to 1983, meaning that a number of large floods which occurred prior to this were not recorded;
- The relative decrease in design rainfall intensity moving from west to east;
- The relative aridity of this catchment, and consequent higher rainfall losses;
- The presence of lakes and anabranched channels which act to store and attenuate floodwaters thus reducing the peak flow of any resulting runoff.



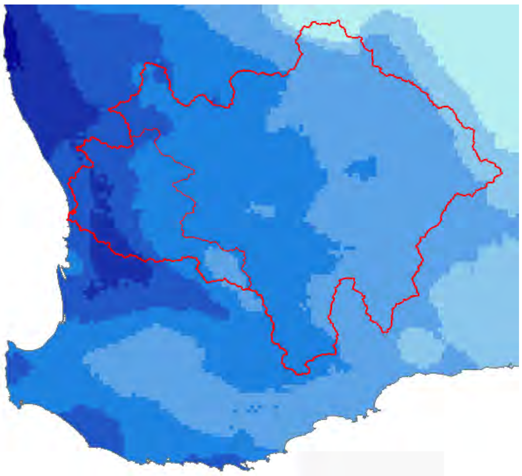
Rank 1: 16/07/1926 – 23/07/1926



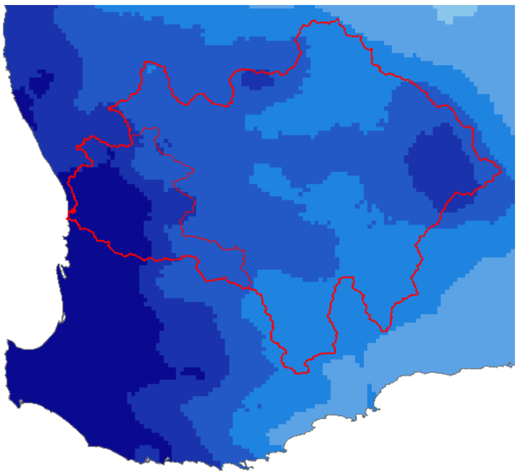
Rank 2: 23/07/1917 – 25/07/1917



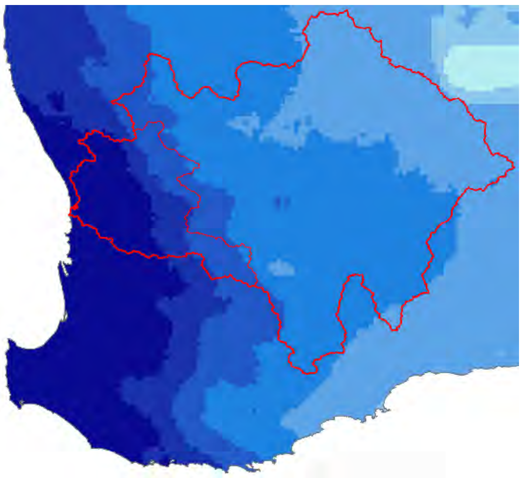
Rank 3: 16/06/1945 – 22/06/1945



Rank 4: 05/06/1946 – 08/06/1946

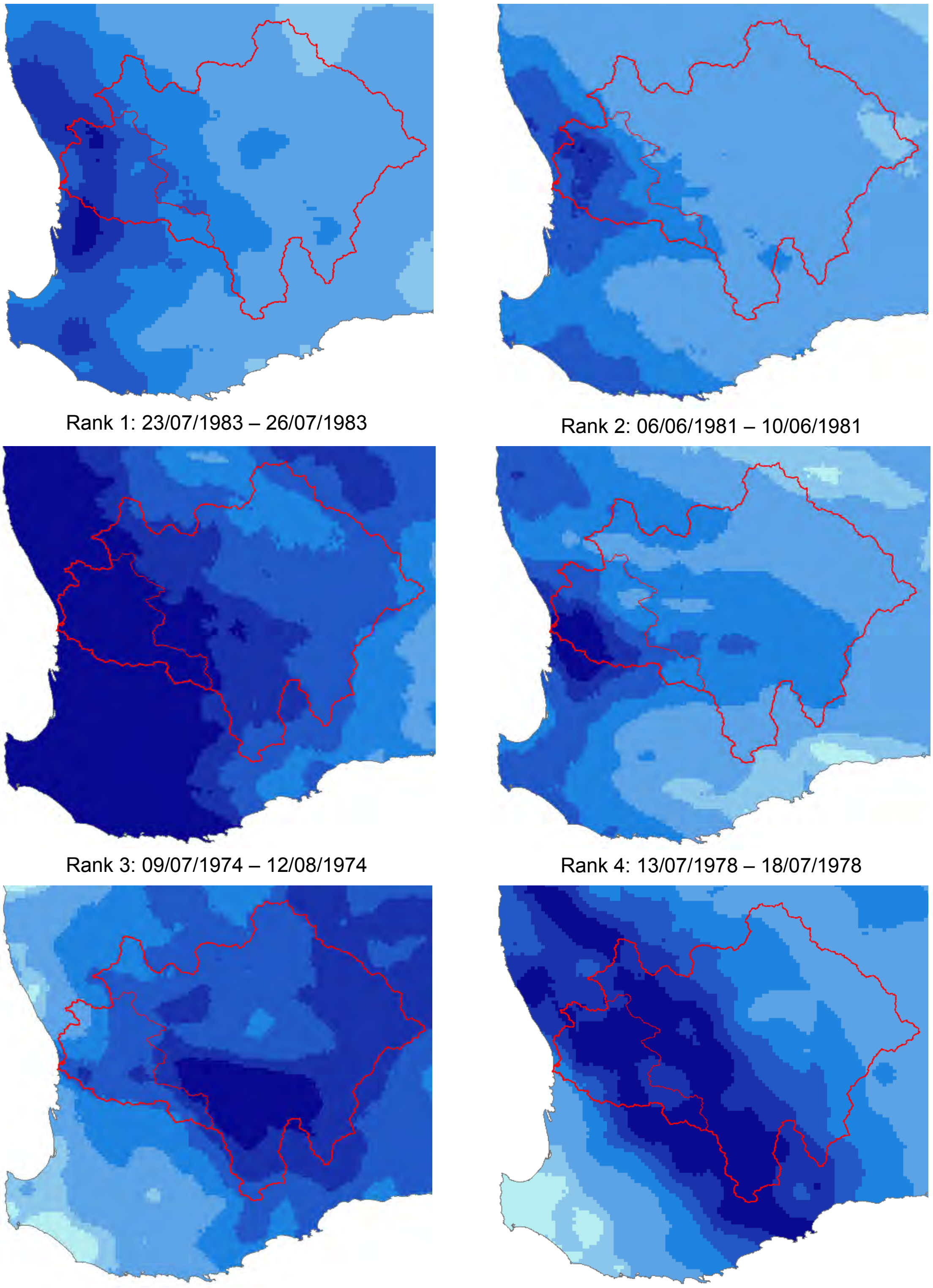


Rank 5: 17/08/1963 – 24/08/1963



Rank 6: 22/07/1958 – 29/07/1958

Figure 4-9: Rainfall spatial patterns for top six pre-gauge flood events



Rank 1: 23/07/1983 – 26/07/1983

Rank 2: 06/06/1981 – 10/06/1981

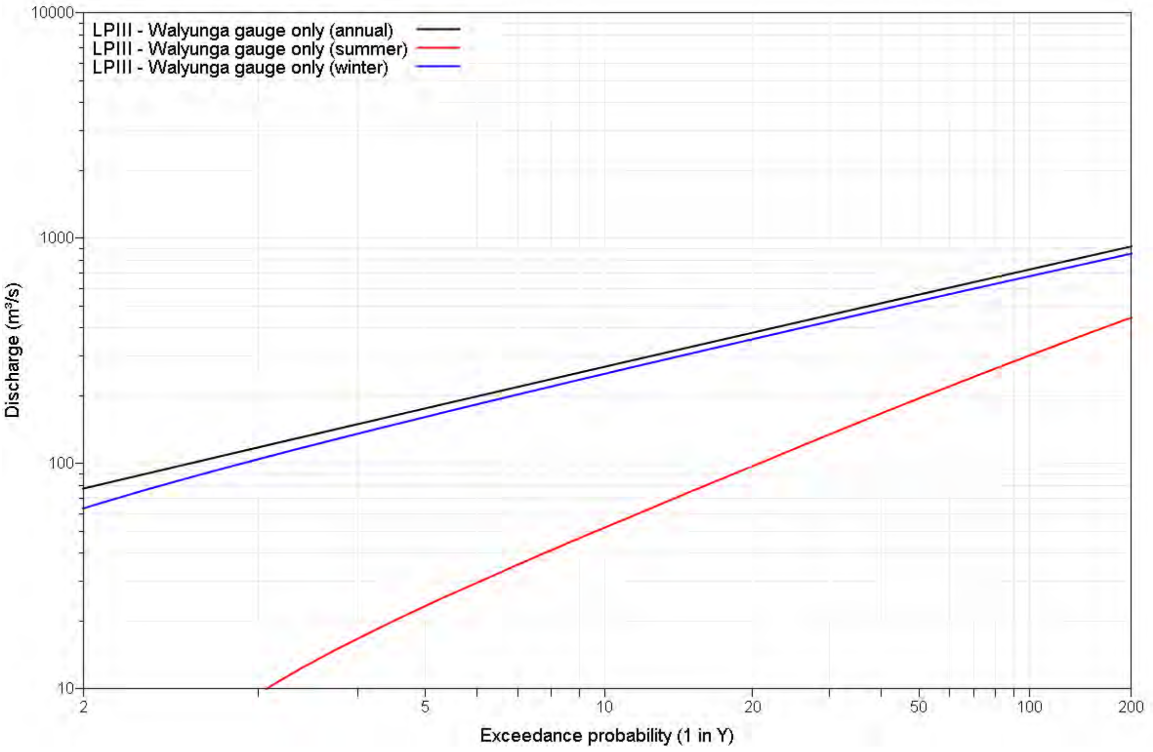
Rank 3: 09/07/1974 – 12/08/1974

Rank 4: 13/07/1978 – 18/07/1978

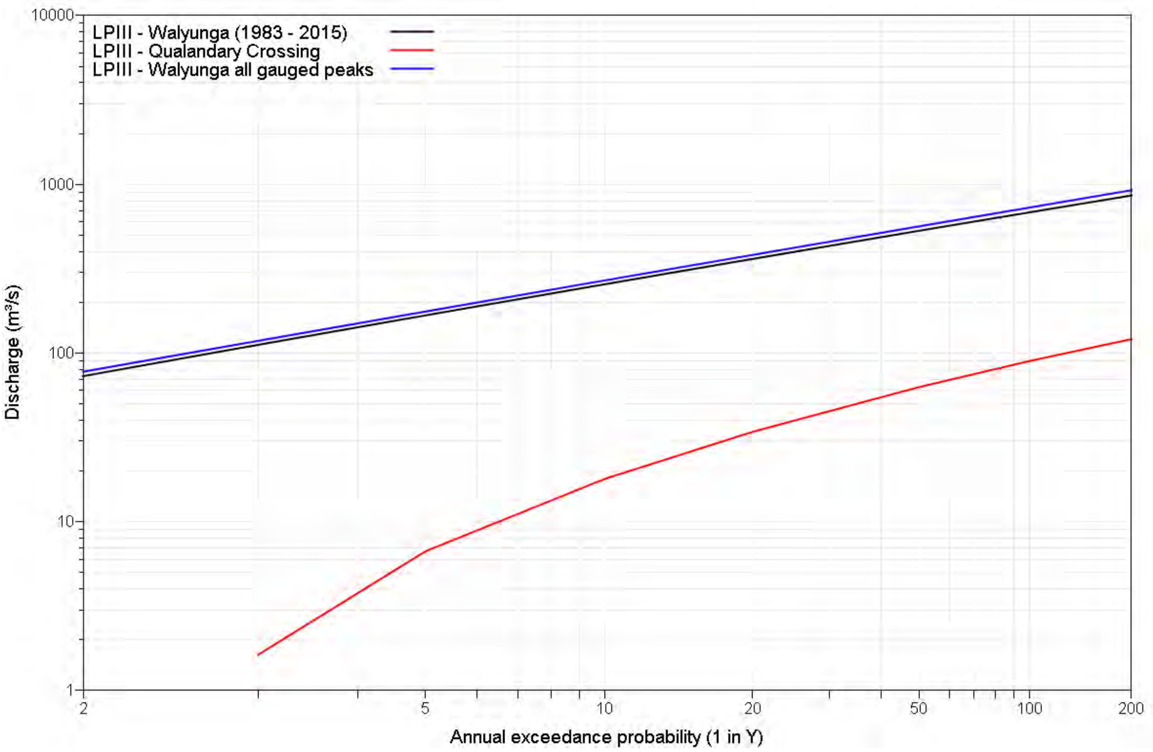
Rank 5: 21/01/2000 – 25/01/2000

Rank 6: 26/01/1990 – 30/01/1990

Figure 4-10: Rainfall spatial patterns for the top six flood events recorded at the gauge



- **Figure 4-11: Annual and seasonal flood frequency analysis at Walyunga**



- **Figure 4-12: Flood frequency analyses at Walyunga and Qualandary Crossing**

4.2.2.3 Whiteman Road

The Whiteman Road gauge is located at the downstream end of the Helena River catchment, and is immediately upstream of its confluence with the Swan River. The location of this gauge provides a useful means for verifying that the RORB model is able to reproduce the observed flood regime in the Helena River. The total catchment area upstream of this gauge is just over 1,646 km^2, however over 90% of this catchment area is affected by Mundaring Dam.

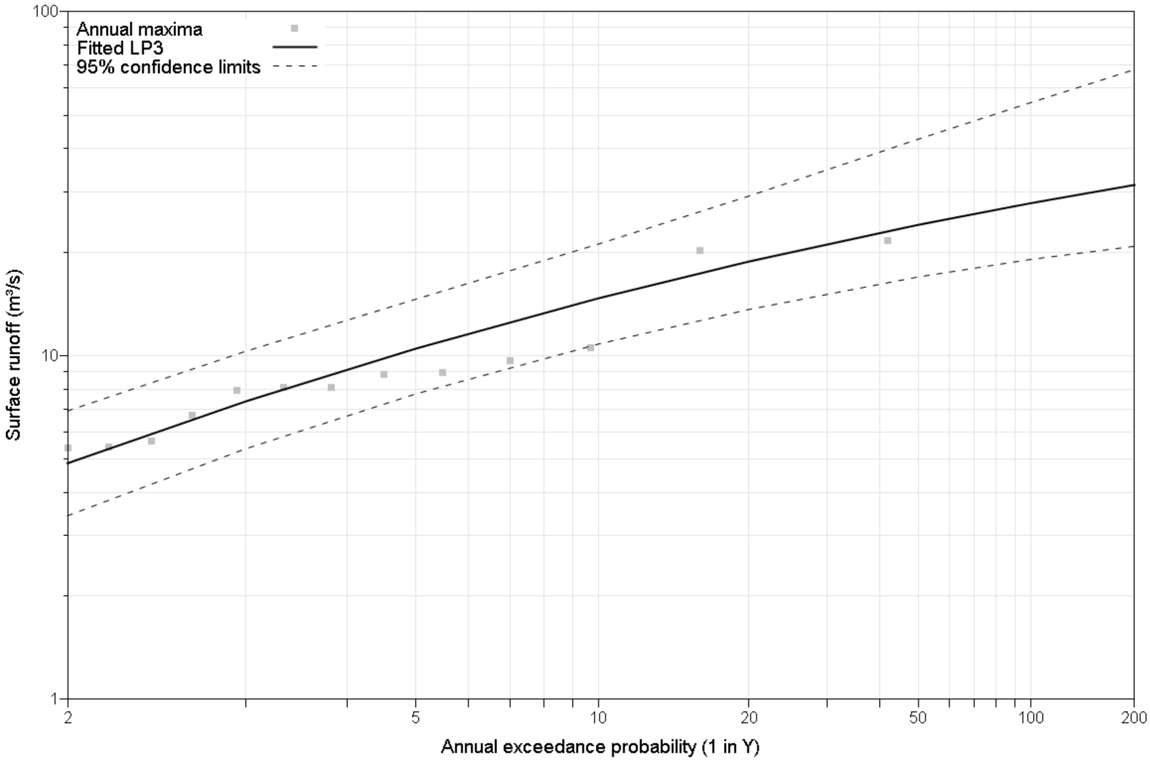
The gauged flow record provided by DoW commences in January 1988, although there is an extended period of missing data between January 1993 and April 1996. It should be noted that this gauge record includes only one flood event which resulted in outflow from Mundaring Dam, in September 1996. For the remainder of the gauge record, the water level in Mundaring Dam has been below full supply level. This suggests that the data at this gauge is likely to underestimate the historic frequency of flooding, as there were a series of significant outflows from Mundaring between 1963 and 1974.

Annual maxima were extracted from the gauge records, and the 10 largest floods at the gauge are shown in Table 4-5.

Table 4-5: Largest gauged floods at Whiteman Road

Rank (gauged floods only)	Event	Season	Gauged peak flow (m^3/s)	Estimated peak surface flow (m^3/s)
1	July 2008	Winter	23.3	21.6
2	July 2001	Winter	20.3	20.2
3	August 1998	Winter	10.8	10.6
4	July 2007	Winter	10.2	9.6
5	August 1991	Winter	9.9	8.9
6	June 1996	Winter	8.8	8.8
7	July 2009	Winter	8.9	8.1
8	September 2013	Winter	9.2	8.1
9	August 2000	Winter	8.2	7.9
10	August 2011	Winter	7.1	6.7

An LP3 probability distribution was fitted to the annual maxima at Whiteman Road using TUFLOW FLIKE. As noted previously, a Bayesian fitting technique was adopted, and maxima less than a threshold of 2 m^3/s were censored to avoid biasing the fit towards these smaller events. The results are shown in Figure 4-13 and Table 4-6.



- **Figure 4-13: Annual flood frequency estimates at Whiteman Road**

- **Table 4-6: Summary of estimated flood quantiles**

AEP	Flood quantile estimate (m^3/s)
10%	14.7
5%	18.8
2%	24.0
1%	27.8

No attempt was made to undertake this analysis on a seasonal (summer/winter) basis, given the relatively short period of record and lack of definition of the annual maxima. However, it is clear from inspection of the largest peaks in Table 4-5 that the available record of the gauge is dominated by winter events.

4.3 Craignish

The Craignish gauge is located on the Helena River approximately 9 km upstream of the Whiteman Road gauge. The catchment area of the gauge is 1,602 km^2, close to that of the Whiteman Road gauge, and similarly close to 90% of the catchment is affected by Mundaring Dam. However, the gauge record spans from 1975 to the current day, so there are an additional 13 years of data, some of which coincide with spills from Mundaring Dam. As such, the flood frequency analysis at this gauge is likely to provide a more accurate representation of the hydrological regime in this catchment. The gauged flow record provided by DoW (via Water Corporation, the gauge owner) commences in late March 1974, with minimal missing data.

Annual maxima were extracted from the gauge records, and the 10 largest floods at the gauge are shown in Table 4-7.

Table 4-7: Largest gauged floods at Craignish

Rank (gauged floods only)	Event	Season	Gauged peak flow (m^3/s)	Estimated peak surface flow (m^3/s)
1	August 1974	Winter	107.3	102.6
2	July 1987	Winter	50.9	50.8
3	July 1978	Winter	23.7	23.6
4	July 1988	Winter	23.4	23.3
5	July 2008	Winter	17.9	16.9
6	September 1996	Winter	17.4	15.0
7	August 1991	Winter	14.3	14.0
8	July 2001	Winter	12.3	12.3
9	July 1986	Winter	12.0	11.9
10	September 1984	Winter	10.3	10.2

An LP3 probability distribution was fitted to the annual maxima at Craignish using TUFLOW FLIKE. As noted previously, a Bayesian fitting technique was adopted, and maxima less than a threshold of 1 m^3/s were censored (based on the multiple Grubbs-Beck test in FLIKE) to avoid biasing the fit towards these smaller events. The results are shown in Figure 4-14 and Table 4-8.

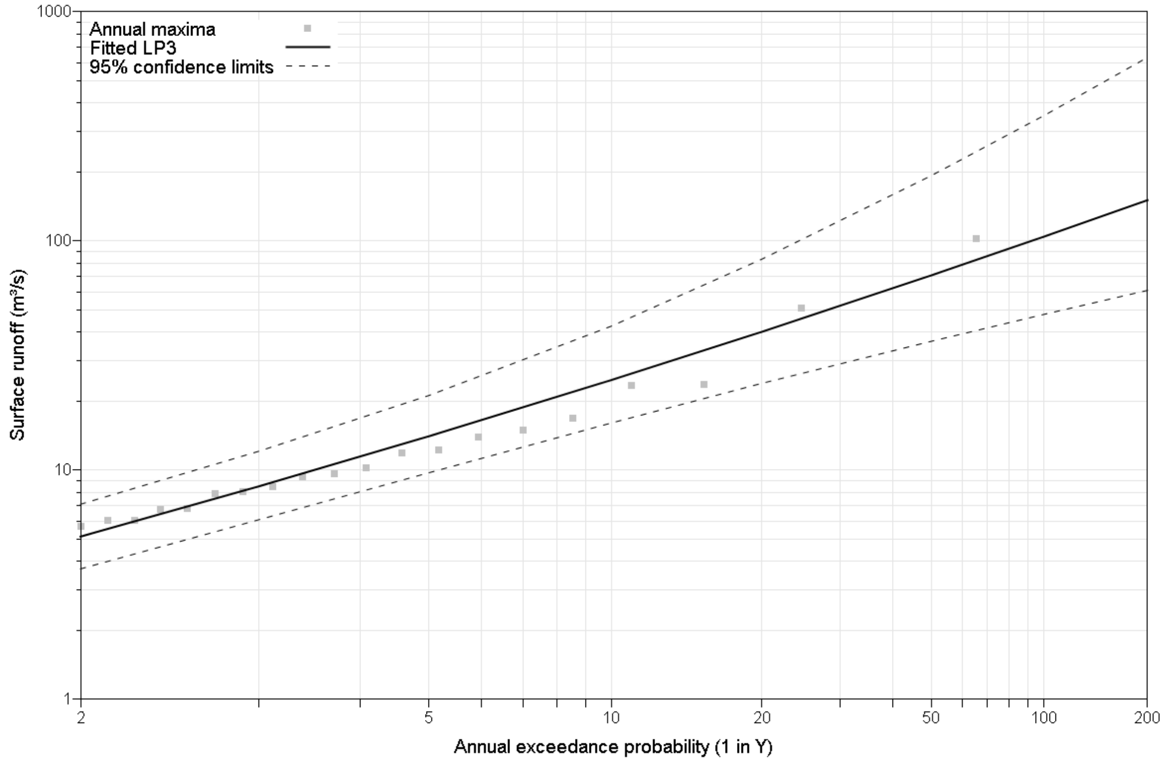
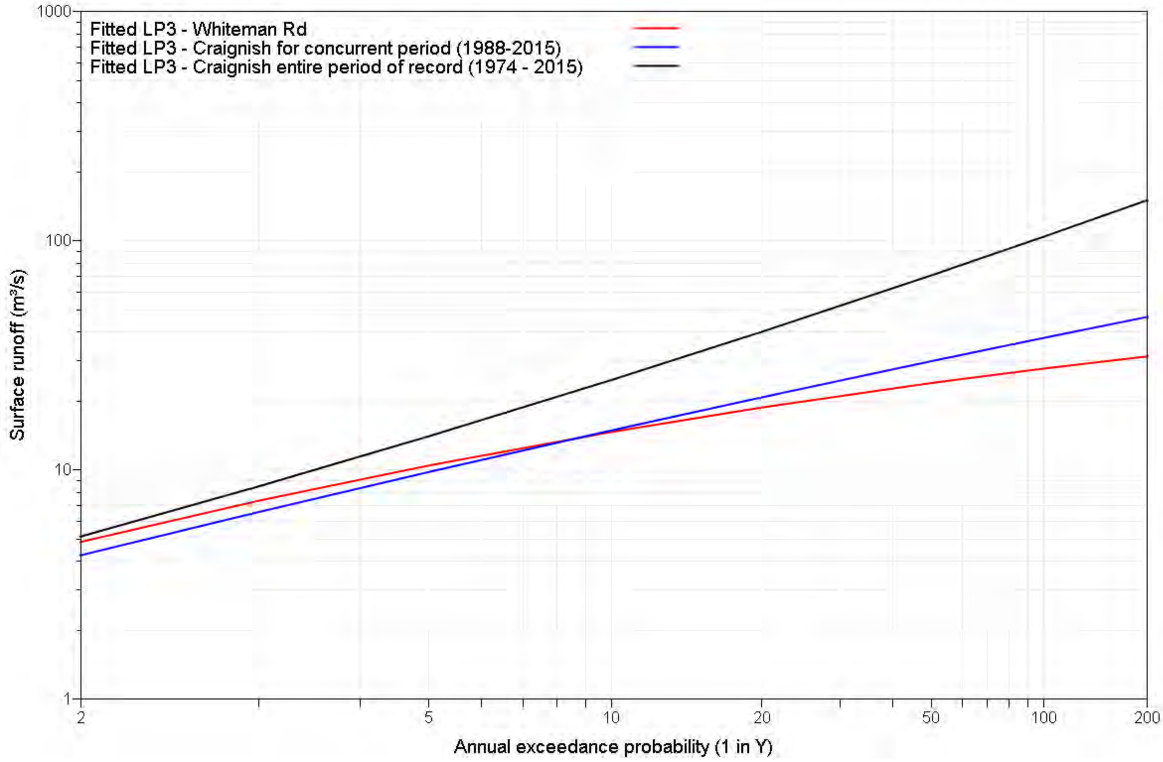


Figure 4-14: Annual flood frequency estimates at Craignish

Table 4-8: Summary of estimated flood quantiles

AEP	Flood quantile estimate (m^3/s)
10%	24.7
5%	40.2
2%	70.8
1%	104.5

A comparison was undertaken between the flood frequency estimates at the Craignish gauge with the Whiteman Road gauge. This was done for both the full period of record as well as the concurrent period of record (1988 to 2015). This comparison is shown in Figure 4-15, and it can be seen that while the frequency curves over the concurrent period are relatively similar (as would be expected from gauges quite close to each other), the frequency curve for the full period of record at Craignish is significantly higher for the larger floods. This demonstrates the value of including the additional 13 years of record available at this gauge.



▪ **Figure 4-15: Whiteman Road and Craignish flood frequency estimates**

As for Whiteman Road, no attempt was made to undertake this analysis on a seasonal (summer/winter) basis. However, it is clear from inspection of the largest peaks in Table 4-7 that the available record of the gauge is dominated by winter events.

4.4 Modelling philosophy

The analysis completed in this section has revealed and confirmed some important points about the flood hydrology of the Swan Avon River catchment. Previous flood studies (e.g. PWD, 1978) identified the concept that the majority of floods experienced downstream of Walyunga were generated from rainfall over the lower reaches of the catchment. As such, the design flood hydrology undertaken in 1978 excluded a large proportion of the catchment on the basis that it did not contribute to the floods of interest.

However, it is clear from the gauged streamflow record at Walyunga that none of the flood events recorded since the gauge commenced were significantly influenced by the large catchment upstream of the Yenyenning Lakes (the Salt, Yilgarn and Lockhart Rivers). The event which has the greatest contribution from this catchment is January 2000, however by the time the outflows from the Yenyenning Lakes reached Walyunga, they had been dominated by a larger peak generated from local runoff.

The limited period of gauged record at Qualandary Crossing indicates that the large catchment upstream of this point plays little or no role in the generation of floods at Walyunga. However, this gauged record is of insufficient duration to capture the influence of this large catchment on major events, especially those prior to 1970. It is therefore unclear whether sufficient runoff was generated from this upstream catchment to influence the peak flows at Walyunga in these large events.

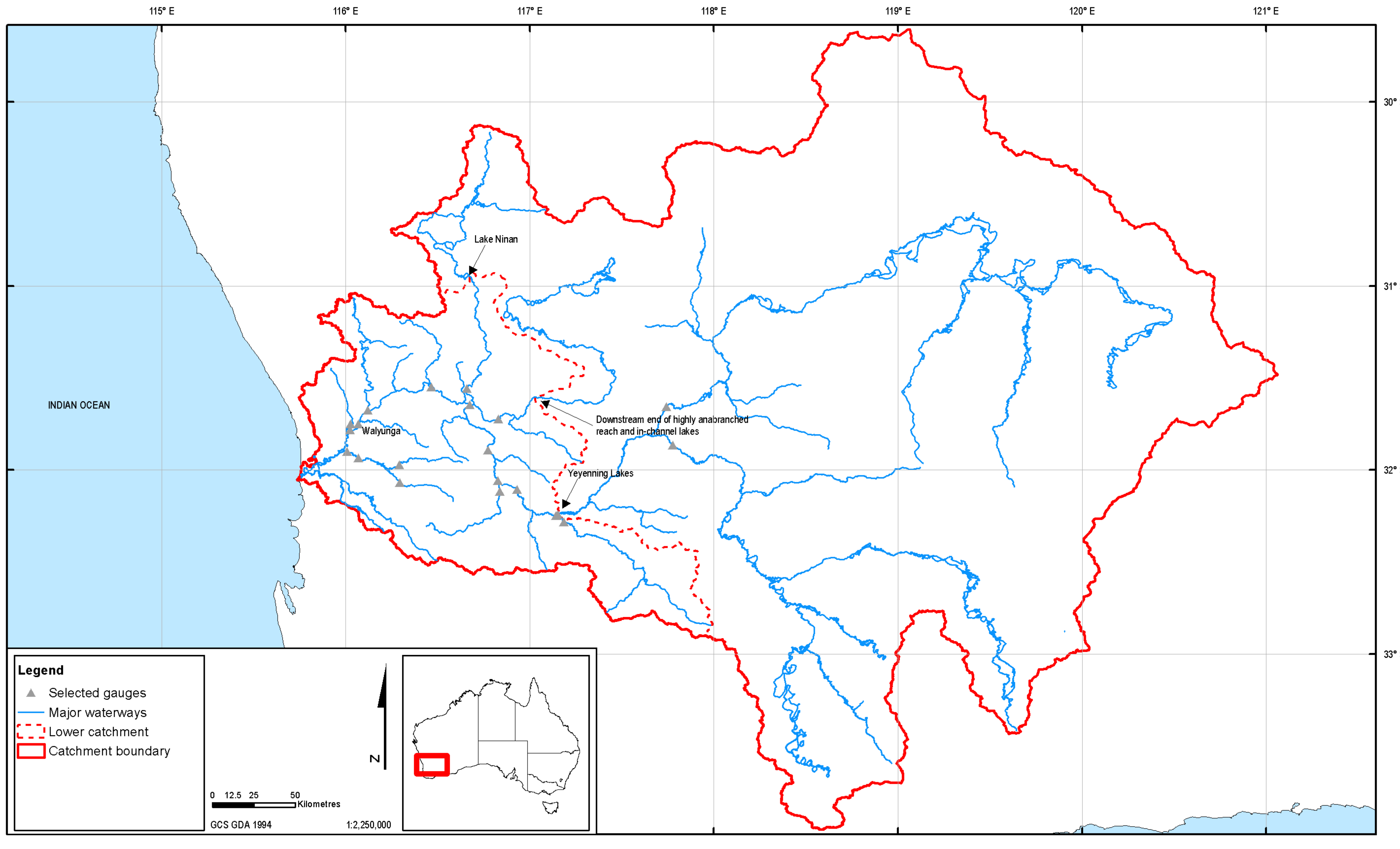
This question has been considered in the previous flood studies of the Swan Avon River, and these studies undertook design flood modelling on the basis of a 'lower' catchment to the gauge at Walyunga. The lower catchment was considered to be that portion of the total catchment where peak flows at Walyunga were generated, and upstream areas were excised from it due to the presence of lakes and hydraulic features which were assumed to result in significant losses or retardation of flows.

In order to define the extent of the lower catchment, reference was made to previous studies, gauged data and other influences such as channel morphology. It was decided to define the lower catchment as ceasing at three points:

- The downstream end of the Yenyenning Lakes on the Salt River
- The outlet of Lake Ninan (a major on-channel lake) on the Mortlock River North Branch
- The downstream end of a long reach of highly anabranched channel on the Mortlock River East Branch.

Based on these assumptions, the catchment area of the lower catchment to Walyunga is 16,100 km^2, as compared to the total catchment area of close to 119,000 km^2. The divide between the lower and upper catchments is shown in detail in Figure 4-16.

To ensure that the questions around the rare floods and PMF contributions could be satisfactorily accounted for, it was also necessary to consider the upper catchment. The following sections of this report demonstrate how the models and design inputs for the lower and upper catchments have been developed, and the relative contributions they have under design conditions for all floods.



■ **Figure 4-16: Lower catchment boundary**

5. Hydrological model development

As noted in Section 4.4, the gauge data record for the Swan Avon River catchment is of insufficient duration to reliably estimate design flood peaks, particularly for floods larger than the 1% AEP. It was therefore necessary to develop a hydrological model of the catchment, which could be used to simulate the runoff and routing associated with a range of design rainfall events. The model adopted for this project was RORB (Laurenson and Mein, 1995), which is a semi-distributed rainfall-runoff model that incorporates features such as channel routing, modelling of special storages such as lakes and dams and the ability to simulate losses along river channels.

5.1 RORB model development

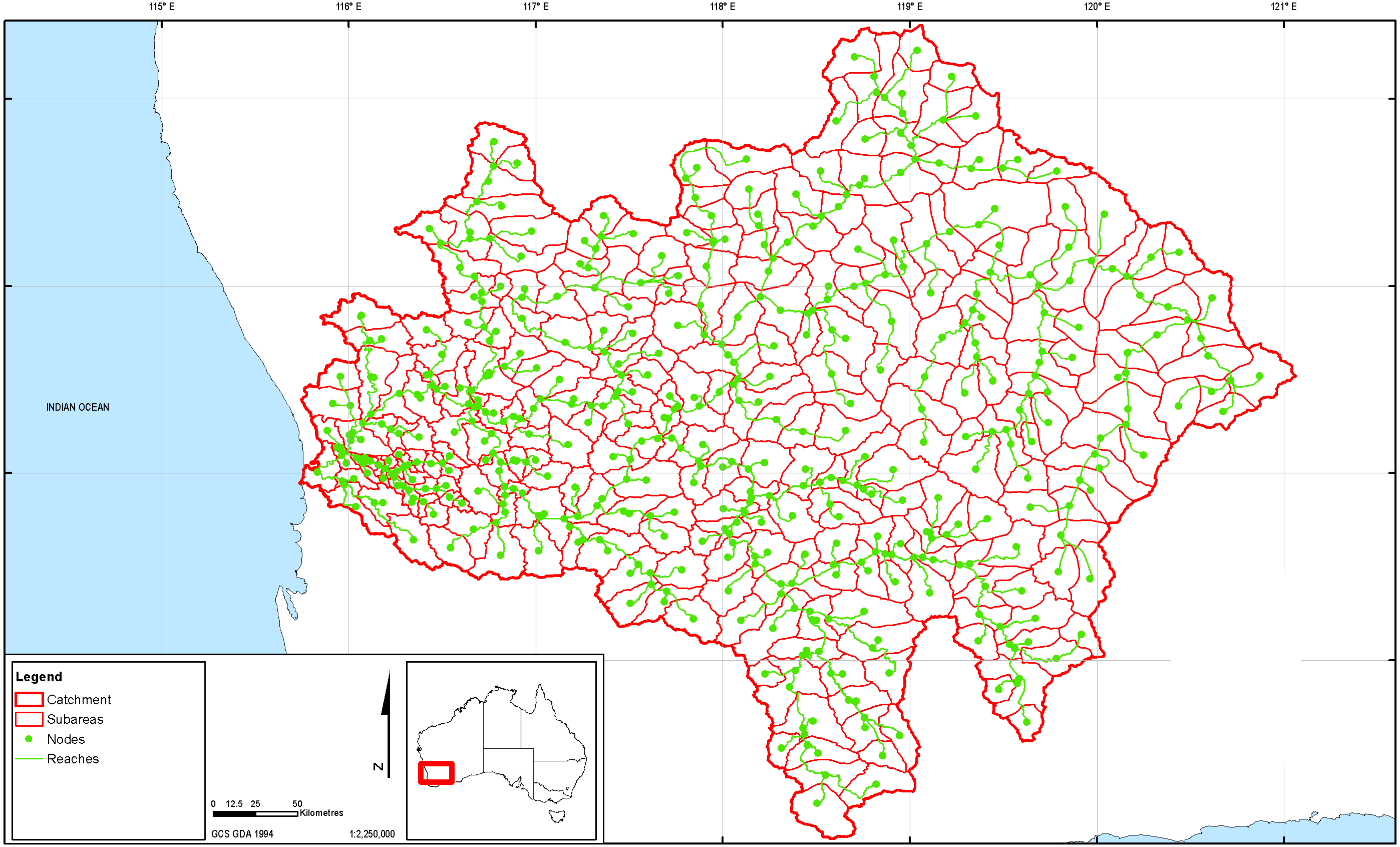
The RORB model was developed based on the topographic data supplied by DoW. The main component of a RORB model is the catchment file, which defines the hierarchy of inflows and channel routing throughout the river catchment. To develop the catchment file, the entire Swan Avon River catchment was sub-divided into 308 sub-areas, each of which acts as a point for the calculation of runoff. Care was taken to ensure that the boundaries of the sub-areas matched key locations such as streamflow gauges, lakes and other points of interest. The sub-areas were then connected by a network of river reaches. Sub-area sizes and reach lengths were calculated using Geographic Information System (GIS) software. The layout of the RORB model is shown in Figure 5-1.

The various model sub-areas were then grouped into interstation areas. Interstation areas are sub-catchments which are considered to have common properties and model parameter values. They are generally associated with streamflow gauge locations, but may also be defined on the basis of particular features of the catchment. A total of 17 interstation areas were adopted for this catchment, primarily as a result of the presence of a number of streamflow gauges along the Avon River. There were two interstation areas (Mortlock River North Branch to Lake Ninan and Mortlock River East Branch) added whose boundaries were defined by the boundary of the upper/lower catchment. These interstation areas do not have streamflow gauges, however they were considered to have hydrological properties which varied from those of the downstream reaches. The adopted interstation areas are shown in Table 5-1 and Figure 5-2.

As noted previously, the Swan Avon River catchment is characterised by the presence of a number of large lake systems. Similarly, there are significant reaches of river channel in the upper parts of the catchment whose morphology is characterised by a number of parallel anabranched channels interspersed with smaller lakes. The presence of these lakes and river channels has a significant impact on the hydrology of the catchment – they act to both attenuate the flood as well as causing volume losses as floods pass through the system.

Table 5-1: Adopted RORB model interstation areas

Area	Interstation area (km²)	Total upstream area (km²)	Average flow distance (km)	Gauge	Maximum gauged flow (m^3/s)
Yilgarn River	58,378	58,378	350.1	Gairdners Crossing (615015)	22.0
Lockhart River	28,398	28,398	244.3	Kwolyn Hill (615012)	56.4
Salt River to Yenyenning Lakes	3,270	90,046	83.3	Qualandary Crossing (615022)	181.1
Avon River to Boyagarra Rd	3,159	3,159	74.6	Boyagarra Road (615063)	7.6
Avon River to Beverley	1,173	94,377	37.1	Beverley Bridge (615025)	180.0
Dale River to Waterhatch Bridge	2,004	2,004	37.7	Waterhatch Bridge (615027)	16.5
Avon River to York	949	97,330	26.7	Balladong Street York (615024)	102.0
Avon River to Northam	785	98,115	29.7	Northam Weir (615062)	273.6
Mortlock River East Branch	8,114	8,114	172.4	N/A	NA
Mortlock River East Branch to O'Driscolls	1,548	9,662	41.2	O'Driscolls (615020)	40.8
Mortlock River North Branch to Lake Ninan	4,503	4,503	95.6	NA	NA
Mortlock River North Branch to Frenches	2,324	6,827	51.3	Frenches (615013)	58.3
Avon River to Toodyay	788	115,392	34.5	Stirling Terrace Toodyay (615026)	266.6
Brockman River	1,543	1,543	67.6	Yalliawirra (616019)	31.1
Avon River to Walyunga	2,196	118,761	55.1	Walyunga (616011)	635.7
Helena River	1,757	1,757	53.0	Whiteman Road (616086)	21.6
Swan River	3,052	123,940	43.8	NA	NA



▪ **Figure 5-1: RORB model layout**

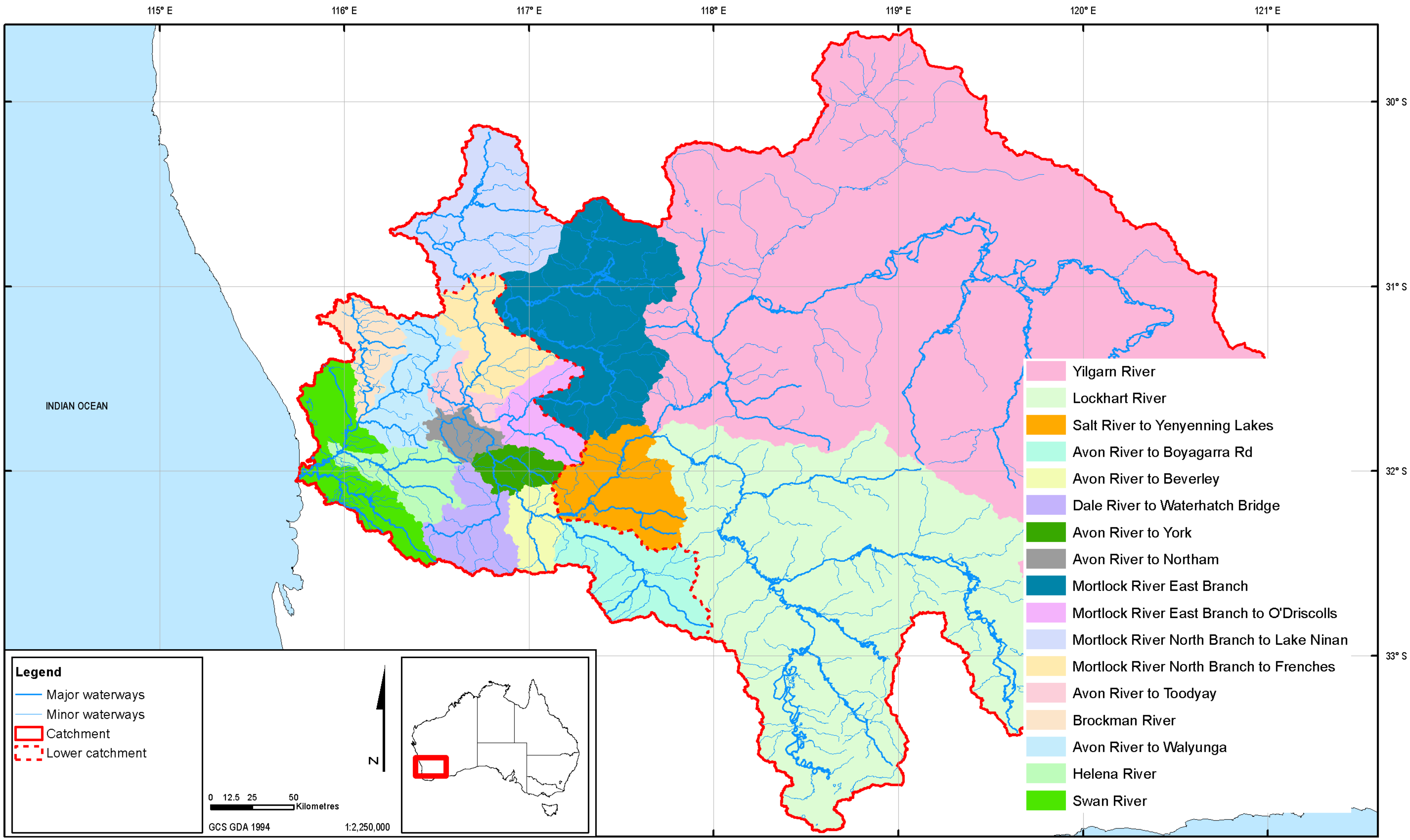


Figure 5-2: Adopted RORB model interstation areas

In order to develop a hydrological model of the catchment, and to better understand the nature of the transition between the upper and lower catchments discussed in Section 4.4, it was necessary to simulate in some form the impact of the lakes and anabranched channels. This analysis focused primarily on those lakes and river reaches at or near the boundary of the upper and lower catchments – it is by no means a complete review of all lakes and anabranched rivers within the Swan Avon River system.

5.2 Effect of anabranched channels

Anabranching is a common feature of lowland Australian streams with relatively flat bed slopes. In the case of the Swan Avon River system, there are significant lengths of highly anabranched channels which also feature intermittent lake systems (both on- and off-line lakes). An example of this type of channel format is shown in Figure 5-3 for the Mortlock River East Branch upstream of the town of Meckering. It can be seen that there is little evidence of a single, well-defined river channel. Rather, the river valley consists of numerous very small channels, which are frequently disconnected from each other. The edge of the valley also demonstrates the presence of off-line lakes.

During a flood event, flows will tend to gradually fill up these small channels, with a significant amount of water being trapped or retained in the cut off channels as well as their associated lakes. Even relatively large floods will take a significant amount of time to move through these systems, and are likely to lose a large percentage of the overall flood volume in retained water.

During the process of preliminary calibration to the January 2000 flood event (refer Section 6.3), various attempts were made to approximate the observed losses and hydrograph delays in these anabranched channels. This event was a useful means of establishing the correct model adjustments to simulate these anabranched channels, as the event was one of the largest on record in the Lockhart and Yilgarn Rivers, as well as being significant on the Mortlock River East Branch. These are the river systems which display the largest degree of anabranching and have the largest number of in-line lake systems.

For the Mortlock River East Branch, it was found that a single minor adjustment was required to the model in order to match the gauged flows at O'Driscolls. This was the introduction of a delay function to the river network, with a delay of the hydrograph by 18 hours immediately upstream of the O'Driscolls gauge. The introduction of this delay function resulted in an excellent match to the O'Driscolls hydrograph, using reasonable routing parameter values. It was found that the effect of losses in the numerous lake systems along this reach could be accounted for by factoring down the adopted runoff coefficient for this area.

In the Yilgarn River catchment, it was found necessary to introduce a number of loss functions to simulate the loss of runoff in a series of large lake systems. It was also found necessary to artificially increase the main channel reach lengths along much of the system in order to achieve a match to the gauged hydrograph at Gairdners Crossing. Similarly, in the Lockhart River system the main channel reach lengths were also factored up in order to achieve a match at the Kwolyn Hill gauge. The introduction of these adjustments, coupled with relatively low runoff coefficients (which presumably account for loss of runoff in on-line lake systems) produced a good match at the Gairdners Crossing, Kwolyn Hill and Yenyenning Confluence gauges.

The introduction of these adjustments to the model catchment file is regarded as justified on the basis that they are a conceptual representation of the effects of the channel morphology in this region. Their use is ultimately justified by the reasonable match to the gauged record that they produce, particularly for the January 2000 flood event. It should be noted that these adjustments were maintained unchanged for all four calibration events, as well as the model verification (refer Section 9). This provides a high level of confidence that the model is representing the historic behaviour of the catchment.

There is some uncertainty surrounding the behaviour of these adjustments under large flood conditions. In particular, it is possible that the three delay functions introduced to the model perform differently in a large flood such as the 0.05% AEP or PMF. As part of the final design flood model runs undertaken for this project, sensitivity analysis was conducted to determine the effect on these larger floods of assuming that the delay and loss functions were removed. This represents the scenario of a large flood where significant antecedent rainfall has resulted in the in-line lake systems filling up prior to the start of the flood event. Refer Section 10 for the results of this sensitivity analysis.

The locations of all adjustments made to the RORB model catchment file are shown on the map in Figure 5-14.

Figure 5-3: Mortlock River East Branch upstream of Meckering

5.3 Yenyenning Lakes

The Yenyenning Lakes are a lake system located at the downstream end of the Salt River catchment. The upstream catchment of the lakes includes both the Lockhart and Yilgarn Rivers, with a total catchment area of over 90,000 km^2. The lakes themselves are a connected series of individual water bodies, which tend to fill and spill in succession. The downstream end of the lake system is a formed road crossing known as Qualandary Crossing. The crossing is elevated somewhat above natural terrain, and has been modified to include a gated low flow culvert to retain the highly saline waters of the Salt River in the lakes. Qualandary Crossing itself is shown in Figure 5-4 and a locality map of the lakes is shown is Figure 5-5.

Figure 5-4: Qualandary Crossing

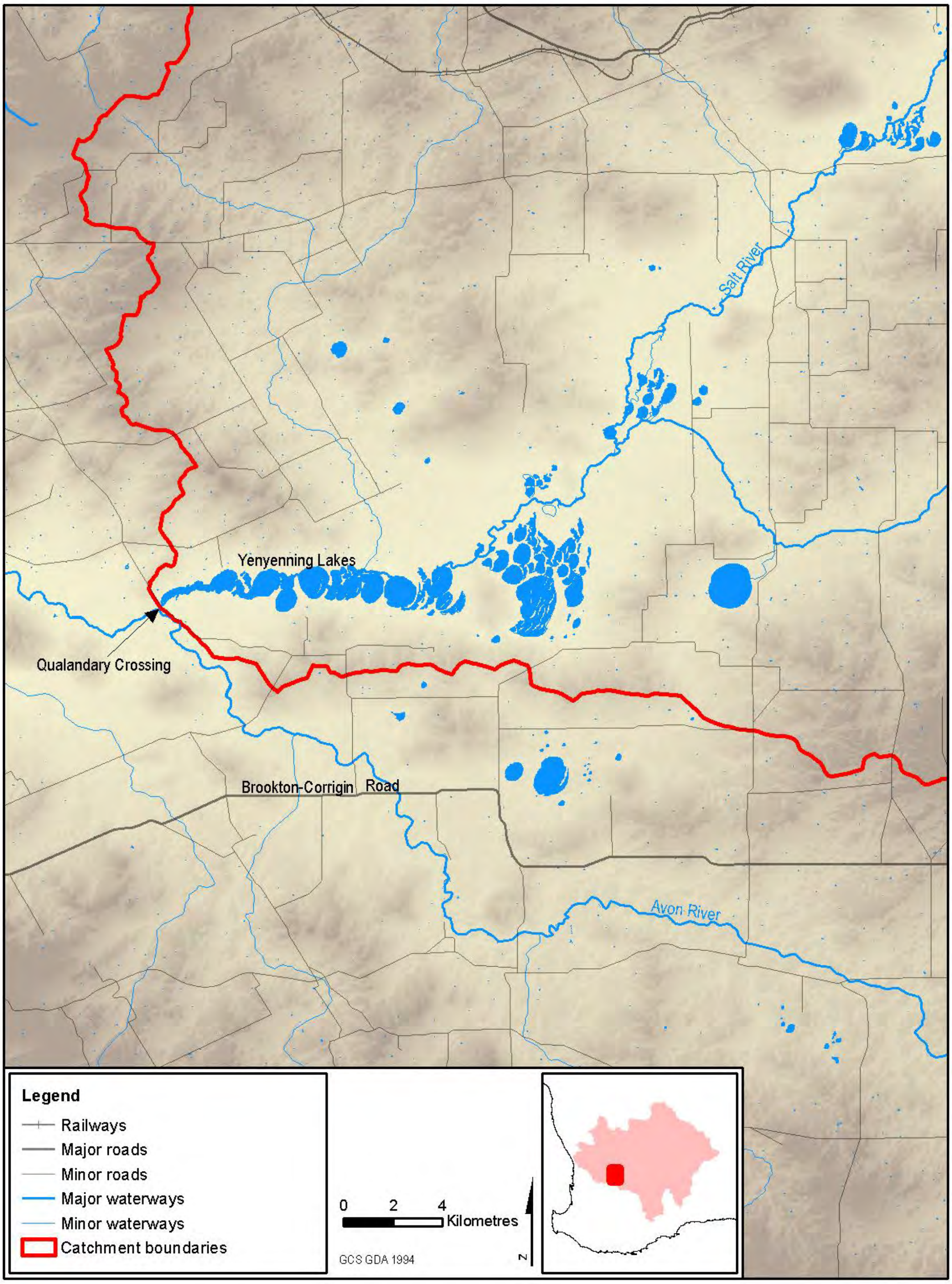


Figure 5-5: Yenyenning Lakes

The Salt River at Qualandary Crossing is gauged (615022), with the period of record commencing in 1982. The gauge is located immediately upstream of the crossing site and a number of flow gaugings have been established to support development of a rating curve. The gauge data indicates that the crossing has overflowed 8 times since records began, with the most recent occurring in January 2000, which is also the flood of record. During that event, a peak flow of approximately 74 m^3/s was estimated to have passed over the crossing.

It is clear that from these gauged records, the large catchment area upstream of the lakes does not contribute significant outflows from the lakes. There are likely to be several reasons for this, including the presence of highly anabranched river systems resulting in significant transmission losses and peak flow attenuation in the catchments upstream of the lakes. Another reason is likely to be the storage and attenuation of flood flows within the lakes themselves. The location of the lakes immediately adjacent to the Avon River was considered of sufficient importance to attempt to quantify the effect of the lakes themselves on flood inflows.

A digital elevation model of the lakes was established from the terrain data provided by DoW. This model was used to extract storage volumes for the lakes as a combined system. This was done by first defining a range of water levels of interest, both above and below the cease-to-flow threshold at Qualandary Crossing, and then undertaking a GIS calculation of the storage volume available below those levels. This data was then complied as a stage-storage relationship for the lakes. This relationship is included as Figure 5-6.

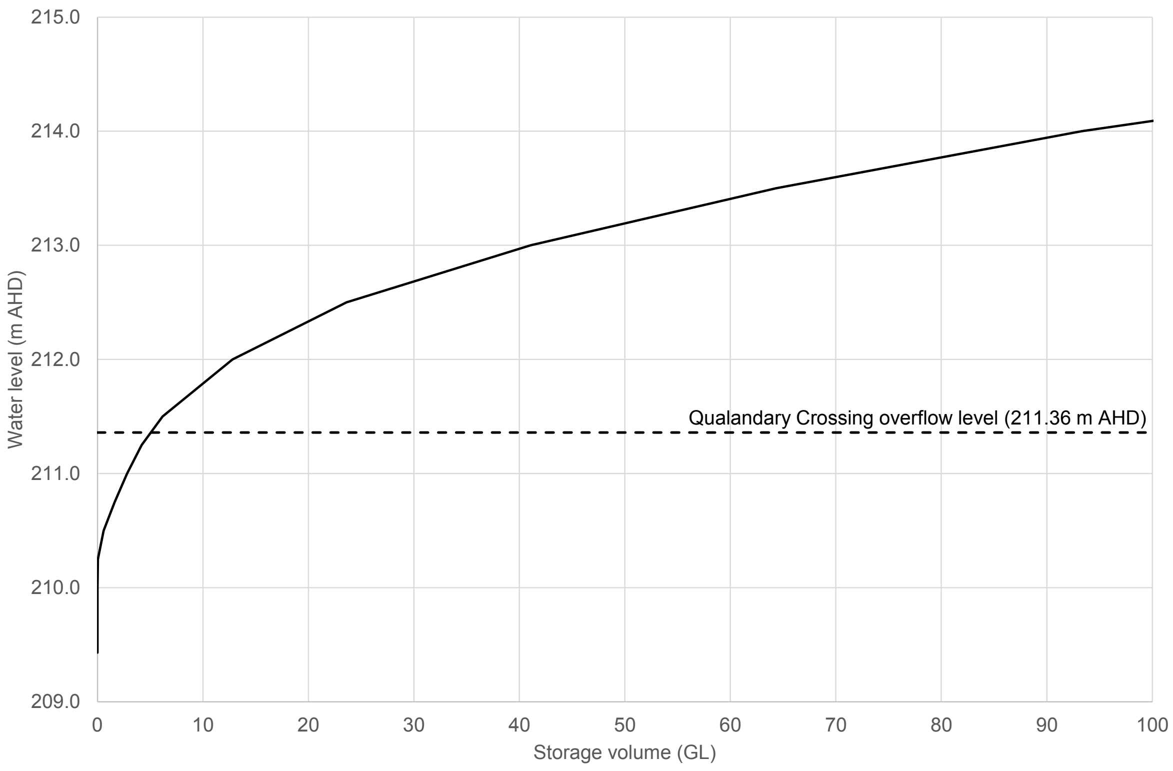


Figure 5-6: Yenyenning Lakes stage-storage relationship

It can be seen that at the cease-to-flow level of Qualandary Crossing, the Yenyenning Lakes can hold a volume of 5 GL, which demonstrates that the road crossing itself does not retain a significant volume of water.

The next stage in this analysis was to determine the outflow rate from the lakes for a range of water levels. Given the location of the streamflow gauge, which was developed using a number of gauged flows, it was decided to adopt the rating curve for this purpose. The primary issue with adoption of the rating curve for this purpose was that it only covers water levels up 1 m over the Qualandary Crossing road level. As such, it was necessary to extend the rating curve. A number of options were investigated do to this, and it was determined that a simple application of the broad-crested weir relationship provided a good fit to the existing rating curve. A uniform coefficient of discharge of 1.9 was adopted based on fitting the weir equation to the existing rating curve. The stage-discharge relationships for the Yenyenning Lakes is included as Figure 5-7.

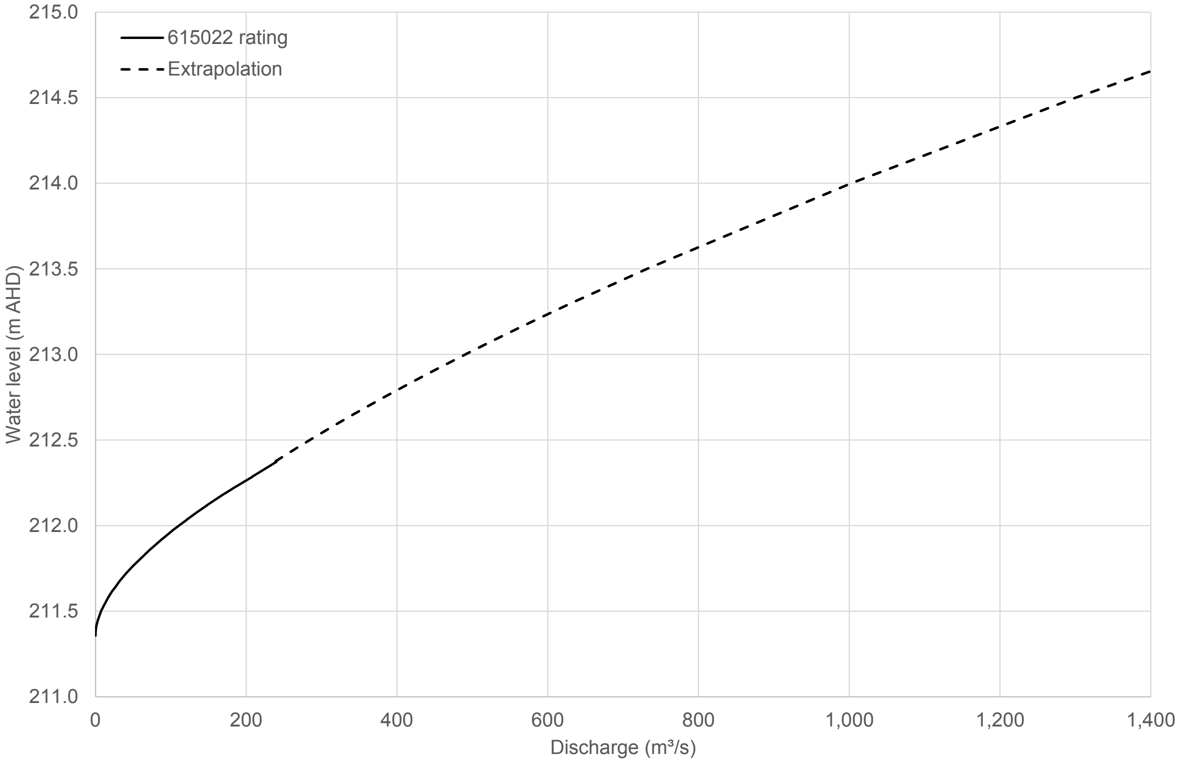


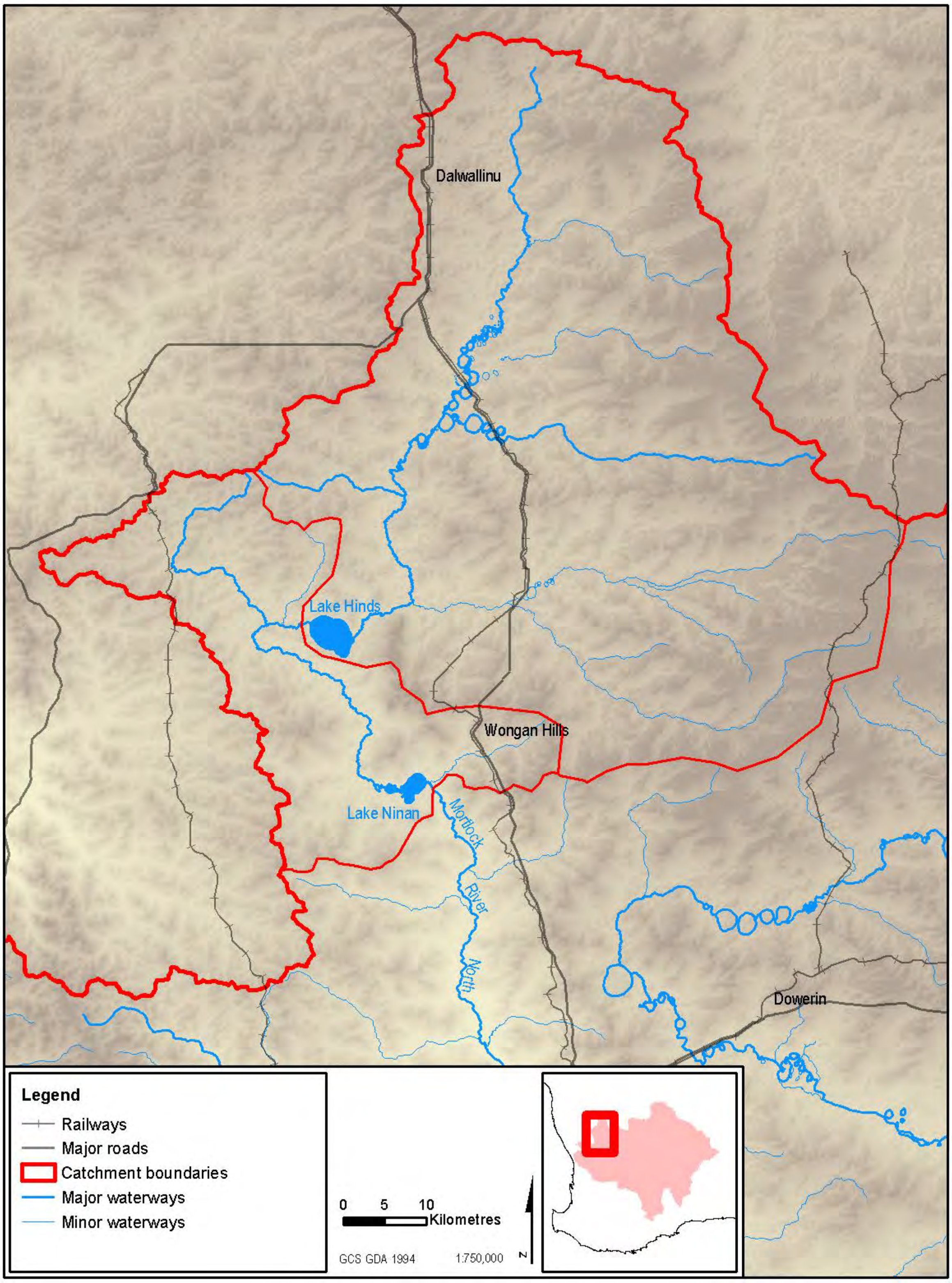
Figure 5-7: Yenyenning Lakes stage-discharge relationship

5.4 Lakes Hinds and Ninan

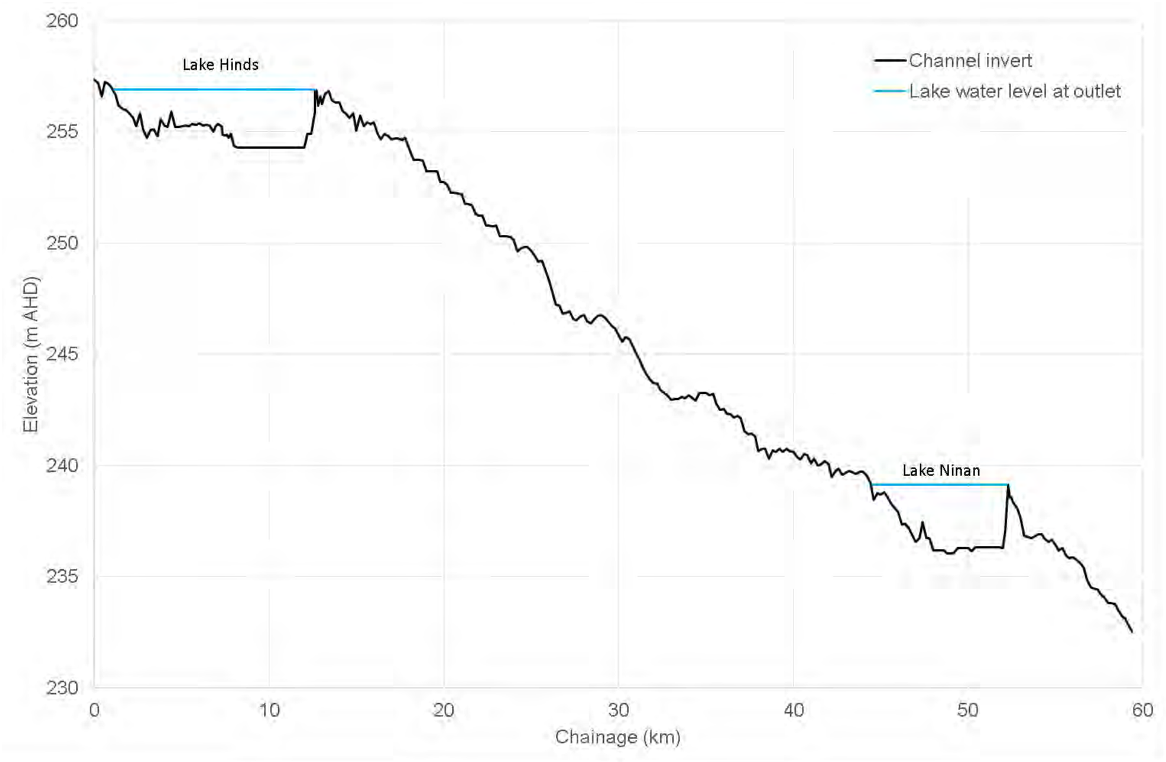
These lakes are located on the Mortlock River North Branch west of the town of Wongan Hills. Lake Hinds is located some 30 km upstream of Lake Ninan. Both lakes have a well-defined channel outlet and significant available storage volumes. The catchment area of the Mortlock River North Branch to Lake Hinds and Lake Ninan is 3,600 km^2 and 4,500 km^2 respectively. The closest streamflow gauge to the lakes is some 80 km downstream of Lake Ninan at Frenches (615013). A locality map of the lakes is shown as Figure 5-8.

In previous studies (PWD, 1978) Lake Ninan was described as a 'salt lake' and the outlet was effectively assumed not to contribute to flood flows in the lower part of the Avon River. In order to test this assumption, it was necessary to analyse the storage and discharge characteristics of the lake. Given the relatively proximity and size of Lake Hinds, both lakes were analysed together.

The digital elevation data supplied by DoW was used to extract a longitudinal section of the river profile though the lakes. This revealed that there are well-defined terrain thresholds in both lakes which will control overflows. The long section is shown as Figure 5-9.



■ **Figure 5-8: Lake Hinds and Lake Ninan**

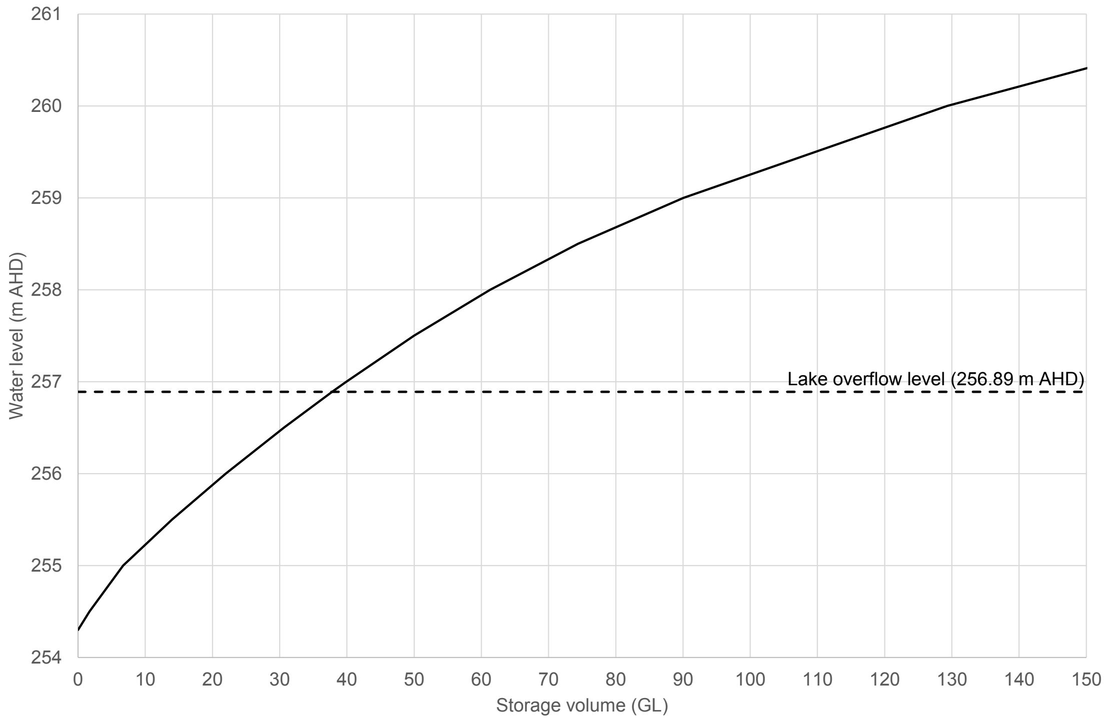


■ **Figure 5-9: Lake Hinds and Lake Ninan longitudinal section**

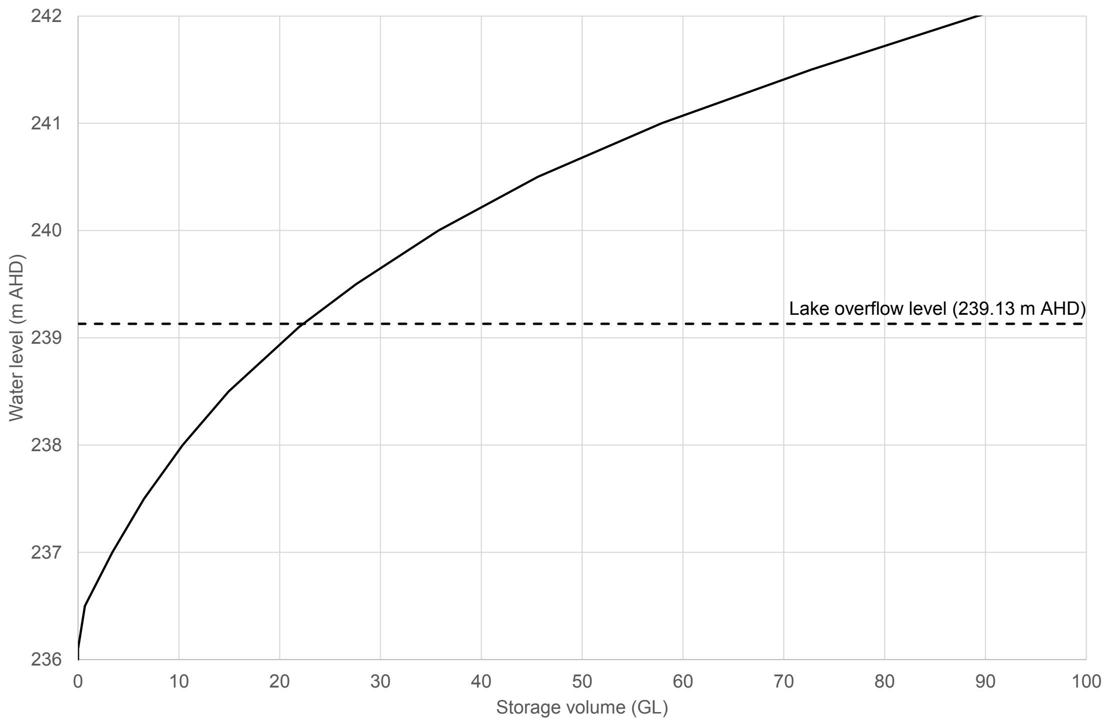
The long section also suggests that the bottom of the lakes was not captured in the digital elevation data, presumably because water was present in the lakes when the elevation source data was captured (aerial photography). As no bathymetric survey of these lakes was available, no attempt was made to define the 'true' bottom terrain levels of the lakes.

It can be seen from the longitudinal section that there are clear terrain thresholds beyond which water will flow out of each lake. Inspection of the elevation data indicated that these levels are 256.89 m AHD and 239.13 m AHD for Lake Hinds and Lake Ninan respectively.

The digital elevation model was then used to extract storage volumes for each lake. This was done by first defining a range of water levels of interest, both above and below the critical terrain threshold for outflows, and then undertaking a GIS calculation of the storage volume available below those levels. This data was then complied as a stage-storage relationship for each lake. The relationships for Lakes Hinds and Ninan are included as Figure 5-10 and Figure 5-11 respectively.



▪ **Figure 5-10: Lake Hinds stage-storage relationship**



▪ **Figure 5-11: Lake Ninan stage-storage relationship**

It can be seen that at the level of the lake outflow, Lake Hinds can hold a volume of 38 GL and Lake Ninan can hold 22 GL. By extension, this implies that if both lakes are empty at the start of a rainfall event, there needs to be more than 60 GL of runoff generated over the catchment before any flow is contributed downstream of Lake Ninan. Averaging this out over the upstream catchment area gives a rainfall excess of over 13 mm, which indicates the relative ratio of available storage in these lakes as compared to the catchment area.

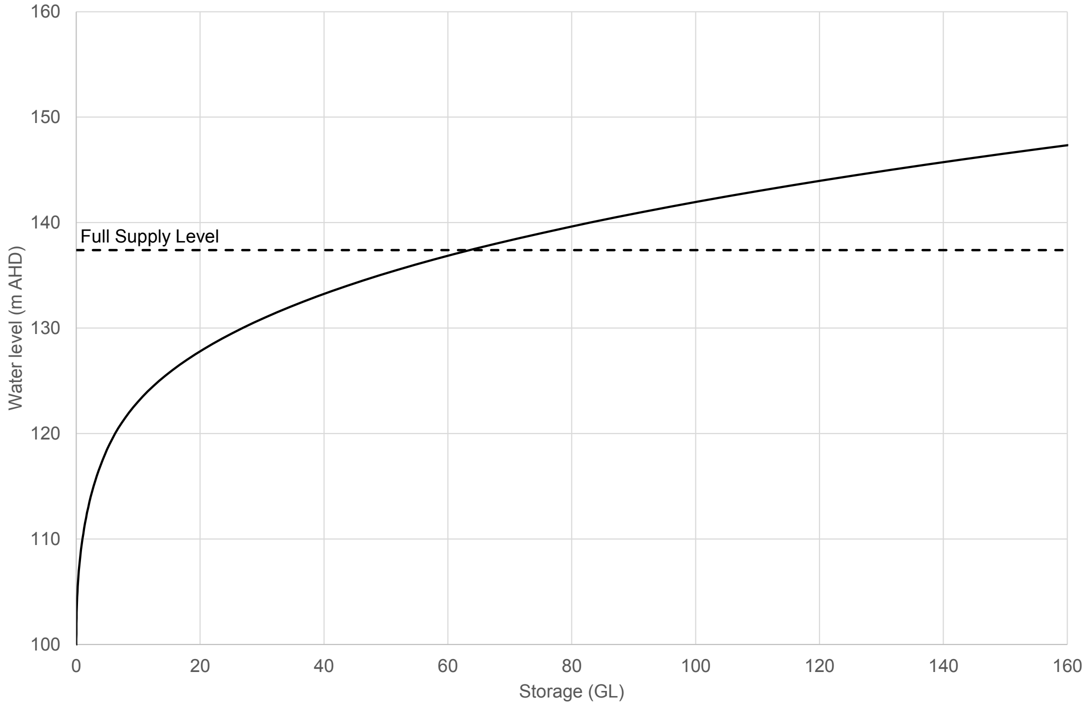
The next stage in this analysis was to determine the attenuating effect of these lakes on an inflow hydrograph. As a part of the preliminary calibration of the January 2000 flood event (refer Section 6.3), a range of possible storage-discharge relationships were trialled for these lakes. The advantage of using the January 2000 event to determine the discharge properties of the lakes is that the gauge downstream of the lakes (Frenches) displays two nearly independent flood peaks – one resulting from local runoff between Lake Ninan and the gauge, and the second resulting from outflows from Lake Ninan. Additionally, data from the South West Wetlands Monitoring Project (SWWMP) report for 2013 (DPW, 2013), which has collected 6 monthly water level readings at these lakes from 1979 onwards, was used to determine the approximate water level in both lakes prior to the start of the flood event. It was found that Lake Hinds was most likely close to full and Lake Ninan some 0.6 metres (4.4 GL) below the spill level in early January 2000.

The results of this modelling demonstrated that these lakes do not add additional attenuation to the flood hydrograph beyond what is already experienced using the normal channel routing functions in RORB. It therefore appears that the primary influence of these lakes on the flood regime is to trap and store a portion of water when they are below their spill level. Once the lakes are full, they do not provide any additional influence on the hydrograph. Therefore, both lakes were modelled as a special storage in RORB, using a storage discharge equation set such that inflow is effectively equal to outflow. For the calibration events, a fixed drawdown volume was used to simulate the loss of storage.

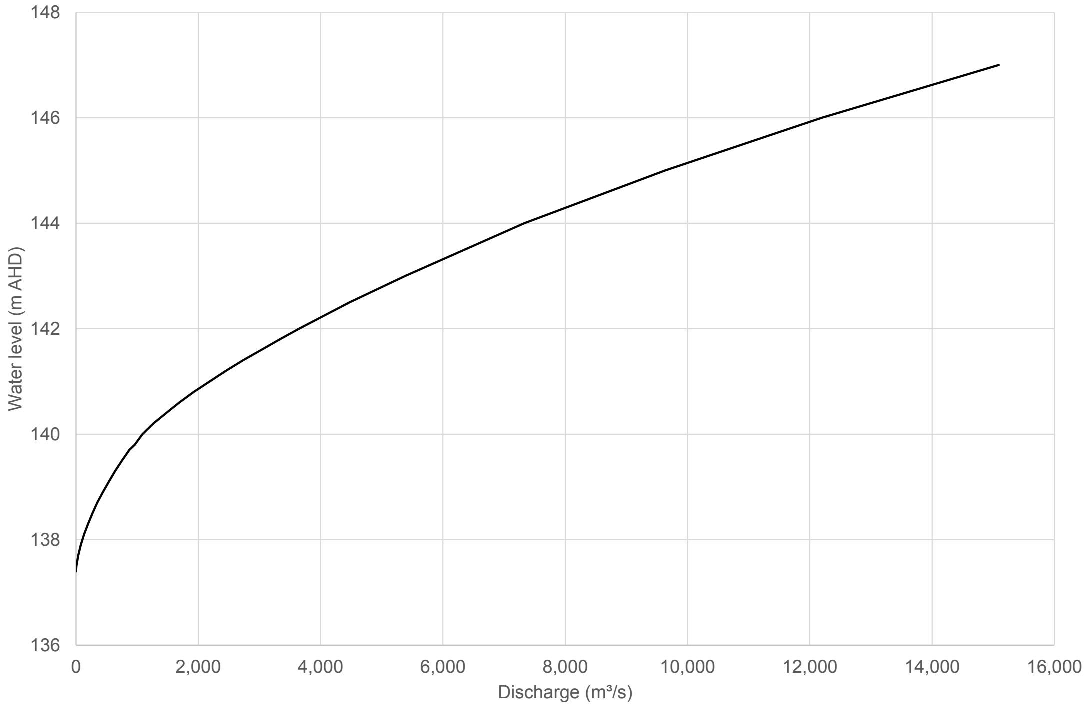
5.5 Mundaring Dam

Mundaring Dam is located on the Helena River, and is owned and operated by Water Corporation. The dam is a 40 m high concrete gravity structure, and was first constructed in 1903 then raised to its current height in 1953. The dam last spilled in October 1996, which is the only time it has spilled since the early 1980s.

The dam was incorporated into the RORB model as a special storage, using a stage-storage relationship and spillway rating curve obtained from DoE (2004a). These relationships are shown in the plots in Figure 5-12 and Figure 5-13.



■ **Figure 5-12: Mundaring Dam stage-storage relationship**



■ **Figure 5-13: Mundaring Dam spillway rating curve**

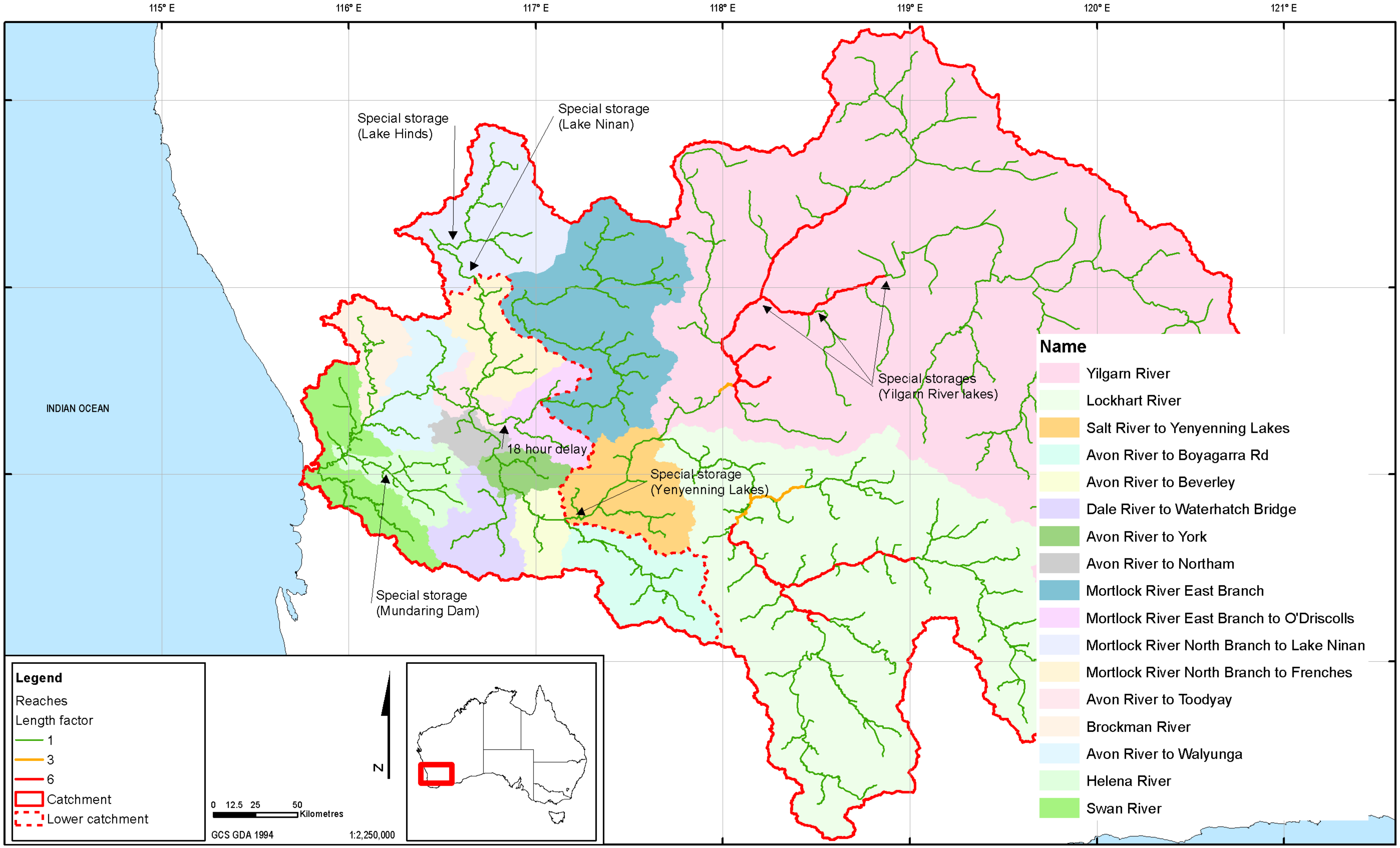


Figure 5-14: RORB model adjustments

6. Model calibration

Calibration of the Swan Avon River RORB model was undertaken to ensure that the model was able to reproduce the historic flood behaviour of the catchment. This process focused on ensuring that the model adjustments described in Section 5 were able to match the observed flood hydrographs at a series of gauging stations given reasonable routing and loss parameter values.

6.1 Selection of calibration events

Four flood events were selected from the historic record for the process of model calibration. When selecting events, the following criteria were used:

- Larger flood events were preferred to smaller ones;
- Events with recorded streamflow at more gauges were preferred;
- Events with simpler (single peaked) hydrographs were preferred to long duration, multi-peaked events.

Ultimately these criteria proved difficult to fulfil. More recent events tend to have a greater record to gauged flow data, but tend to be smaller than some historic events. It proved very difficult to isolate large flood events which did not consist of multiple, non-independent peaks. Eventually, the following four events were selected:

- January 2000. This event has a wide range of gauged streamflow data, with most of the current gauging network in operation. The event is also the largest on record in the upper Avon.
- July 1983. Although part of a series of flood peaks over winter 1983, this event demonstrates sufficient independence of the peaks before and after it to be treated as effectively a single peaked event. It also has a reasonable coverage of gauged data and is the flood of record in the mid to lower Avon.
- July 1974. This event is an extremely long duration, multi-peaked storm. The only gauge record in the Avon River catchment is at Walyunga. However, the event also features significant flooding over the Helena River catchment upstream of Mundaring Dam, where there is a reasonable spread of gauged data.
- July 1946. This event occurred prior to any gauged records in the catchment. It was included as a model validation event on the basis of the estimated peak flow of 935 m^3/s discussed in Section 4.2.2.2, which is the largest flood at Walyunga for which an estimate of peak flow exists.

6.2 Selection of loss model

Traditionally, there are two main choices for rainfall loss modelling in design flood hydrology: initial loss/continuing loss and initial loss/proportional loss. Both models have two components, the first of which (initial loss) represents the loss of water at the start of the flood event. Initial losses may be caused by seepage into the unsaturated soil zone, pondage into local depression storages, interception by vegetation and evaporation. The second component of these models (continuing loss or proportional loss) accounts for the ongoing loss of rainfall throughout the course of the design storm (typically as a result of seepage into the unsaturated zone).

The initial loss/continuing loss model is generally preferred for design flood modelling, as the continuing loss rate is regarded as invariant with rainfall AEP. This means, for example, that the same continuing loss rate (typically in the range of 2 to 6 mm per hour) can be used for floods with AEPs between 10% and 0.2%. However, for very long duration events such as the storms of interest in this catchment, there are some conceptual drawbacks to the adoption of continuing loss. The first is that conceptually, the continuing loss rate will tend to decline as the storm continues. This is generally not of importance for rainfall events with durations up to 24-48 hours, but could be significant for longer storms. Secondly, the continuing loss rate is dependent on the timestep at which the modelling is undertaken. For very long duration events, typically modelled at a timestep of 6 hours or more, the continuing loss rate would be a fraction of the value expected if the timestep was 1 hour.

Adoption of a proportional loss model avoids some of these challenges. Proportional loss is applied by calculating the loss in each timestep as a proportion of the total rainfall depth, typically within a range of 40% to 90%. The difficultly introduced by proportional loss is that it is considered to vary with AEP, such that an increase in the magnitude of the rainfall depth will result in a decrease in the proportional loss rate.

There is a third choice of loss model available, which is a conceptual soil storage model. One version of such an approach developed in south-west Western Australia is called SWMOD (WRC, 2003). This model attempts to define and mimic the water holding properties of the various soil types encountered in each catchment, and has proved to be effective in the high water capacity soils of the Darling Scarp. Whilst SWMOD models have been traditionally based on soil type information available only over these Darling Scarp catchments, recent approaches have attempted to generalise the model to Australia-wide soil datasets.

For the Swan Avon River catchment, attempts were made during the calibration of the RORB model to fit all three types of loss model. These attempts focused primarily on the January 2000 event, as this was the event with the largest amount of recorded data available. It was found that attempting to fit an initial loss/continuing loss model resulted in the calibrated continuing loss rates being unrealistically low. This presumably reflects the fact that the event is relatively long duration (3 days) and that over this time period, loss rates would exhibit a significant decline as the upper soil profile approaches saturation.

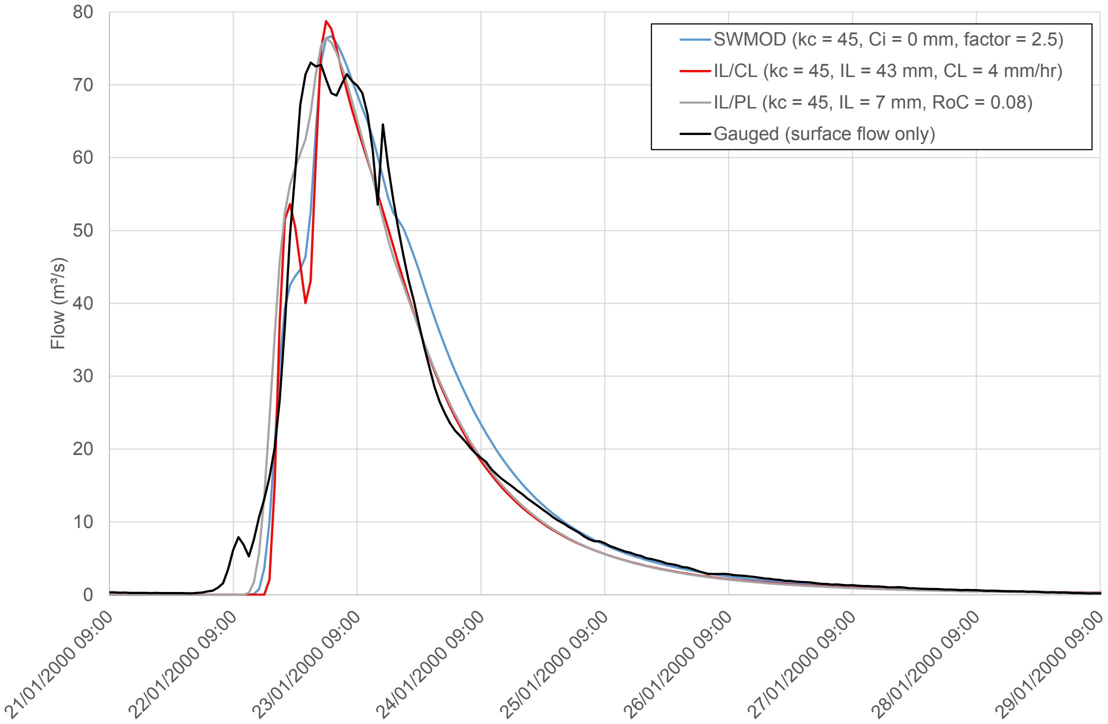
Inputs for an SWMOD model of the complete catchment were also prepared so that this loss model could be trialled. DoW provided access to the CALM System 6 soil maps of Perth and the Darling Scarp, however this information was of insufficient extent to cover the entire catchment. It was therefore decided to use the digital Atlas of Australian Soils (Northcote et al, 1960-1968) soil classification layer available from Geoscience Australia to define sub-area soil types for input into SWMOD. The model requires that upper, median and lower moisture holding capacity be defined for each soil type, together with a factor describing the linearity of the soil type's response to saturation. These were defined for each soil type using the profile information from McKenzie et al (2000) in accordance with the procedure outlined in Australian Rainfall and Runoff Revision Project 6: Loss Models for Catchment Simulation – Rural Catchments Stage 2 Report. (Hill *et al*, 2013).

As part of the preparation of this data, comparisons were made between the resolution and definition of soil types in the CALM System 6 maps as compared to the Northcote soil classifications – it was generally found that the Northcote data had significantly lower resolution than the System 6 data. This is a cause of some uncertainty in the application of SWMOD to the entire Swan Avon River catchment.

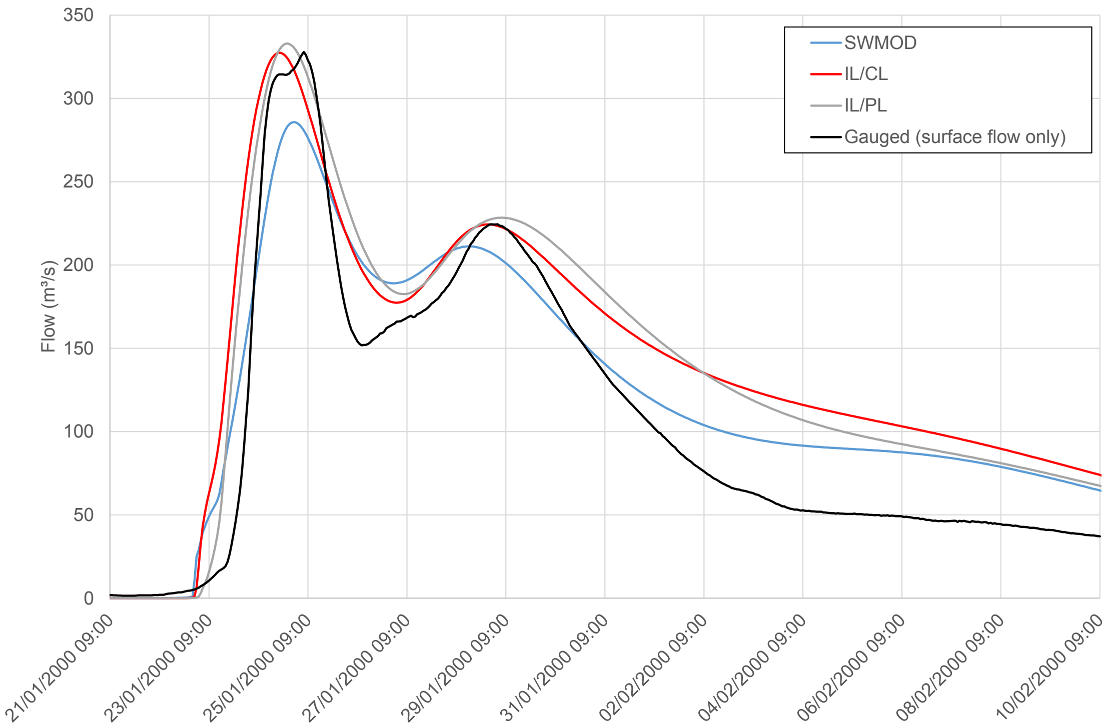
The developed SWMOD model was then trialled to the January 2000 event. It was found that a reasonable fit could be obtained along the majority of the Avon River with the initial soil store set to between 0 and 10 mm and a factor on the soil water holding capacity (as per the second parameter described in Hill *et al*, 2013) of between 0.6 and 2.5, which is within the ranges of factor values reported in Project 6 (Hill *et al*, 2013). It was found necessary to factor up the soil moisture capacity in the Lockhart, Yilgarn and upper reaches of the Mortlock River by a factor of 10 to 25 in order to obtain results that reflected the gauged information.

Of the three loss models trialled, the most success was obtained with an initial loss/proportional loss model. It was found that adoption of this framework resulted in significantly better fits of the model results to gauged hydrographs, with a generally reasonable range of loss and routing parameter values. It was therefore decided to adopt initial loss/proportional loss as the preferred loss model for this project. The main concern with this decision is the ability to select a proportional loss value which is sufficiently flexible as to be able to simulate both relatively frequent events (10% AEP) and rare events (0.02% AEP). This issue is dealt with in further detail in the verification of the model to gauged flood frequency estimates discussed in Section 9.

An objective evaluation of the performance of these loss models is difficult given the highly complex nature of this catchment. Changing loss models often results in a concurrent need to change routing parameter values, and as such it can be difficult to separate the performance benefits or otherwise solely resulting from the loss model. It was therefore decided to use the gauge on the Dale River at Waterhatch Bridge as a suitable location to demonstrate the results achieved from each loss model. This 2,000 km^2 catchment is located in the upper reaches of the Avon River, but appears to respond uniformly with minimal evidence of multiple peaks. For completeness, a comparison of the gauged data and best model fits achieved using all three loss models is shown at Waterhatch Bridge (Figure 6-1) and the gauge on the Avon River at the Great Northern Highway (Figure 6-2).



▪ **Figure 6-1: Loss model comparison at Dale River at Waterhatch Bridge**



▪ **Figure 6-2: Loss model comparison at Avon River at Great Northern Highway**

6.3 January 2000 event

The January 2000 flood event was defined as beginning at 9 am on January 21, 2000 and ending at 9 am on February 10, 2000. Note that the actual period of intense rainfall was significantly shorter than this, and mostly occurred on the 21st, 22nd and 23rd of January.

6.3.1 Rainfall inputs

Total rainfall depths for each RORB model subarea were derived from AWAP data covering the period of interest. The AWAP grids representing each day's rainfall were accumulated and used to define the rainfall depth for each sub-area centroid. A map showing the accumulated rainfall depth for this event is shown in Figure 6-3.

Six minute rainfall data from a number of Bureau of Meteorology and Department of Water pluviograph stations was extracted and used to define the rainfall temporal pattern. A total of 13 stations with usable recorded data for the event were found, mainly located around the downstream end of the catchment. The locations of these pluviographs are also shown in the map in Figure 6-3.

The pluviograph data at each station was aggregated to a one hour timestep, and then regularised to percentage of total storm depth. Cumulative plots of the rainfall temporal distribution at each pluviograph station are shown in Figure 6-4 and Figure 6-5. Allocation of the pluviographs to each model subarea was initially based on proximity, however this was changed during the course of the calibration process to improve the fit. The adopted pluviograph stations for each subarea are summarised in Table 6-1.

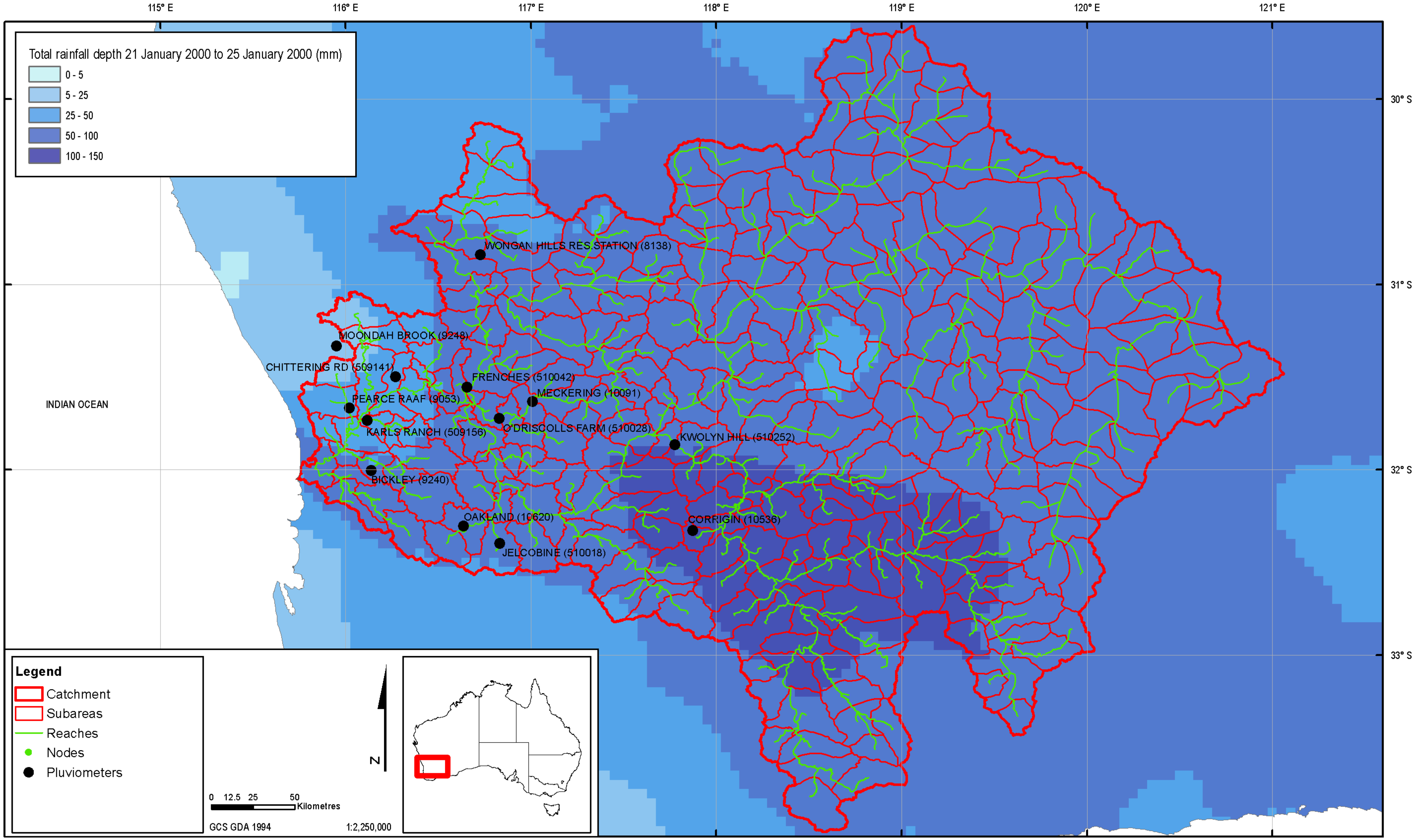
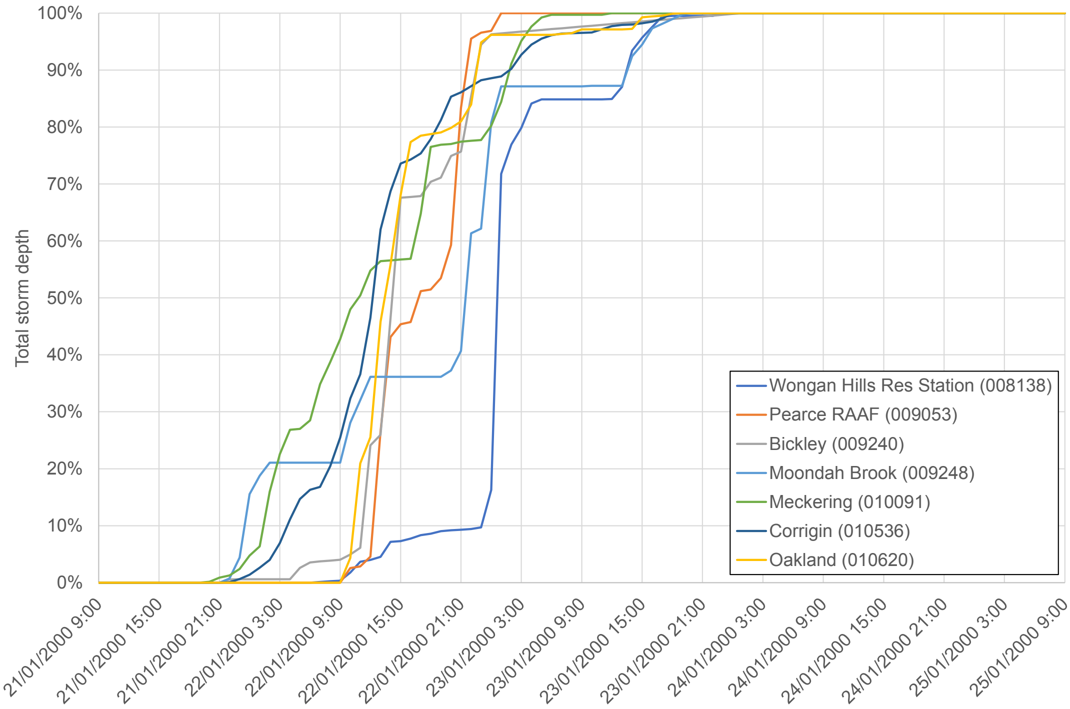
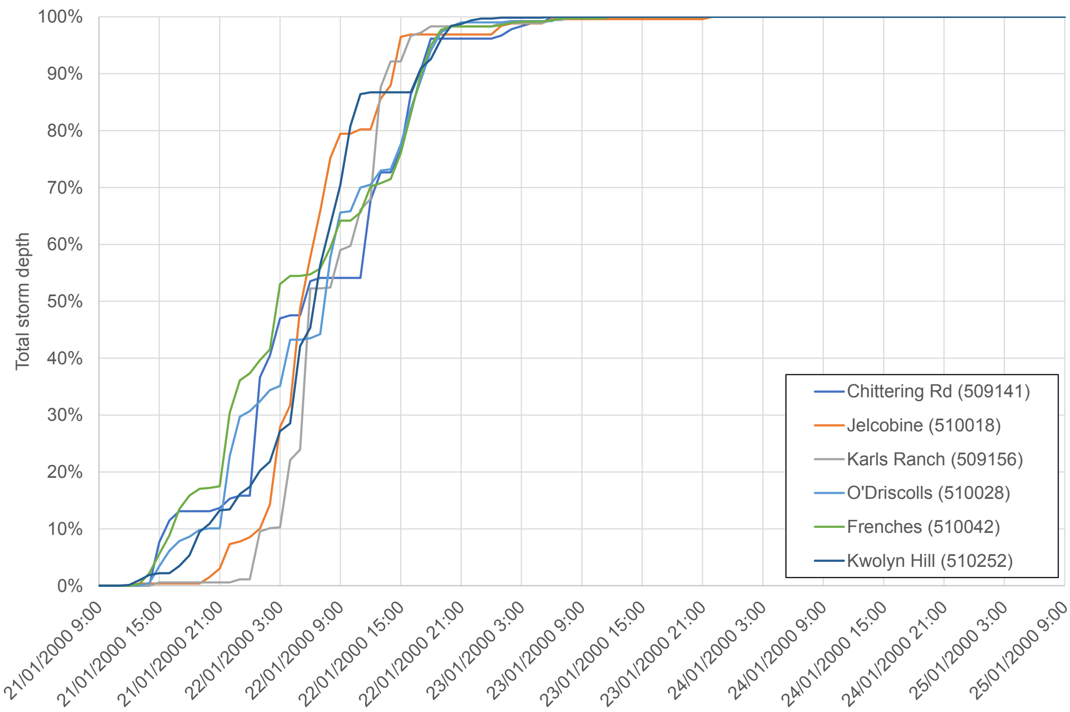


Figure 6-3: January 2000 flood event rainfall depths



■ **Figure 6-4: January 2000 Bureau of Meteorology pluviograph data**



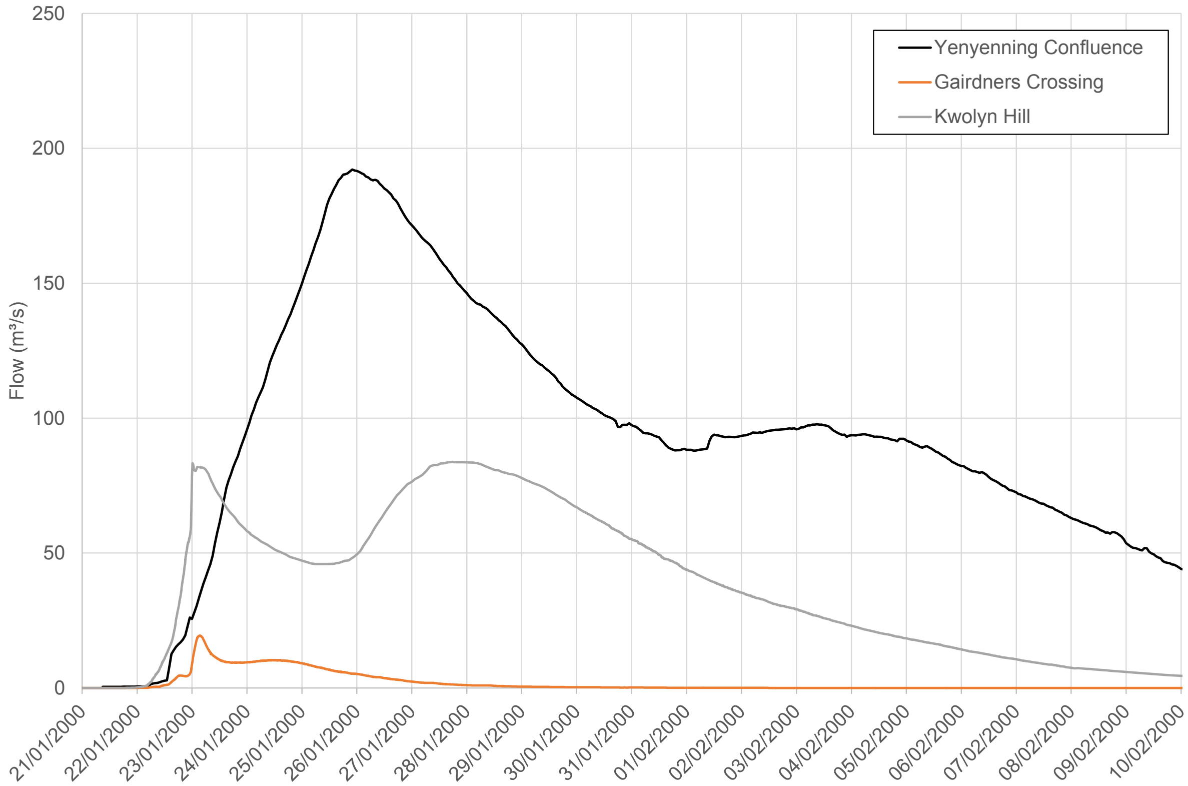
■ **Figure 6-5: January 2000 Department of Water pluviograph data**

Table 6-1: January 2000 event assigned pluviograph stations

Interstation area	Pluviograph station
Yilgarn River	Meckering (010091)
Lockhart River	Meckering (010091)
Salt River to Yenyenning Lakes	Meckering (010091)
Avon River to Boyagarra Rd	Corrigin (010536)
Avon River to Beverley	O'Driscolls (510028)
Dale River to Waterhatch Bridge	Oakland (010620)
Avon River to York	Oakland (010620)
Avon River to Northam	Oakland (010620)
Mortlock River East Branch	Meckering (010091) – upper 14 subareas Oakland (010620) – lower 10 subareas4
Mortlock River East Branch to O'Driscolls	O'Driscolls (510028)
Mortlock River North Branch to Lake Ninan	Frenches (510042)
Mortlock River North Branch to Frenches	Frenches (510042) – upper 3 subareas Meckering (010091) – lower 4 subareas
Avon River to Toodyay	Frenches (510042) – upper subarea Wongan Hills Res Station (008138) – lower 3 subareas
Brockman River	Wongan Hills Res Station (008138)
Avon River to Walyunga	Wongan Hills Res Station (008138)
Helena River	Bickley (009240)
Swan River	Pearce RAAF (009053) – upper 4 subareas Bickley (009240) – lower 5 subareas

6.3.2 Streamflow data

This event was recorded at a number of streamflow gauges throughout the catchment, making it one of the most useful calibration events. Upstream of the Yenyenning Lakes, relatively small flows were recorded at Gairdners Crossing on the Yilgarn River. At Kwolyn Hill on the Lockhart River this event resulted in the flood of record. Significant floodwaters were also recorded at Yenyenning Confluence. This event resulted in the flood of record at this gauge, with a peak flow of over 180 m^3/s recorded close to 10 pm on January 25th. Hydrographs for these three sites are shown in Figure 6-6. These hydrographs demonstrate that a significant flood volume was generated in the Lockhart River catchment, with large volumes also being generated from local runoff in the Salt River catchment immediately upstream of the Yenyenning Lakes and the upper Avon River catchment. It should be noted that although heavy rainfall in the area ceased by the 23rd of January, the peak flood outflow from the Yenyenning Lakes did not occur until January 25th, illustrating the very long travel times characteristic of the system.



- **Figure 6-6: Yilgarn, Lockhart and Yenyenning Confluence hydrographs**

Further downstream in the Avon River, the outflow from the lakes appears as a consistent signal at a series of gauges including Beverley, Brouns Farm, York, Northam, Toodyay, Walyunga and Great Northern Highway. It is clear however, that the Yenyenning Lakes outflow has taken considerable time to move down through the lower Yilgarn, lower Lockhart and Salt Rivers. The rainfall burst which resulted in these large outflows from the lakes also caused significant local runoff in the Avon River and its tributaries downstream of the lakes. The gauged record for all the Avon gauges clearly shows two significant first ('local' runoff) peaks and a third peak resulting from the Yenyenning Lakes outflows.

The behaviour is apparent in the gauged hydrographs at Qualandary Crossing, Beverley, Brouns Farm, York and Northam, as shown in Figure 6-7.

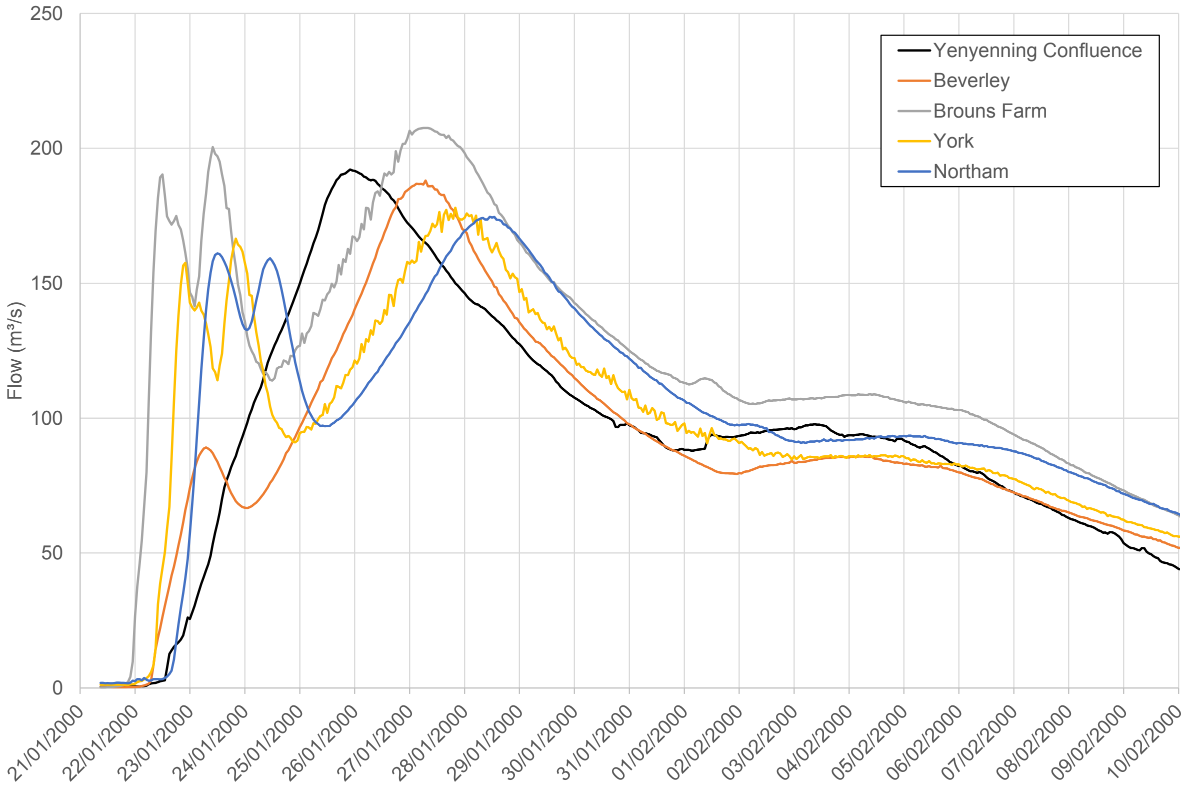


Figure 6-7: Avon River hydrographs from Yenyenning Confluence to Northam

The flood behaviour recorded in these hydrographs is remarkably consistent, save for the exception of the data at Brouns Farm. At this location, all three flood peaks are considerably larger than at both the upstream and downstream gauges. Whilst this is theoretically possible for the first two (local runoff) peaks, there is no rainfall after the 23rd of January which could contribute to increasing the third (Yenyenning Lakes outflow) peak. The source of this apparent inconsistency in the gauged flow data is not apparent. The gauge at Brouns Farm has been gauged up to a flow of 227 m^3/s, which should provide some certainty in the rating curve. The gauges at Beverley and York have been rated up to flows of 180 and 102 m^3/s respectively, which would tend to place more reliability on the data at Brouns Farm. Given the apparent lack of a physical explanation associated with the inconsistent data at Brouns Farm, this gauge was not used for calibration.

Downstream of Northam, significant floods were also recorded on the Mortlock River East Branch at O'Driscolls and the Mortlock River North Branch at Frenches. Both gauges demonstrate double peaked hydrographs, with the first peak generated from local runoff in the area immediately upstream of the gauge, and the second peak originating from runoff generated in the upper reaches of these catchments. It should be noted that the available water level data (DPW, 2013) for Lakes Ninan and Hind on the Mortlock River North Branch indicates that Lake Ninan at least was overflowing prior to and during this event. The inflows from these tributaries served to enhance all three flood peaks recorded at Northam. This enhancement is shown in the comparison of the recorded hydrographs at Northam and Toodyay in Figure 6-8.

There is some uncertainty surrounding the data at the Walyunga gauge for this event. DoW indicated that there was an issue flagged at the gauge for this event, and as such the quality code for the data was increased (*pers. comm.* Simon Rodgers, DoW, 11/1/16). When the hydrograph from Walyunga is plotted alongside the hydrographs from Toodyay (upstream) and Great Northern Highway (immediately downstream), both the shape of the rising limb and the peak flows appear anomalous (shown in Figure 6-9). As such, data for the gauge at Great Northern Highway was substituted for the Walyunga gauge during the calibration process.

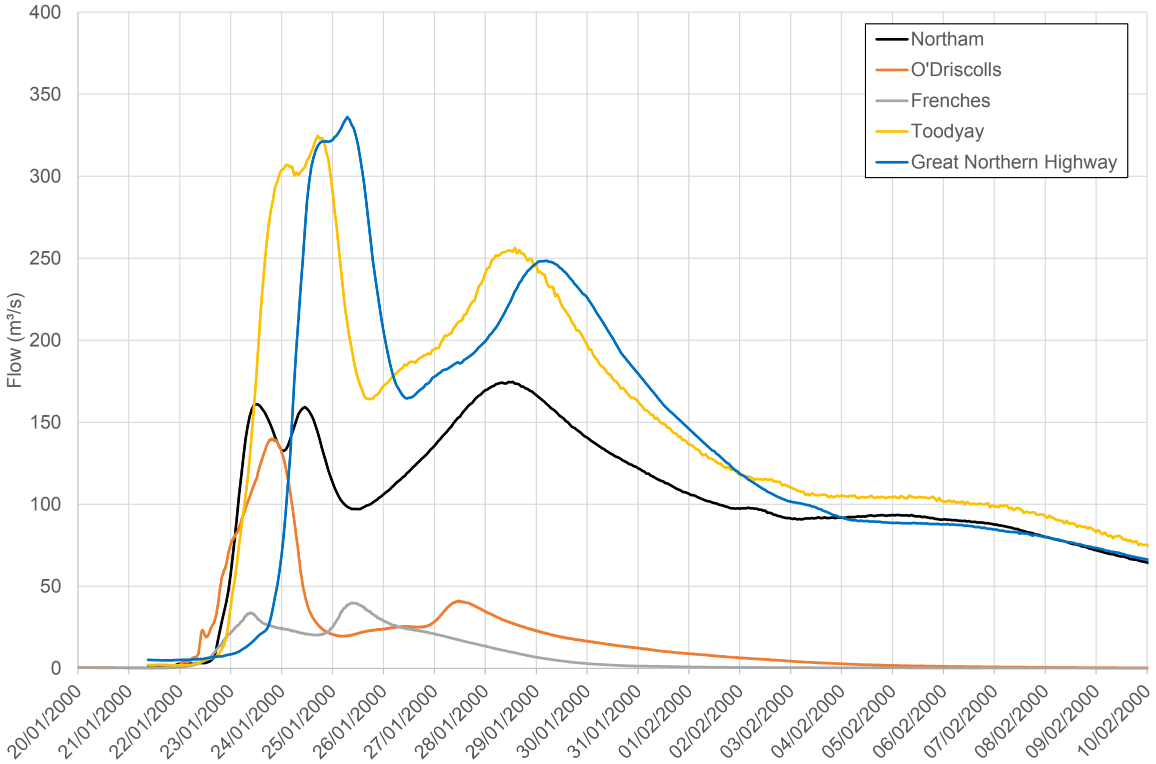
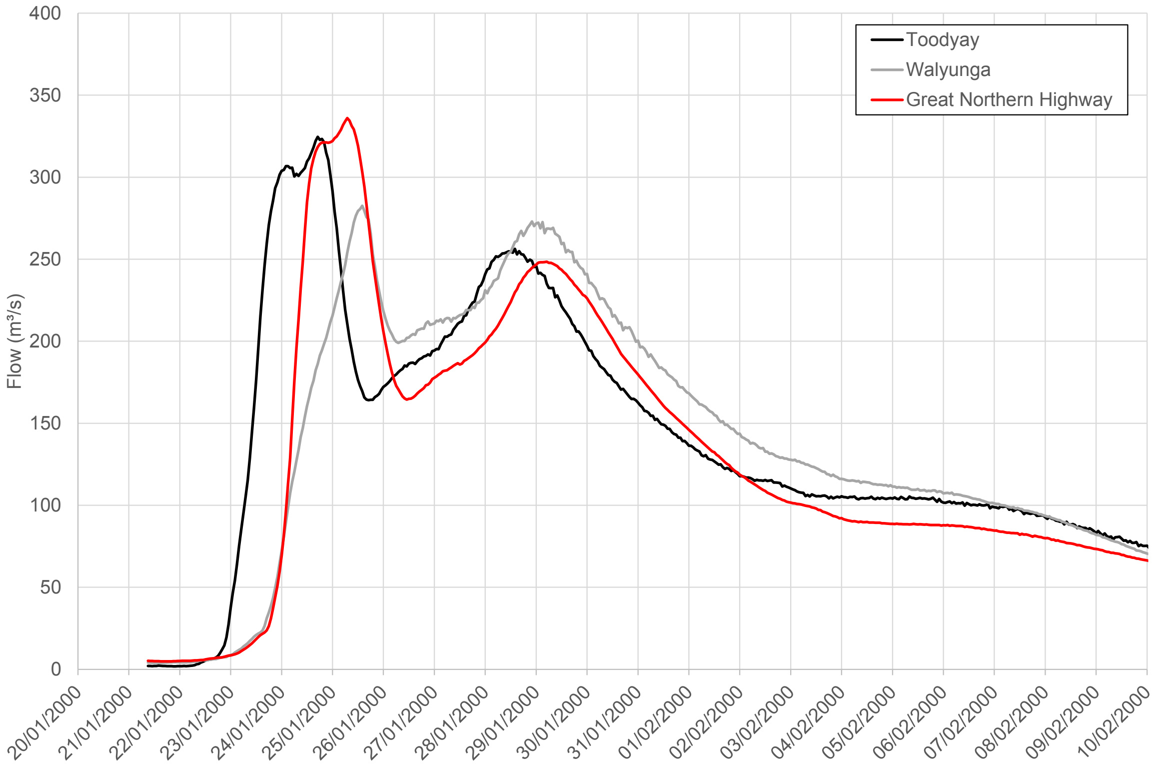


Figure 6-8: Swan and Avon River hydrographs from Northam to Great Northern Highway

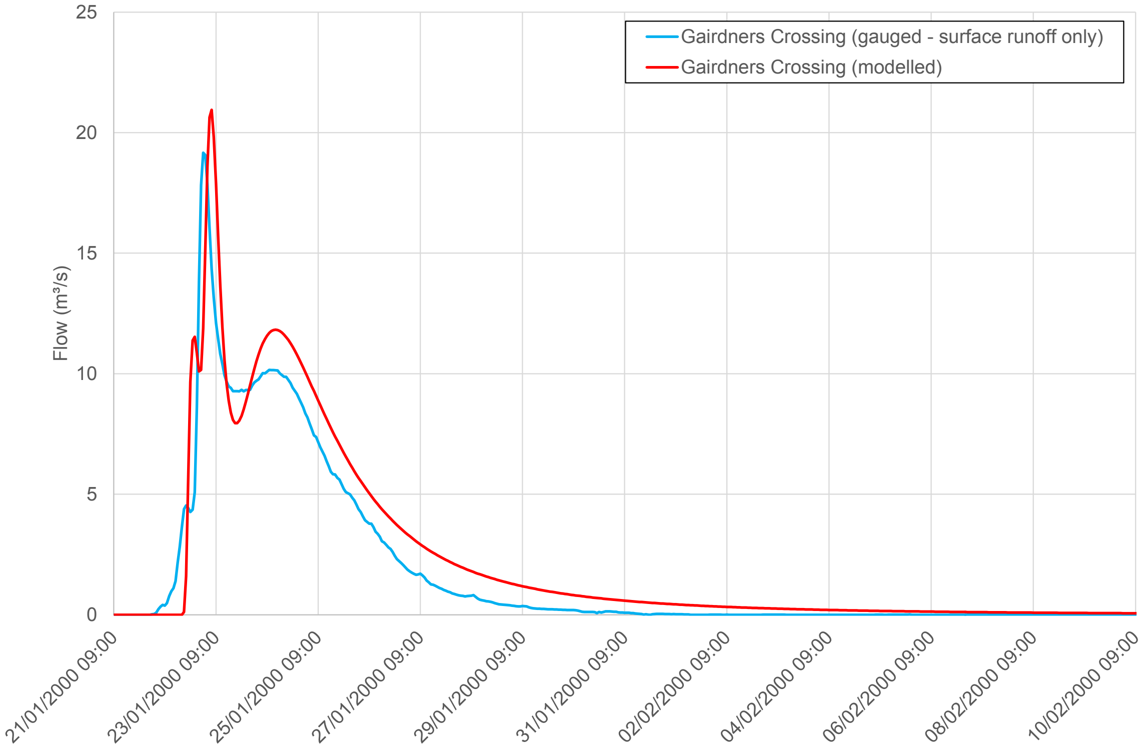


■ **Figure 6-9: Toodyay, Walyunga and Great Northern Highway hydrographs**

Baseflow separation for this event was undertaken using the procedures outlined in Section 4.2.1.

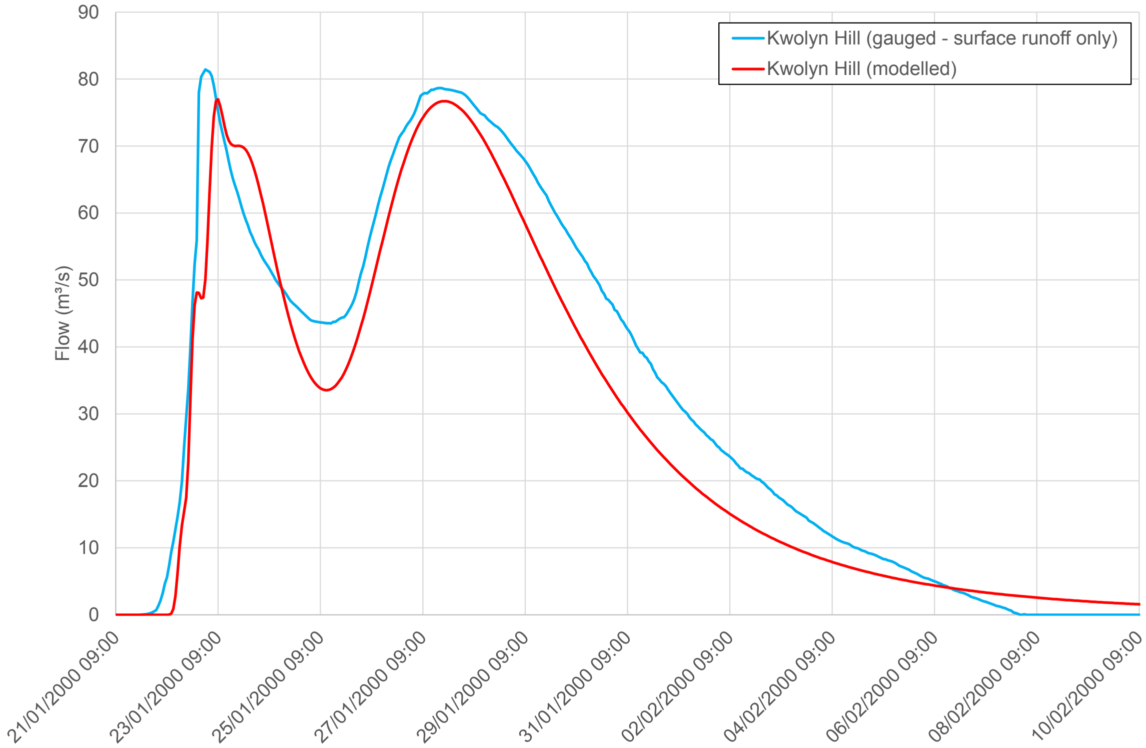
6.3.3 Calibration results

The results of the calibration for the January 2000 event are presented as plots and summary tables showing a comparison of key hydrograph characteristics at each selected gauge location. An overall table of adopted model parameter values and some discussion is provided at the end of the section.



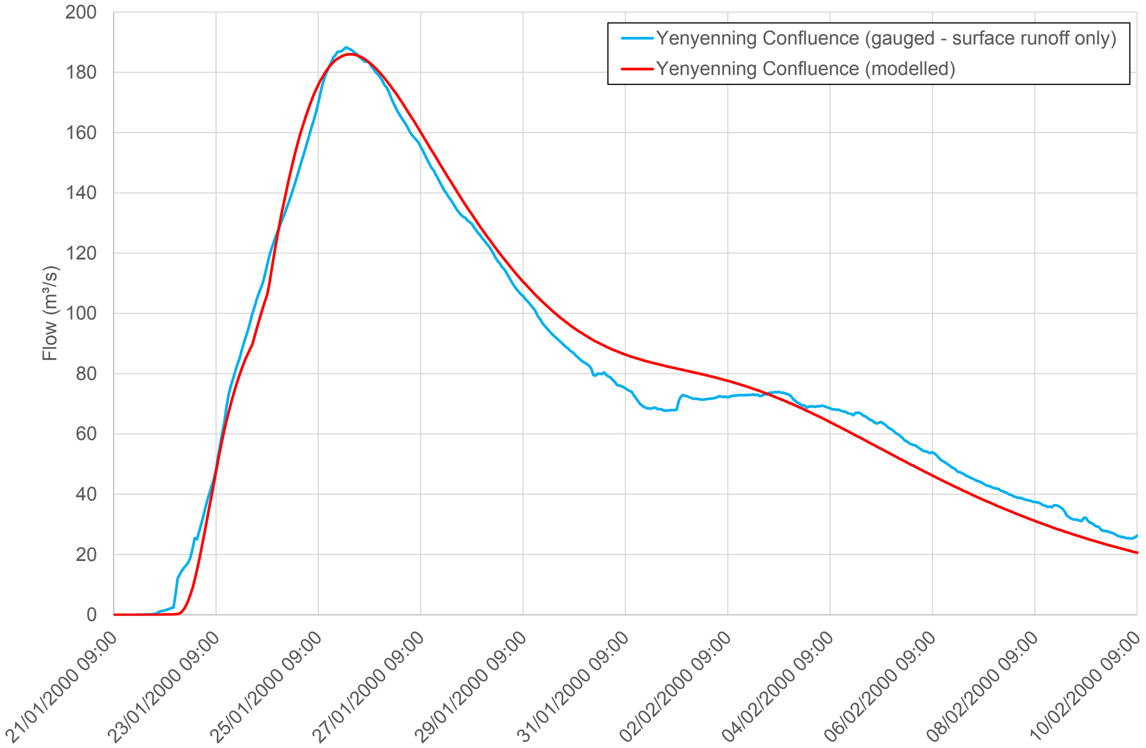
- **Figure 6-10: January 2000 model calibration at Gairdners Crossing**
- **Table 6-2: January 2000 model calibration at Gairdners Crossing**

	Gauged	Modelled	Difference
Peak 1 magnitude (m^3/s)	19.16	20.95	1.79 m^3/s
Peak 1 timing	23/1/00 3:00	23/1/2000 7:00	4 hours
Peak 2 magnitude (m^3/s)	10.15	11.82	1.67 m^3/s
Peak 2 timing	24/1/00 10:00	24/1/2000 13:00	3 hours
Total volume (GL)	3.16	4.07	28.7%



- **Figure 6-11: January 2000 model calibration at Kwolyn Hill**
- **Table 6-3: January 2000 model calibration at Kwolyn Hill**

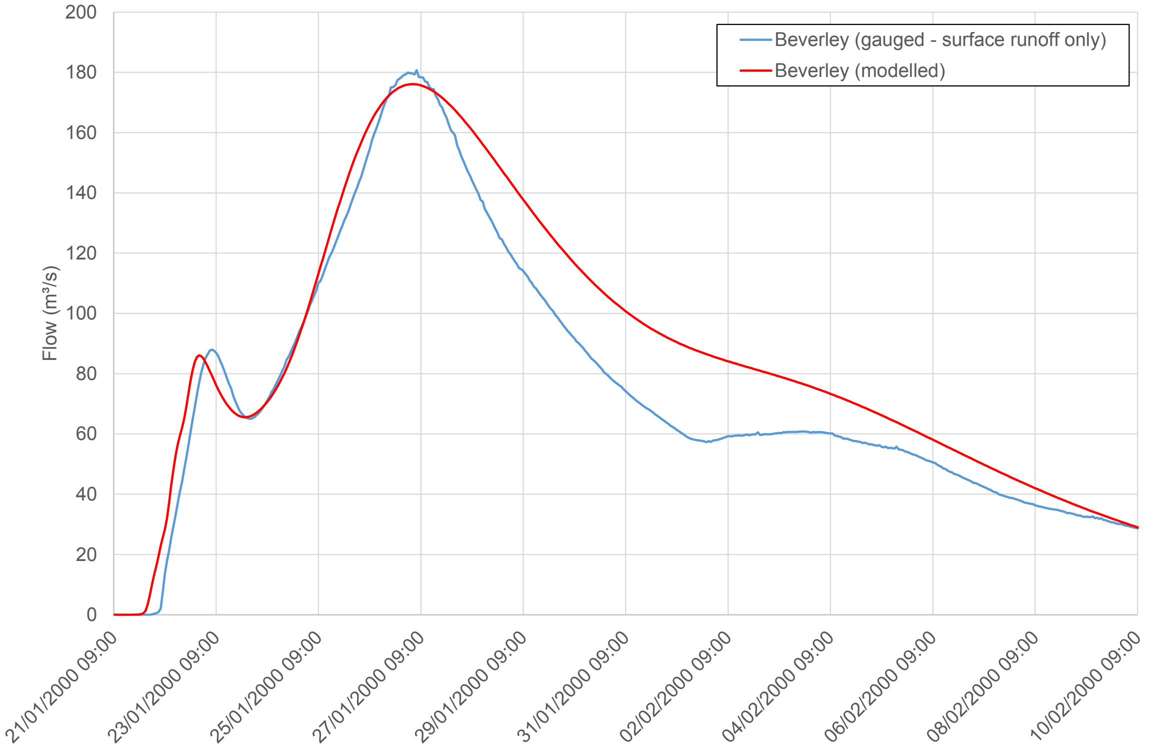
	Gauged	Modelled	Difference
Peak 1 magnitude (m^3/s)	81.45	76.96	-4.48 m^3/s
Peak 1 timing	23/1/00 3:00	23/1/2000 9:00	6 hours
Peak 2 magnitude (m^3/s)	78.65	76.70	-1.95 m^3/s
Peak 2 timing	27/1/00 17:00	27/1/2000 19:00	2 hours
Total volume (GL)	56.31	48.84	-13.3%



- **Figure 6-12: January 2000 model calibration at Yenyenning Confluence**

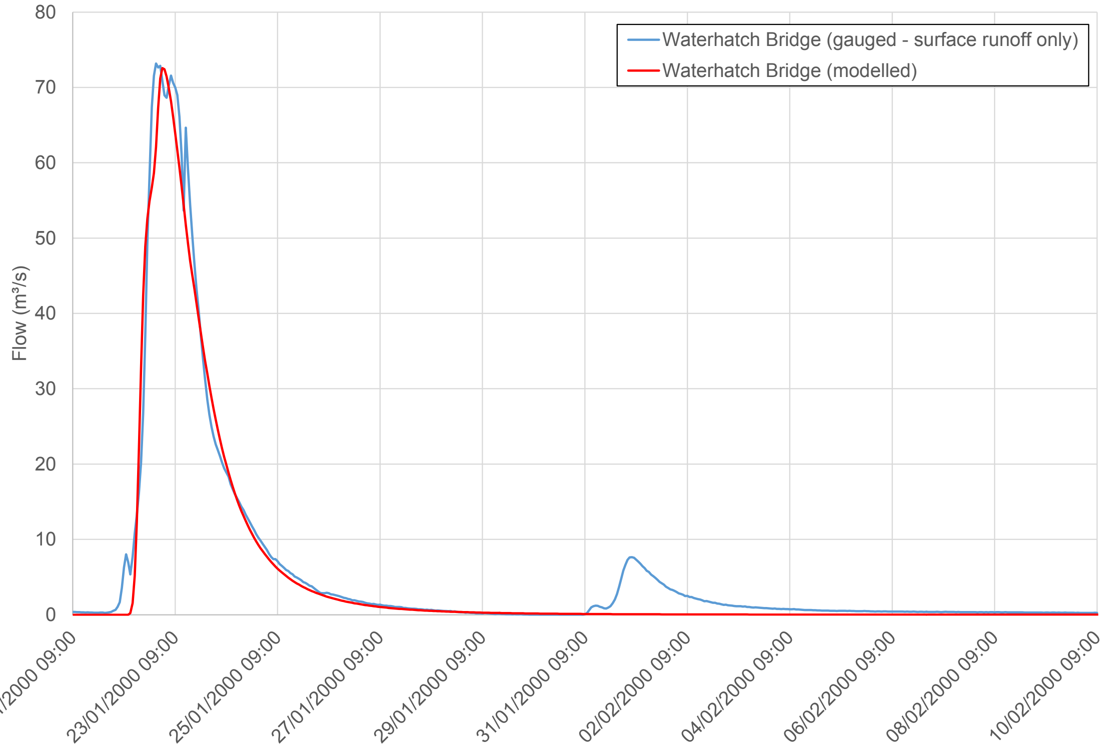
- **Table 6-4: January 2000 model calibration at Yenyenning Confluence**

	Gauged	Modelled	Difference
Peak magnitude (m^3/s)	188.32	186.0.0	-2.32 m^3/s
Peak timing	25/1/00 22:00	26/1/00 0:00	2 hours
Total volume (GL)	137.98	140.29	1.7%



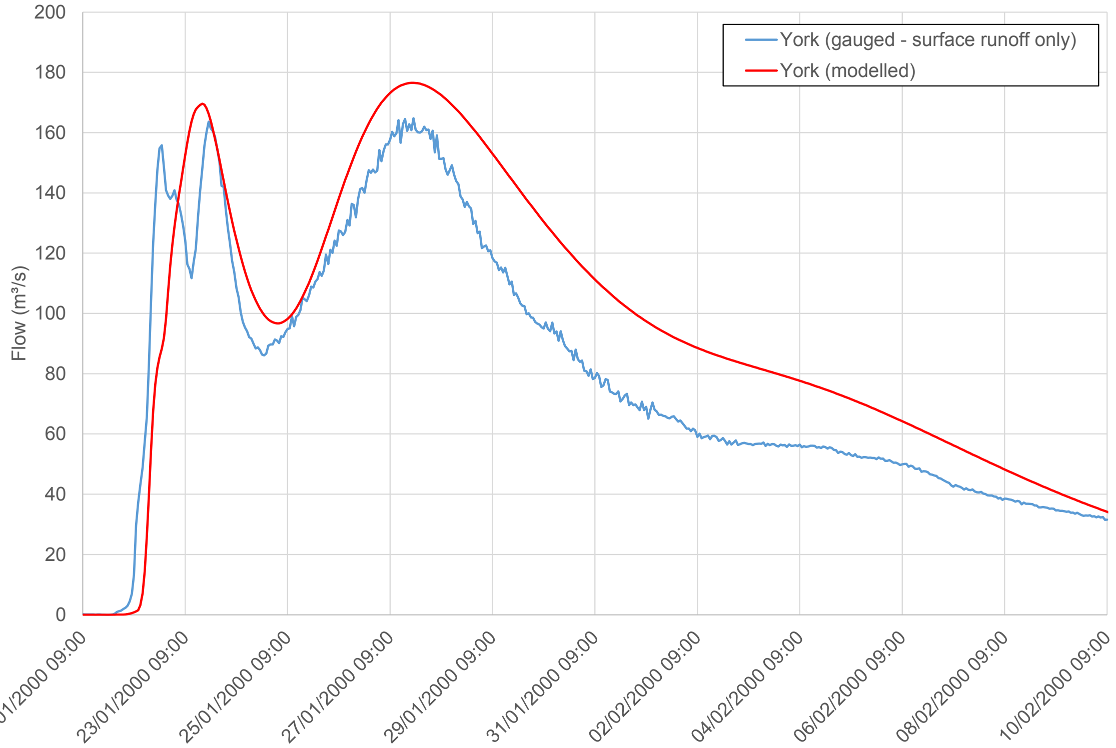
- **Figure 6-13: January 2000 model calibration at Beverley**
- **Table 6-5: January 2000 model calibration at Beverley**

	Gauged	Modelled	Difference
Peak 1 magnitude (m^3/s)	87.98	86.07	-1.92 m^3/s
Peak 1 timing	23/1/00 7:00	23/1/2000 1:00	-6 hours
Peak 2 magnitude (m^3/s)	180.70	176.09	-4.61 m^3/s
Peak 2 timing	27/1/00 7:00	27/1/2000 5:00	-2 hours
Total volume (GL)	130.37	153.18	17.5%



- **Figure 6-14: January 2000 model calibration at Waterhatch Bridge**
- **Table 6-6: January 2000 model calibration at Waterhatch Bridge**

	Gauged	Modelled	Difference
Peak magnitude (m^3/s)	73.18	72.58	-0.60 m^3/s
Peak timing	23/1/00 00:00	23/1/00 03:00	3 hours
Total volume (GL)	9.13	8.72	-4.5%



- **Figure 6-15: January 2000 model calibration at York**
- **Table 6-7: January 2000 model calibration at York**

	Gauged	Modelled	Difference
Peak 1 magnitude (m^3/s)	155.82	No distinct peak modelled	
Peak 1 timing	22/1/2000 22:00		
Peak 2 magnitude (m^3/s)	163.64	169.61	5.97 m^3/s
Peak 2 timing	23/1/2000 20:00	23/1/2000 17:00	-3 hours
Peak 3 magnitude (m^3/s)	164.78	176.52	11.74 m^3/s
Peak 3 timing	27/1/2000 20:00	27/1/2000 20:00	0 hours
Total volume (GL)	138.03	168.38	22.0%

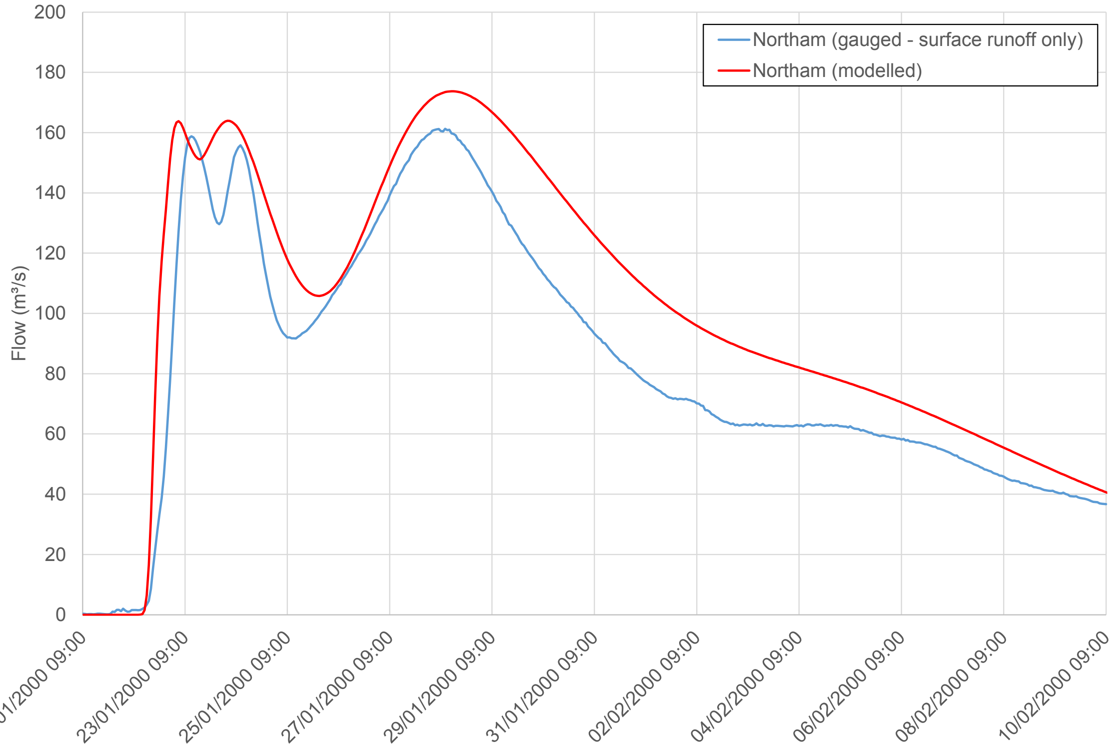
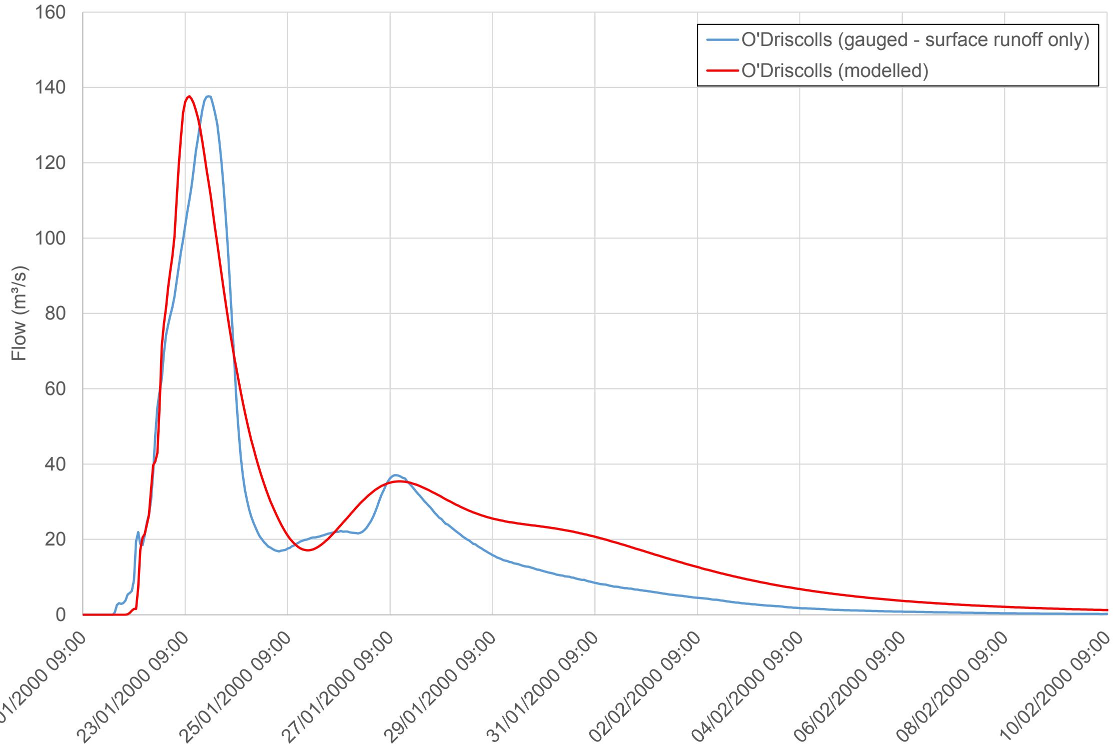


Figure 6-16: January 2000 model calibration at Northam

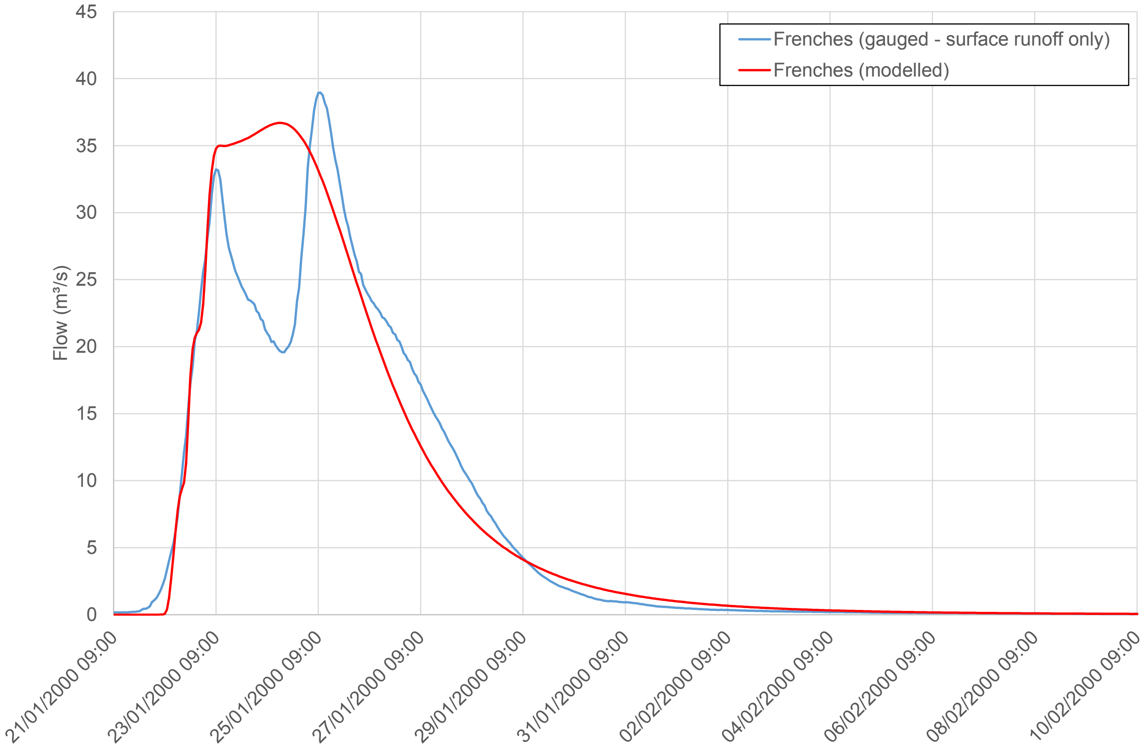
Table 6-8: January 2000 model calibration at Northam

	Gauged	Modelled	Difference
Peak 1 magnitude (m^3/s)	158.80	163.82	5.02 m^3/s
Peak 1 timing	23/1/2000 12:00	23/1/2000 6:00	-6 hours
Peak 2 magnitude (m^3/s)	155.82	163.91	8.09 m^3/s
Peak 2 timing	24/1/2000 11:00	24/1/00 5:00	-6 hours
Peak 3 magnitude (m^3/s)	161.26	173.72	12.46 m^3/s
Peak 3 timing	28/1/2000 11:00	28/1/2000 14:00	3 hours
Total volume (GL)	144.55	178.86	23.7%



- **Figure 6-17: January 2000 model calibration at O'Driscolls**
- **Table 6-9: January 2000 model calibration at O'Driscolls**

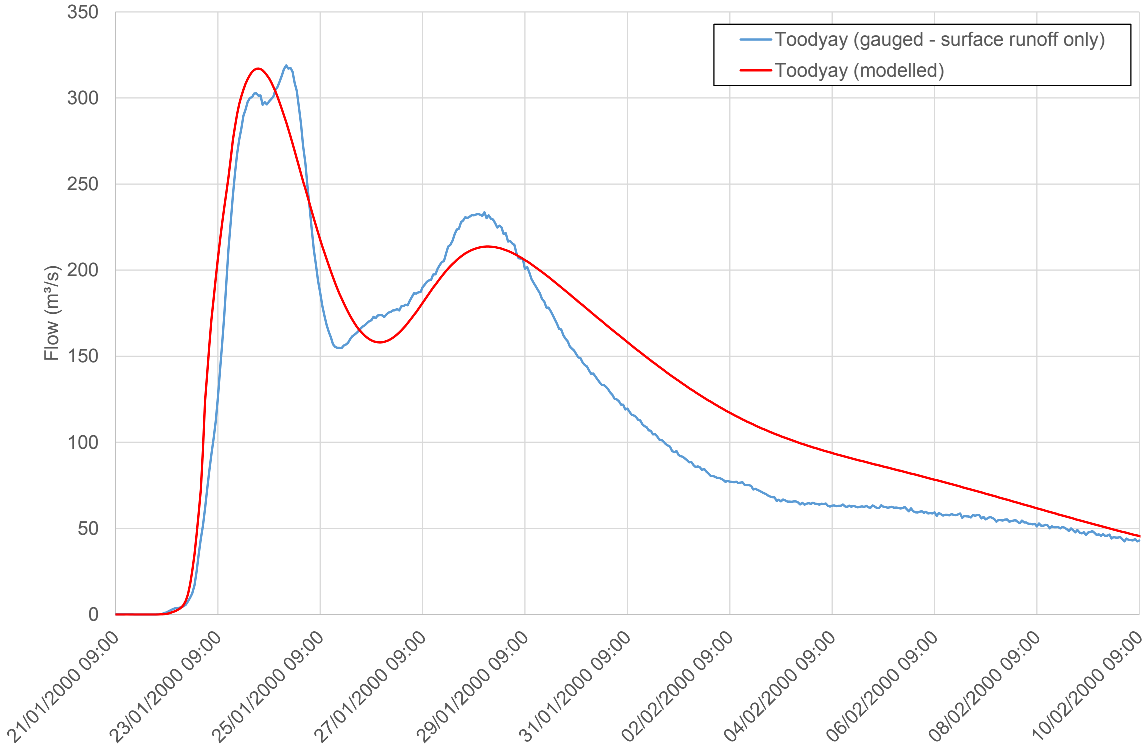
	Gauged	Modelled	Difference
Peak 1 magnitude (m^3/s)	137.66	137.64	-0.02 m^3/s
Peak 1 timing	23/1/2000 20:00	23/1/2000 11:00	-9 hours
Peak 2 magnitude (m^3/s)	37.04	35.40	-1.63 m^3/s
Peak 2 timing	27/1/2000 11:00	27/1/2000 13:00	2 hours
Total volume (GL)	29.62	37.97	28.2%



- **Figure 6-18: January 2000 model calibration at Frenches**

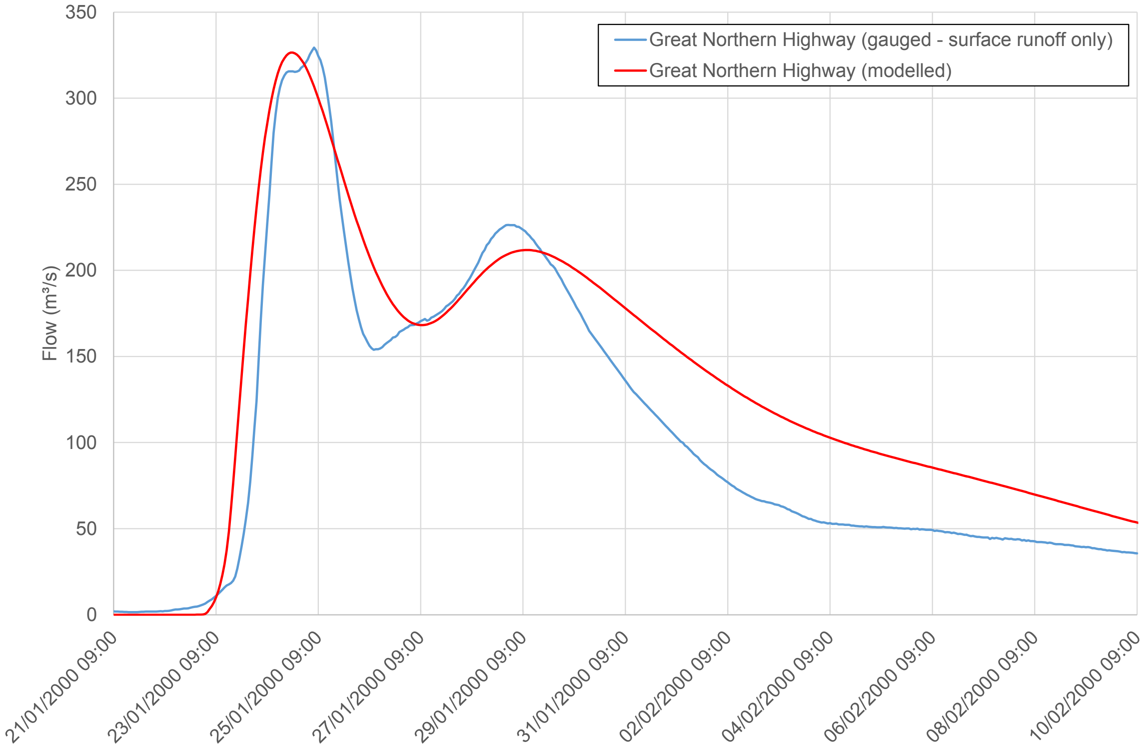
- **Table 6-10: January 2000 model calibration at Frenches**

	Gauged	Modelled	Difference
Peak 1 magnitude (m^3/s)	33.22	35.00	1.78 m^3/s
Peak 1 timing	23/1/2000 9:00	23/1/2000 11:00	2 hours
Peak 2 magnitude (m^3/s)	38.97	36.70	-2.27 m^3/s
Peak 2 timing	25/1/2000 10:00	24/1/2000 15:00	-19 hour
Total volume (GL)	12.65	13.50	6.8%



- **Figure 6-19: January 2000 model calibration at Toodyay**
- **Table 6-11: January 2000 model calibration at Toodyay**

	Gauged	Modelled	Difference
Peak 1 magnitude (m^3/s)	318.94	317.08	-1.85 m^3/s
Peak 1 timing	24/1/2000 17:00	24/1/2000 4:00	-13 hours
Peak 2 magnitude (m^3/s)	233.59	213.68	-19.90 m^3/s
Peak 2 timing	28/1/2000 14:00	28/1/2000 16:00	2 hours
Total volume (GL)	200.24	232.59	16.2%



- **Figure 6-20: January 2000 model calibration at Great Northern Highway**
- **Table 6-12: January 2000 model calibration at Great Northern Highway**

	Gauged	Modelled	Difference
Peak 1 magnitude (m^3/s)	329.39	326.50	-2.89 m^3/s
Peak 1 timing	25/1/2000 7:00	24/1/2000 21:00	-10 hours
Peak 2 magnitude (m^3/s)	226.38	211.82	-14.56 m^3/s
Peak 2 timing	29/1/2000 2:00	29/1/2000 11:00	9 hours
Total volume (GL)	185.12	236.71	27.9%

Table 6-13: January 2000 model calibration summary

Interstation area	Average flow distance (km)	k_c	$C_{0.8}$	Initial loss (mm)	Runoff coefficient
Yilgarn River	1,102.3	500	0.45[1]	40	0.04
Lockhart River	745.4	270	0.36[1]	45	0.04
Salt River to Yenyenning Lakes	83.3	230	2.76	10	0.16
Avon River to Boyagarra Rd	74.6	190	2.55	27	0.30
Avon River to Beverley	37.1	95	2.56	5	0.22
Dale River to Waterhatch Bridge	37.7	45	1.19	7	0.08
Avon River to York	27.0	40	1.48	5	0.14
Avon River to Northam	29.7	45	1.52	10	0.35
Mortlock River East Branch	172.4	190	1.10	15	0.04
Mortlock River East Branch to O'Driscolls	41.2	57	1.38	5	0.19
Mortlock River North Branch to Lake Ninan	95.6	65	0.68	20	0.10
Mortlock River North Branch to Frenches	51.3	55	1.07	22	0.05
Avon River to Toodyay	34.5	35	1.01	25	0.15
Brockman River	67.7	Insufficient streamflow to enable credible calibration			
Avon River to Walyunga	55.1	65	1.18	30	0.25
Helena River	53.0	Insufficient streamflow to enable credible calibration			
Swan River	43.8	No streamflow gauge			

[1] Average flow distance values in these interstation areas are affected by reach length factoring

The results for this event have demonstrated a reasonable fit to a complex event. The nature of the event, with multiple hydrographs passing through the catchment from different sources, makes it difficult to calibrate to. The effect of this is seen in particular at the Northam, Toodyay and Great Northern Highway gauges, where the model does not accurately simulate the shape of the first two (local runoff) peaks.

It was noted that the calibrated initial loss values for this summer event are somewhat lower than would typically be expected, particularly in the mid and lower reaches of the Avon River. This is presumably the result of rainfall antecedent to the event leading to higher than normal soil moisture levels. The daily rainfall gauge at Northam (010111) indicates that there was a rainfall event between January 15 and 17 with a total depth of over 20 mm. This event would potentially have contributed to higher soil moisture levels and may help explain these unusually low summer initial loss values.

6.4 July 1983 event

The July 1983 event was defined as commencing at 9 am on 23 July and ending at 9 am on 12 August. The period of heavy rainfall which resulted in the main peak ceased by 9 am on July 26. There were additional rainfall bursts in later July and early August which resulted in some small rises in the hydrographs, but these bursts were of negligible value for calibration purposes and were not modelled.

6.4.1 Rainfall inputs

Total rainfall depths for each RORB model subarea were derived from AWAP data covering the period of interest. The AWAP grids representing each day's rainfall were accumulated and used to define the rainfall depth for each sub-area centroid. A map showing the accumulated rainfall depth for this event is shown in Figure 6-22.

Six minute rainfall data from a number of Bureau of Meteorology and Department of Water pluviograph stations was extracted and used to define the rainfall temporal pattern. A total of six stations with usable recorded data for the event were found, mainly located around the downstream end of the catchment as shown in the map in Figure 6-22.

The pluviograph data at each station was aggregated to a one hour timestep, and then regularised to percentage of total storm depth. Cumulative plots of the rainfall temporal distribution at each pluviograph station are shown in Figure 6-21. Allocation of the pluviographs to each model subarea was initially based on proximity, however this was changed during the course of the calibration process to improve the fit. The adopted pluviograph stations for each interstation area are shown in Table 6-14.

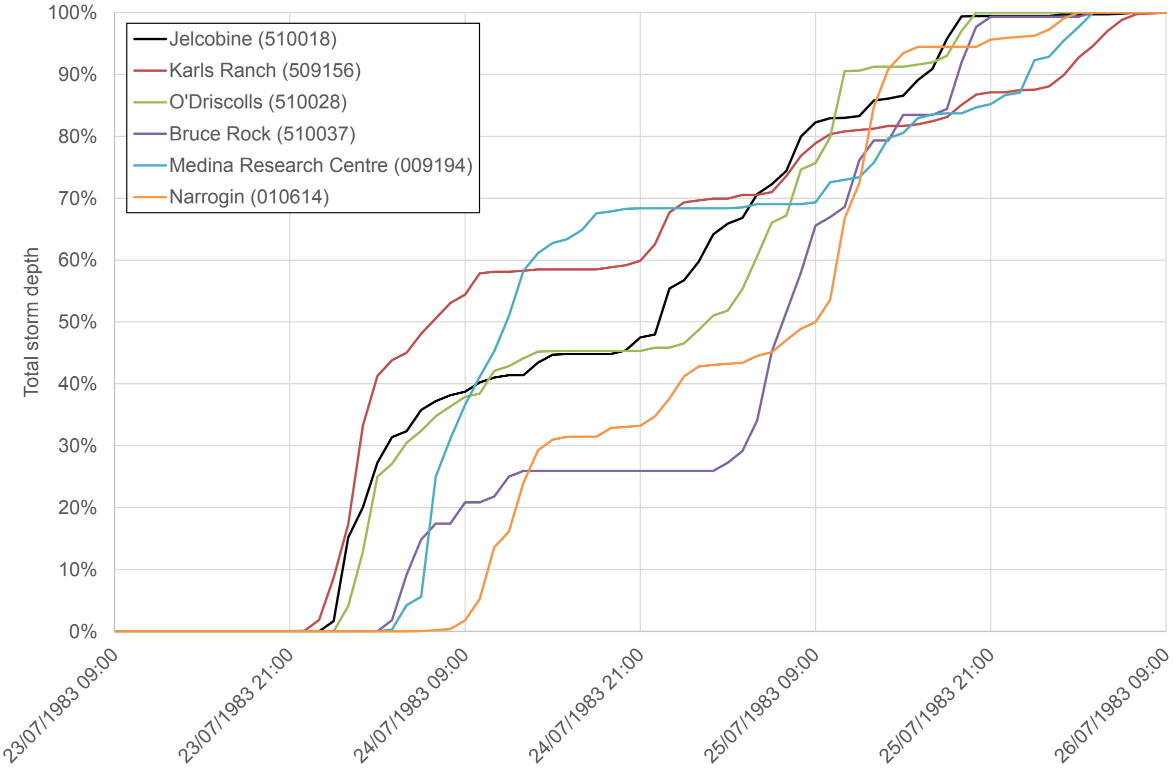


Figure 6-21: July 1983 pluviograph data

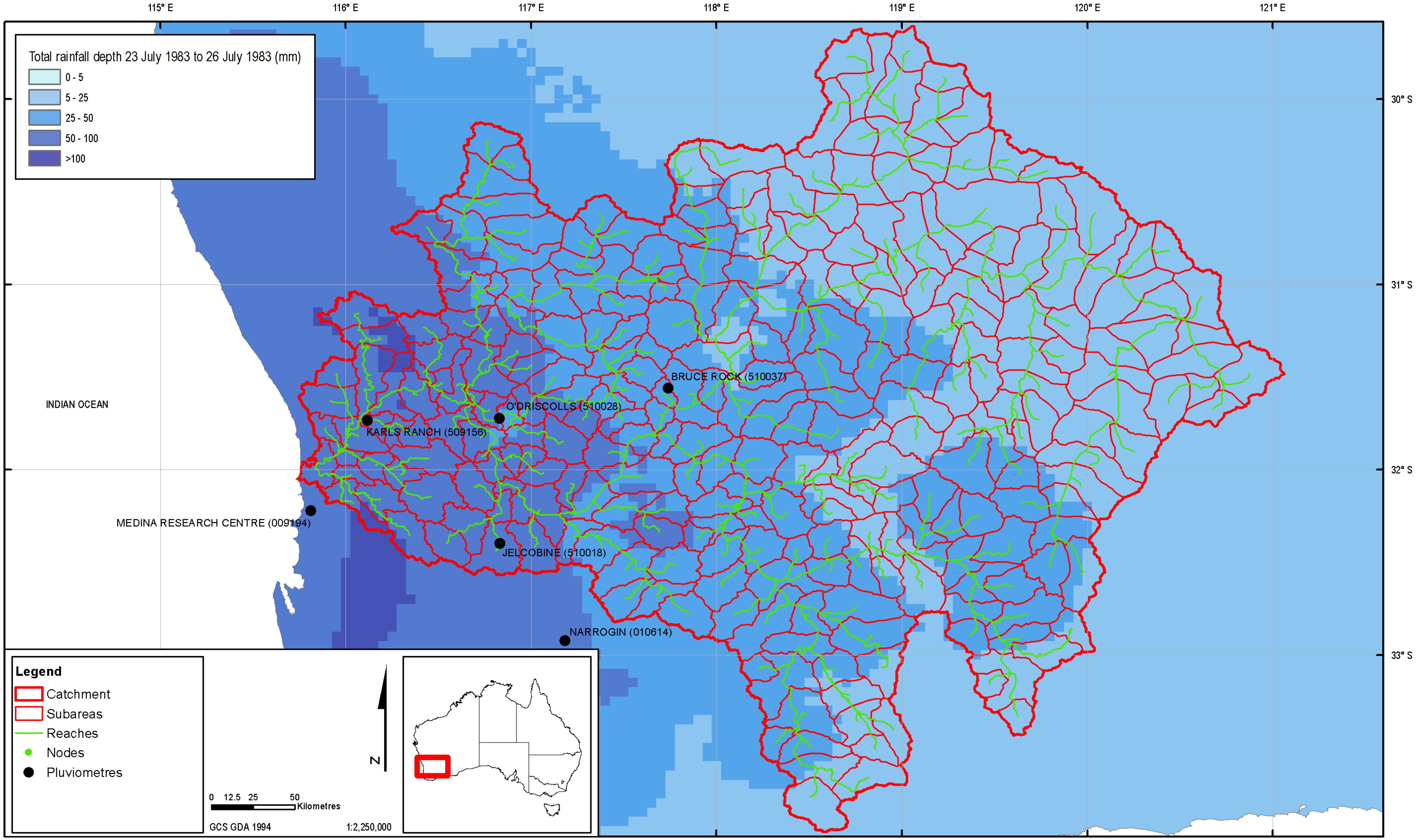


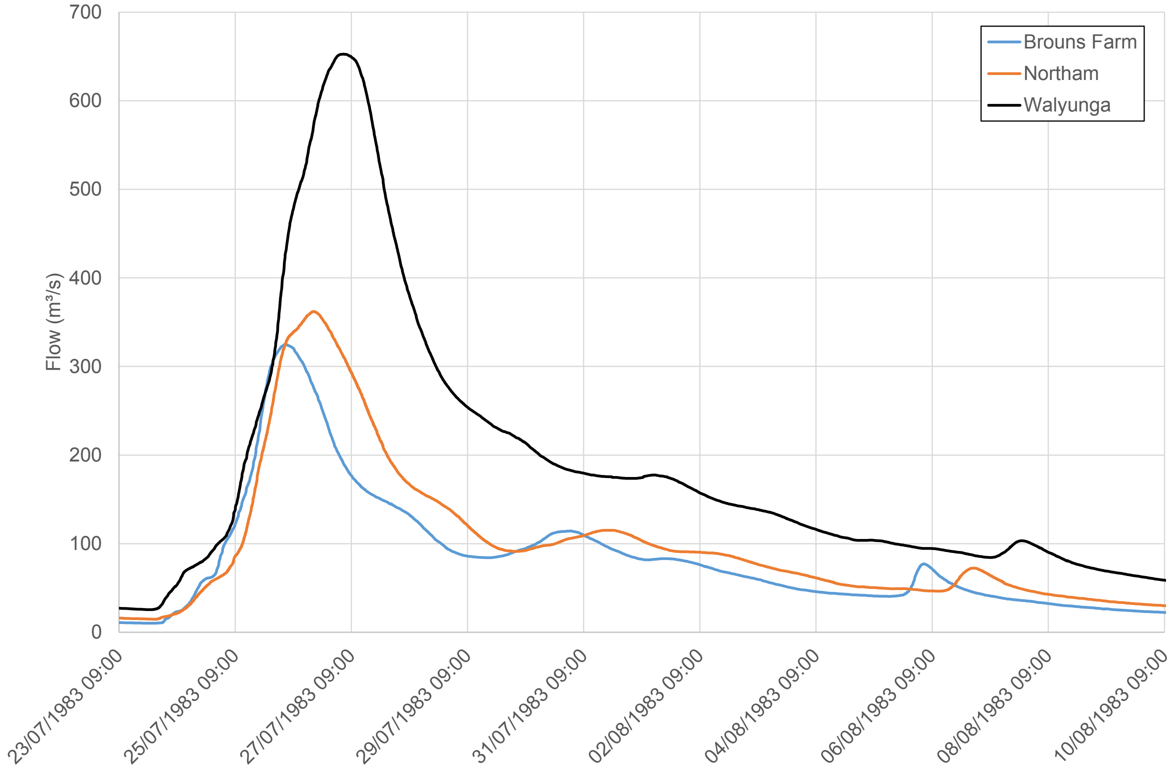
Figure 6-22: July 1983 flood event rainfall depths

Table 6-14: July 1983 event assigned pluviograph stations

Interstation area	Pluviograph station
Yilgarn River	Bruce Rock (510037)
Lockhart River	Bruce Rock (510037)
Salt River to Yenyenning Lakes	Bruce Rock (510037)
Avon River to Boyagarra Rd	Jelcobine (510018)
Avon River to Beverley	Jelcobine (510018)
Dale River to Waterhatch Bridge	O’Driscolls (510028)
Avon River to York	O’Driscolls (510028)
Avon River to Northam	O’Driscolls (510028)
Mortlock River East Branch	O’Driscolls (510028)
Mortlock River East Branch to O’Driscolls	O’Driscolls (510028)
Mortlock River North Branch to Lake Ninan	O’Driscolls (510028)
Mortlock River North Branch to Frenches	O’Driscolls (510028)
Avon River to Toodyay	Karls Ranch (509156)
Brockman River	Karls Ranch (509156)
Avon River to Walyunga	Karls Ranch (509156)
Helena River	Karls Ranch (509156)
Swan River	Karls Ranch (509156)

6.4.2 Streamflow inputs

The July 1983 event was recorded at three main gauges along the Avon River: Brouns Farm, Northam and Walyunga. In addition to this, gauged hydrographs are available on the Mortlock River at Frenches and O’Driscolls. Some data is available for the gauge at Qualandary Crossing, however the peak of the hydrograph appears to be missing and so was not used for calibration. Upstream of the Yenyenning Lakes, the gauges at Gairdners Crossing and Kwolyn Hill were operational but the flows recorded there for this event were negligible. The July 1983 hydrographs along the Avon River are shown in Figure 6-23.

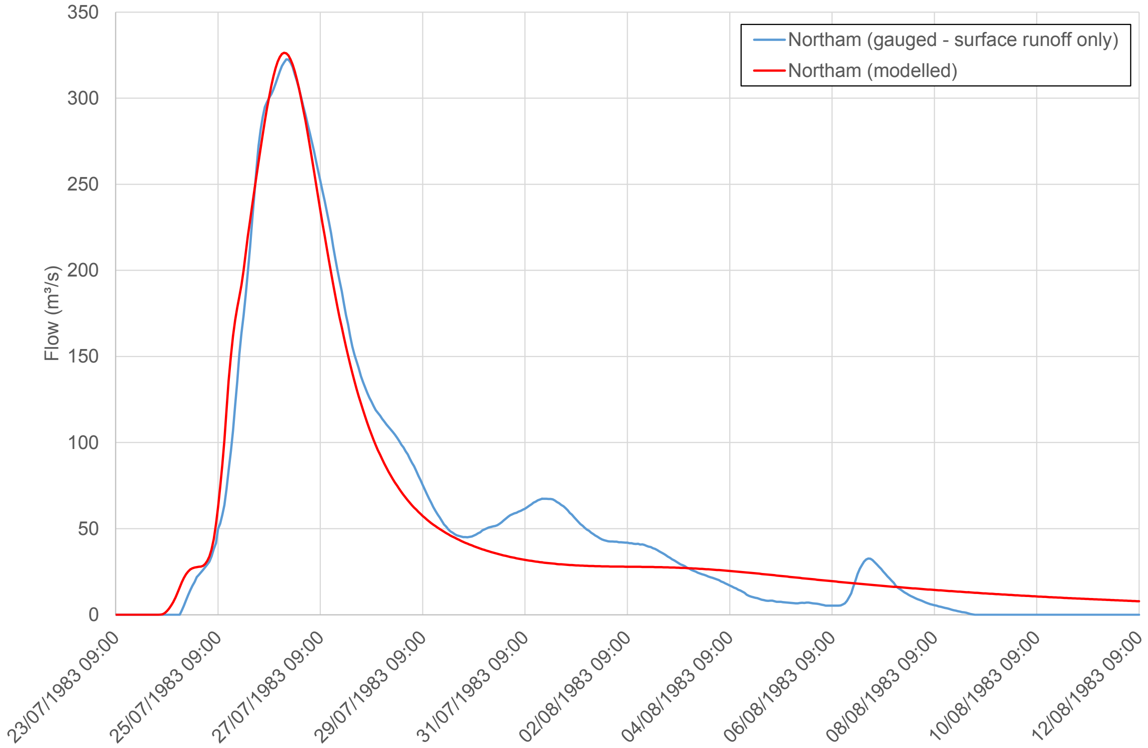


- **Figure 6-23: July 1983 Avon River hydrographs**

Baseflow separation was undertaken on the gauged hydrographs for this event using the procedures outlined in Section 4.2.1.

6.4.3 Calibration results

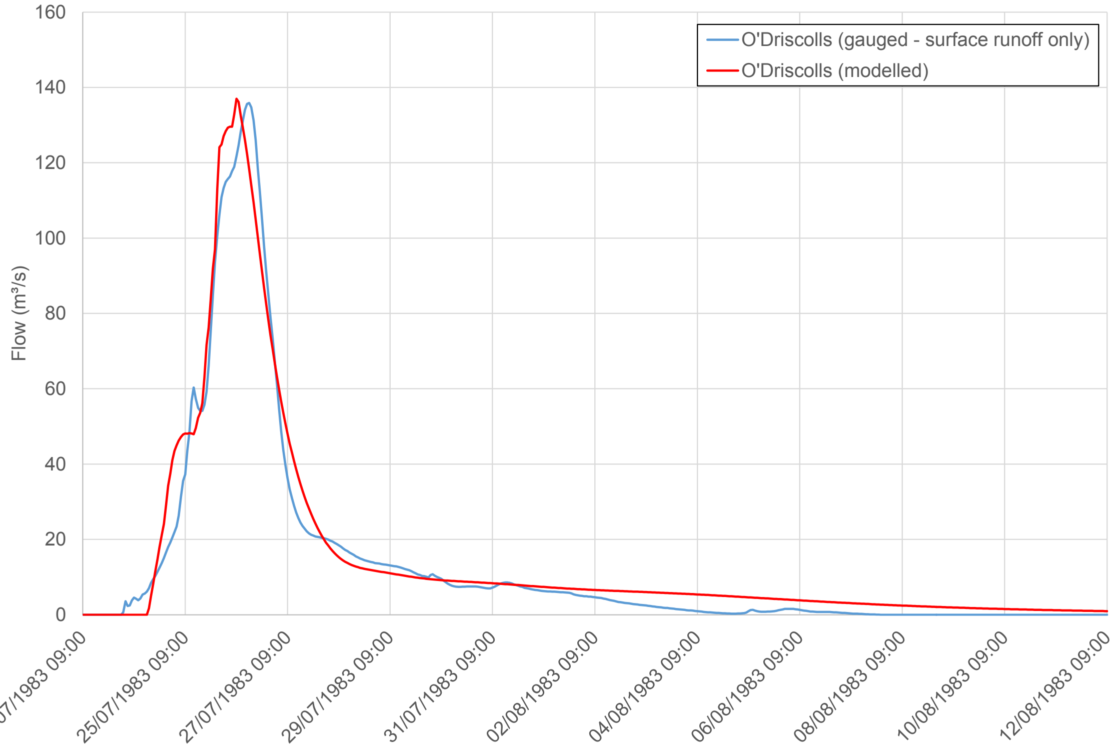
The results of the calibration for the July 1983 event are presented as plots and summary tables showing a comparison of key hydrograph characteristics at each selected gauge location. An overall table of adopted model parameter values and some discussion is provided at the end of the section.



- **Figure 6-24: July 1983 model calibration at Northam**

- **Table 6-15: July 1983 model calibration at Northam**

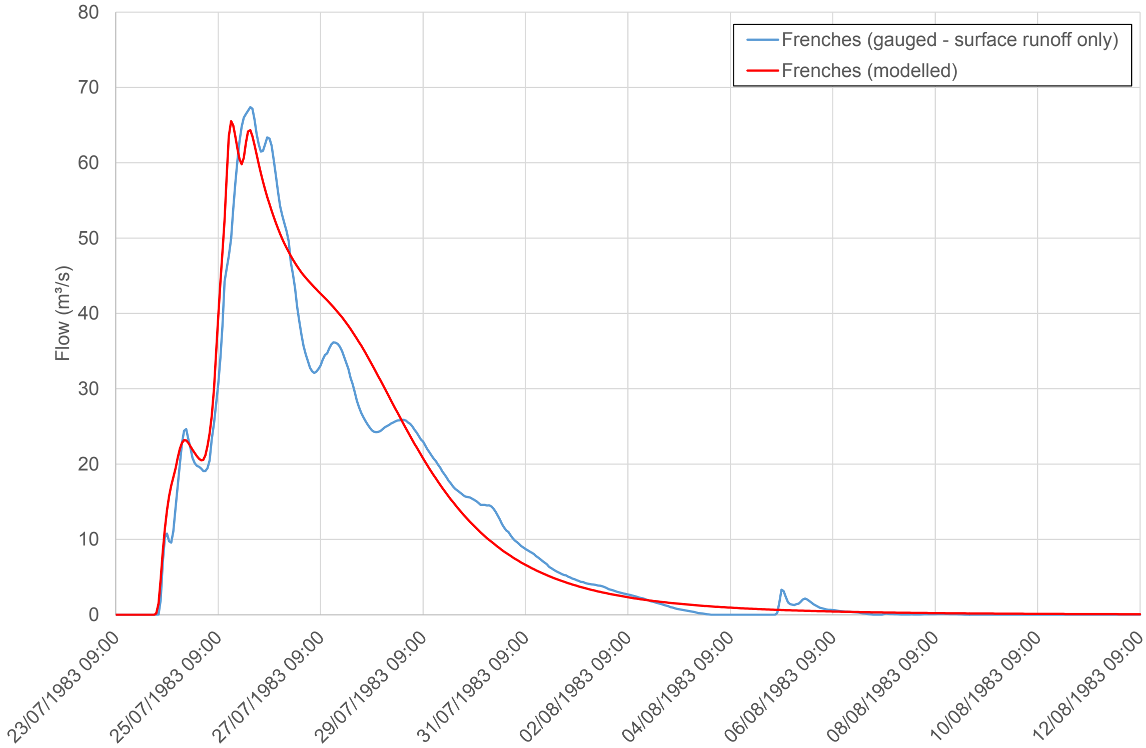
	Gauged	Modelled	Difference
Peak magnitude (m^3/s)	322.61	326.39	3.77 m^3/s
Peak timing	26/7/83 17:00	26/7/83 16:00	-1 hour
Total volume (GL)	95.25	92.93	-2.4%



- **Figure 6-25: July 1983 model calibration at O'Driscolls**

- **Table 6-16: July 1983 model calibration at O'Driscolls**

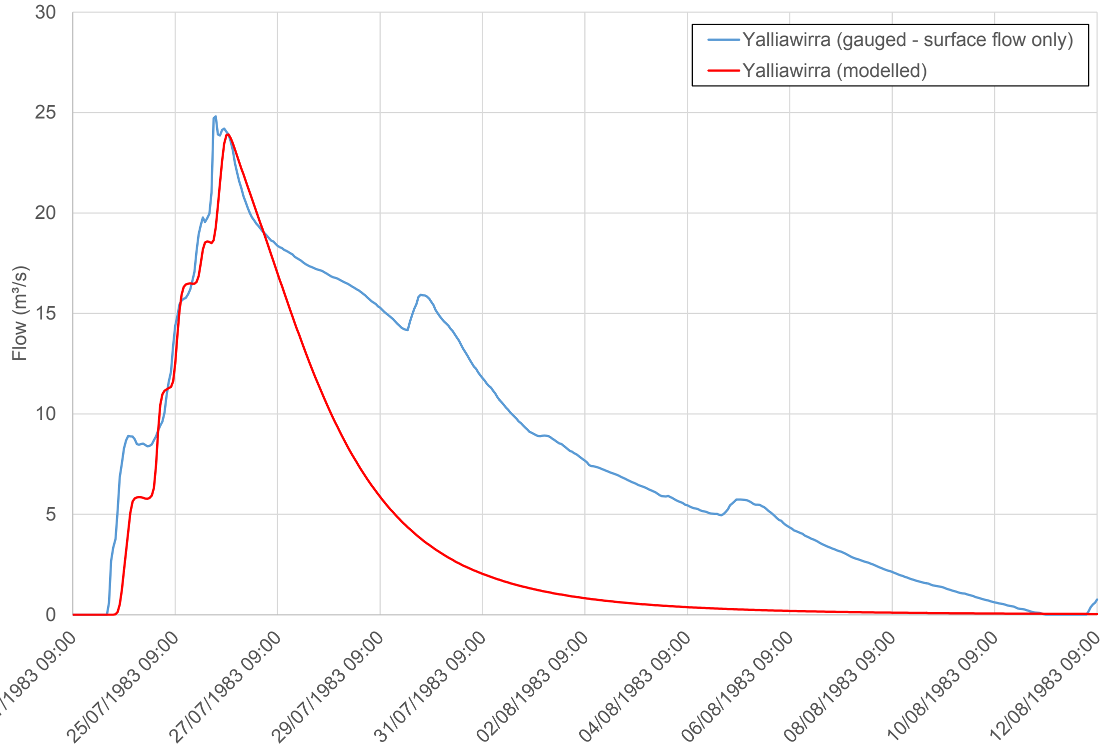
	Gauged	Modelled	Difference
Peak magnitude (m^3/s)	138.86	136.98	1.11 m^3/s
Peak timing	26/7/83 15:00	26/7/83 9:00	-6 hours
Total volume (GL)	23.47	26.67	13.6%



- **Figure 6-26: July 1983 model calibration at Frenches**

- **Table 6-17: July 1983 model calibration at Frenches**

	Gauged	Modelled	Difference
Peak magnitude (m^3/s)	67.38	65.53	-1.85 m^3/s
Peak timing	26/7/83 0:00	25/7/83 15:00	-9 hours
Total volume (GL)	19.33	20.21	4.5%

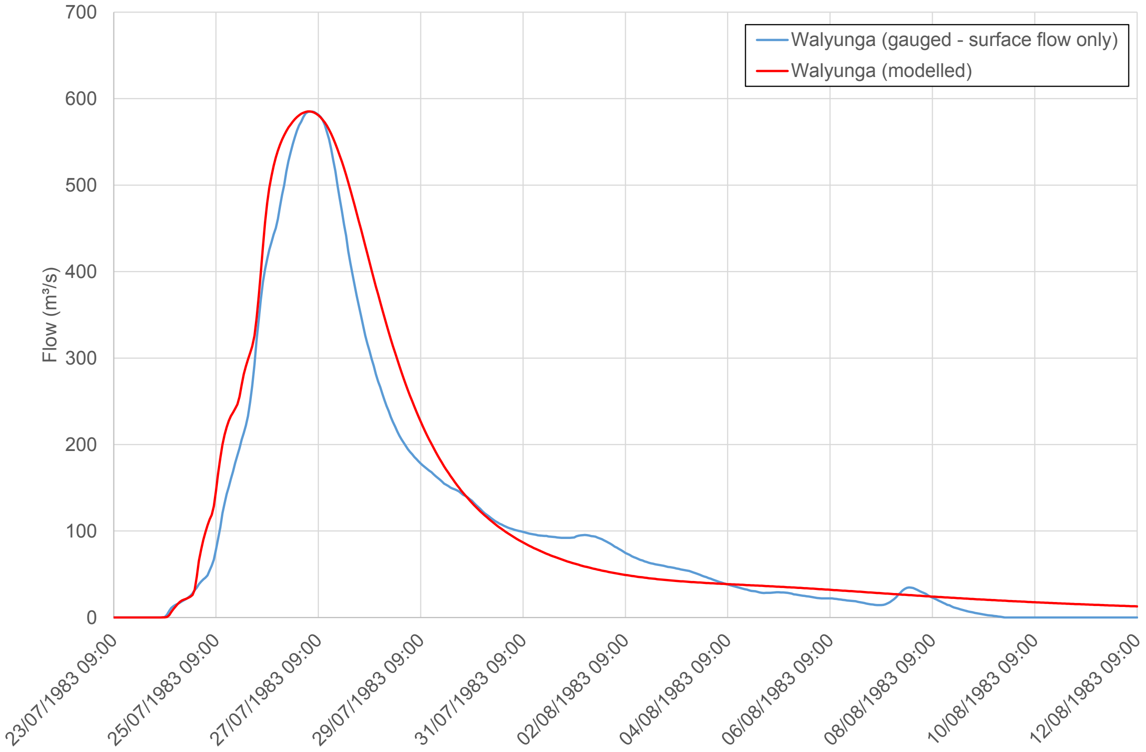


- **Figure 6-27: July 1983 model calibration at Yalliawirra**

- **Table 6-18: July 1983 model calibration at Yalliawirra**

	Gauged	Modelled	Difference
Peak magnitude (m^3/s)	24.81	23.91	-0.90 m^3/s
Peak timing	26/7/83 4:00	26/7/83 10:00	6 hours
Total volume (GL)	14.24	6.87	-51.7%

It can be seen that a reasonable match has been achieved to the peak flow at Yalliawirra, however the overall volume is underestimated. This is likely to be the result of the presence of Lakes Chittering and Needonga, which act to attenuate runoff from the upper portion of the catchment. These lakes have not been explicitly modelled, and so it is likely that the actual runoff coefficient for this interstation area is somewhat higher than the calibrated value, which would add additional volume to the hydrograph. There is little justification for adding additional complexity to the model in this area, as the peak flow and volume from the Brockman River catchment is negligible in comparison to the hydrograph at Walyunga.



- **Figure 6-28: July 1983 model calibration at Walyunga**
- **Table 6-19: July 1983 model calibration at Walyunga**

	Gauged	Modelled	Difference
Peak magnitude (m^3/s)	585.11	584.49	-0.61 m^3/s
Peak timing	27/7/83 5:00	27/7/83 6:00	1 hour
Total volume (GL)	184.94	196.56	6.3 %

Table 6-20: July 1983 model calibration summary

Interstation area	Average flow distance (km)	k_c	$C_{0.8}$	Initial loss (mm)	Runoff coefficient
Yilgarn River	1,102.3	Insufficient streamflow to enable credible calibration			
Lockhart River	745.4	Insufficient streamflow to enable credible calibration			
Salt River to Yenyenning Lakes	83.3	230	2.76	30	0.20
Avon River to Boyagarra Rd	74.6	190	2.55	20	0.20
Avon River to Beverley	37.1	95	2.56	20	0.34
Dale River to Waterhatch Bridge	37.7	45	1.19	20	0.35
Avon River to York	27.0	35	1.30	20	0.35
Avon River to Northam	29.7	40	1.35	20	0.35
Mortlock River East Branch	172.4	190	1.10	20	0.04
Mortlock River East Branch to O'Driscolls	41.2	50	1.21	10	0.31
Mortlock River North Branch to Lake Ninan	95.6	60	0.63	25	0.10
Mortlock River North Branch to Frenches	51.3	60	1.17	10	0.11
Avon River to Toodyay	34.5	35	1.01	40	0.60
Brockman River	67.7	85	1.26	40	0.08
Avon River to Walyunga	55.1	70	1.27	50	0.48
Helena River	53.0	Insufficient streamflow to enable credible calibration			
Swan River	43.8	No streamflow gauge			

This event fits well to a single peaked hydrograph generated primarily off the lower catchment. The peaks of the very small flows in the Yilgarn and Lockhart Rivers are matched but there is little confidence in the model's estimate of hydrograph shape or timing for these catchments. On the Avon River, good matches are achieved at Northam and Walyunga using routing parameter values which are remarkably consistent with those adopted for the January 2000 event. It was noted that a good match to rising limb and peak flow was achieved on the Brockman River at Yalliawirra, however the falling limb and overall hydrograph volume was underestimated. This is presumably due to the effect of Lakes Chittering and Needonga, which capture a significant proportion of the catchment runoff and attenuate it. Due to their relatively small influence, these lakes have not been specifically modelled.

It was noted that the calibrated runoff coefficient values in the Avon River interstation area between Northam and Toodyay was unusually high (0.60). There is no apparent explanation for this, but it may be due to a number of factors including underestimates of the gauged rainfall depths in this area.

6.5 July 1974 event

The July 1974 flood event was defined as beginning at 9 am on July 9, 1974 and ending at 9 am on August 27, 1974. This is an extremely long duration flood event marked by a sustained rise in the Avon River at Walyunga, with numerous individual smaller peaks. Period of intermittent heavy rainfall occurred through July and August 1974 over the catchment.

6.5.1 Rainfall inputs

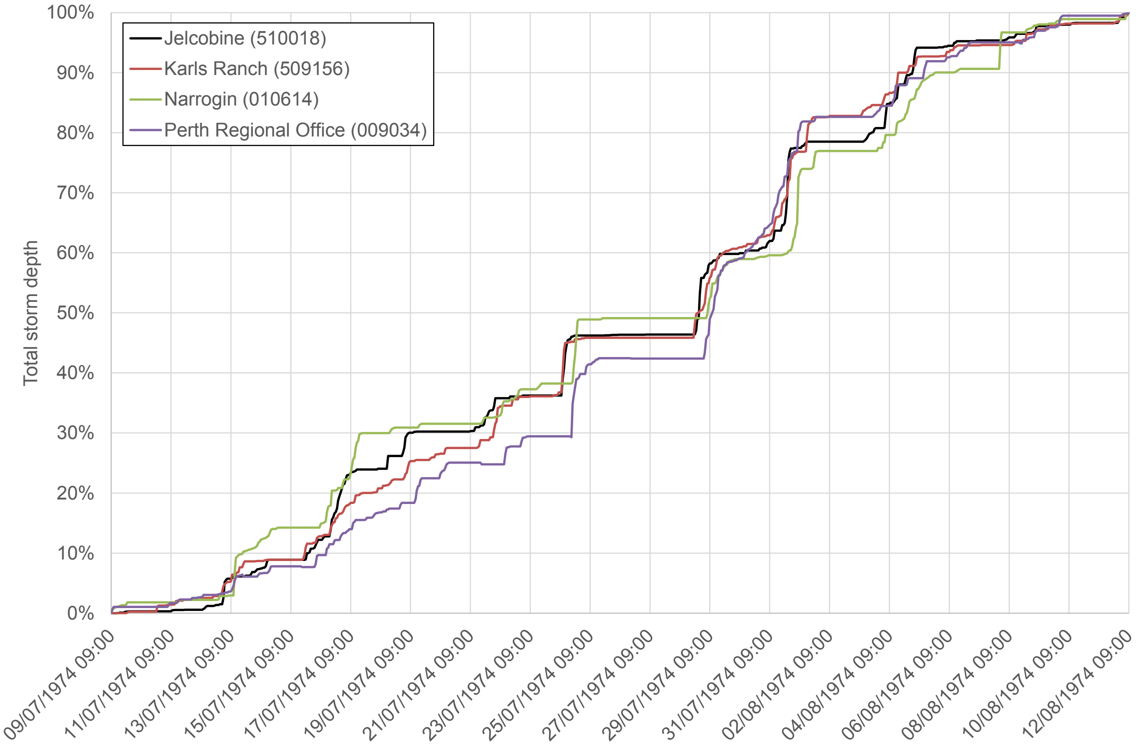
Total rainfall depths for each RORB model subarea were derived from AWAP data covering the period of interest. The AWAP grids representing each day's rainfall were accumulated and used to define the rainfall depth for each sub-area centroid. A map showing the accumulated rainfall depth for this event is shown in Figure 6-30.

Six minute rainfall data from a number of Bureau of Meteorology and Department of Water pluviograph stations was extracted and used to define the rainfall temporal pattern. A total of 4 stations with usable recorded data for the event were found, mainly located around the downstream end of the catchment. All four pluviograph stations are located in or near the western part of the catchment as per the locations shown in the map in Figure 6-30.

The pluviograph data at each station was aggregated to a one hour timestep, and then regularised to percentage of total storm depth. Cumulative plots of the rainfall temporal distribution at each pluviograph station are shown in Figure 6-29: July 1974 pluviograph data. Allocation of the pluviographs to each model subarea was initially based on proximity, however this was changed during the course of the calibration process to improve the fit. The adopted pluviograph stations for each interstation area are shown in Table 6-21.

It should be noted that this event was modelled as a single rainfall 'burst' in RORB. This means that the initial loss parameter has only a minor influence on an event this large. There is a clear separation between rainfall bursts over the period July 25 to July 29, which would potentially allow the initial loss to reset somewhat. Due to the limitations on input data size imposed within RORB however, it was not possible to easily model this event as two separate bursts, and so the effect of recovering initial loss was ignored.

Notwithstanding this, a second run of this event was also undertaken which covered only the period from July 28 to August 12. This second run was used solely to calibrate the Helena River interstation area, where it was found that the short response time of the catchment necessitated that the recovery of initial loss between July 25 and 29 be accounted for.



- **Figure 6-29: July 1974 pluviograph data**

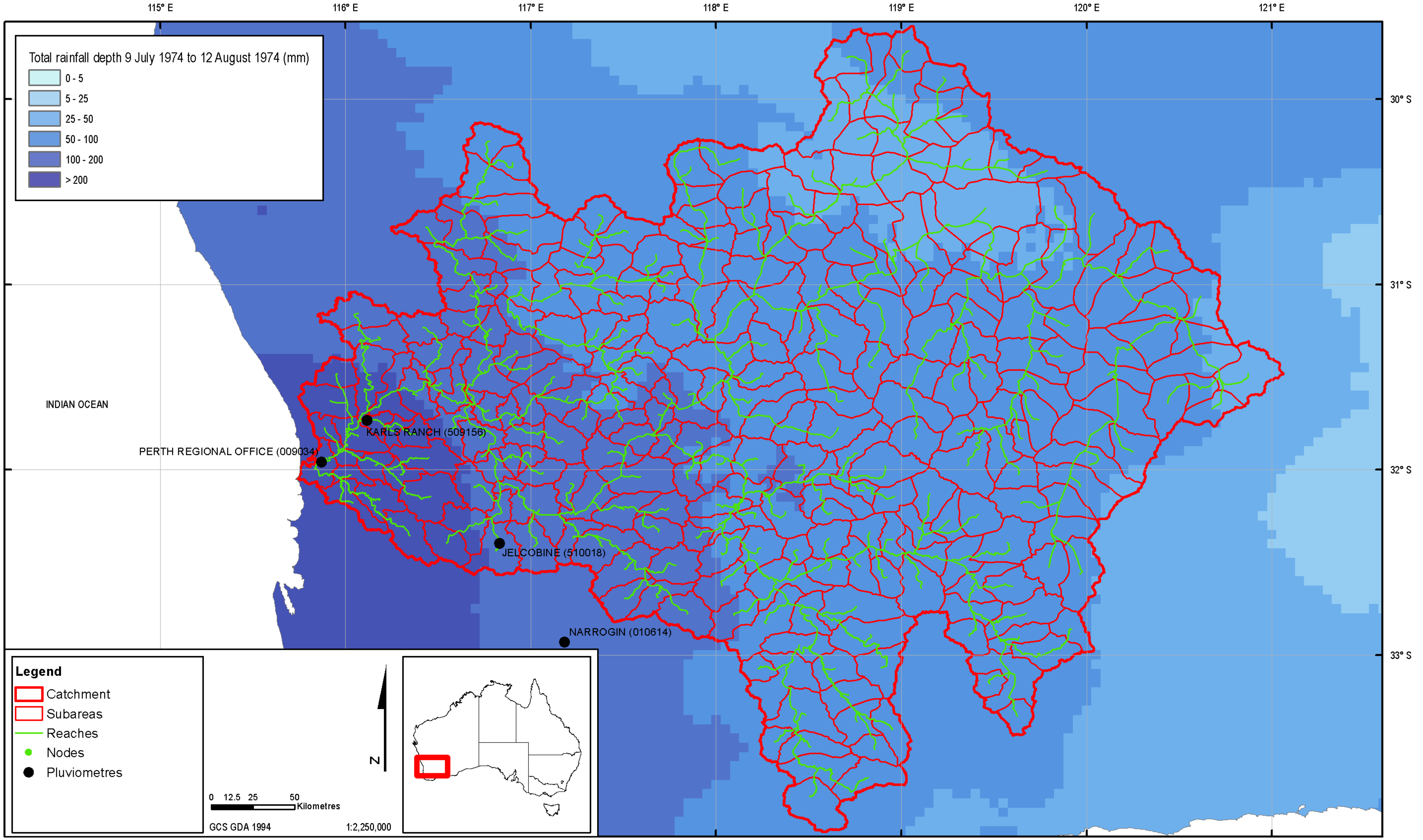


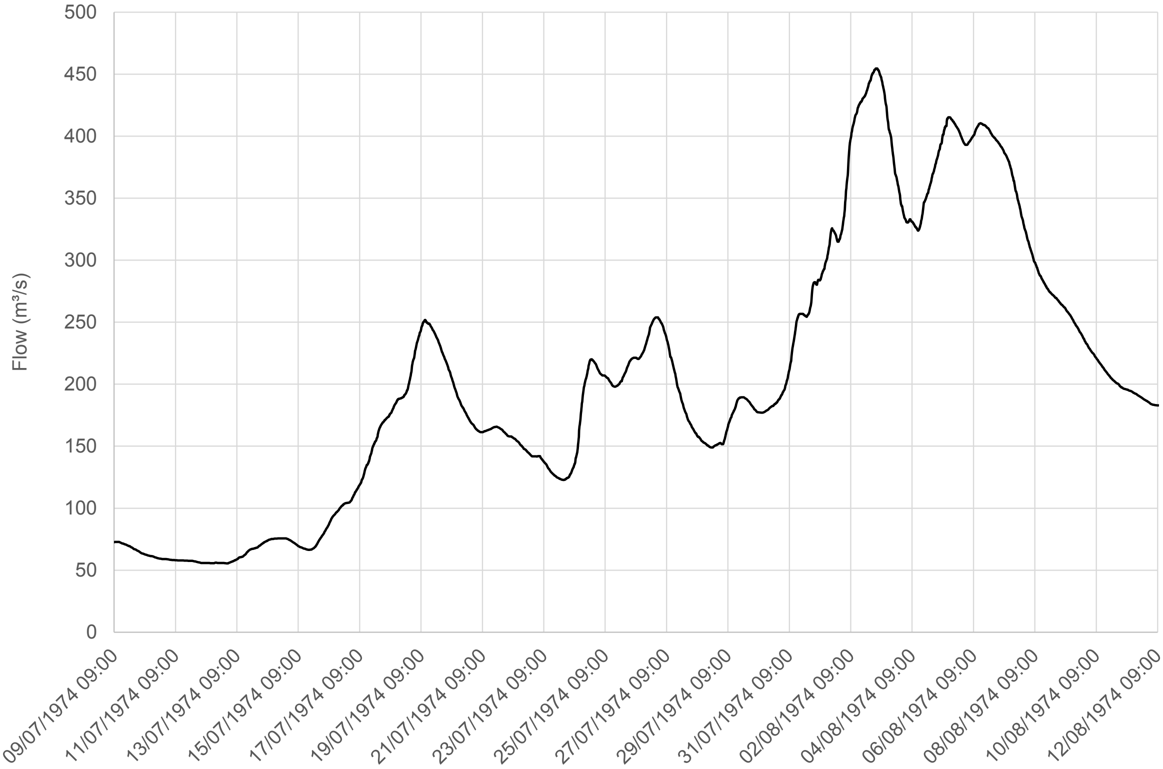
Figure 6-30: July 1974 flood event rainfall depths

Table 6-21: July 1974 event assigned pluviograph stations

Interstation area	Pluviograph station
Yilgarn River	Narrogin (010614)
Lockhart River	Narrogin (010614)
Salt River to Yenyenning Lakes	Narrogin (010614)
Avon River to Boyagarra Rd	Narrogin (010614)
Avon River to Beverley	Narrogin (010614)
Dale River to Waterhatch Bridge	Narrogin (010614)
Avon River to York	Narrogin (010614)
Avon River to Northam	Narrogin (010614)
Mortlock River East Branch	Jelcobine (510018)
Mortlock River East Branch to O'Driscolls	Jelcobine (510018)
Mortlock River North Branch to Lake Ninan	Narrogin (010614)
Mortlock River North Branch to Frenches	Jelcobine (510018)
Avon River to Toodyay	Jelcobine (510018)
Brockman River	Jelcobine (510018)
Avon River to Walyunga	Jelcobine (510018)
Helena River	Karls Ranch (509156) – upper 10 sub-areas Perth Regional Office (009034) – lower 16 sub-areas
Swan River	Perth Regional Office (009034)

6.5.2 Streamflow inputs

This event was recorded by a limited number of streamflow gauges. Along the Avon River at its tributaries, the only gauge operating at the time of the event was Walyunga. The hydrograph at Walyunga for this event is shown in Figure 6-31. It can be seen that over the period of the event, there are four main distinct flood peaks. It is also clear that there is a significant baseflow signal, with over 50 m^3/s of flow in the river at the start of the event.



- **Figure 6-31: July 1974 flood event at Walyunga**

The July 1974 event also caused significant flooding in the Helena River catchment, and there were a number of gauges upstream of Mundaring Dam which recorded the event. Data from two of these gauges was selected and was used to ensure that this section of the model was able to reproduce the recorded flood data. The data at the selected gauges (Helena River at Poison Lease and Darkin River at Pine Plantation) is shown in Figure 6-32.

Baseflow was separated from the gauged hydrographs for this event using the procedures outlined in Section 4.2.1.

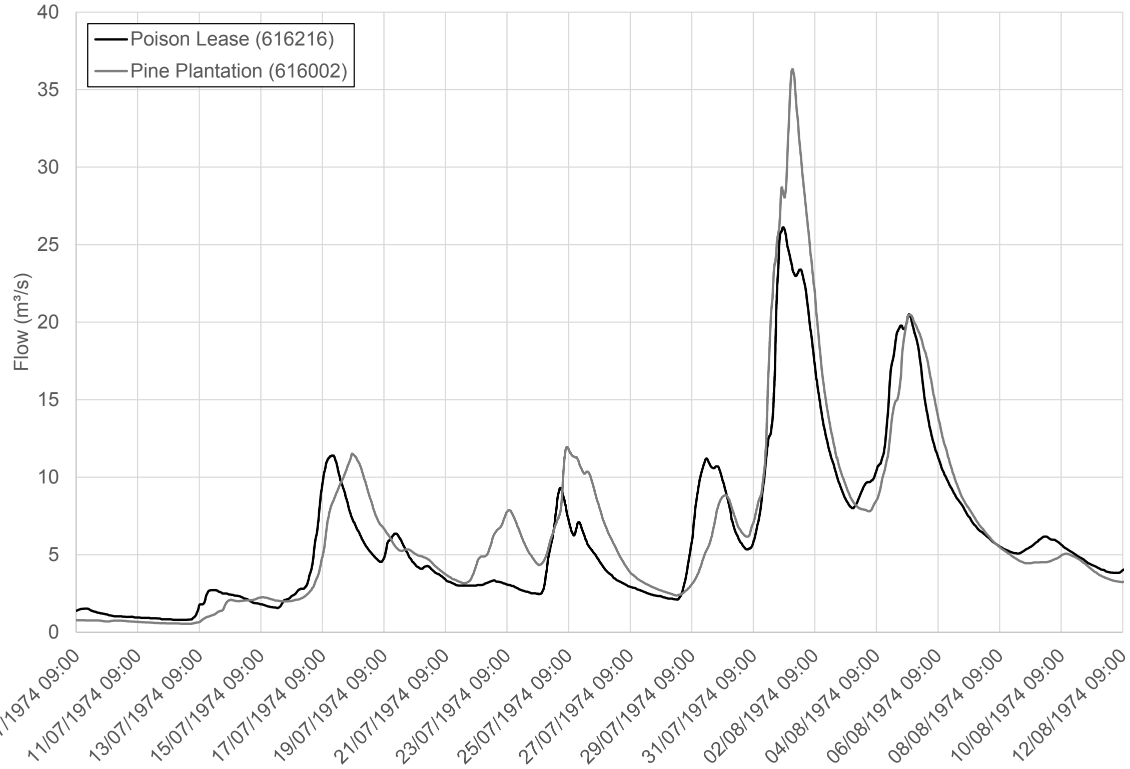
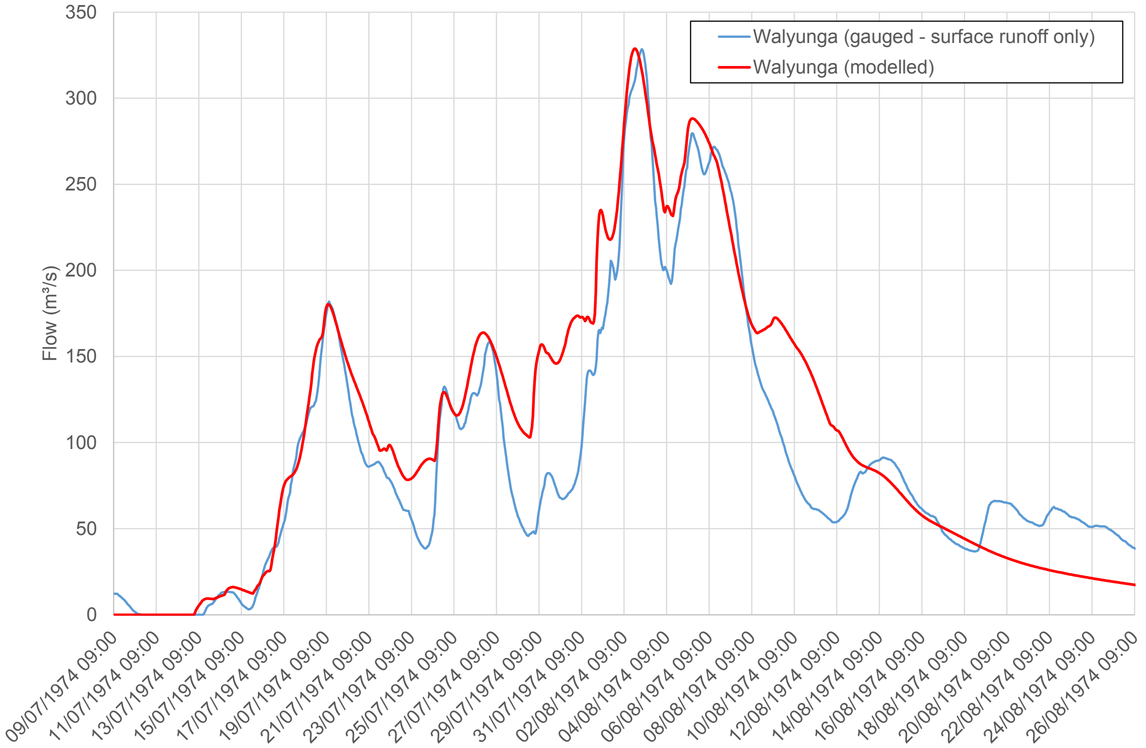


Figure 6-32: July 1974 flood event – Helena River gauges

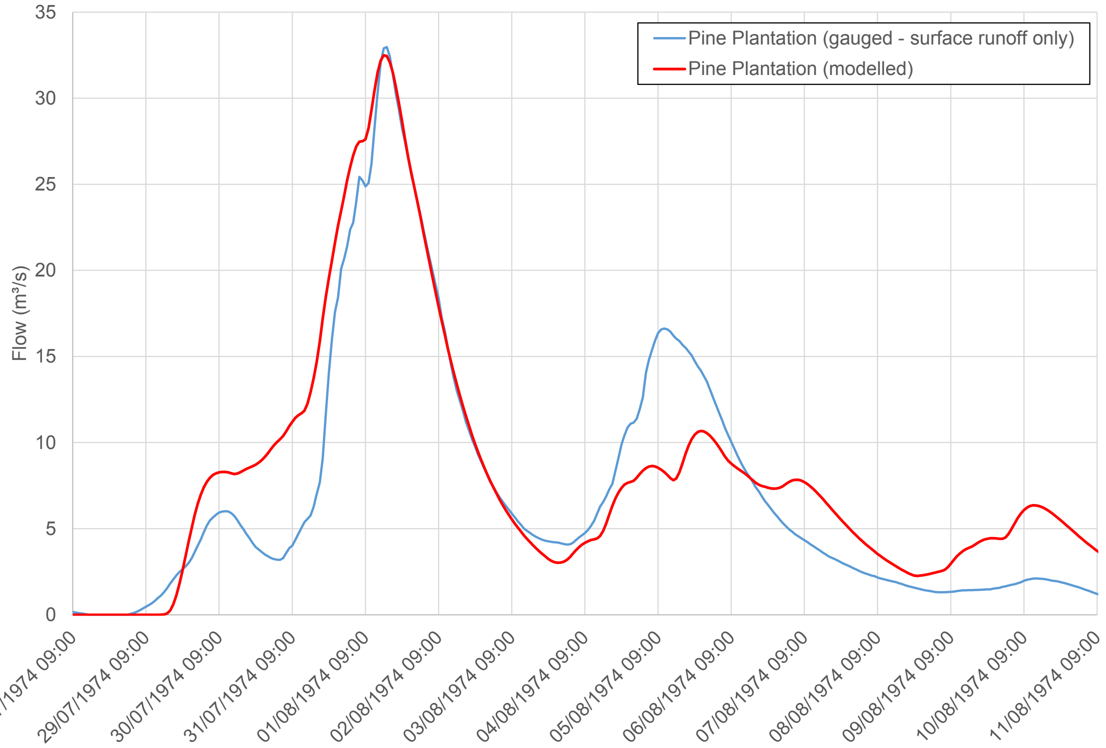
6.5.3 Calibration results

The results of the calibration for the July 1974 event are presented as plots and summary tables showing a comparison of key hydrograph characteristics at each selected gauge location. An overall table of adopted model parameter values and some discussion is provided at the end of the section.



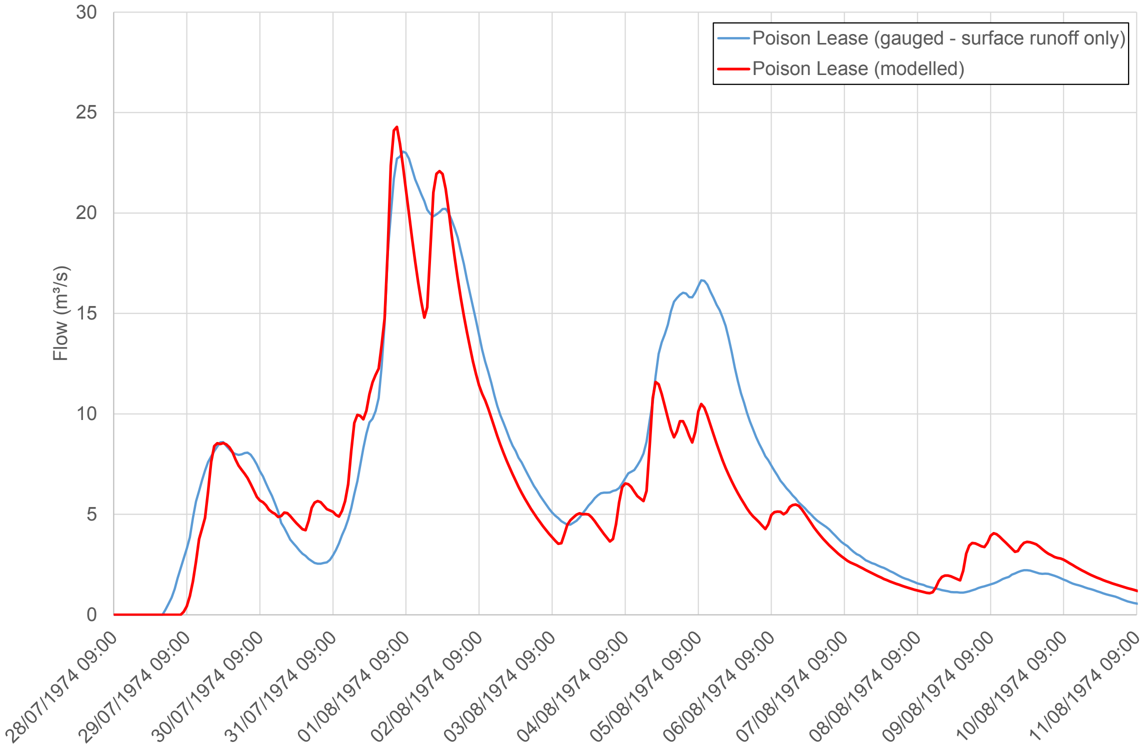
- **Figure 6-33: July 1974 model calibration at Walyunga**
- **Table 6-22: July 1974 model calibration at Walyunga**

	Gauged	Modelled	Difference
Peak 1 magnitude (m^3/s)	181.86	180.38	-1.48 m^3/s
Peak 1 timing	19/7/74 12:00	19/7/74 11:00	-1 hour
Peak 2 magnitude (m^3/s)	158.41	163.85	5.43 m^3/s
Peak 2 timing	27/7/74 1:00	26/7/74 18:00	-7 hours
Peak 3 magnitude (m^3/s)	328.51	328.78	0.26 m^3/s
Peak 3 timing	3/8/74 5:00	2/8/74 21:00	-8 hours
Peak 4 magnitude (m^3/s)	279.65	288.25	8.60 m^3/s
Peak 4 timing	5/8/74 14:00	5/8/74 14:00	0 hours
Total volume (GL)	388.75	439.83	13.1%



- **Figure 6-34: July 1974 model calibration at Pine Plantation**
- **Table 6-23: July 1974 model calibration at Pine Plantation**

	Gauged	Modelled	Difference
Peak magnitude (m^3/s)	32.97	32.50	-0.47 m^3/s
Peak timing	1/8/74 16:00	1/8/74 15:00	-1 hour
Total volume (GL)	10.61	10.33	-2.9%



- **Figure 6-35: July 1974 model calibration at Poison Lease**
- **Table 6-24: July 1974 model calibration at Poison Lease**

	Gauged	Modelled	Difference
Peak magnitude (m^3/s)	23.06	24.29	1.53 m^3/s
Peak timing	1/8/74 8:00	1/8/74 6:00	-2 hour
Total volume (GL)	10.21	7.48	-26.7%

Table 6-25: July 1974 model calibration summary

Interstation area	Average flow distance (km)	k_c	$C_{0.8}$	Initial loss (mm)	Runoff coefficient
Yilgarn River	1,102.3	500	0.45[1]	45	0.02
Lockhart River	745.4	270	0.36[1]	45	0.02
Salt River to Yenyenning Lakes	83.3	230	2.76	45	0.05
Avon River to Boyagarra Rd	74.6	190	2.55	25	0.15
Avon River to Beverley	37.1	95	2.56	25	0.15
Dale River to Waterhatch Bridge	37.7	40	1.06	25	0.15
Avon River to York	27.0	35	1.30	25	0.15
Avon River to Northam	29.7	40	1.35	25	0.15
Mortlock River East Branch	172.4	50	0.29	25	0.10
Mortlock River East Branch to O'Driscolls	41.2	35	0.85	10	0.30
Mortlock River North Branch to Lake Ninan	95.6	65	0.68	20	0.10
Mortlock River North Branch to Frenches	51.3	55	1.07	10	0.20
Avon River to Toodyay	34.5	30	0.87	5	0.10
Brockman River	67.7	85	1.26	5	0.10
Avon River to Walyunga	55.1	55	1.00	5	0.10
Helena River	53.0	100	1.78	20	0.13
Swan River	43.8	No streamflow gauge			

[1] Average flow distance values in these interstation areas are affected by reach length factoring

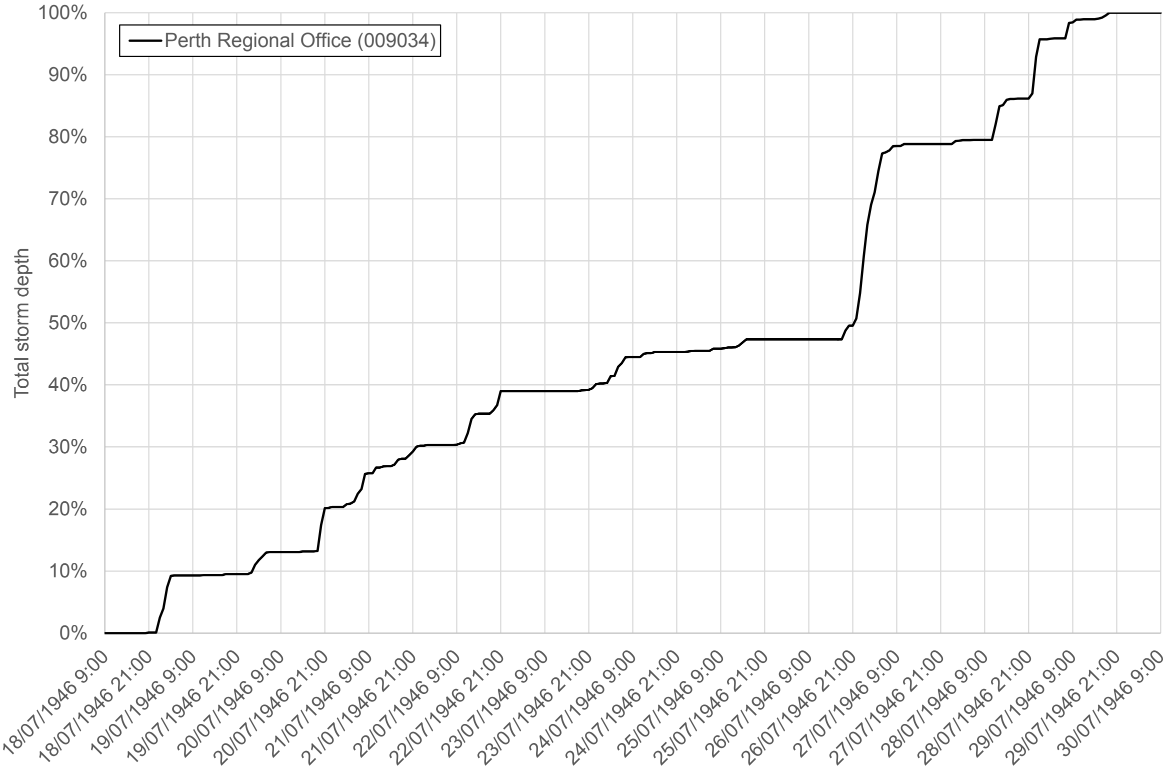
This event demonstrated a remarkably good match to an extremely long duration, multi-burst event. There is reasonable consistency in the loss parameter values, with the upper reaches of the Avon and Mortlock Rivers having higher initial losses and lower runoff coefficients. It was noted for this event that the calibrated runoff coefficient values for some of the lower Avon River interstation areas appear unusually low (0.10). There is no apparent justification for this – it may represent error or overestimation of the local gauged rainfall depths.

6.6 July 1946 event

There is minimal data to support model calibration for this event as it occurred prior to the commenced of streamflow gauging records in the Swan and Avon River system. The only data available was sourced from Binnie (1985), which notes that the event had an estimated peak flow of 935 m^3/s at Walyunga. The source of this information is not clear, but it is presumed to be based on a recorded peak flood level which was then converted to a peak flow using the gauge rating curve. No timing information is available, except that the peak occurred in June.

This event was primarily selected for calibration as it is the largest flood for which there is a reliable estimate of peak flow at Walyunga. However, given the relative lack of information on the timing and shape of the hydrograph, it was decided to treat this event as a ‘validation’ event. The model routing parameter values derived from the previous three events were used and the model loss parameter values were adjusted until the peak flow was matched at Walyunga. The aim was to ensure that the model was capable of reproducing a flood of this magnitude with reasonable loss parameter values. Note that as per the discussion in Section 4.2.2.2, no attempt was made to calculate or separate baseflow from this peak flood estimate.

Coverage of sub-daily rainfall data for this event was also limited. The only pluviograph station with available data was the Bureau of Meteorology gauge at Perth Regional Office (009034). The data from this station was analysed along with daily rainfall records at Northam and Beverley to determine the start and end dates of the event. June and July of 1946 were both months of significant rainfall in this region, and it is difficult to determine when the most intense storm occurred over this period. The storm occurring over the period July 18 to July 30 appeared to contain the most intense rainfall, and was adopted as the calibration event. The Perth Regional Office pluviograph data for this period was aggregated to a 1 hour timestep and normalised before being used in the model. The data is shown in Figure 6-36.



- **Figure 6-36: July 1946 pluviograph data**

Sub-area rainfall depths for this event were extracted from AWAP data. The total rainfall depth over the catchment is shown on the map in Figure 6-37.

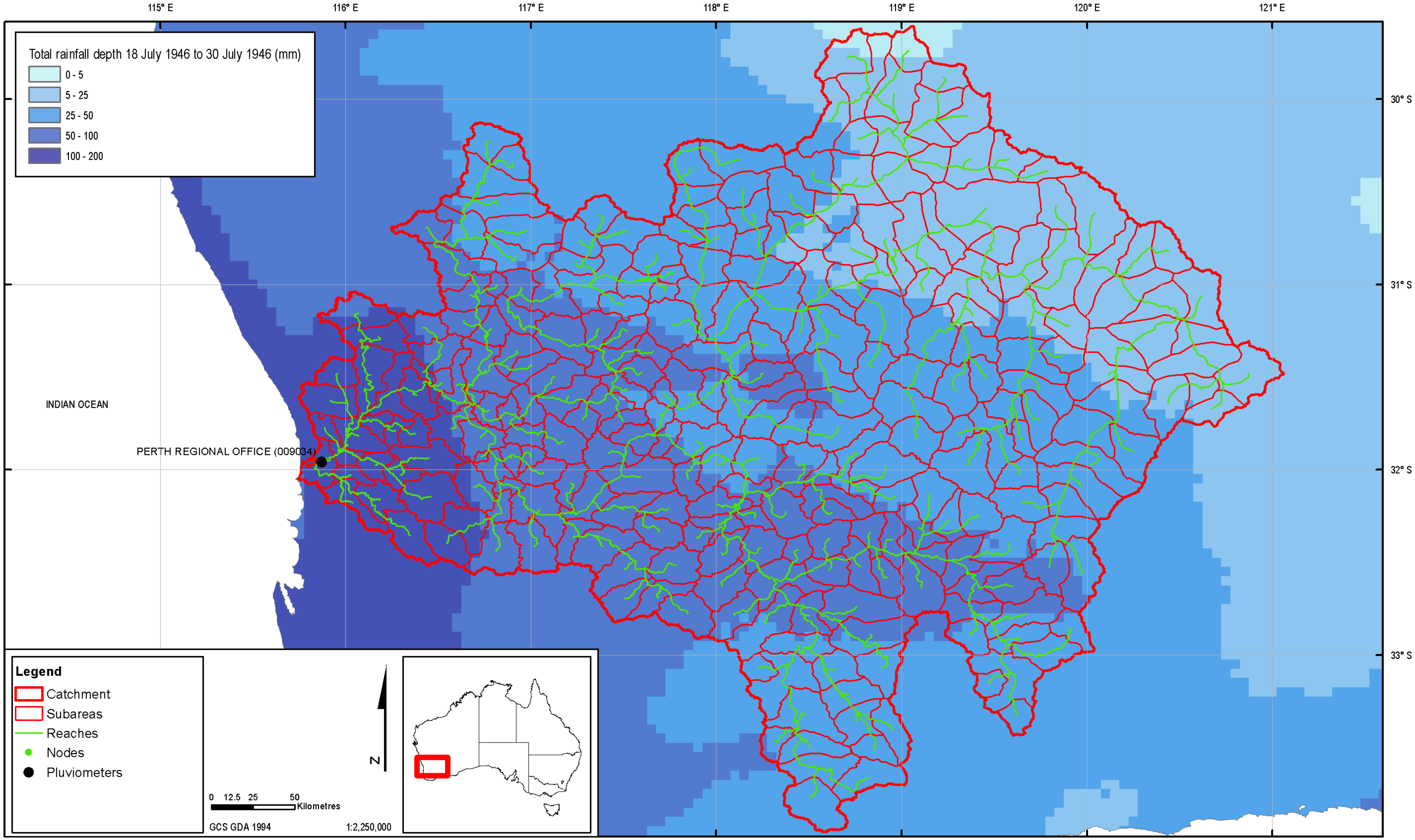


Figure 6-37: July 1946 flood event rainfall depths

The model was run with these rainfall inputs and routing parameter values consistent with those subsequently adopted for design (refer Section 6.7). The loss model parameters were then varied uniformly across the entire model domain until the modelled peak flow at Walyunga approximated the recorded estimate. It was found that an initial loss of 10 mm and a runoff coefficient of 0.35 were sufficient to generate a peak flow of 935 m^3/s. The modelled hydrograph for this event is shown in Figure 6-38.

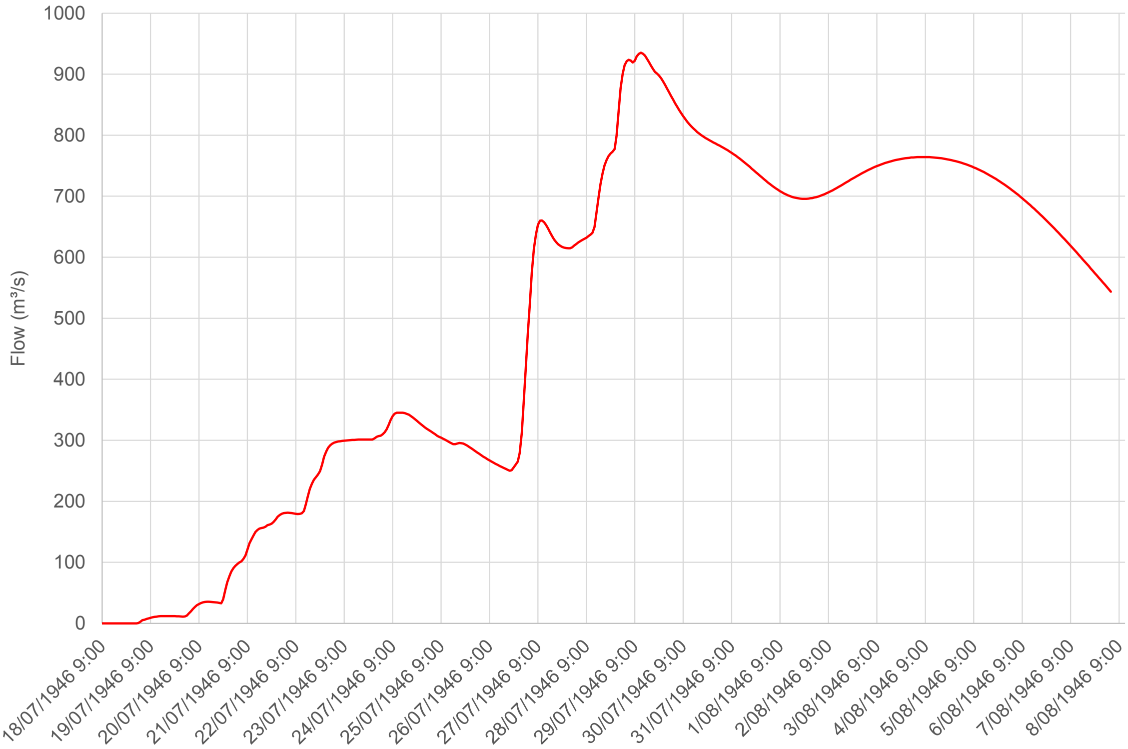


Figure 6-38: July 1946 modelled hydrograph at Walyunga

Given that the adopted loss parameter values for this event are well within the expected range, it was concluded that this event was an acceptable validation of the model performance.

6.7 Summary of calibration parameters

The calibration process has provided a wealth of valuable information on the routing parameter values which were subsequently used for design. The calibrated k_c values and adopted values for design are shown in Table 6-26, and it can be seen that there is a remarkable degree of consistency in these values from event to event. The table also shows the average flow distance (d_{av}) for each interstation area. This is provided as an 'unadjusted' value (i.e. calculated directly from GIS) and an 'adjusted' value (i.e. including the manual modifications to certain reach lengths described in Section 5.2). The $C_{0.8}$ value (the ratio of k_c to d_{av}) has also been calculated for the adjusted and unadjusted d_{av} values. Further detail on the selection and justification of the design k_c values and the expected range of design loss values is provided in the sections below.

Table 6-26: Summary of routing parameter values

Interstation area	Unadjusted d_{av} (km)	Adjusted d_{av} (km)	Calibrated k_c values 1974	1983	2000	Adopted for design	Unadjusted $C_{0.8}$	Adjusted $C_{0.8}$
Yilgarn River	350.1	1,102.3	500	N/A	500	500	1.43	0.45
Lockhart River	244.3	745.4	270	N/A	270	270	1.11	0.36
Salt River to Yenyenning Lakes	83.3	83.3	230	230	230	230	2.76	2.76
Avon River to Boyagarra Rd	74.6	74.6	190	190	190	190	2.55	2.55
Avon River to Beverley	37.1	37.1	95	95	95	95	2.56	2.56
Dale River to Waterhatch Bridge	37.7	37.7	40	45	45	45	1.19	1.19
Avon River to York	27.0	27.0	35	35	40	35	1.30	1.30
Avon River to Northam	29.7	29.7	40	40	45	40	1.35	1.35
Mortlock River East Branch	172.4	172.4	50	190	190	190	1.10	1.10
Mortlock River East Branch to O'Driscolls	41.2	41.2	35	50	57	50	1.21	1.21
Mortlock River North Branch to Lake Ninan	95.6	95.6	65	60	65	65	0.68	0.68
Mortlock River North Branch to Frenches	51.3	51.3	55	60	55	55	1.07	1.07
Avon River to Toodyay	34.5	34.5	30	35	35	35	1.01	1.01
Brockman River	67.7	67.7	85	85	N/A	85	1.26	1.26
Avon River to Walyunga	55.1	55.1	55	70	65	65	1.18	1.18
Helena River	53.0	53.0	100	N/A	N/A	100	1.89	1.89
Swan River	43.8	43.8	N/A	N/A	N/A	52	1.19	1.19

Yilgarn River

A k_c value of 500 was adopted for this interstation area for all three calibration events. Based on the unadjusted d_{av} of 350.1 km, this results in a $C_{0.8}$ value of 1.43. Pearse *et al* (2002) analysed a number of calibrated RORB models and concluded that $C_{0.8}$ typically ranges from 0.61 to 2.13, with an expected value of 1.14. The adopted k_c for the Yilgarn River is thus well within that range.

The calibration results demonstrated that the adopted initial losses for this interstation area are generally higher than for the lower reaches of the model. Similarly, the runoff coefficient values are significantly lower. This reflects the relatively arid nature of this interstation area, however these loss values are also likely to be compensating to some degree for catchment processes such as loss of flood volume within lakes and anabranched channels. Nevertheless, a reasonable fit was obtained to the large flood in January 2000, and so these model parameter values can be regarded as credible. The initial loss values ranged from 30 to 45 mm, and the runoff coefficient varied from 0.02 to 0.04.

Lockhart River

A k_c value of 270 was adopted for this interstation area for all three calibration events. Based on the unadjusted d_{av} of 244.3 km, this results in a $C_{0.8}$ value of 1.11, which is within the range of the Pearse *et al* (2002) values and close to the expected value. The calibration undertaken for the July 1983 event suggests that the Lockhart River hydrograph for this event is over-attenuated, but this is a relatively small flood compared to the January 2000 event, for which a good match was obtained.

The calibration results demonstrated that the adopted initial losses for this interstation area are generally higher than for the lower reaches of the model. Similarly, the runoff coefficient values are significantly lower. This reflects the relatively arid nature of this interstation area, however these loss values are also likely to be compensating to some degree for catchment processes such as loss of flood volume within lakes and anabranched channels. Nevertheless, a reasonable fit was obtained to the large flood in January 2000, and so these model parameter values can be regarded as credible. The initial loss values ranged from 35 to 45 mm, and the runoff coefficient varied from 0.02 to 0.05.

Salt River to Yenyenning Lakes

A k_c value of 230 was adopted for this interstation area for all three calibration events. Based on the d_{av} of 83.3 km, this results in a $C_{0.8}$ value of 2.76, which is outside the range of the Pearse *et al* (2002) estimates. The likely explanation for this is that the interstation area contains the Yenyenning Lakes. An attempt has been made to specifically include the most downstream of these lakes as a special storage, but there are numerous additional lakes located right along the main channel of this interstation area.

The calibration results indicated that the hydrologic response for this catchment varies from event to event. In some cases, the losses are relatively similar to those adopted for the Yilgarn and

Lockhart Rivers (high initial loss, low runoff coefficient). In other cases, this interstation area responds more like the interstation areas on the Avon River catchment (lower initial losses, higher runoff coefficient). This is presumably related to a number of factors such as rainfall intensity and antecedent conditions (i.e. if the lakes are full at the start of the rainfall burst, losses will tend to be much lower). It is also possible that the higher losses recorded in the July 1974 event are compensation for uncertainty in losses in other interstation areas, as there is no local gauge data in this area in 1974 to confirm the adopted values. Based on this, the initial loss is expected to vary between 10 and 45 mm, and the runoff coefficient between 0.1 and 0.2 for design.

Avon River to Boyagarra Rd

The adopted $C_{0.8}$ value for this interstation area is 2.55, which is above the higher range of the Pearse *et al* (2002) values. There is a degree of uncertainty in this value, as none of the calibration events had recorded data at the Boyagarra Rd gauge.

The loss parameter values for this interstation area reflect the more responsive lower catchment. Initial losses ranged from 10 to 30 mm and the runoff coefficient ranged from 0.15 to 0.35.

Avon River to Beverley

The adopted $C_{0.8}$ value for this interstation area is 2.56, which is higher than the range of the Pearse *et al* (2002) values. It does not vary from event to event and there is some confidence in the adopted value.

The loss parameter values for this interstation area reflect the more responsive lower catchment. Initial losses ranged from 5 to 25 mm and the runoff coefficient ranged from 0.15 to 0.35.

Dale River to Waterhatch Bridge

The adopted $C_{0.8}$ value for this interstation area is 1.19, which is well within the range of the Pearse *et al* (2002) values and close to the expected value.

The loss parameter values for this interstation area reflect the more responsive lower catchment. Initial losses ranged from 5 to 25 mm and the runoff coefficient ranged from 0.1 to 0.35.

Avon River to York

The adopted $C_{0.8}$ value for this interstation area is 1.30, which is well within the range of the Pearse *et al* (2002) values and close to the expected value.

The loss parameter values for this interstation area reflect the more responsive lower catchment. Initial losses ranged from 5 to 25 mm and the runoff coefficient ranged from 0.15 to 0.35.

Avon River to Northam

The adopted $C_{0.8}$ value for this interstation area is 1.35, which is well within the range of the Pearse *et al* (2002) values and close to the expected value.

The loss parameter values for this interstation area reflect the more responsive lower catchment. Initial losses ranged from 10 to 25 mm and the runoff coefficient ranged from 0.15 to 0.35.

Mortlock River East Branch

This interstation area is within the upper catchment and is dominated by the presence of a large lake system at Cowcowing and numerous smaller lakes and anabranched channels. The adopted k_c value of 190 was found to match well for the 2000 and 1983 events, but had to be significantly reduced for the 1974 event. The adopted k_c results in a $C_{0.8}$ of 1.10, which is well within the range of the Pearse *et al* (2002) values and close to the expected value.

The loss parameter values for this catchment reflect the more arid areas within the upper catchment, and potentially are also accounting for a degree of storage and loss in the lakes and anabranched channels. The initial loss values varied from 15 to 25 and the runoff coefficient values varied from 0.05 to 0.10.

Mortlock River East Branch to O'Driscolls

The calibrated k_c values for this interstation area varied from event to event, which presumably reflects the large variability in flow from the upstream interstation area based on rainfall intensity, losses and the effects of lakes and anabranched channels. The adopted k_c value was selected from within the range of the calibrated values, and resulted in an unadjusted $C_{0.8}$ of 1.21, which is well within the range of the Pearse *et al* (2002) values.

The loss parameter values for this interstation area reflect the more responsive lower catchment. Initial losses ranged from 5 to 10 mm and the runoff coefficient ranged from 0.2 to 0.35.

Mortlock River North Branch to Lake Ninan

This interstation area is within the upper catchment, and contains both Lake Hinds and Lake Ninan. The adopted k_c value is 65, which results in a $C_{0.8}$ of 0.68. This low value reflects the fact that the k_c is only impacting the small catchment between Lakes Hinds and Ninan, as Lake Hinds has a large storage capacity. The d_{av} is effectively over-estimated, therefore resulting in a low $C_{0.8}$.

The loss parameter values for this catchment reflect the more arid areas within the upper catchment, and potentially are also accounting for a degree of storage and loss in the lakes. The initial loss values varied from 20 to 25 and the runoff coefficient values varied were generally around 0.1.

Mortlock River North Branch to Frenches

The calibrated k_c values for this interstation area were relatively consistent from event to event, resulting in an adopted, unadjusted $C_{0.8}$ value of 1.07, which is well within the Pearse et al (2002) range and close to the expected value.

The loss parameter values for this interstation area reflect the more responsive lower catchment. Initial losses ranged from 10 to 25 mm and the runoff coefficient ranged from 0.05 to 0.35.

Avon River to Toodyay

The adopted $C_{0.8}$ value for this interstation area is 1.01, which is close to the expected value in Pearse *et al* (2002).

The loss parameter values for this interstation area reflect the more hydrologically responsive reaches of the lower Avon River. Initial losses ranged from 5 to 40 mm and the runoff coefficient ranged from 0.1 to 0.6.

Brockman River

This tributary of the lower Avon River exhibits unusual behaviour evidenced in the gauged record for the July 1983 event. Whilst the rising limb and peak of the hydrograph has been matched, there is a long receding limb that the model does not simulate, resulting in an overall under-prediction of the hydrograph volume. It is likely that this effect is caused by the presence of Lakes Chittering and Needonga, which capture and attenuate a significant proportion of the catchment but have not be specifically modelled. Notwithstanding this, the relative contribution of this interstation area to flows at Walyunga is typically small, so there is little value to be gained in exploring this issue further. The adopted k_c value results in a $C_{0.8}$ of 1.26, well within the Pearse et al (2002) range. Initial losses varied from 5 to 40 mm and the runoff coefficient varied from 0.1 to 0.35.

Avon River to Walyunga

The adopted $C_{0.8}$ value for this interstation area is 1.18, which is well within the range of the Pearse *et al* (2002) values and close to the expected value.

The loss parameter values for this interstation area reflect the more hydrologically responsive reaches of the lower Avon River. Initial losses ranged from 5 to 50 mm and the runoff coefficient ranged from 0.1 to 0.5.

Helena River

The Helena River interstation area was calibrated to a single event, and it was found that a k_c value of 100 gave a reasonable fit, resulting in a $C_{0.8}$ of 1.89, which is towards the upper end of the Pearse *et al* (2002) values. It was noted that this value was above the range of k_c values typically adopted for south-west WA catchments (on the basis of catchment area) reported in DoE (2004b).

The initial loss adopted for the 1974 event was 20 mm and the runoff coefficient was 0.13. This is within the range of the volumetric runoff coefficients reported in DoE (2004a)

Swan River

The design k_c value for the Swan River interstation area was set using the adopted $C_{0.8}$ value for the Avon River to Walyunga interstation area. There is limited additional gauging information that could be used to improve this estimate, and the sensitivity to the overall model results is likely to be low. As for the Avon River to Walyunga, initial loss values are likely to vary in the range of 5 to 50 mm and the runoff coefficient values vary between 0.1 and 0.5.

It should be noted that the value of the routing exponent parameter (*m*) was set to 0.8 for all calibration and design runs in all interstation areas.

7. Method for design flood modelling

7.1 Event based and Monte-Carlo approaches

The estimation of design floods has traditionally been based on the “design event” approach, in which all parameters other than rainfall are input as fixed, single values. This concept is illustrated in Figure 7-1 for the case where a distribution of design rainfalls is combined with fixed values of losses, rainfall temporal patterns and spatial patterns. Considerable effort is made to ensure that the single values of the adopted parameters are “AEP-neutral”, that is, they are selected with the objective of ensuring that the resulting flood has the same annual exceedance probability as its causative rainfall.

This approach suffers from the limitations that:

- the AEP-neutrality of some inputs can only be tested on frequent events for which independent estimates are available
- for more extreme events, the adopted values of AEP-neutral inputs must be conditioned by physical and theoretical reasoning
- the treatment of more complex interactions (such as the seasonal variation of inputs) becomes rapidly more complex and less easy to defend

Joint probability techniques offer an improvement to the traditional design event method. These techniques recognise that any design flood characteristics (e.g. peak flow) could result from a variety of combinations of flood producing factors, rather than from a single combination. For example, the same peak flood could result from a moderate storm on a saturated catchment, or a large storm on a dry catchment. In probabilistic terms, a 1 in 100 AEP flood could be the result of a 1 in 50 AEP rainfall on a very wet catchment, or a 1 in 200 AEP rainfall on a dry catchment. Joint probability approaches attempt to mimic “mother nature” in that the influence of all probability distributed inputs are explicitly considered, thereby providing a more realistic representation of the flood generation processes.

The method is easily adapted to focus on only those aspects that are most relevant to the problem. For example as illustrated in Figure 7-2 it is possible to adopt single “AEP-neutral” values for some inputs (in this case the manner in which rainfalls are spatially distributed over the catchment), and full distributions for other more important inputs, such as losses and temporal patterns.

The application of joint probability approaches to flood estimation is widely acknowledged to be a more thorough and defensible approach to design flood estimation than the design event approach in Australian practice, and has been incorporated in the current version of Australian Rainfall and Runoff (ARR).

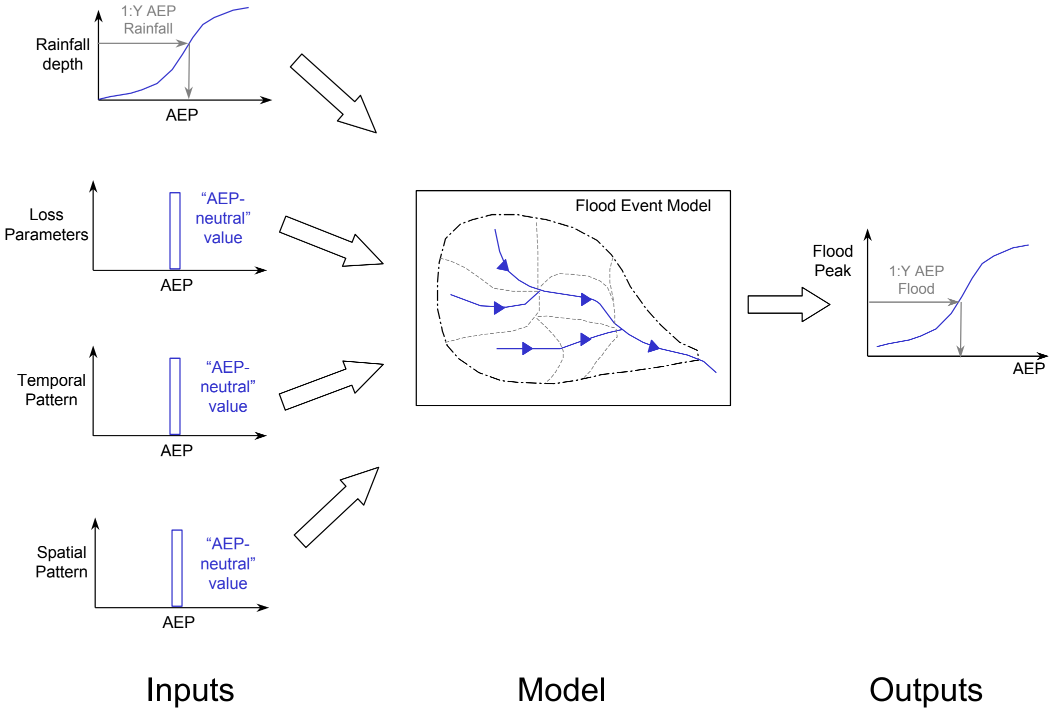


Figure 7-1: Schematic illustration of the design event approach

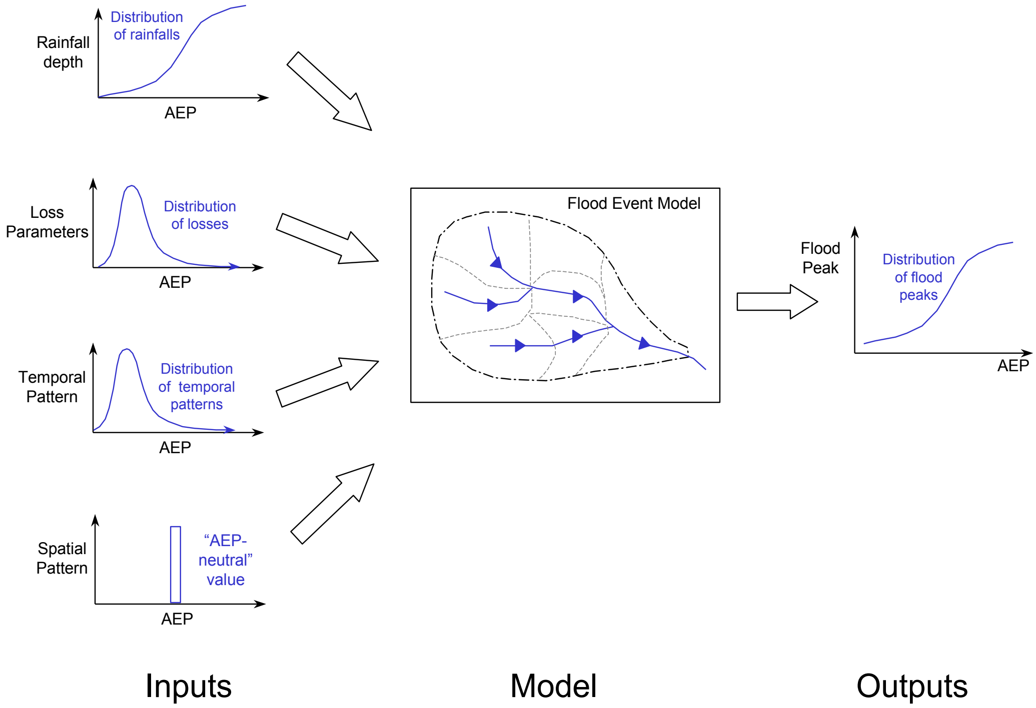


Figure 7-2: Schematic illustration of the joint probability approach

7.2 Overview of adopted joint probability framework

The joint probability framework adopted was developed by Nathan *et al* (2002, 2003) and is summarised in Figure 7-3. In essence the approach involves undertaking numerous model simulations, where the model inputs are sampled from non-parametric distributions that are based either on readily available design information or on the results of recent research.

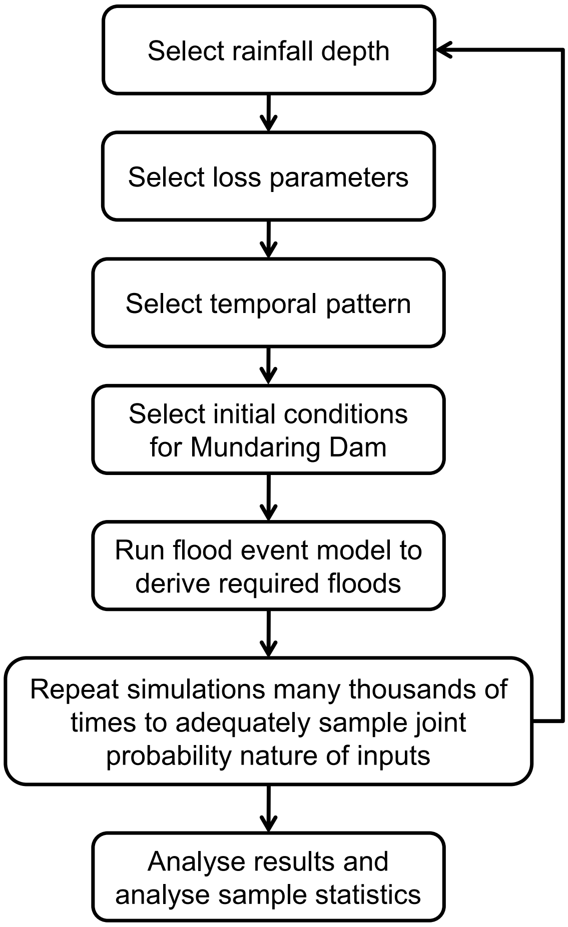


Figure 7-3: Overview of adopted joint probability framework

In developing the joint probability framework particular attention was given to ensuring that the model inputs and the manner in which they were incorporated was consistent with current datasets and guidance available as part of the revision of Australian Rainfall and Runoff. At the time of writing, however, these procedures were in a state of flux as the revision project has not yet been completed. Thus, whilst the adopted model inputs are consistent with the current state of practice, this is expected to change in the immediate future. The following briefly describes the main inputs, and how they relate to established design information.

Select rainfall depth. Rainfall depths are stochastically sampled from a cumulative distribution of rainfall depths. The relationship between rainfall depth and AEP for a given burst duration is based directly on the ARR guidelines, though additional information is obtained from ARR procedures to derive rainfalls for AEPs as frequent as 1 in 2.

Select storm losses: Storm initial losses for a given season are stochastically sampled from a nonparametric distribution that was determined from the analysis of a large number catchments across Australia (Hill et al, 2014). There is little information regarding the correlation between initial and proportional loss rates, and since antecedent conditions have most influence on initial loss rates, in this study the proportional loss rates were held constant within each season.

Select temporal pattern. Temporal patterns are randomly selected from a sample of temporal patterns relevant to the catchment area and duration of the storm. The temporal patterns are derived from large historic storms that have been observed in the region.

Select initial conditions in Mundaring Dam. For the Helena River model runs, the initial water level (and thus airspace volume below full supply level) in Mundaring Dam was sampled from a distribution derived from historic storage volumes. This is documented further in Section 8.5.1.

Monte-Carlo simulation. Simulations are undertaken using a stratified sampling approach in which the sampling procedure focuses selectively on the probabilistic range of interest. Thus, rather than undertake many millions of simulations in order to estimate an event with, say, a 1% probability of exceedance, a reduced number of simulations are undertaken over a specified number of probability intervals. In this study, the rainfall frequency curve was divided into 100 intervals uniformly spaced over the standardised normal probability domain, and 200 simulations were taken within each division. Thus, a total of 20,000 simulations were undertaken to derive the frequency curve corresponding to each storm duration considered.

7.3 Seasonality

All design model runs undertaken as part of this project were done on an annual basis. That is to say, the variability in flood producing factors such as losses and reservoir/lake water levels from summer to winter was not considered explicitly. It was considered that the additional complexity resulting from the introduction of seasonally based inputs was not justified given the very strong dominance of winter events observed in the gauged records at all gauges considered for flood frequency analysis (refer Table 4-1, Table 4-3, Table 4-4, Table 4-5, Table 4-7). These records clearly indicate that despite the likelihood of more intense rainfall occurring in summer (as per the largest rainfall events shown in Figure 4-2), the conditions of the catchment are such that in over 100 years of gauge and pre-gauge records at Walyunga, there is minimal evidence of large summer floods. The implications of this are that losses (particularly initial loss) in summer is sufficiently high that it results in even relatively large rainfall events generating only relatively small flow events.

8. Design inputs

This section documents the design inputs which were derived for the Swan Avon River RORB model. Three separate sets of inputs were required; for the upper and lower catchment and the Helena River interstation area.

8.1 Lower catchment

Design rainfalls for the lower catchment area upstream of the gauge at Walyunga (total area 16,469 km^2) were calculated using the following procedures.

8.1.1 Burst depths

This section describes how design rainfalls depths were derived for the lower catchment for burst durations of 12 hours and longer. Shorter durations were not considered likely to produce the critical duration for this catchment and so were not estimated.

1 in 2 to 1 in 100 AEP

Point design rainfall depths for burst durations between 12 and 168 hours, and AEPs from 50% to 1%, were estimated for the centroid of each RORB model sub-area using the IFD 2013 analysis available from the Bureau of Meteorology website (http://www.bom.gov.au/water/designRainfalls/revised-ifd/). These were then converted to a catchment average point value.

1 in 100 to 1 in 2000 AEP

The CRC-FORGE method (Durrant and Bowman, 2004) provides growth factors that extrapolate the 2% AEP point rainfall estimate for a particular duration, to the 1%, 0.5%, 0.2%, 0.1% and 0.05% AEPs. These growth factors were extracted from gridded data sets available for standard storm durations (24, 48, 72, 96 and 120 hours). Growth factors for 36 hour rainfalls were interpolated from the 24 and 48 hour results, and growth factors for 120 hours were also applied to the 144 and 168 hour rainfalls. These growth factors were then re-scaled, such that the 1% AEP point rainfall depth matched the updated IFD analysis.

Areal reduction factors

Areal reduction factors (ARFs) are required to convert point rainfall estimates into catchment wide values. Recent work has been undertaken by Podger *et al* (2015) to derive updated estimates of areal reduction factors for short and long duration storms. This work offers the significant advantage over previous available information that it covers catchment areas up to 30,000 km^2 for long duration storms between 12 and 168 hours. Additionally, the factors are provided on a regional basis, with the south-west WA region covering the Swan Avon River catchment. The equations from Podger *et al* (2015) were therefore used to calculate areal reduction factors for all AEPs between 50% and 1%.

Areal reduction factors for AEPs rarer than 1% were extrapolated out using the Podger *et al* (2015) relationships. It is understood that this procedure is consistent with the advice under development for the revision of Australian Rainfall and Runoff. The adopted areal reduction factors are shown in Figure 8-1.

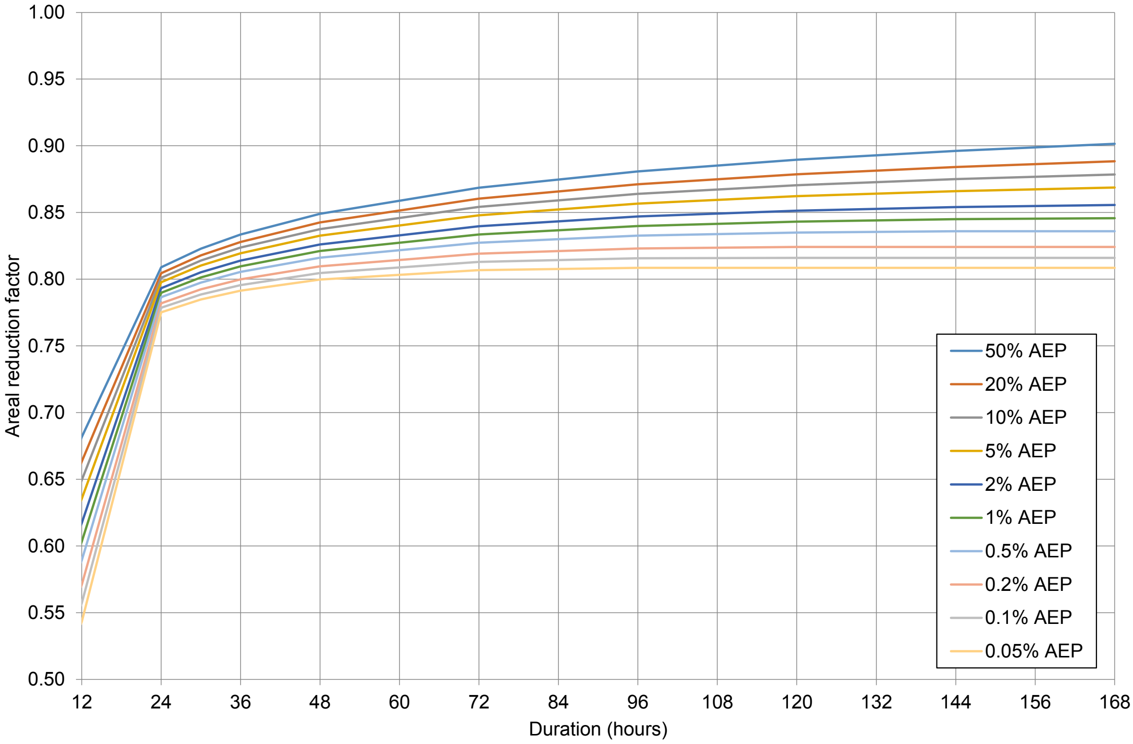


Figure 8-1: Lower catchment areal reduction factors

PMP estimates

PMP rainfall depths for burst durations between 24 and 120 hours were obtained using the GTSMR method for estimating PMP depths (BoM, 2003). For the 140 and 168 hour durations, the PMP depths were estimated by extrapolating the relationship between depth and duration between 96 and 120 hours. It was noted that this extrapolation produces a relatively flat response – this was consistent with the plotted depth-area curves for these durations which appear in Figure A3.1 for the GTSMR document (BoM, 2003). The adopted PMP depths are shown in Table 8-1.

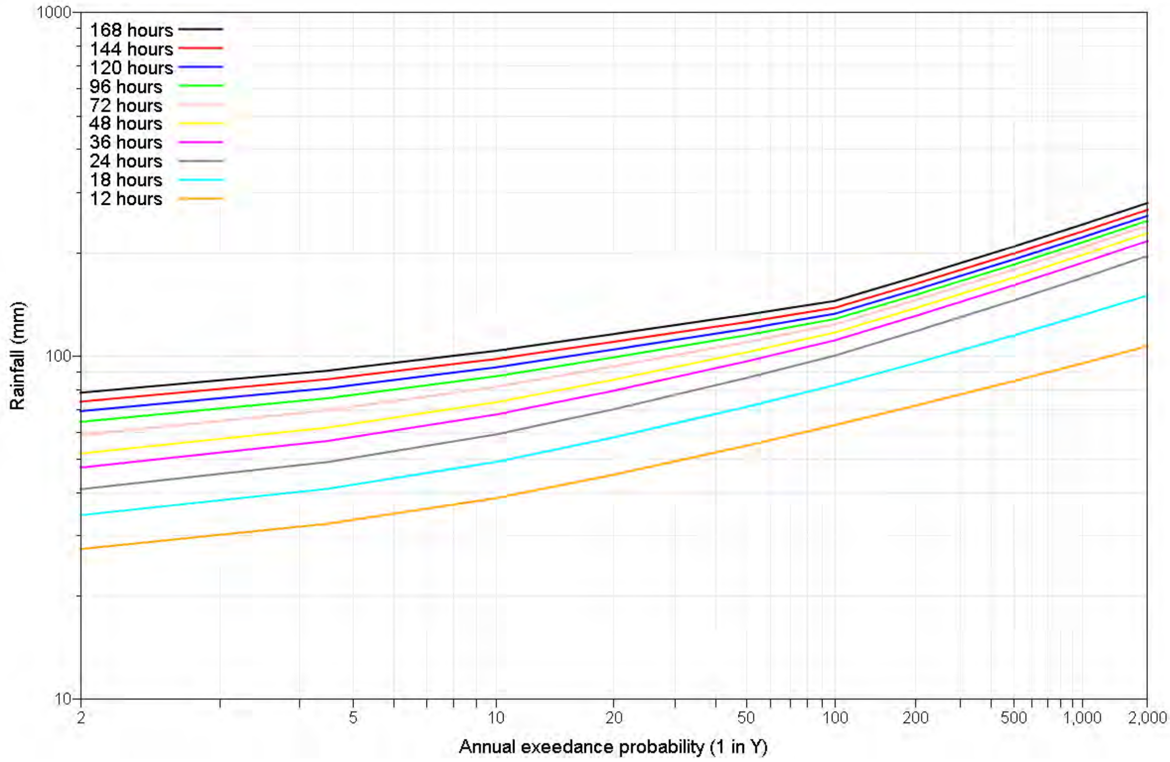
As the PMP estimates will only be used to estimate the PMF, no AEP was assigned for the PMP.

Table 8-1: PMP depths for the lower catchment

	PMP depth (mm)							
Duration	24 h	36 h	48 h	72 h	96 h	120 h	140 h	168 h
	370	440	510	630	710	740	750	755

Summary

The adopted design rainfall depths are summarised in Figure 8-2 and Table 8-2.



- **Figure 8-2: Design rainfall depths for the lower catchment**
- **Table 8-2: Design rainfall depths for the lower catchment**

AEP (1 in Y)	Design rainfall depth (mm)									
	12 h	18 h	24 h	36 h	48 h	72 h	96 h	120 h	144 h	168 h
10	38.6	49.2	59.1	67.6	73.4	81.3	87.4	92.8	98.1	103.6
20	45.1	58.0	70.0	79.3	85.3	93.3	99.2	104.5	110.1	116.0
50	54.8	71.1	86.3	96.5	102.5	109.8	115.0	120.0	125.6	131.9
100	62.9	82.3	100.3	111.1	117.1	123.7	128.2	132.8	138.2	144.8
200	71.7	95.4	118.2	131.0	137.9	145.6	150.7	156.0	162.4	170.0
500	84.6	114.9	145.3	160.8	169.5	178.6	184.6	191.3	199.0	208.4
1000	95.2	131.6	168.9	187.0	197.0	207.3	214.3	221.8	230.8	241.7
2000	106.9	150.1	195.6	216.4	227.9	239.7	247.6	256.1	266.5	279.1

8.1.2 Pre-burst

There is limited information available on pre-burst depths or temporal patterns in the GTSMR region. It is understood that a dataset is currently being prepared as part of the revision of

Australian Rainfall and Runoff, but as this data was not available and in lieu of any other information, pre-burst was not modelled.

8.1.3 Spatial patterns

The spatial pattern adopted for all AEPs was based on the relative proportion of the catchment average 1% AEP rainfall depth to the 1% AEP rainfall depth in each model sub-area. This calculation was undertaken for each duration modelled using the 2013 IFD data, and is consistent with the approach which will be recommended in the revision of Australian Rainfall and Runoff. The spatial pattern is therefore similar in nature to the spatial variability of IFD shown in Figure 4-1.

8.1.4 Temporal patterns

A sample of temporal patterns from the catalogue of storms used to develop the GTSMR method for estimating PMP depths was used in the Monte-Carlo modelling of design floods for all storm durations and AEPs. Similarly to estimates of pre-burst, it is understood that a new sample of temporal patterns is being prepared as part of the ARR revision, but this data was not available at the time of writing. In lieu of this information, the GTSMR temporal patterns were considered to provide a reasonable sample of variability.

8.2 Complete catchment

Estimation of design rainfalls for the complete catchment was undertaken in a different manner as compared to the lower catchment. The size of the complete catchment is such that it is difficult to estimate reliable areal reduction factors, and even the new advice on ARFs provided by Podger *et al* (2015) only covers catchment areas up to 30,000 km^2. It was therefore decided to use the Bureau of Meteorology's Australian Water Availability Product (AWAP) daily rainfall grids to estimate design rainfalls.

8.2.1 AWAP burst depths

The AWAP rainfall dataset is a gridded estimate of daily (9 am to 9 am) rainfall at a 0.05° × 0.05° resolution for all of Australia for each day after 1/1/1900. The AWAP data was used to directly estimate a daily time series of areal rainfall over the Swan Avon River complete catchment. From this daily time series annual maxima were extracted. The annual maxima were defined on a 1st October to 29th September basis as to reduce the likelihood of a storm occurring over the boundary between years.

Generalised extreme value (GEV) distributions were fitted to the maxima (using LH-moments) in order to define the areal rainfall estimates. Note that in spite of the curves being fitted independently for each duration it is known that the tails of the fitted GEV distributions should not cross (as seen at an AEP of 1% in Figure 8-3). This additional constraint can be added to the fitting procedure to get more appropriate estimates of GEV parameters.

The fitting procedure was adapted by smoothing sample L1 and L2 values across durations for 3 – 7 day bursts, as well as adopting a fixed L-Skew for these durations (Figure 8-4). The LH-moment fitting for 1 – 2 day bursts exhibited different behaviour than the other durations, therefore the unsmoothed GEV curves for 1 – 2 days were adopted.

The AWAP data is collected on a 9am to 9am basis. Given this, the maxima inferred from this data will be lower than truth because any events that occur around 9am will be split between time-steps. This difference is larger when the duration of interest is closest to the data resolution. Factors to convert datasets from restricted to unrestricted estimates of rainfall were developed as part of the ARR IFD project (Green et. al., 2012) and were presented in Stensmyr and Babister (2015). These factors (shown in Table 8-3) were applied to the derived AWAP GEV distributions.

Table 8-3: Restricted to unrestricted sampling conversion factors

Restricted x Factor = Unrestricted							
Duration (hours)	24	48	72	96	120	144	168
Factor	1.15	1.11	1.07	1.05	1.04	1.03	1.02

The adopted AWAP areal rainfall estimates are shown in Figure 8-5.

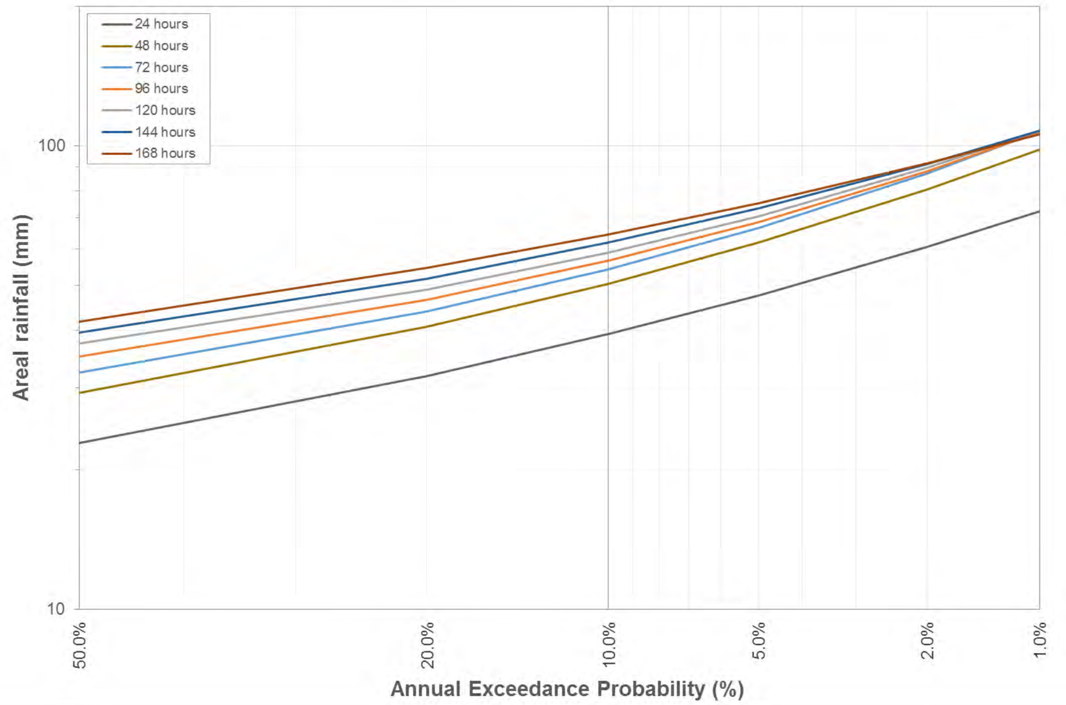
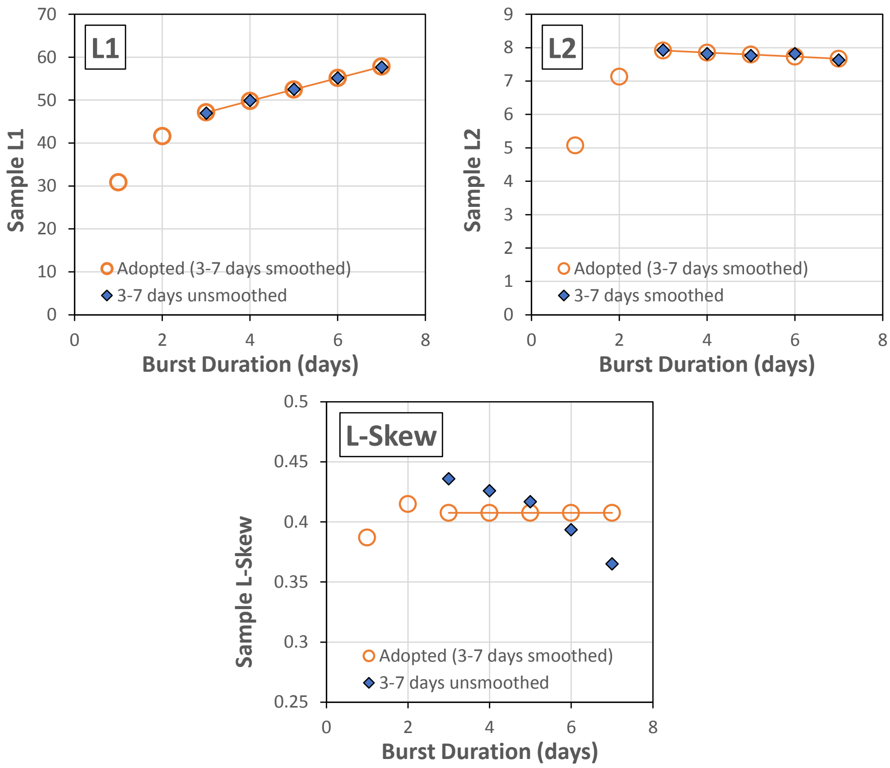
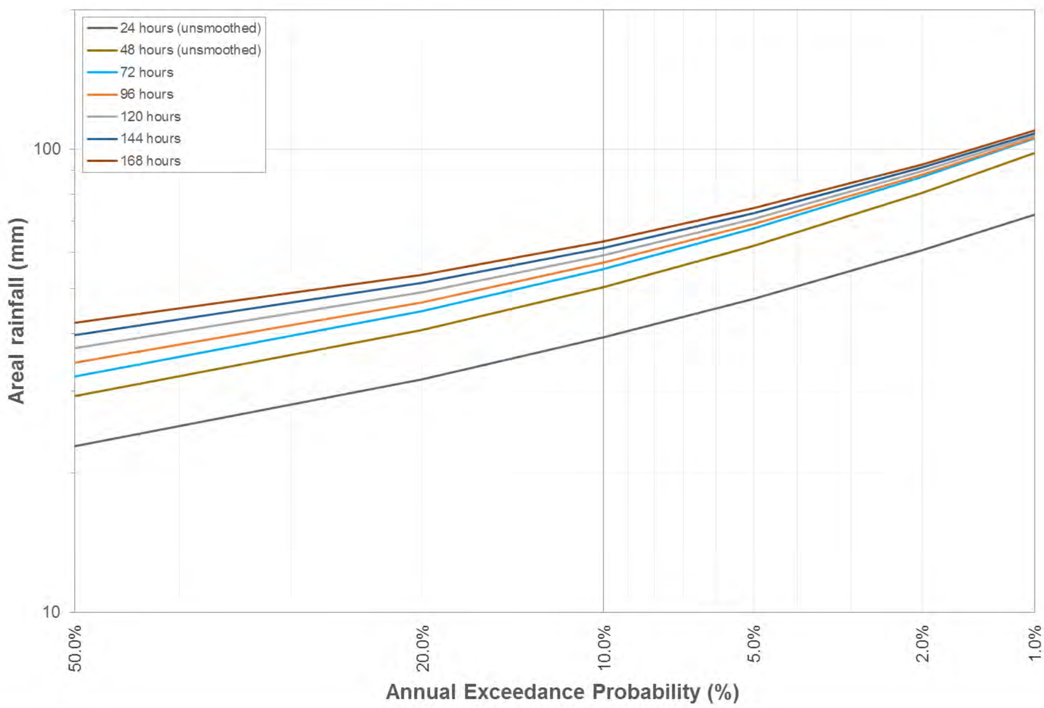


Figure 8-3: Unsmoothed AWAP areal rainfall estimates



■ **Figure 8-4: LH-Moments procedure with smoothing across duration**



■ **Figure 8-5: Smoothed AWAP areal rainfall estimates**

8.2.2 BoM burst depths

In order to benchmark the IFD estimates obtained from AWAP data, burst depths were also extracted from the Bureau of Meteorology IFD 2013 dataset. Point design rainfall depths for burst durations between 12 and 168 hours, and AEPs from 50% to 1%, were estimated for the centroid of each RORB model sub-area using the IFD 2013 analysis available from the Bureau of Meteorology website (http://www.bom.gov.au/water/designRainfalls/revised-ifd/). These were then converted to a catchment average point values.

Areal reduction factors for the complete catchment were then calculated by extrapolating the new ARF equations from Podger *et al* (2015). Whilst these equations were only derived for areas of up to 30,000 km^2, they were regarded as the most current source of information on ARFs for very large catchments. For completeness, ARFs were also calculated using the equation coefficients provided in the WA CRC-FORGE report (Durrant and Bowman, 2004). These factors are typically assumed to be invariant with AEP, but only apply up to catchment areas of 10,000 km^2, so the equation results had to be extrapolated. Calculated ARFs for the complete catchment are shown in Figure 8-6.

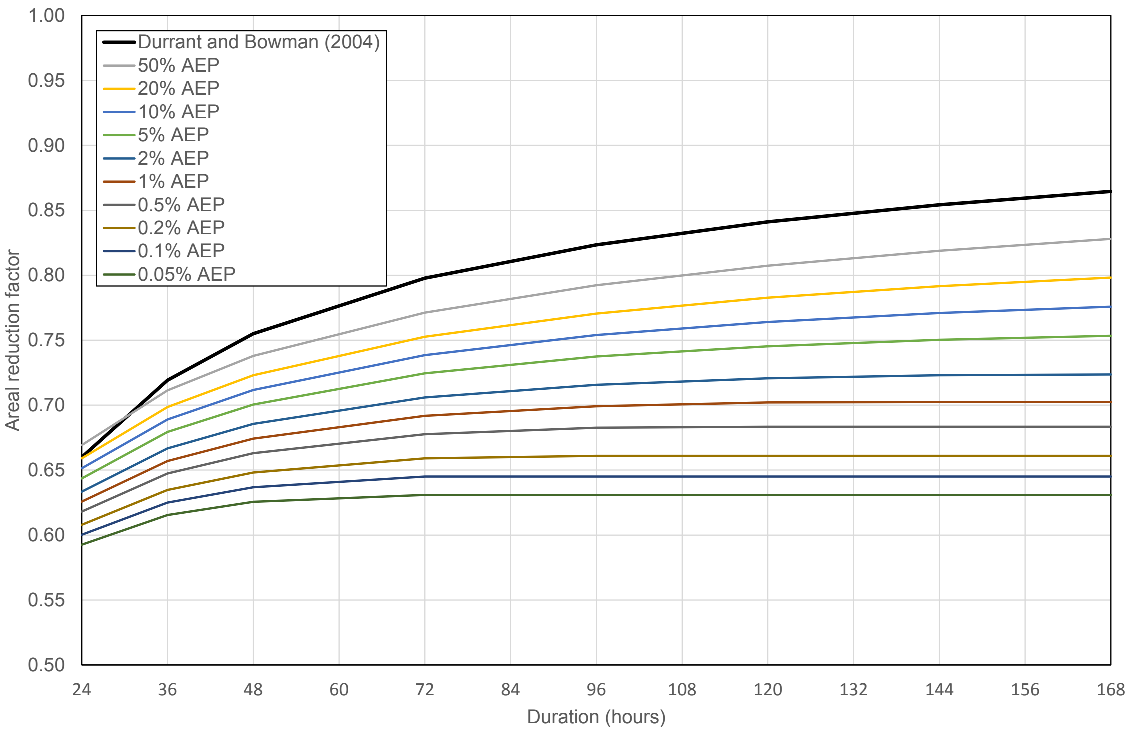
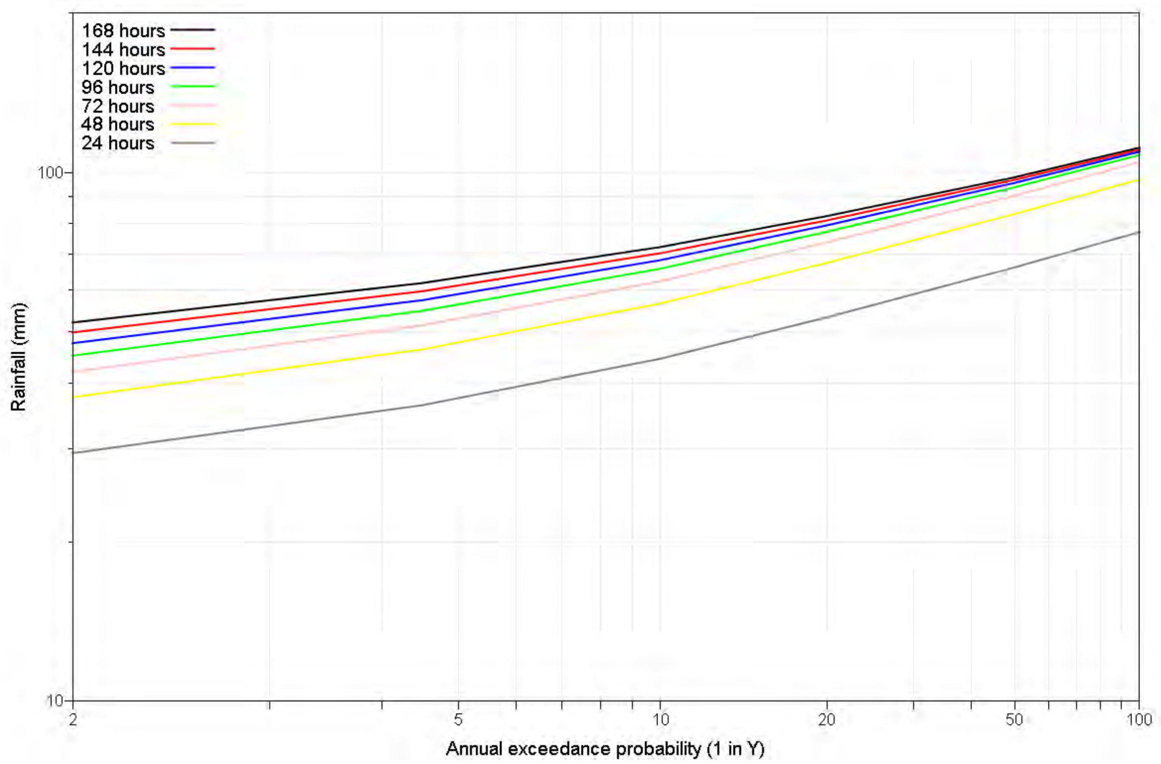


Figure 8-6: Podger *et al* (2015) and Durrant and Bowman (2004) ARF estimates

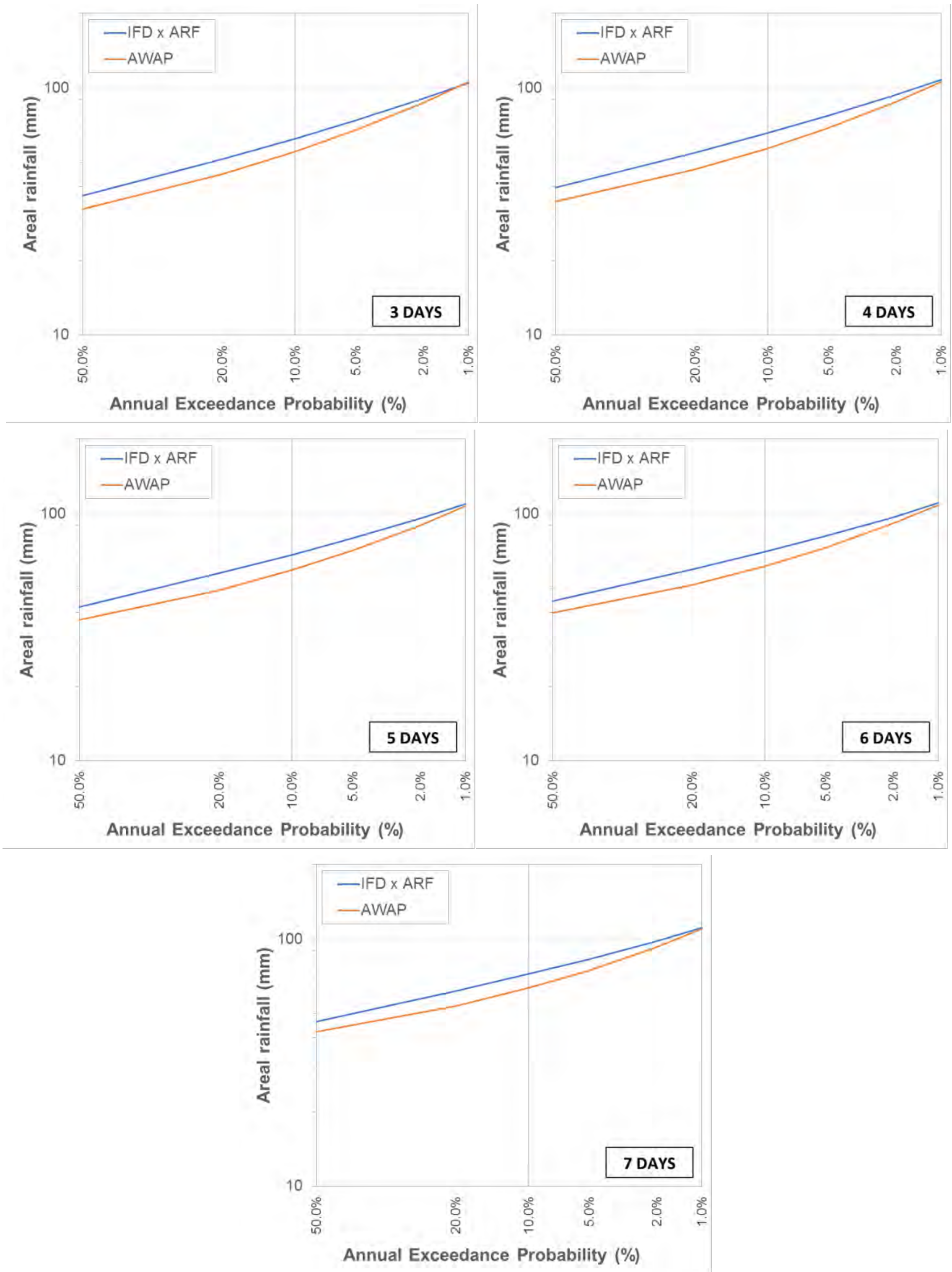
The extrapolated Podger *et al* (2015) ARF estimates were then applied to the BoM burst rainfall depths to estimate catchment average rainfall depths. These are shown in Figure 8-7.



- **Figure 8-7: BoM IFD and extrapolated ARFs rainfall depths**

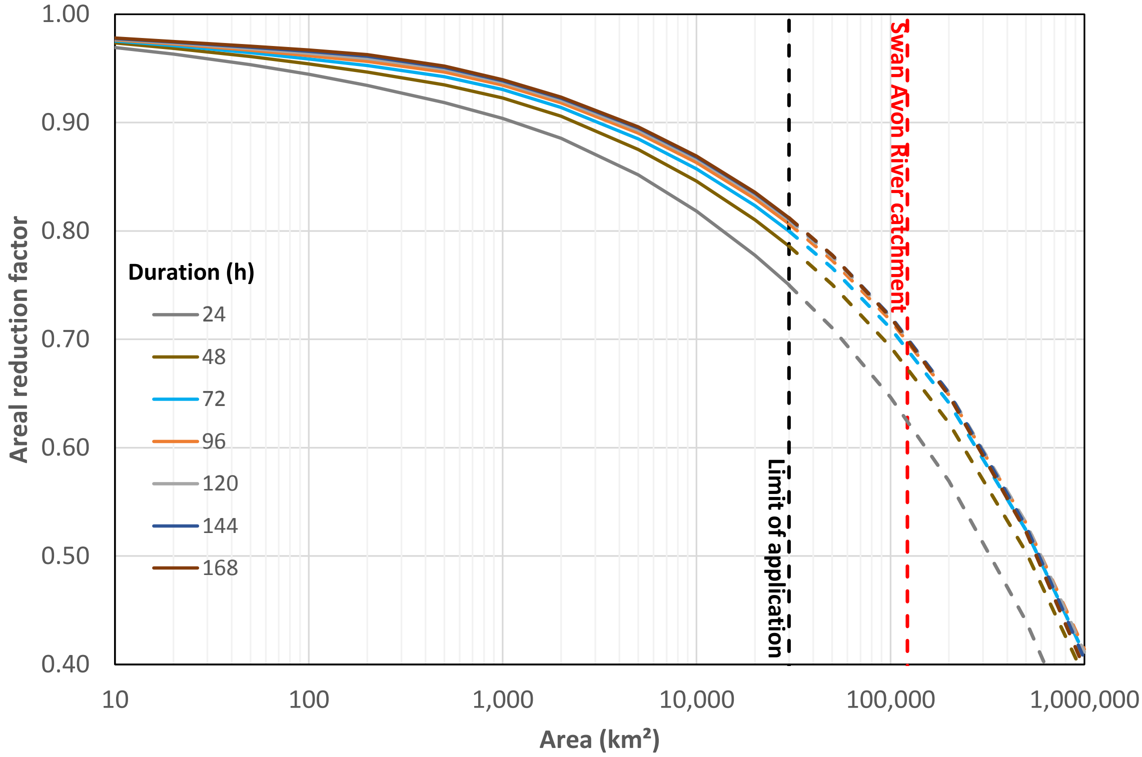
8.2.3 Comparison of methods

A comparison of the IFD and AWAP areal rainfalls is shown in Figure 8-8. It can be seen that the AWAP method results in a lower estimate of areal rainfall than the IFDs using extrapolated ARFs.



▪ **Figure 8-8: Comparison of IFD and AWAP areal rainfall estimates**

The extrapolated Podger et al (2015) ARFs for the 1% AEP are shown in Figure 8-9. It can be seen that they drop off steeply in what appears to be a conceptually consistent manner



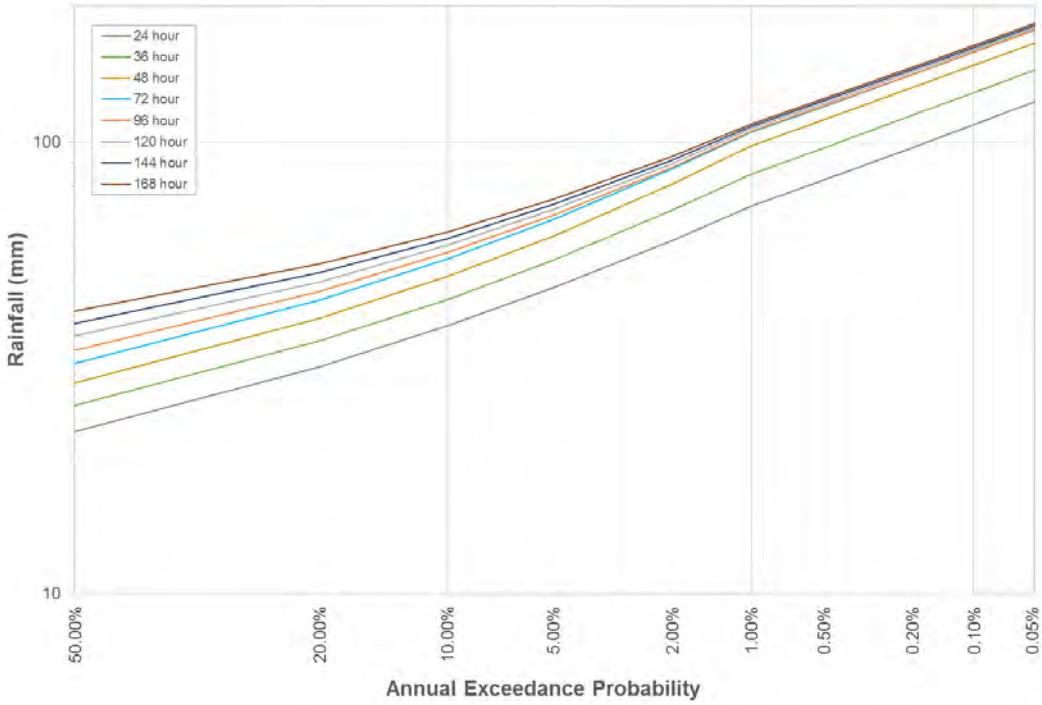
▪ **Figure 8-9: Change in ARFs with respect to area**

8.2.4 Burst depths for AEPs from 1% to 0.05%

The AWAP areal rainfall estimates were extended from an AEP of 1% to an AEP of 0.05% using the CRC-FORGE method.

The CRC-FORGE method (Durrant and Bowman, 2004) provides growth factors that extrapolate the 2% AEP point rainfall estimate for a particular duration, to the 1%, 0.5%, 0.2%, 0.1% and 0.05% AEP. These growth factors were extracted from gridded data sets available for standard storm durations (24, 48, 72, 96 and 120 hours). Growth factors for 36 hour rainfalls were interpolated from the 24 and 48 hour results, and growth factors for 120 hours were also applied to the 144 and 168 hour rainfalls. These growth factors were then re-scaled, such that the 1 in 100 AEP areal rainfall depth matched the AWAP analysis.

The AWAP areal rainfalls with an AEP of 1% were scaled using the CRC-FORGE growth factors, and are shown in Figure 8-10. Note that in order to estimate floods with an AEP of 0.05% in a Monte Carlo framework rainfalls an order of magnitude rarer (i.e. an AEP of 1 in 20,000) are required for sampling. These rainfalls have been estimated through extrapolation of the CRC-FORGE estimates. This extrapolation does not have a significant impact on the modelled floods.



- **Figure 8-10: Adopted design rainfall curves**

8.2.5 Probable maximum precipitation

Probable maximum precipitation (PMP) rainfall depths for burst durations between 24 and 96 hours were obtained using the GTSMR method for estimating PMP depths (BoM, 2003). PMP depths for durations of 120, 144 and 168 hours were estimated by extrapolating the curve of PMP depths versus burst duration (Table 8-4).

- **Table 8-4: Adopted PMP depths**

Duration	24 h	36 h	48 h	72 h	96 h	120 h	140 h	168 h
PMP depth (mm)	200	240	280	340	390	430	450	460

The AEP assigned to the PMP is a function of the method used in its derivation. The recommendations by Laurenson and Kuczera (1999) are for a lower limit of 1 in 10^7 for catchments less than 100 km², and for the AEP of the PMP to vary as a power function of catchment area up to an AEP of 1 in 10^4 for a catchment area of 100 000 km². The Swan Avon River catchment is larger than this upper bound, and would require site specific analysis to assign an AEP to the PMP. However, in this study the PMP is only required for estimation of the probable maximum flood (which typically cannot be assigned an AEP). Consequently, no AEP has been assigned to the PMP.

8.2.6 Pre-burst

There is limited information available on pre-burst depths or temporal patterns in the GTSMR region. It is understood that a dataset is currently being prepared as part of the revision of Australian Rainfall and Runoff, but as this data was not available and lieu of any other information, pre-burst was not modelled.

8.2.7 Spatial patterns

The spatial pattern adopted for all AEPs was based on the relative proportion of the catchment average 1% AEP rainfall depth to the 1% AEP rainfall depth in each model sub-area. This calculation was undertaken for each duration modelled using the 2013 IFD data, and is consistent with the approach which will be recommended in the revision of Australian Rainfall and Runoff. The spatial pattern is therefore similar in nature to the spatial variability of IFD shown in Figure 4-1.

8.2.8 Temporal patterns

A sample of temporal patterns from the catalogue of storms used to develop the GTSMR method for estimating PMP depths was used in the Monte-Carlo modelling of design floods for all storm durations and AEPs. Similarly to estimates of pre-burst, it is understood that a new sample of temporal patterns is being prepared as part of the ARR revision, but this data was not available at the time of writing. In lieu of this information, the GTSMR temporal patterns were considered to provide a reasonable sample of variability.

8.3 Helena River catchment

Design rainfalls were also estimated for the Helena River catchment (total area 1,646 km^2) so that design flow estimates could be provided for this catchment independently of the Swan Avon River catchment.

8.3.1 Burst depths

This section describes how design rainfalls depths were derived for the Helena River catchment for burst durations between 1 and 120 hours.

1 in 2 to 1 in 100 AEP

Point design rainfall depths for burst durations between 1 and 120 hours, and AEPs from 50% to 1%, were estimated for the centroid of each RORB model sub-area using the IFD 2013 analysis available from the Bureau of Meteorology website (http://www.bom.gov.au/water/designRainfalls/revised-ifd/). These were then converted to a catchment average point values.

1 in 100 to 1 in 2000 AEP

For short durations (1 hour to 12 hours), a regional approach for estimating design rainfalls developed by Jordan *et al* (2005) was adopted for AEPs between 1% and 0.05%. Jordan *et al* (2005) obtained rainfall records from the Bureau of Meteorology for twelve continuously recording rain gauges located around Australia, and used these records to estimate regional growth factors for short duration burst depths with an AEP between 2% and 0.05%.

For long durations, the CRC-FORGE method (Durrant and Bowman, 2004) provides growth factors that extrapolate the 2% AEP point rainfall estimate for a particular duration, to the 1%, 0.5%, 0.2%, 0.1% and 0.05% AEPs. These growth factors were extracted from gridded data sets available for standard storm durations (24, 48, 72, 96 and 120 hours). Growth factors for 36 hour rainfalls were interpolated from the 24 and 48 hour results. These growth factors were then re-scaled, such that the 1% AEP point rainfall depth matched the updated IFD analysis.

Areal reduction factors

Areal reduction factors (ARFs) are required to convert point rainfall estimates into catchment wide values. Recent work has been undertaken by Podger *et al* (2015) to derive updated estimates of areal reduction factors for short and long duration storms. As for the lower catchment, the equations from Podger *et al* (2015) were used to calculate areal reduction factors for all AEPs between 50% and 1%.

Areal reduction factors for AEPs rarer than 1% were extrapolated out using the Podger *et al* (2015) relationships. It is understood that this procedure is consistent with the advice under development for the revision of Australian Rainfall and Runoff. The adopted areal reduction factors are shown in Figure 8-11.

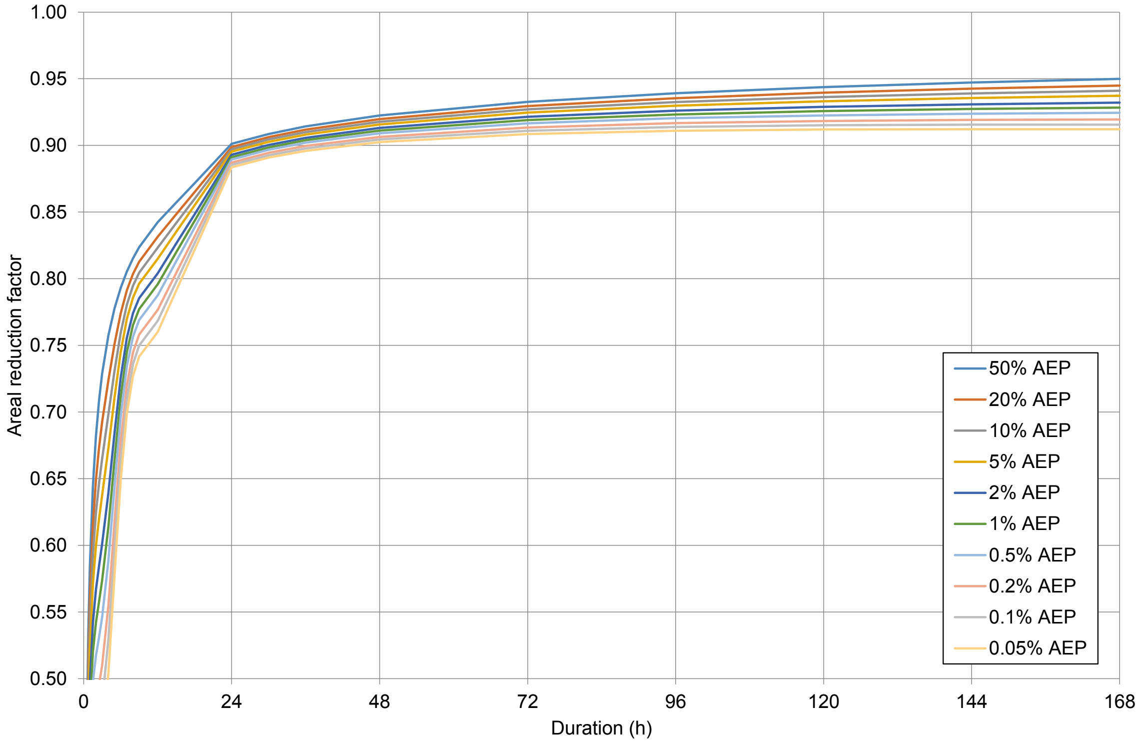


Figure 8-11: Helena River catchment areal reduction factors

PMP estimates

PMP estimates for 1 to 6 hour duration storms were obtained by applying the Generalised Short Duration Method (GSDM) as described by the Bureau of Meteorology (2003). For longer durations (24 to 120 hours), the GTSMR (BoM, 2003) method was used. An envelope curve was fitted by eye for the intermediate durations (9 to 18 hours). The adopted PMP depths are shown in Table 8-5.

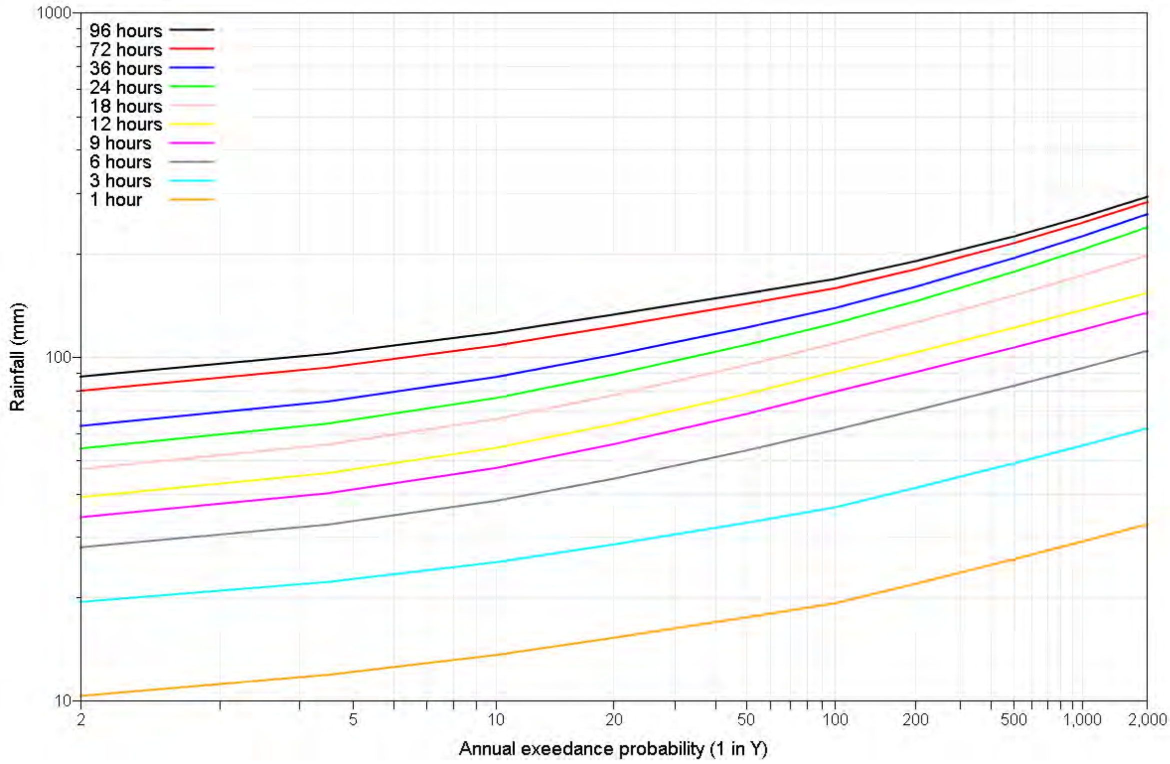
As the PMP estimates will only be used to estimate the PMF, no AEP was assigned for the PMP.

Table 8-5: PMP depths for the Helena River catchment

	PMP depth (mm)											
Duration	1 h	2 h	3 h	6 h	12 h	18 h	24 h	36 h	48 h	72 h	96 h	120 h
	206	284	368	489	560	600	640	750	860	1,040	1,190	1,250

Summary

The adopted design rainfall depths are summarised in Figure 8-12 and Table 8-6.



- **Figure 8-12: Design rainfall depths for the Helena River catchment**
- **Table 8-6: Design rainfall depths for the Helena River catchment**

AEP (1 in Y)	Design rainfall depth (mm)										
	1 h	2 h	3 h	6 h	12 h	18 h	24 h	36 h	48 h	72 h	96 h	120 h
10	13.6	20.4	25.4	38.3	54.6	66.2	76.2	87.7	96.0	108.1	117.9	126.8
20	15.3	22.9	28.6	44.4	64.0	77.6	89.2	101.7	110.4	122.9	133.1	142.9
50	17.5	26.3	33.0	53.6	78.2	94.9	108.7	122.0	130.7	143.0	153.3	164.0
100	19.3	29.0	36.6	61.5	90.7	109.8	125.5	139.0	147.3	158.7	168.9	180.2
200	22.0	33.1	41.7	70.1	103.4	126.3	145.5	160.3	169.5	180.3	190.4	201.9
500	25.9	39.0	49.2	82.7	121.9	151.3	177.2	194.3	204.6	214.7	224.6	236.6
1000	29.2	43.9	55.4	93.1	137.2	173.1	205.7	225.0	236.4	246.1	255.8	268.4
2000	32.7	49.3	62.2	104.4	154.0	197.8	238.9	260.7	273.5	282.8	292.8	306.2

8.3.2 Pre-burst

There is limited information available on pre-burst depths or temporal patterns in the GTSMR region. It is understood that a dataset is currently being prepared as part of the revision of Australian Rainfall and Runoff, but as this data was not available and lieu of any other information, pre-burst was not modelled.

8.3.3 Spatial patterns

The spatial pattern adopted for all AEPs was based on the relative proportion of the catchment average 1% AEP rainfall depth to the 1% AEP rainfall depth in each model sub-area. This calculation was undertaken for each duration modelled using the 2013 IFD data, and is consistent with the approach which will be recommended in the revision of Australian Rainfall and Runoff. The spatial pattern is therefore similar in nature to the spatial variability of IFD shown in Figure 4-1.

8.3.4 Temporal patterns

For short duration storms, temporal patterns for the Helena River catchment were extracted from the database of short duration temporal patterns provided in the GSDM.

For long duration storms, a sample of temporal patterns from the catalogue of storms used to develop the GTSMR method for estimating PMP depths was used in the Monte-Carlo modelling of design floods for all storm durations and AEPs.

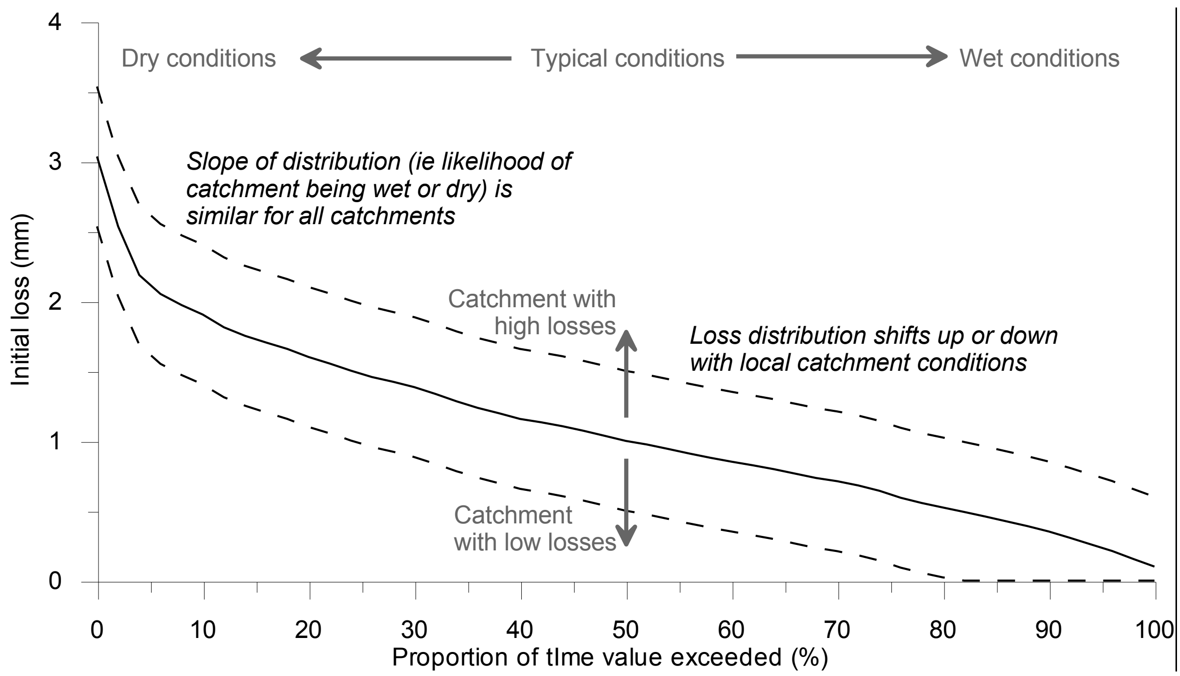
Similarly to estimates of pre-burst, it is understood that a new sample of temporal patterns is being prepared as part of the ARR revision, but this data was not available at the time of writing. In lieu of this information, the GSDM and GTSMR temporal patterns were considered to provide a reasonable sample of variability.

8.4 Losses

The shape of the initial loss distribution used in the design flood modelling was derived by Hill *et al* (2014) from flood modelling results for a large number of catchments across Australia. Hill *et al* (2014) developed a non-dimensional distribution of initial loss values for each catchment, by representing initial losses as a proportion of the median loss. This allowed the distributions of initial losses across different catchments to be directly compared. The standardised distributions exhibited a high degree of consistency, and suggested that while the magnitude of initial losses may vary between different catchments, the shape of the distribution does not. That is, while it may be expected that typical loss rates vary from one catchment to another, the likelihood of a catchment being in a relatively dry or wet state is similar for all catchments. This concept is illustrated in Figure 8-13. The initial losses reported by Hill et al (1997) for southeast Australia, Waugh (1991) for Western Australia, and Ilahee (2005) for Queensland follow a similar distribution when standardised as a proportion of median losses.

The correlation between initial losses and proportional losses is not well understood, but continues to be an area of active research. Current practice is for initial losses to be sampled from a distribution, while the proportional loss is held constant; this approach was used for the design flood modelling.

Values for the median initial loss and constant proportional loss rate for the Swan Avon River catchment were estimated by verifying the flood quantiles produced by RORB to flood frequency analyses of observed flood peaks and volumes at Walyunga. Details of the verification process and the adopted losses are given in Section 9.



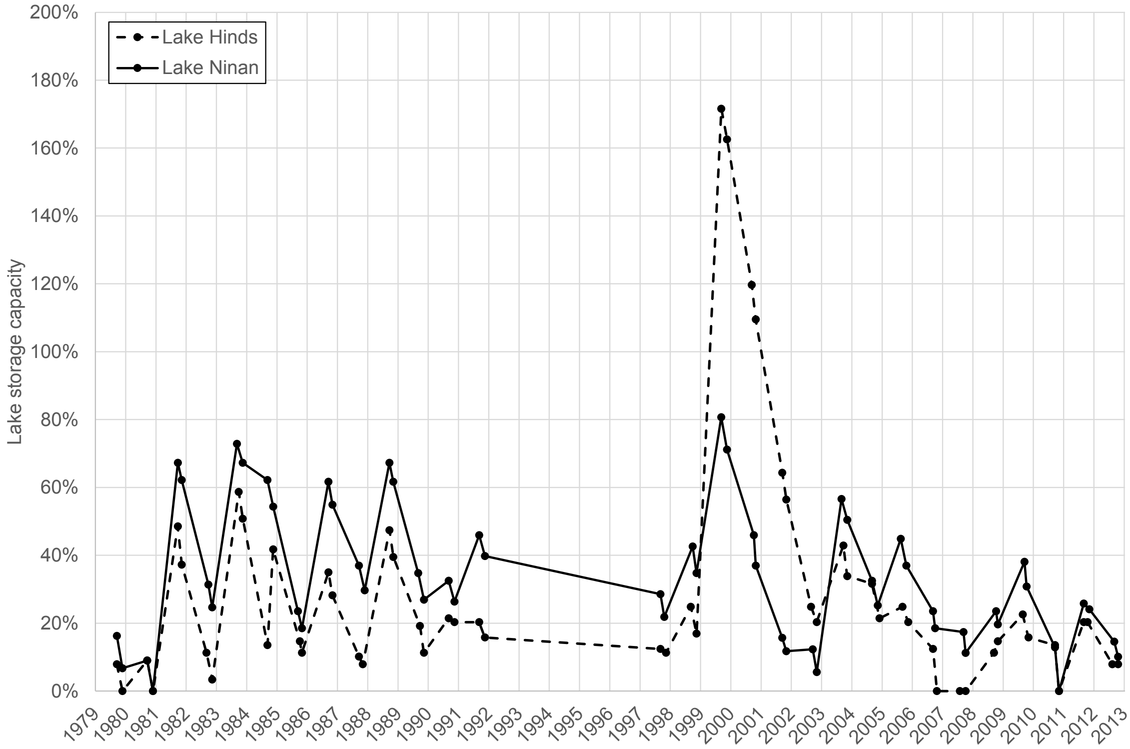
- **Figure 8-13: Schematic illustration of the variation in location but not shape of initial loss distributions for Australian catchments**

8.5 Lake and reservoir starting levels

The initial water level in lakes and reservoirs at the start of a flood event can have a significant impact on the degree to which the flood hydrograph loses volume and is attenuated as it passes through the waterbody. The initial water level in a waterbody is often considered independent to other flood producing variables such as initial loss, although both are a product of the antecedent conditions. Typically, lake and reservoir water levels are controlled by long term (months) climatic variability, whereas initial loss is more likely to vary with short term (days to weeks) climatic conditions.

Excluding the special case of Mundaring Dam, which is discussed in Section 8.5.1, the waterbodies of interest in the Swan Avon River catchment are natural lakes. As noted previously, three of these lakes (Hinds, Ninan and the Yenyenning system) have been explicitly included in the RORB model. The influence of the remaining lakes is implicitly addressed through the use of loss functions and lower than expected runoff coefficient values.

The extent to which the variability in starting water level, and thus drawdown (or the volume of airspace in the lake prior to spilling) can be defined for these natural lakes is limited. The formal gauging of lake levels available was in the South West Wetlands Monitoring Project report (DPW, 2013), which provides plotted values of six-monthly recorded water levels at numerous lakes from 1977 to 2012. For Lake Hinds and Ninan, this data was digitised, converted to a percentage of storage volume (with 100% being the spill level of the lake) and plotted, as shown in Figure 8-14.



- **Figure 8-14: Lakes Hinds and Ninan six-monthly water level records**

It can be seen that over the period of record, Lake Hinds has spilled once (during the January 2000 event), and Lake Ninan has not spilled. This assumes that a spill event was not missed during the interval between the six-monthly readings, or the period of missing data between 1992 and 1997. There is a reasonable correlation between the water level at both lakes, and the typical (or median) water level in the lakes is approximately 25 – 30% of the storage volume below the spill threshold.

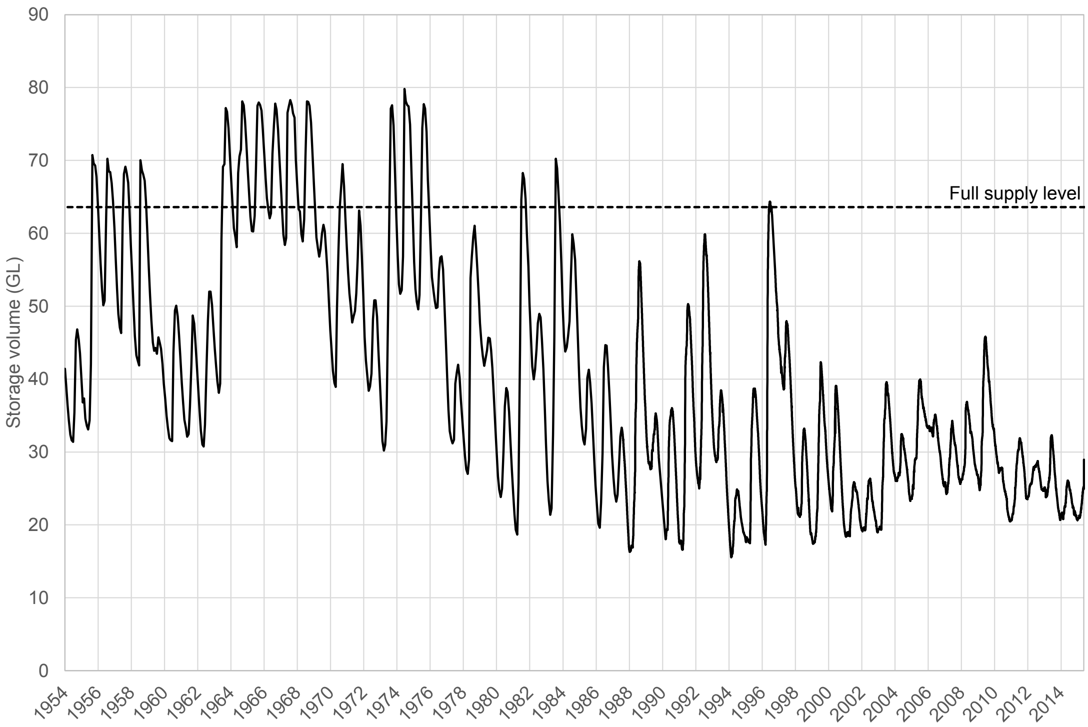
A number of trials were undertaken with the RORB model in design mode to determine the influence of the initial water level in these lakes on design peak flows at Walyunga. It was found that the initial level in Lakes Hinds and Ninan has minimal influence over modelled flows at Walyunga, and therefore a degree of conservatism was applied to the design model runs by setting the initial water level at the spill level of the lakes. The same assumption was applied for the Yenyenning Lakes, as the volume of storage available there below the spill threshold is negligible.

For the three loss functions which are incorporated in the upper reaches of the Yilgarn River, there was minimal information available on how variability in water level at the large lakes systems upstream of Merredin would impact these loss volumes. Initial runs of the RORB model indicated that the volumes assumed for these loss functions had a significant impact on estimated design flows at Walyunga, however this impact could be dampened somewhat by adjustment of the runoff coefficient value over the Yilgarn River interstation area. It is suggested that, given the results of the verification discussed in Section 9.2, that this issue would be worthy of further study. For the design and verification runs, it was ultimately decided to assume that these loss functions behaved in the same manner as was used for the calibration events, in lieu of better information.

8.5.1 Mundaring Dam

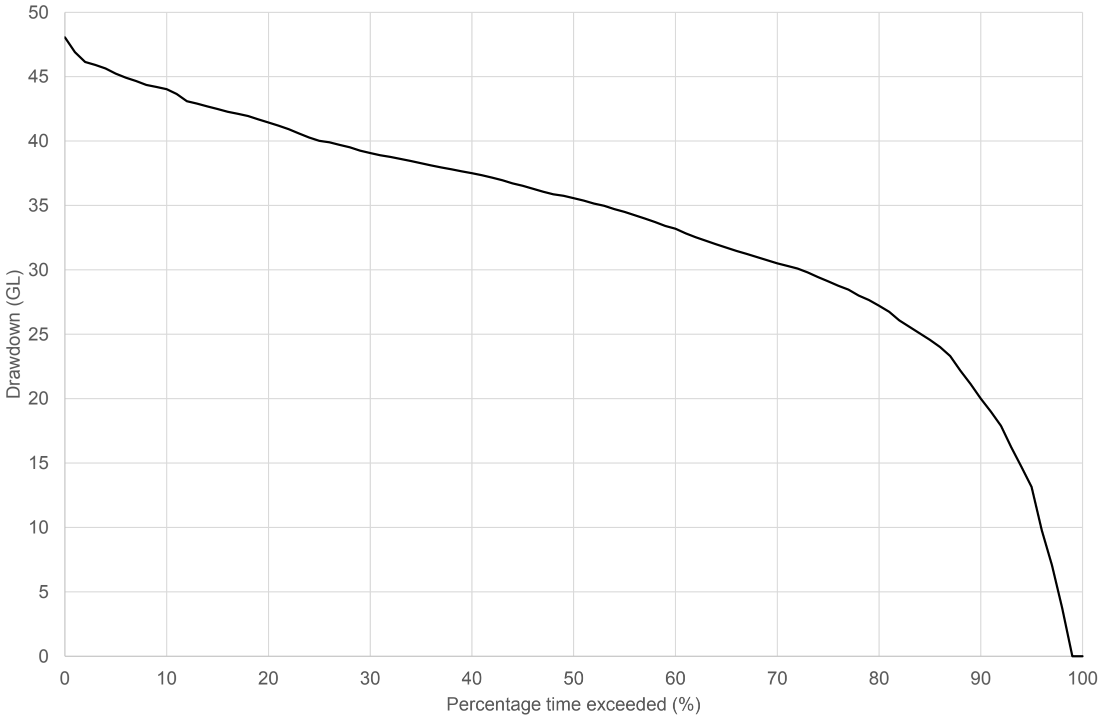
As noted in Section 5.5, Mundaring Dam has only spilled once since the early 1980s. The implications of this are that the initial storage volume in the dam is very influential on flood estimates at the Whiteman Road gauge, as the dam impounds 90% of the Helena River catchment.

Historic daily storage volumes were provided for Mundaring Dam by Water Corporation. This data was analysed and used to determine the daily storage volume in the dam, as well as the storage volume below full supply level (63.6 GL). Only data from the period 1954 onwards was used, as the dam was raised in 1953. The time series of storage volume is shown in Figure 8-15, and the exceedance curve of volume below full supply level, which was sampled in the RORB model, is shown in Figure 8-16.



- **Figure 8-15: Mundaring Dam storage volume**

It is potentially conservative the use the full period of record between 1954 and 2015. The time series plot in Figure 8-15 shows that since the early 1980s there has been a significant decrease in typical storage volumes within the dam. Using the full period of record will therefore tend to underestimate the available airspace in comparison to recent conditions.



■ **Figure 8-16: Mundaring Dam exceedance curve of storage below FSL 1954-2015**

8.6 Baseflow

Appropriately simulating the baseflow expected during floods is a key component of the estimation of complete hydrographs suitable for use in the hydraulic modelling component of a flood study. Baseflow was estimated for the Swan River at Walyunga using the following steps:

- The annual maxima total gauged flow of each year of the Walyunga gauge record was found, and the baseflow on this day and three days either side was extracted from the digital filter results (as described in Section 4.2.1).
- The maximum baseflow in each seven day window was selected.
- A generalised extreme value (GEV) distribution for each season was then fitted to these baseflows.

The results of this process were GEV distributions of the baseflow expected during the annual maximum flood at Walyunga. The distributions were capped at values 50% greater than the 1% annual exceedance probability. This approach is slightly more conservative than recommended in Book VI of ARR (Nathan and Weinmann, 2000) which suggests adopting baseflows 20% - 50% higher than the maximum observed when modelling extreme floods, however as noted previously the gauge record potentially underestimates baseflow as it does not include a number of large events prior to the start of the record.

The adopted baseflow distribution is shown in Figure 8-17.

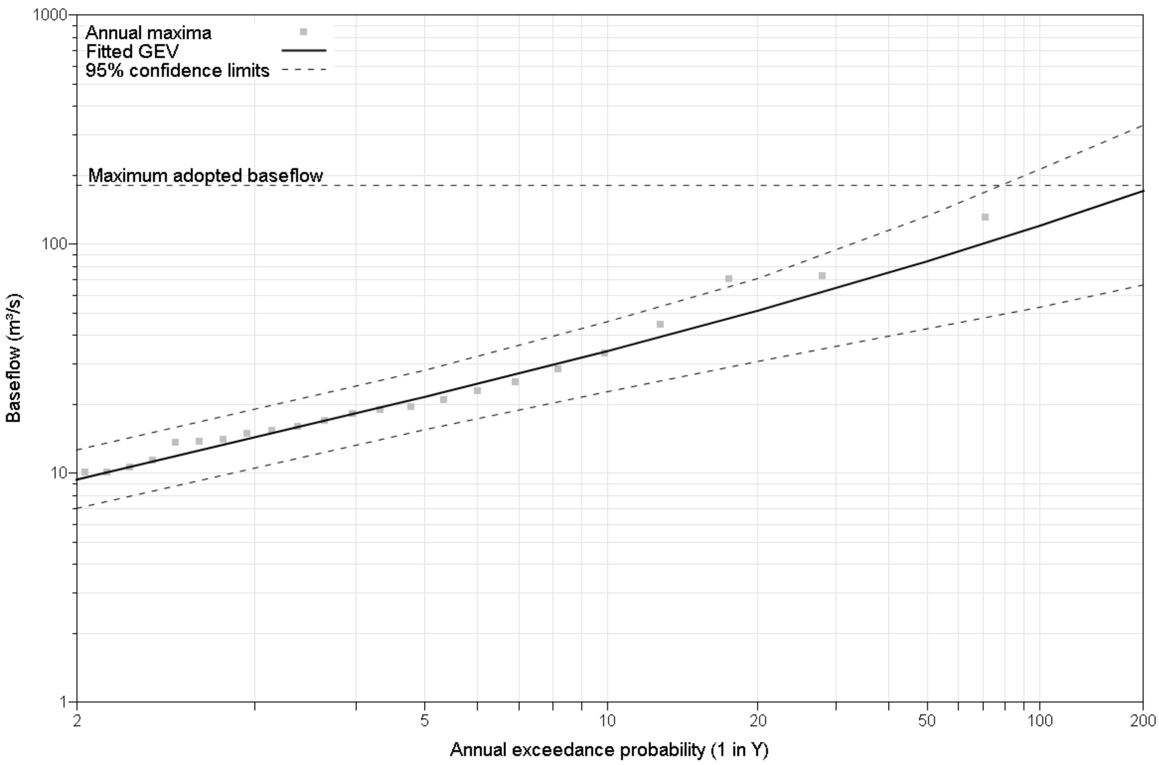


Figure 8-17: GEV distribution of baseflow at Walyunga

9. Model verification

The model calibration documented in Section 6 demonstrates that the model can reproduce specific historic events using reasonable routing and loss parameter values. However, it can be seen from the calibration results that loss parameter values (unlike the routing parameter values) vary significantly from event to event due to factors such as antecedent wetness and rainfall intensity. By running the model with the design inputs and for the full flood frequency curve, it is possible to select a set of loss parameter values such that the modelled flood frequency curve replicates the gauged flood frequency curve. This process is referred to as verification, and provides additional objective reliability in the adopted loss parameters. The verification is discussed further in this section.

9.1 Approach

The verification of the Swan Avon River lower and complete catchment models was complicated by the uncertainty associated with the flood frequency analysis at Walyunga (refer Section 4.2.2.2). Typically, the verification process relies on the model loss parameter values being adjusted until the model results match the gauged flood frequency estimates. However, in this case it was decided that there was insufficient robustness in the flood frequency estimates to act as the sole point of truth. The verification was therefore undertaken by setting loss parameter values that appeared reasonable given the calibration results, and to ensure that the slope of the modelled flood quantiles was similar to the slope of the gauged frequency estimates (which are relatively similar regardless of the assumptions made about pre-gauge floods). For the Helena River catchment, the model was verified by adjusting the loss parameter values such that the modelled flood quantiles matched those at the Craignish gauge, which has a significantly longer period of record than Whiteman Road.

9.2 Lower and complete catchment results

The RORB model for the Swan Avon River lower catchment was run with different combinations of initial loss and proportional loss, until the slope of the RORB model results approximated the slope of the flood quantiles from the annual flood frequency analysis at Walyunga. The model runs were undertaken using:

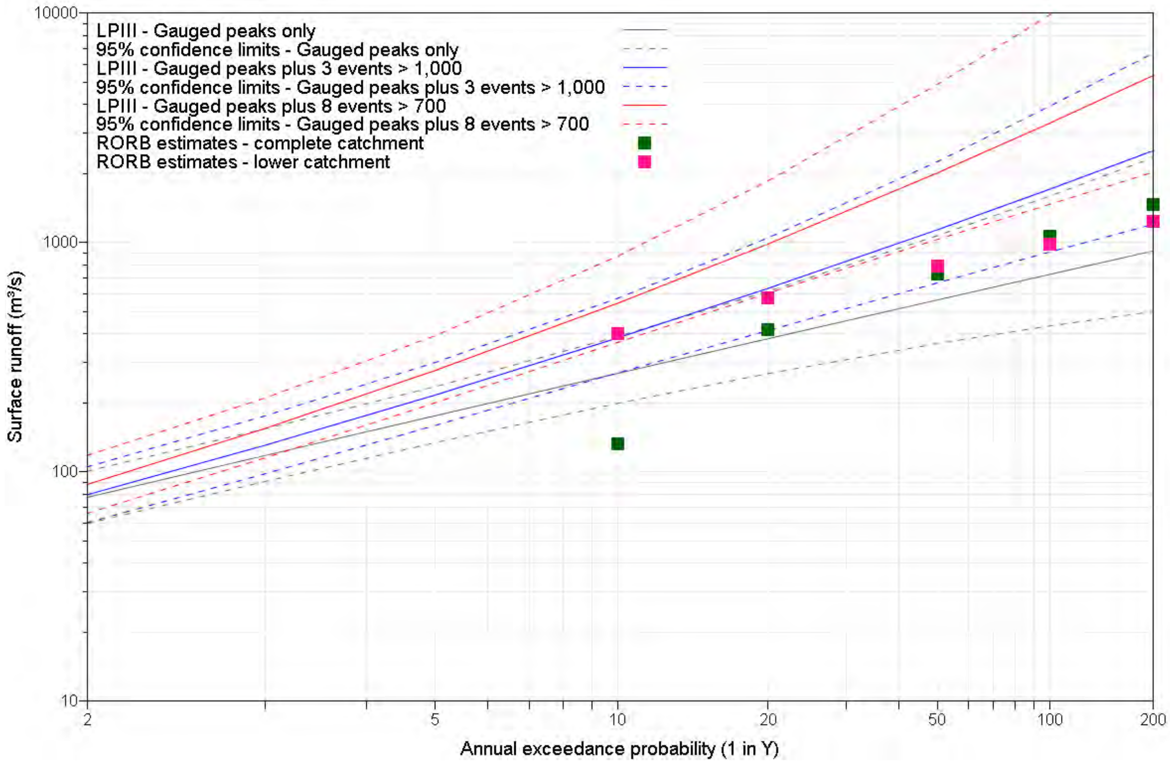
- Routing parameter values as specified in Section 6.7
- Design rainfall bursts for the lower catchment area, calculated in the manner described in Section 8.1
- A spatial pattern that varied with duration, with each spatial pattern based on the 1% AEP rainfall estimated by the Bureau of Meteorology's IFD 2013
- A sample of temporal patterns from the catalogue of storms used to develop the GTSMR method for estimating PMP depths.

Using the design inputs specified above, it was found that a combination of a runoff coefficient value of 0.35 (similar to the 'typical' results obtained during calibration) and an initial loss value of 100 mm provided a reasonable match to the slope of the gauged flood frequency estimates. Whilst this initial loss value is somewhat higher than the 'typical' values obtained from the four calibration events, this is not necessarily unexpected. Selection of the calibration events tends to be biased towards larger flood events, whereas significant rainfall events which did not generate large floods are ignored. Lowering the initial loss value for design was found to overemphasise the magnitude of the more frequent floods (between the 10% and 5% AEPs), causing the slope of the modelled frequency curve to be overly flat. The parameter values adopted produced model results somewhat higher than the frequency curve which excluded the pre-gauge events, but somewhat lower than including three pre-gauged events larger than 1,000 m^3/s. This results appears reasonably consistent with the available evidence, including the calibration events and the gauged frequency estimates. It demonstrates that it is possible for large floods to be generated from the lower catchment only, with this model not including any inflows from the upper catchment.

To further investigate the role of the upper catchment, the complete catchment model was run with the same loss parameter values over the lower catchment. For the upper catchment, the loss values were increased to an initial loss of 120 mm and a runoff coefficient of 0.20. The selection of these values was based on the evidence from the calibration events that losses tended to be higher in the upper catchment, as well as the need to maintain the slope of the modelled frequency curve. The model runs were undertaken using:

- Routing parameter values as specified in Section 6.7
- Design rainfall bursts for the complete catchment area, calculated in the manner described in Section 8.2
- A spatial pattern that varied with duration, with each spatial pattern based on the 1% AEP rainfall estimated by the Bureau of Meteorology's IFD 2013
- A sample of temporal patterns from the catalogue of storms used to develop the GTSMR method for estimating PMP depths
- Zero drawdown at all lakes and storages at the start of the model run.

Figure 9-1 shows the range of flood frequency curves at Walyunga, together with the RORB model results for the lower and complete catchments.



- **Figure 9-1: Lower and complete model verification results**

The outcome of the verification demonstrates that floods at Walyunga between AEPs of 2% and 0.5% can be generated from either the lower or complete catchments. Although the design rainfall depths adopted for the complete catchment are significantly lower (and the adopted loss values for the upper catchment are significantly higher) than for the lower catchment, the sheer volume of water generated from the larger upper catchment does provide a sufficient mechanism to generate floods of a similar magnitude to the lower catchment by itself. These results match the anecdotal and gauged evidence of the flood regime. Since gauged records began in 1970, there is only one flood where significant outflows were recorded from the Yenyenning Lakes (January 2000). Similarly, in comparison to the pre-gauged flood peaks, the AEPs of the gauged annual maxima are typically more frequent than about 2%. There is little anecdotal evidence to indicate where the pre-gauged floods were generated, however review of Figure 4-9 indicates that at least some of these events recorded significant rainfall over the lower reaches of the Yilgarn and Lockhart Rivers (for example July 1917, June 1946 and August 1963). Indeed, it appears that as the magnitude of flood events grows larger, the influence of the complete catchment becomes greater. There appears to be a point for AEPs between 2% and 1%, where either the lower or complete catchment is able to generate floods of the same magnitude. The full model runs (Section 10.1.1) demonstrate that for floods rarer than the 2% AEP, the complete catchment tends to produce flows in the order of 10% larger than the lower catchment.

Further investigation was therefore undertaken to determine which parts of the catchment generated the largest contributions to flows at Walyunga. This analysis was undertaken for the 1% AEP quantile, and is shown in Table 9-1.

Table 9-1: Contribution of flows to peak at Walyunga

Interstation area	Lower catchment 1% AEP		Complete catchment 1% AEP	
	Peak flow (m^3/s)	Critical duration (hours)	Peak flow (m^3/s)	Critical duration (hours)
Yilgarn River	0	N/A	975	120
Lockhart River	0	N/A	475	72
Salt River to Yenyenning Lakes	0	N/A	1,035	144
Avon River to Boyagarra Rd	140	144	75	168
Avon River to Beverley	160	168	1,025	144
Dale River to Waterhatch Bridge	390	36	245	48
Avon River to York	510	48	1,020	144
Avon River to Northam	480	72	1,020	144
Mortlock River East Branch	0	N/A	105	144
Mortlock River East Branch to O'Driscolls	190	48	110	72
Mortlock River North Branch to Lake Ninan	0	N/A	145	72
Mortlock River North Branch to Frenches	270	48	170	72
Avon River to Toodyay	840	72	1,045	144
Brockman River	190	48	120	72
Avon River to Walyunga	970	120	990	144

This analysis demonstrates that there are two different mechanisms of flood generation from these catchments. The lower catchment results tend to increase from upstream to downstream, presumably as additional local runoff is added into the hydrograph. This effect tends to dominate the routing of the hydrograph as it moves along the river. Conversely, for the complete catchment, the largest peak flows are generated upstream of the Yenyenning Lakes. As the hydrograph moves downstream, the additional inflows from local runoff are overwhelmed by attenuation of the large flood peak from upstream.

The range of critical durations estimated from these results is also of interest. The modelled design flood quantiles and their critical durations for the both the lower and complete catchments are shown in Table 9-2.

Table 9-2: Modelled critical durations at Walyunga

AEP	Lower catchment		Complete catchment	
	Peak flow at Walyunga (m^3/s)	Critical duration (hours)	Peak flow at Walyunga (m^3/s)	Critical duration (hours)
10%	400	144	150	168
5%	585	144	400	168
2%	780	96	665	168
1%	970	120	990	144

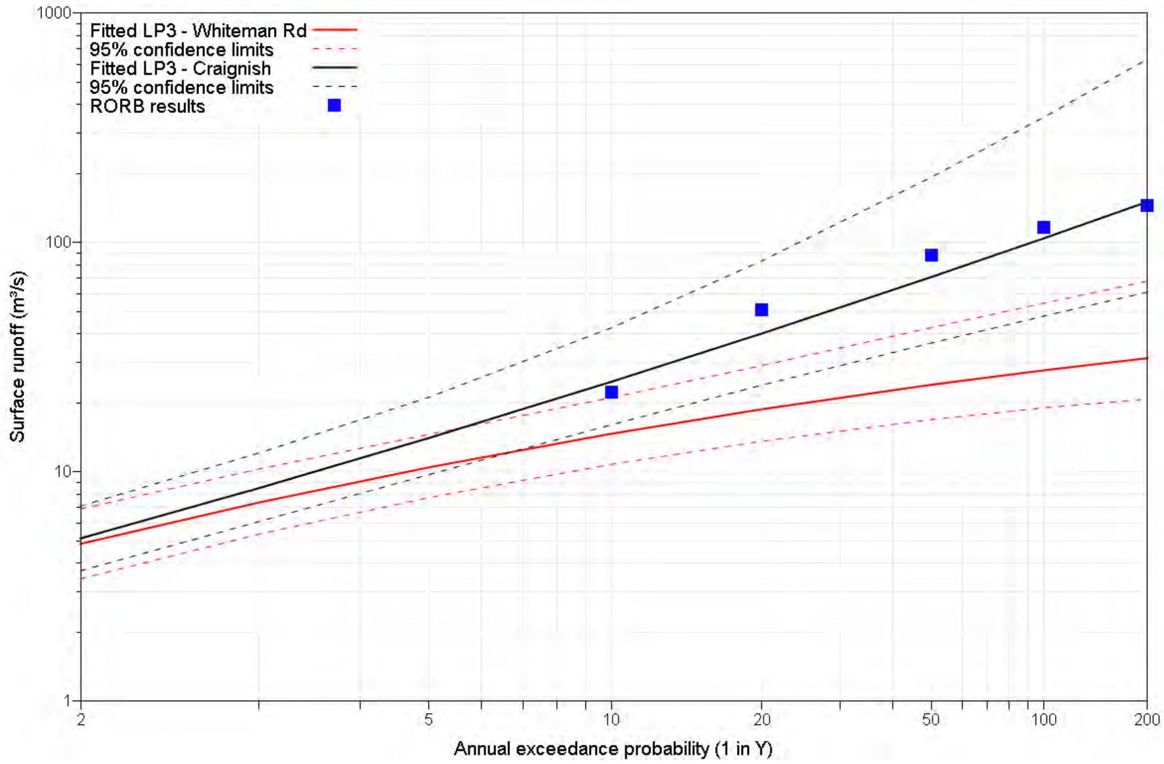
9.3 Helena River catchment

The RORB model for the Swan Avon River was run with different combinations of initial loss and proportional loss, until the RORB model results matched satisfactorily with the 10%, 5%, 2% and 1% AEP quantiles from the annual flood frequency analysis at Craignish. The model runs were undertaken using:

- Routing parameter values as specified in Section 6.7
- Design rainfall bursts for the Helena River catchment area, calculated in the manner described in Section 8.3
- A spatial pattern that varied with duration, with each spatial pattern based on the 1% AEP rainfall estimated by the Bureau of Meteorology's IFD 2013
- A sample of temporal patterns from the catalogue of storms used to develop the GTSMR method for estimating PMP depths
- Drawdown in Mundaring Dam sampled from the distribution derived in Section 8.5.1.

The modelling results showed that an initial loss of 80 mm and a runoff coefficient of 0.27 resulted in a reasonable match to the flood frequency curve at Craignish. This is regarded as being the more accurate comparison given that the gauge at Whiteman Road does not include a number of the events during which Mundaring Dam overtopped. The verification results are shown in Figure 9-2.

It is noted that the adopted runoff coefficient value is lower and the initial loss value is higher for the Helena River catchment than the values used in the lower and complete catchment model verifications. This represents the quantitatively different hydrologic response of the Helena River catchment. The catchment is located on the sandy soils of the Darling Scarp, and heavily forested. Previous modelling indicates that these catchments tend to have very low runoff coefficients, and the value of 0.27 that was adopted is consistent with the volumetric runoff coefficients estimated as part of the extreme flood studies for Mundaring Dam (DoE, 2004a) and the Helena River Pumpback Weir (DoE, 2004b).



■ **Figure 9-2: Helena River model verification results**

The critical durations estimated from the modelling results are significantly longer than would normally be expected for a 1,700 km^2 catchment. This effect is caused by the presence of Mundaring Dam, which requires a large volume to force spills from the dam under the majority of conditions. These large volumes are typically generated by very long duration storms. For the 10% AEP event, the dam does not spill, which is consistent with the recent history of the catchment. A summary of the critical durations is shown in Table 9-3.

■ **Table 9-3: Modelled critical durations at Craignish**

AEP	Peak flow at Craignish (m^3/s)	Critical duration (hours)
10%	24	168
5%	57	168
2%	92	168
1%	120	168

9.4 Adopted loss parameters

To derive a final set of loss parameters for use in the design runs, it was decided to adopt the design rainfalls and losses associated with the lower catchment model. Despite some quantitative uncertainty surrounding the estimates of peak flow at Walyunga prior to the commencement of the gauge record, the flood frequency analysis associated with these peaks is regarded as more representative of the historic behaviour of the catchment. For AEPs rarer than about 0.5%, the complete catchment model generates peak flows approximately 10% larger than those from the lower catchment only. Given the very high degree of uncertainty associated with these flow estimates, it was decided to still adopt the lower catchment flows for all AEPs up to 0.05%.

A summary of the adopted loss parameter values is included as Table 9-4. Note that these losses were applied uniformly across the lower catchment, with the exception of the Helena River interstation area, where the loss values were adopted from the catchment-specific verification. In all cases, the losses were assumed to be invariant with both storm duration and AEP. There is reasonable evidence to suggest that initial loss behaves in this manner (e.g. ARR Book VI), however there is considerable uncertainty around the behaviour of proportional loss with AEP. Conceptually, it would be expected that proportional loss would reduce for less frequent AEPs. Despite this, the overall comparison of the modelled results with the gauged flood frequency curve suggests that the adopted values of proportional loss are valid for AEPs up to 1%. Beyond this, there may be a case for decreasing the proportional loss, however there was scant evidence to support such an adjustment, so the verified value was adopted for all AEPs.

Table 9-4: Adopted loss parameters for all AEPs

<table>
<tr><th>Interstation area</th><th>Initial loss (mm)</th><th>Runoff coefficient</th></tr>
<tr><td>Yilgarn River</td><td rowspan="3">N/A</td><td rowspan="15">0.35</td></tr>
<tr><td>Lockhart River</td></tr>
<tr><td>Salt River to Yenyenning Lakes</td></tr>
<tr><td>Avon River to Boyagarra Rd</td><td rowspan="5">100</td></tr>
<tr><td>Avon River to Beverley</td></tr>
<tr><td>Dale River to Waterhatch Bridge</td></tr>
<tr><td>Avon River to York</td></tr>
<tr><td>Avon River to Northam</td></tr>
<tr><td>Mortlock River East Branch</td><td>N/A</td></tr>
<tr><td>Mortlock River East Branch to O'Driscolls</td><td>100</td></tr>
<tr><td>Mortlock River North Branch to Lake Ninan</td><td>N/A</td></tr>
<tr><td>Mortlock River North Branch to Frenches</td><td rowspan="4">100</td></tr>
<tr><td>Avon River to Toodyay</td></tr>
<tr><td>Brockman River</td></tr>
<tr><td>Avon River to Walyunga</td></tr>
<tr><td>Helena River</td><td>80</td><td>0.27</td></tr>
</table>

Comparisons of the loss values used for verification/design and those used in calibration should be undertaken with some caution. The process of selection of calibration events tends to be heavily biased towards those events which have favourable flood-producing conditions (ie intense rainfalls and low losses). As such, the range of calibrated loss parameter values does not include cases such as intense rainfall coupled with high losses, as these cases do not tend to produce large floods. However, such events are implicitly contained within the gauged frequency analysis and so need to be addressed when setting design losses for verification. Additionally, the wealth of information available at the different gauge locations during calibration allows for a higher degree of spatial variability in the loss rates. For verification, the only gauge where there is a sufficient length of record and evidence of pre-gauge events to allow for robust estimates of losses is at Walyunga. Therefore, the spatial variability of the losses is much more lumped than for calibration.

Given this, the adoption of a runoff coefficient value of 0.35 for the lower catchment is regarded as a reasonable balance between the variability observed during calibration and the gauged and pre-gauged evidence of flood behaviour at Walyunga. During the 1983, 1946 and to a lesser extent January 2000 event, a number of the interstation areas in the lower catchments used runoff coefficients of close to this value. The initial loss value of 100 mm adopted for verification/design is considerably higher than any of the values observed during calibration, but as noted above this is not unexpected. Initial loss is the model parameter most subject to variability as a result of antecedent conditions, and as such is expected to demonstrate the most change from calibration to verification. It is also noted that the initial loss needs to reflect a mixture of summer and winter conditions, whereas the calibration events were predominately winter (with the exception of January 2000, which as noted previously had reasonable antecedent rainfall). It can be seen from Figure 4-2 that the majority of large rainfall events occur in summer, whereas virtually all of the large floods recorded at Walyunga (refer Table 4-3 and Table 4-4) occurred in winter. This suggests that summer losses are significantly higher than winter losses, and as observed in the verification results, lowering of the initial loss value tends to overly flatten the slope of the modelled frequency curve, overestimating floods around the 10% AEP.

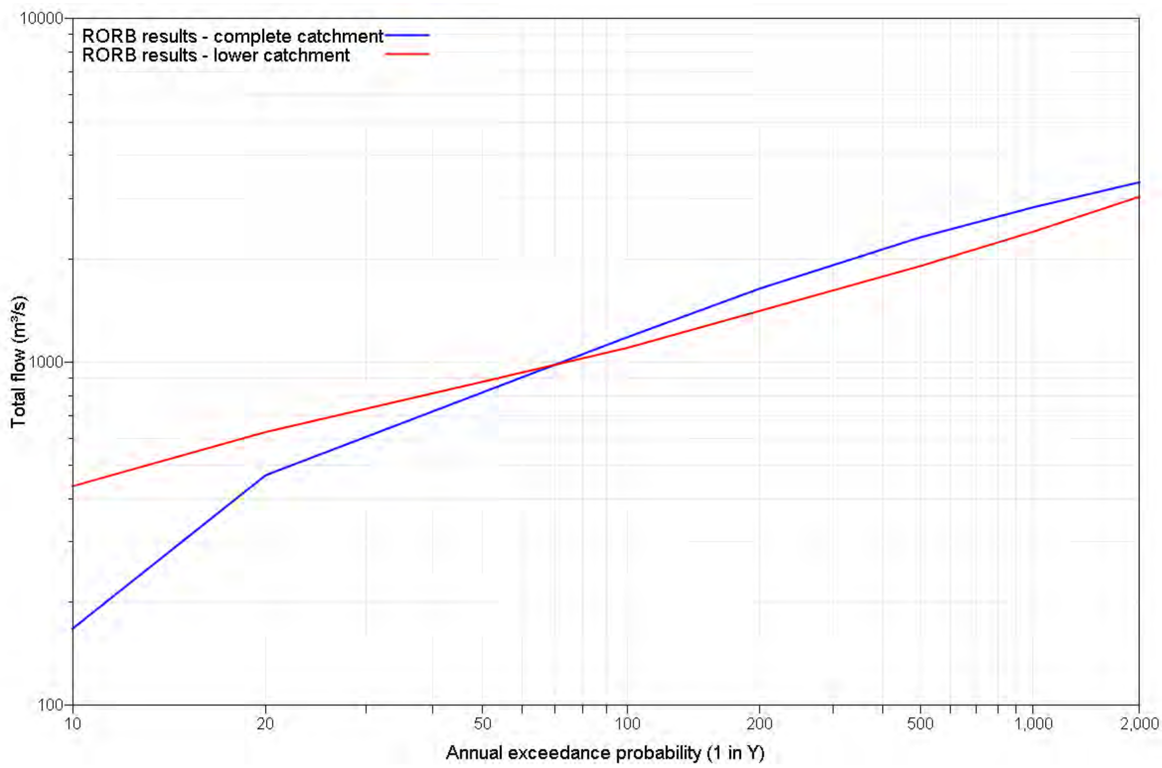
10. Design flood hydrology

In order to generate estimates of design flood peaks and hydrographs for the study area, the RORB models were run in design mode using the inputs and adopted parameter values described in the previous sections.

10.1 Modelling results

10.1.1 Swan River at Walyunga

To determine the design flood quantiles and hydrographs at Walyunga, the lower catchment model was run for all AEPs up to and including 0.05%. To provide a point of comparison, the complete catchment model was also run. The design models are basically the same as those used for verification, with the addition of an AEP-varying, time-constant baseflow input at Walyunga based on the analysis discussed in Section 8.6. The results of the design model runs are shown as a flood frequency curve in Figure 10-1 and tabulated values in Table 10-1.



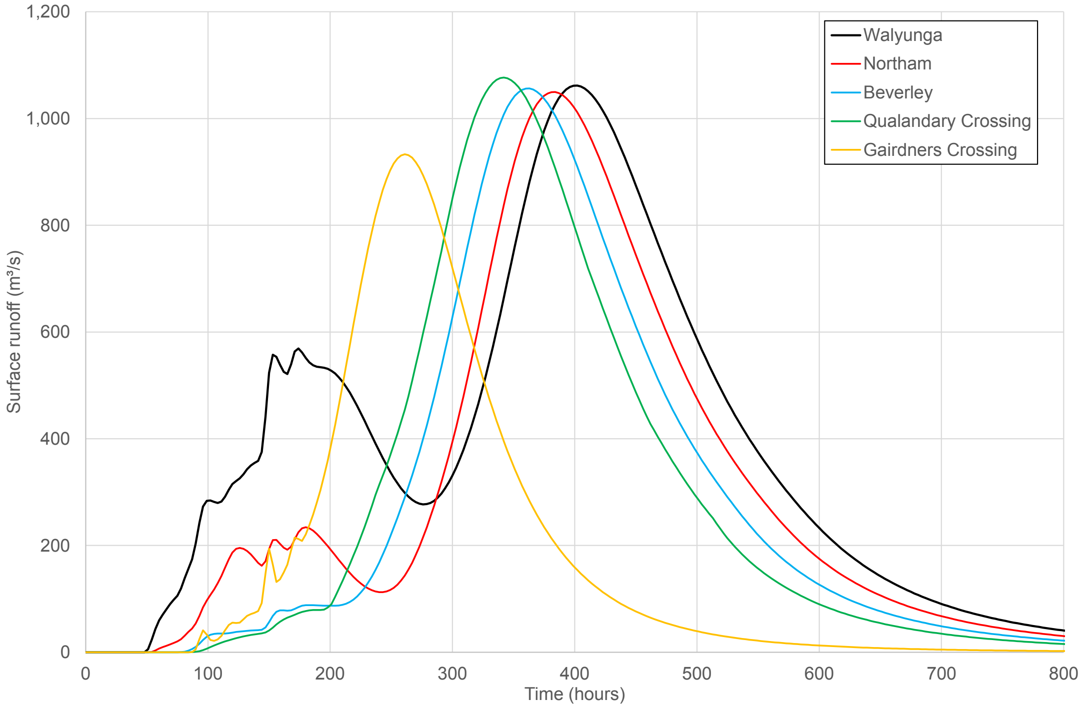
- **Figure 10-1: Design flood frequency curve at Walyunga**

It can be seen from Figure 10-1 that the complete catchment flows are significantly less than those estimated using the lower catchment model for AEPs up to 2%. The influence of the complete catchment tends to increase with AEP, particularly for AEPs rarer than 0.5%, however the effect of this is subject to considerable uncertainty and the increase in peak flow estimates is only of the order of 10%.

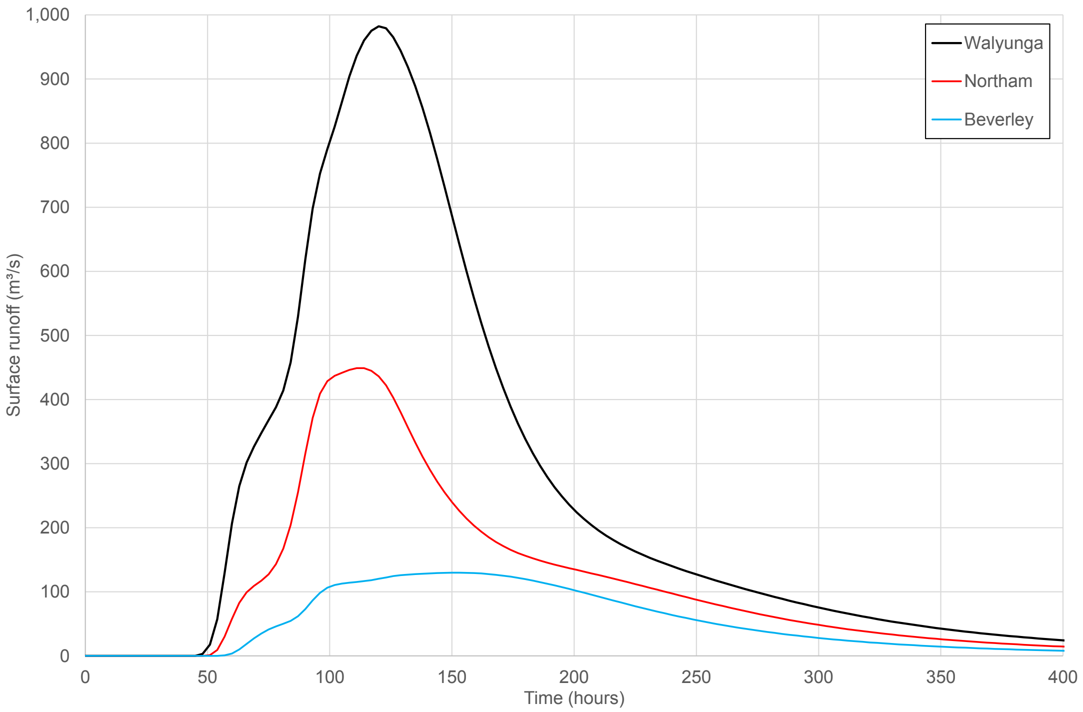
Table 10-1: Design flood peak quantiles at Walyunga

AEP	Peak total flow (m^3/s)	Critical duration (hours)
10%	435	144
5%	635	144
2%	865	96
1%	1,100	96
0.2%	1,900	48
0.05%	3,200	48

Typical hydrographs for the 1% AEP event for the critical duration at Walyunga were also extracted from the lower (120 hour duration) and complete (168 hour duration) catchment RORB models results at key locations. These hydrographs are shown in Figure 10-2 (complete catchment) and Figure 10-3 (lower catchment), and they demonstrate the differing flood generation mechanisms at play. Note that these hydrographs do not include baseflow. The peak at Walyunga in the lower catchment is primarily generated from rainfall towards the downstream (western) end of the catchment, with the hydrograph peak increasing in magnitude from Beverley to Walyunga. This effect is primarily driven by the spatial distribution of rainfall within the design runs, which is in turn based on the spatial distribution of IFD depths (one example of which is shown in Figure 4-1). Conversely, in the complete catchment example, the hydrograph at Walyunga is principally generated from runoff upstream of Qualandary Crossing. While runoff from the lower catchment does act to mitigate the impact of attenuation, by and large the peak flow is generated in the upper catchment.



- **Figure 10-2: Typical hydrographs for 1% AEP flood – complete catchment (note baseflow not included)**



- **Figure 10-3: Typical hydrographs for 1% AEP flood – lower catchment (note baseflow not included)**

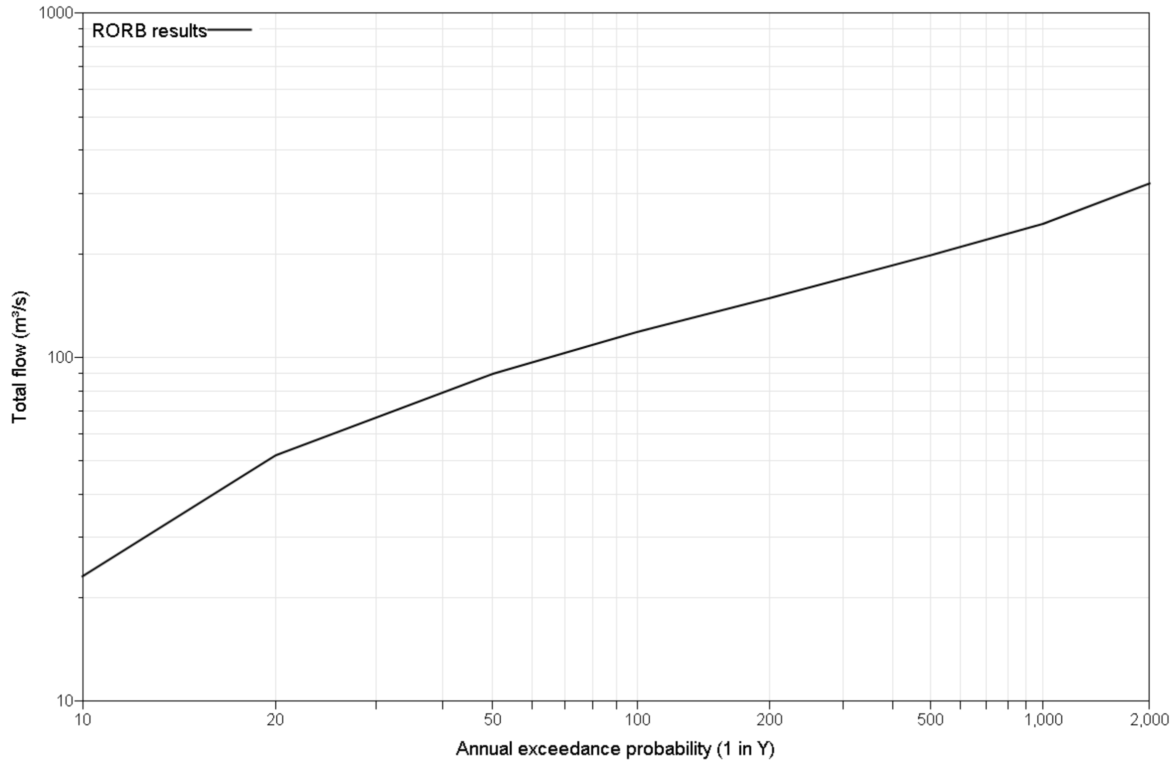
The design flood estimates at Walyunga generated as part of this project were compared to the previous estimates, generated in PWD (1985) and Binnie (1985). It was found that the design flood peak values have decreased significantly from previous estimates. This presumably reflects the impacts of 30 years of additional rainfall and streamflow data, as well as revised techniques and guidance on the estimation of floods. The additional 30 years of streamflow data, in particular, contains few examples of large floods, which has tended to draw the gauged flood frequency estimates down from previous analyses. The previous and current peak flow estimates are shown in Table 10-2.

Table 10-2: Comparison of current and previous design peak flows

AEP	Previous estimate (1985)	Current estimate
10%	642	435
2%	1,320	865
1%	1,700	1,100

10.1.2 Helena River at Whiteman Road

To determine the design flood quantiles and hydrographs at Whiteman Road, the Helena River component of the model was run. The design model is basically the same as the model used for verification, with the addition of a AEP-varying, time-constant baseflow input at Whiteman Road based on the analysis discussed in Section 8.6. The results of the design model runs are shown as a flood frequency curve in Figure 10-4 and tabulated values in Table 10-3.



- **Figure 10-4: Design flood frequency curve at Whiteman Road**
- **Table 10-3: Design flood peak quantiles at Whiteman Road**

AEP	Peak total flow (m^3/s)	Critical duration (hours)
10%	23	168
5%	55	168
2%	89	168
1%	116	168
0.2%	198	168
0.05%	322	48

It was noted that the longest critical duration modelled (168 hours) was assessed as the critical duration for events with AEPs between 5% and 0.2%. Unusually, the 0.05% AEP critical duration then decreases back to 48 hours. This reflects the impact of Mundaring Dam; between the 5% and 0.2% AEPs; the volume of drawdown in the dam requires a very long duration hydrograph to cause it to fill and spill. The variability of peak flow versus storm duration was investigated for these smaller floods, and it was found that between durations of 144 and 168 hours there is minimal increase in the modelled peak flows with duration. It was therefore concluded that modelling of longer and longer storm durations would not significantly increase the peak flow estimates for AEPs between 5% and 0.2%.

10.2 Climate change

The latest available evidence suggests that there will be an increase in design rainfall intensities as a result of climate change. This increase is bounded by the current advice from the Bureau of Meteorology that climate change may not impact PMP estimates.

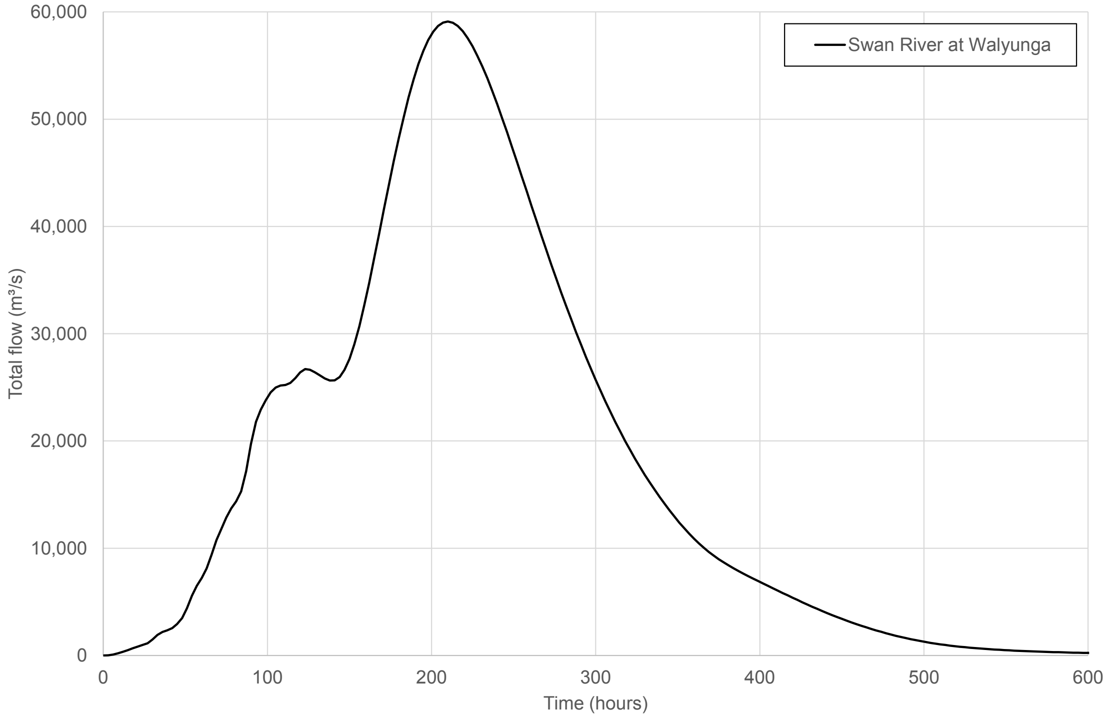
The revised version of ARR (Bates *et al*, 2016) offers preliminary advice on estimation of the increase in design rainfall intensities associated with a range of climate change scenarios. Considering a planning horizon out to 2060, it is more likely than not that design rainfall intensities will increase in the order of 5%. Beyond that horizon is more speculative, however increases of up to 12% may be possible by 2090. This analysis is highly uncertain, and relies (amongst other things) on the downscaling of coarse resolution global climate models to estimate rainfall events which occur over relatively short scales of space and time.

Additionally, climate change is likely to have other indirect impacts on the flood producing factors. These include generally lower average rainfalls, resulting in generally higher loss rates and lower average reservoir and lake levels. Thus, it is by no means certain that increases in design rainfall will *inter alia* result in increases in flood estimates.

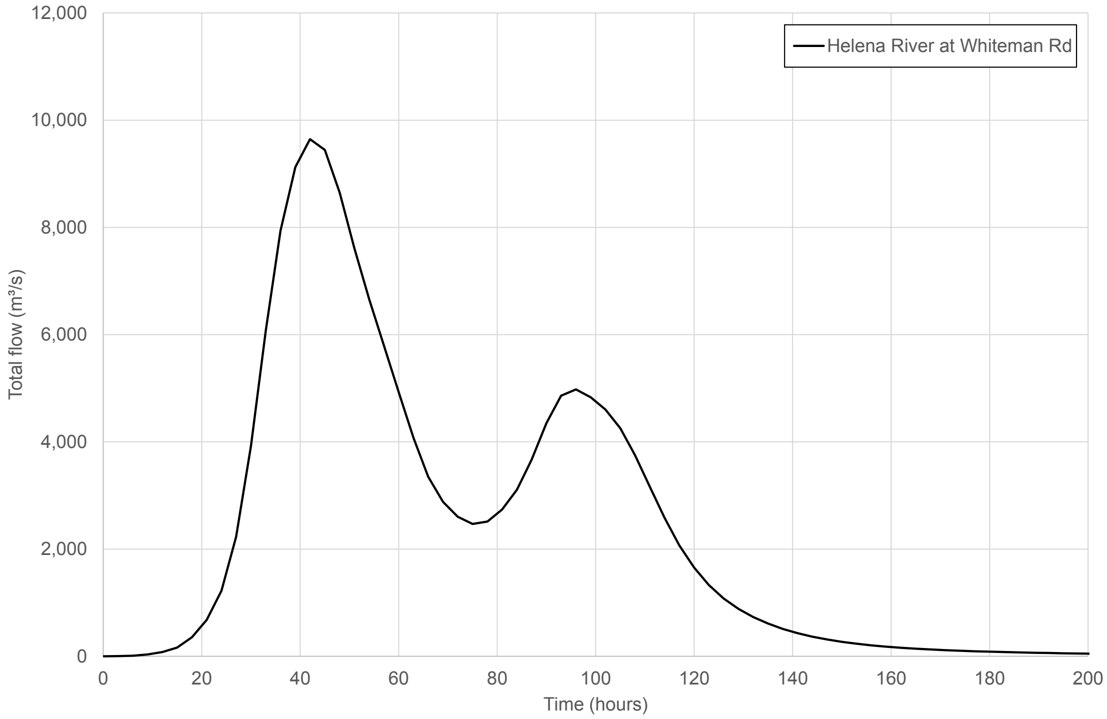
10.3 Probable maximum flood

As well as the design peak flows for AEPs up to 0.05%, an estimate was also made of the probable maximum flood (PMF). To do this, the PMP rainfalls were run in the complete catchment model using a constant initial loss of 0 mm and a runoff coefficient of 0.75 for the Swan Avon River catchment and 0.95 for the Helena River catchment. The value of the runoff coefficient was selected by first modelling the PMF with an initial loss/continuing loss model, with losses set to 0 mm initial loss and 1 mm continuing loss. Then, then runoff coefficient value was selected which approximated the peak flow generated by this model run. This is consistent with the procedures recommended in ARR Book VI (Nathan and Weinmann, 2000). Temporal patterns were sampled from the set used for design, and the temporal pattern which produced the largest peak flow at the point of interest was adopted. These inputs and parameter values are considered consistent with the ARR guidance on estimation of extreme floods, but care should be taken with the interpretation of these results. They are regarded as the maximum upper limit of floods which could, under highly adverse circumstances, be generated from the Swan and Helena Rivers catchments.

The resulting estimates of the PMF peak flow at Walyunga was 59,000 m^3/s, with a critical duration of 120 hours. The estimate of the PMF peak flow at Whiteman Road was 9,650 m^3/s, with a critical duration of 96 hours. The PMF hydrographs for are included as Figure 10-5 and Figure 10-6.

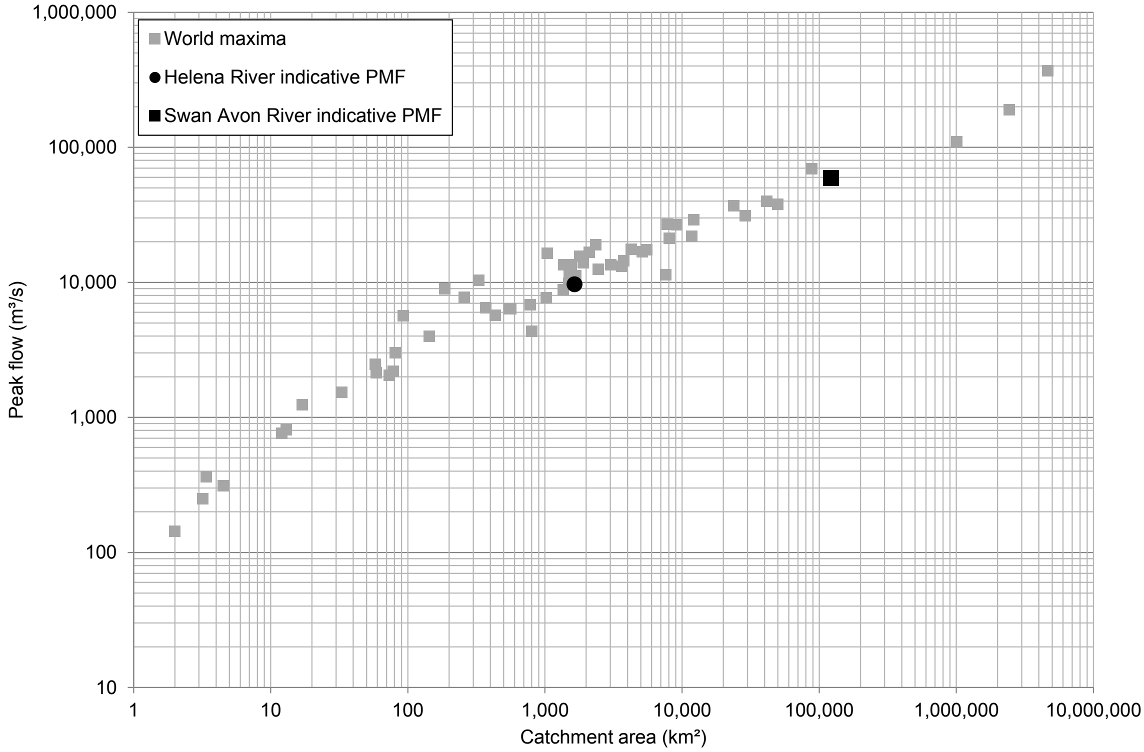


▪ **Figure 10-5: Indicative PMF hydrograph at Walyunga**



▪ **Figure 10-6: Indicative PMF hydrograph at Whiteman Road**

In order to ensure that the PMF estimates were reasonable, they were compared to historic maximum flood estimates from around the world. This comparison, shown in Figure 10-7, indicates that the derived PMF peak flows are comfortably within the range expected based on these worldwide events, some of which have been recorded on catchments larger than the Swan Avon River.



■ **Figure 10-7: Comparison of indicative PMF estimates to world maxima**

11. Conclusion

The primary focus of this project has been the development, calibration and verification of a hydrological model of the Swan Avon and Helena River catchments. The modelling results demonstrate that both selected historic flood events and complete gauged flood frequency relationships can be reproduced using the model with reasonable loss and routing parameter values. Although there is considerable uncertainty surrounding the evaluation of the gauged flood frequency at Walyunga, the model results have demonstrated consistency with the gauged information as well as anecdotal evidence on flood behaviour. The modelling results at both the Walyunga and Craignish gauges are regarded as suitable for use in a future hydraulic modelling study of the Swan and Helena Rivers.

Notwithstanding this, the complexity of the Swan Avon River catchment and current state of flux of guidance in design flood modelling have resulted in a number of uncertainties which could be better understood through additional work. These include issues such as:

- Design rainfall depths. It is understood that at the time of writing, the Bureau of Meteorology are on the verge of releasing revised design rainfall IFD data. Where possible, this information should be incorporated into the verification and design modelling undertaken as part of this work.
- Other design inputs. Similarly, there is significant work currently ongoing as part of the revision of Australian Rainfall and Runoff to estimate pre-burst temporal patterns and rainfall depths, and provide consistent sets of burst temporal patterns across Australia. This information is likely to have a minor impact on the results presented here, but should be incorporated as part of a future revision of this work.
- Explicit modelling of lake systems. A significant outcome of this project was that design floods at Walyunga are significantly influenced by runoff from the Lockhart and Yilgarn Rivers. These two systems are very large and complex catchments, with minimal gauged data records. There are numerous, interconnected lake systems scattered throughout these catchments, with minimal information available on historic variability in water level. Whilst this project has attempted to account for this complexity implicitly, the magnitude of runoff generated from these catchments warrants further attention.
- Improved Monte Carlo sampling of rainfall variability. Whilst some attempt has been made to sample rainfall spatial and temporal variability over the large Lockhart and Yilgarn River catchments, a more detailed approach could be undertaken whereby design 'space-time' patterns of rainfall are generated from the AWAP data and sampled probabilistically. Incorporating these techniques in a future update would improve confidence in the final design results.

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