Ginninderra Creek Flood Study

ROADS ACT, ACT GOVERNMENT

Project Report – 1 in 50 and 1 in 100 AEP flood extents

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Contents

Important note about your report

The sole purpose of this report and the associated services performed by Jacobs is to summarise the findings from the flood hydrology study for the Ginninderra Creek dams, which was completed in accordance with the scope of services set out in the contract between Jacobs and the Australian Capital Territory (ACT). That scope of services was developed with the ACT Government. This document provides a summary of the results of that broader study, presenting the outcomes specifically for the 1 in 50 and 1 in 100 flood events.

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1. Introduction

1.1 Ginninderra Creek Catchment

Ginninderra Creek is located in Canberra, in the north-west direction from the city centre. The catchment covers approximately 32,000 hectares through highly urbanised suburbs such as Gungahlin, Belconnen and Latham, and rural land in the lower reaches. The creek travels west from the extent of the Canberra urban development to cross the border into NSW and flows into the Murrumbidgee River.

Urban development in the catchment has occurred reasonably quickly over the past few decades. Preliminary planning for urban development in the late 1970s already recognised the earlier impact of grazing in the catchment, in particular to the Ginninderra Creek waterway. Increased runoff from the local grazing properties was noted to have caused deep scouring in the Creek. Since then, further changes to the flow regime have occurred due to the rapid urbanisation. This has had significant implications on the flow regime and water quality conditions within Ginninderra Creek.

Three dams were constructed on Ginninderra Creek to provide water quality, flow retarding and flood mitigation control functions, to help manage the change in runoff characteristics from the urban developments in the catchment. These dams, namely Yerrabi, Gungahlin and Ginninderra Dams are situated within the urban area, in the reaches immediately upstream of Belconnen Town Centre.

Future proposed development in the West Belconnen area has heightened the need to accurately estimate the flood extents that result in the Ginninderra Creek catchment as a result of the 1 in 50 and 1in 100 Annual Exceedance Probability (AEP) flood events.

A map of the general study area is shown in Figure 1-1.

1.2 Previous Studies

Flooding and drainage within the Ginninderra Creek catchment have been identified as important issues for decades, with relevant studies undertaken since the early 1990s. Chang and Knee (1991) prepared a flood study for ACTEW that looked at inundation with and without a dambreak at Ginninderra Dam.

Later, flood studies were considered by Ecowise in 1998 and more recently in 2002, which included the application of XP-RAFTS and HEC-RAS models and preparation of flood inundation maps.

Each of the three dams in the catchment, Yerrabi, Gungahlin and Ginninderra, have documented comprehensive surveillance reports and dam safety reports over the dam history. Those surveillance reports that contained information relevant to the current study, such as discharge relationships, were utilised where appropriate.

1.3 This Project

Jacobs has recently completed an update of the hydrology, dambreak analyses and consequence assessment of the three dams along Ginninderra Creek. The focus of the study was the estimation of extreme floods for assessing spillway adequacy for each of the dams. The methods used to develop the hydrologic and hydraulic inputs to the inundation modelling in that project were also used to develop the flood extents for the 1 in 50 and 1 in 100 AEP floods. Full details are reported in the Ginninderra Creek Flooding and Dams Assessment Final Report on Hydrology, Dambreak and Consequence Assessment (Jacobs, 2014).

The current report describes these methods and the outcomes from the hydrologic modelling of the Ginninderra Creek catchment, primarily focussing on the flood extents along Ginninderra Creek produced by the 1 in 100 and 1 in 50 AEP floods.

Figure 1-1 : Map of study area

2. Storage characteristics

Understanding the relationship between storage volume, water level and outflow of the dams with the catchment is critical to routing floods through the storages in an accurate manner. In this section, existing and derived storage-elevation and elevation-discharge relationships of Yerrabi, Gungahlin and Ginninderra Dams are presented.

Some key features of the dams are shown in Table 2-1.

Table 2-1 : Key characteristics of Ginninderra Creek dams

Dam	Dam Type	Full Supply Level (FSL)	Primary Spillway Level (mAHD)	Secondary Spillway Level (mAHD)	Embankment Crest Level (mAHD)	Construction Date
Yerrabi	Zoned, Earth-fill, clay core	612.15	612.15	614.00	618.20	1993-1994
Gungahlin	Zoned, Earth-fill	596.20	596.20	598.00	604.00	1992
Ginninderra	Earth-fill	577.27	577.27	582.83 (embankment crest)	582.83 (embankment crest) 583.83 (parapet wall)	1975

2.1 Storage-elevation relationships

2.1.1 Yerrabi

The elevation-storage relationship for Yerrabi Dam was sourced from Ecowise & Bill Guy and Partners (2002).

The adopted storage-elevation relationship for Yerrabi dam is shown in Figure 2-1. Full Supply Level (FSL) is at 612.15 mAHD.

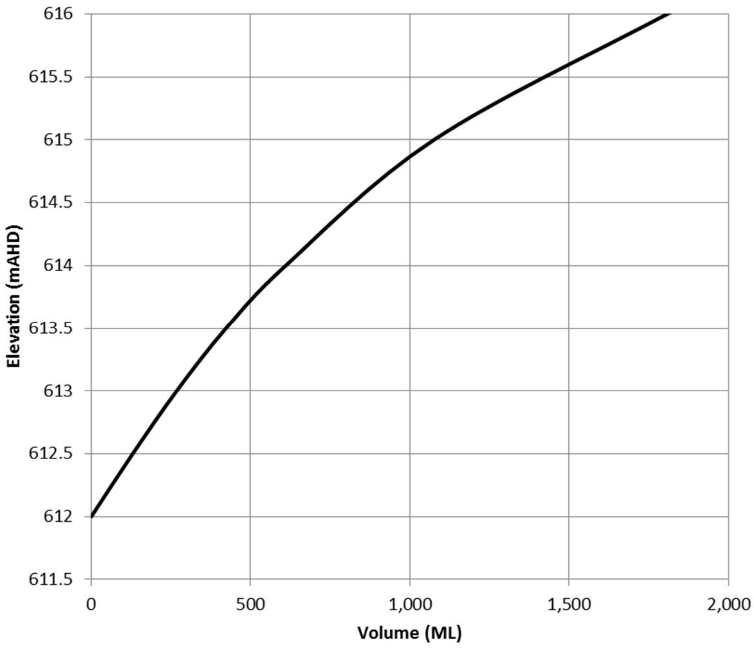


Figure 2-1 : Storage-elevation relationship for Yerrabi Dam with key elevations

2.1.2 Gungahlin

Elevation – volume relationships for Gungahlin Dam were identified in the following references:

- Ecowise & Bill Guy and Partners (2002) Coordination of Lake Ginninderra DAM PMF Protection with Gungahlin Flood Retardation Planning. (page 3 of Appendix C)
- ActewAGL (2010) Gungahlin Dam Comprehensive Surveillance Report (page 13)

These relationships are presented in Figure 2-2, and confirm that there is general consistency between these two sources. The actual data used was from the ActewAGL report. FSL is at 596.2 mAHD

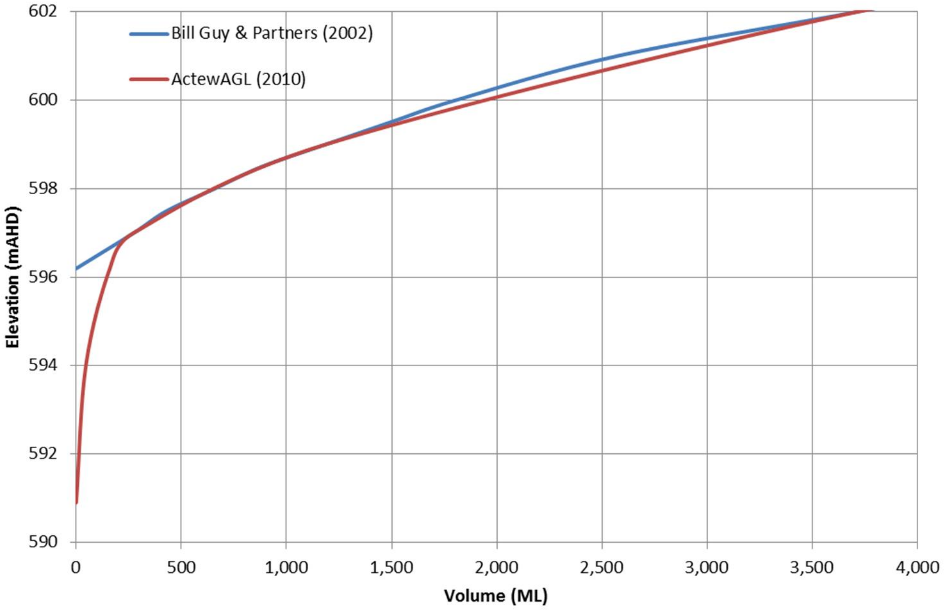


Figure 2-2 : Elevation-volume relationship for Gungahlin Dam with key elevations

2.1.3 Ginninderra

Elevation – volume relationship data for Ginninderra Dam was identified within two reports:

- Ecowise & Bill Guy and Partners (2002) Coordination of Lake Ginninderra Dam PMF Protection with Gungahlin Flood Retardation Planning. (Table of data provided on page 1 of Appendix C)
- ActewAGL (2006) Ginninderra Dam Comprehensive Surveillance Report (page 4).

These two data sets are presented in Figure 2-3, with the ActewAGL data digitised from the chart presented in that report. While it appears that these two data sets differ at the lower end of the relationship, it should be noted that the data table in the Ecowise and Guy & Partners (2002) study provides no data points for elevations between 565.4 mAHD and 577.27 mAHD. This means that there is no curvature in the relationship at lower storage levels and the differences in the two sets of data are due to the limited representation of points in the Ecowise report. It was considered likely that these two data sets represent a consistent relationship.

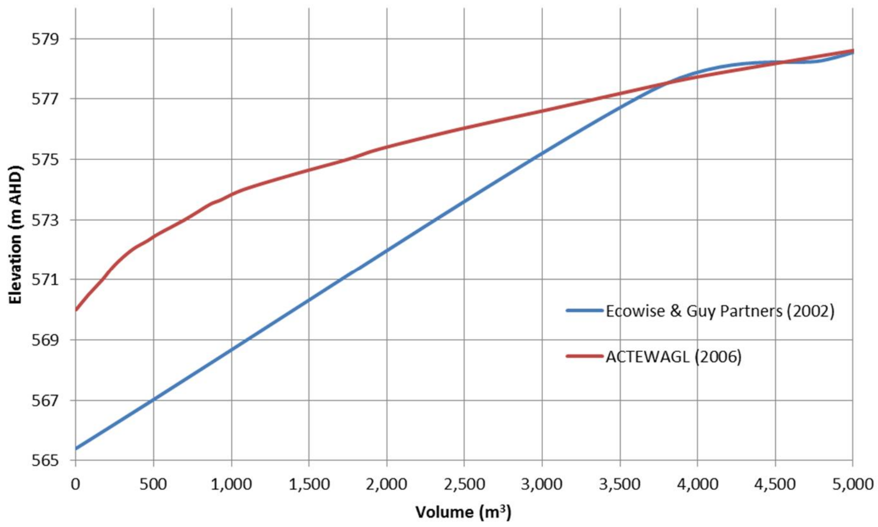


Figure 2-3 : Elevation-volume relationship sourced for Ginninderra Dam

The data from the ActewAGL report was used in the modelling within this report. FSL for Ginninderra Dam is 577.27m.

2.2 Discharge relationships

2.2.1 Yerrabi Dam

Elevation-discharge relationships for the Yerrabi Dam were identified in the following references:

- Willing and Partners (1992) Yerrabi Pond Water quality control pond and associated works. Final Sketch Plan Report (page 14 and Figure 5).
- Ecowise (1999) Dam Failure and Flood Inundation Studies for Ginninderra Creek (page 7)

Figure 2-4 presents these relationships, and confirms that they are generally consistent. The Willing and Partners (1992) report provides detail of the primary and secondary spillways under culvert inlet control and under weir control, however only the combined spillway discharge relationship is shown.

The Ecowise (1999) data was used for the modelling within this report and assumes that the culverts are unblocked.

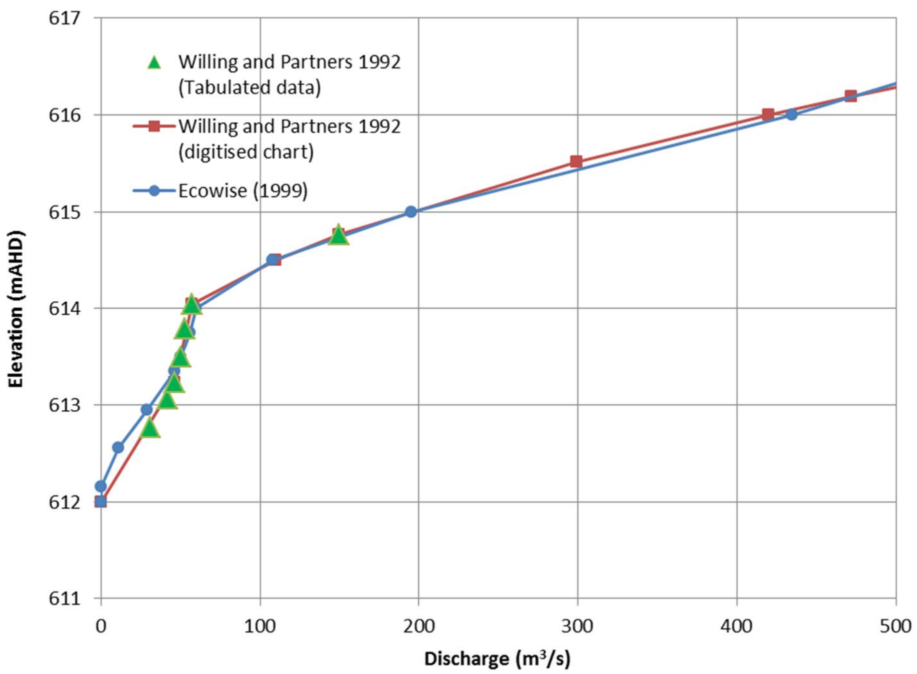


Figure 2-4 : Elevation-discharge relationship for Yerrabi (excluding flows over dam embankment)

2.2.2 Gungahlin Dam

A number of spillway discharge relationships for Gungahlin Dam were found in previous reports. The most recent of these include:

- Ecowise (1999) Dam Failure and Flood Inundation Studies for Ginninderra Creek (page 8) , adopted from Chang *et al* (1993)
- ActewAGL (2010) Gungahlin Dam Comprehensive Surveillance Report (page 14)

The data available in these reports has been reviewed and are consistent over the range of data provided. A number of older data sets (Scott and Furphy, 1990 and ACT Electricity & Water, 1999) are also available, but each of these provide a different relationship. Given these older studies were prepared at early design and pre-construction stages of the dam development, they may not fully represent the final as-constructed spillway explaining the differences with the more recent data. As such, they are not relevant for the current hydrology assessment.

Figure 2-5 compares the data presented in the Ecowise (1999) and ActewAGL (2010) studies.

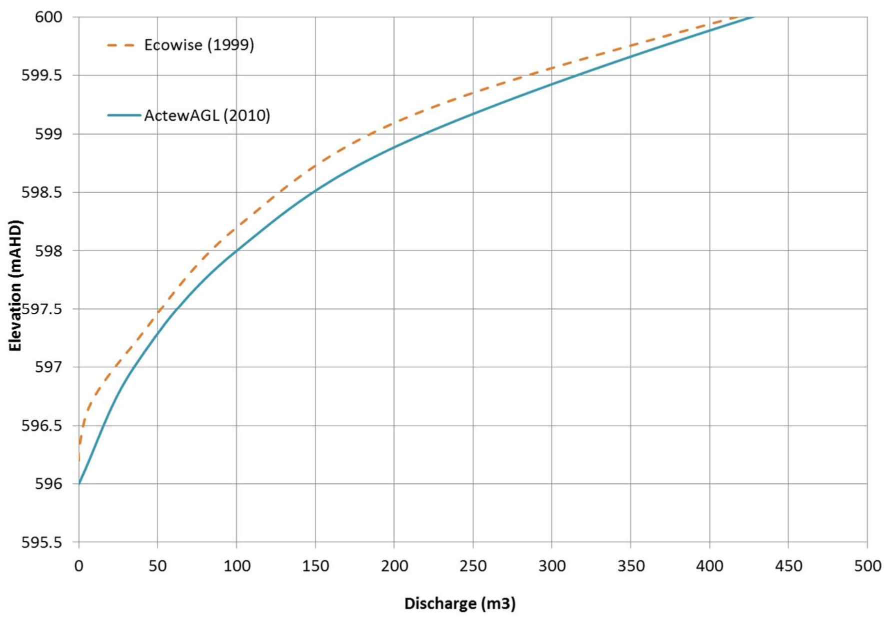


Figure 2-5 : Previous elevation-discharge relationship for Gungahlin Dam

The detail surrounding the derivation of the above elevation-discharge relationships for Gungahlin was unclear in the previous reports studied. This is particularly important for Gungahlin Dam, where the secondary spillway (and potentially an informal tertiary spillway) is established through the local topographic conditions rather than through physical structures. Chang *et al* (1993) is referenced by Ecowise (1999) as the source of their spillway relationship. Jacobs has not obtained this document and as such, the calculation approach for the available spillway discharge relationships is unknown. It is unclear as to the details of this original document, as full references are not provided in the Ecowise (1999) reference list. It is possible that it should refer to a Chang / ACT Electricity and Water (1990) report *(Flood Inundation Study resulting from Ginninderra Pond no 1 dambreak*) which does include a design stage discharge curve. However this curve differs from that presented in Ecowise (1999) and the basis of the relationship is not described.

Given these uncertainties it was decided that Jacobs would derive a secondary spillway relationship based on HEC-RAS modelling in order to explicitly incorporate the varying topography into the discharge relationship.

This activity incorporated a number of key tasks, specifically:

- Model development. As the spillway channel is fairly linear and lends itself well to a 1D approach, HEC-RAS was used to model the spillway dynamics. LiDAR data was used to extract cross sections of the spillway and channel downstream (Figure 2-6).
- Other geometric features of the spillway and different model parameters, boundary conditions, and other details such as Manning's coefficients were required to be incorporated into the model also.
- Run the HEC-RAS model and export results.

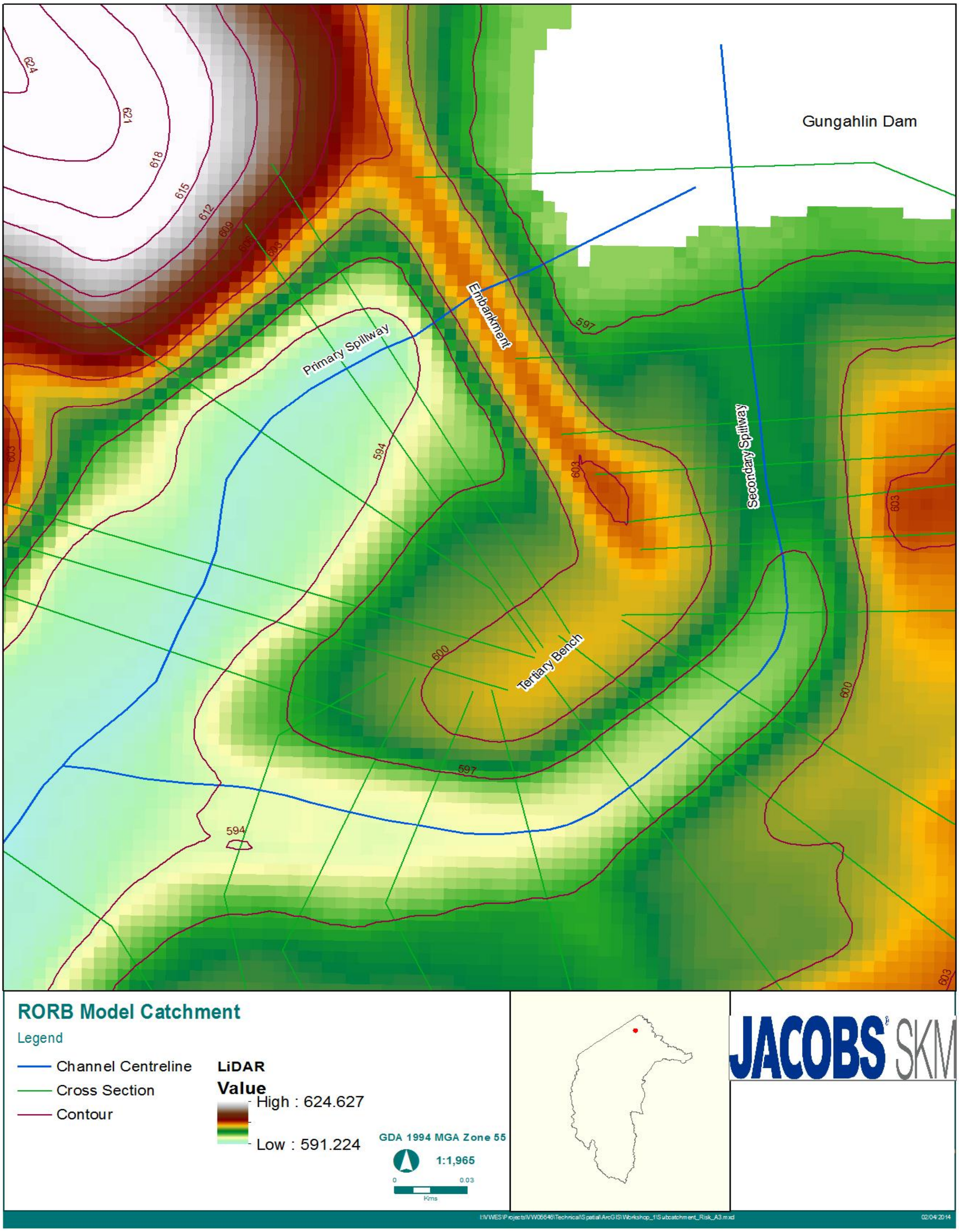


Figure 2-6 : Gungahlin Dam spillway representation with cross sections and centreline used for HEC-RAS model

2.2.3 Gungahlin Spillway Modelling – Model Development

LiDAR data was used to extract cross section and centreline information for the Gunghalin Dam spillways and was used as input into HEC-RAS. There were two centrelines; one that represents the main channel of Ginninderra Creek from the primary spillway (the 'primary channel'), the other that represents the channel beginning at the secondary spillway (the 'secondary channel'). These two centrelines are connected at the confluence where the secondary channel meets the primary channel and continue downstream.

Cross sections were drawn with the intent of capturing any important hydraulic structures (constrictions, sharp bends etc.). Cross sections were extended out away from the centreline usually to at an elevation equal to at least the elevation of the embankment (604mAHD). There were instances, particularly along the eastern side of the secondary channel, where this was not practically possible due to the topography of the surrounding land. Where this occurred, the cross sections were extended out as far as was practical. Elevation data at each cross section was then extracted using a GIS and imported to HEC-RAS. An example of one of the extracted cross section plots is shown in Figure 2-7.

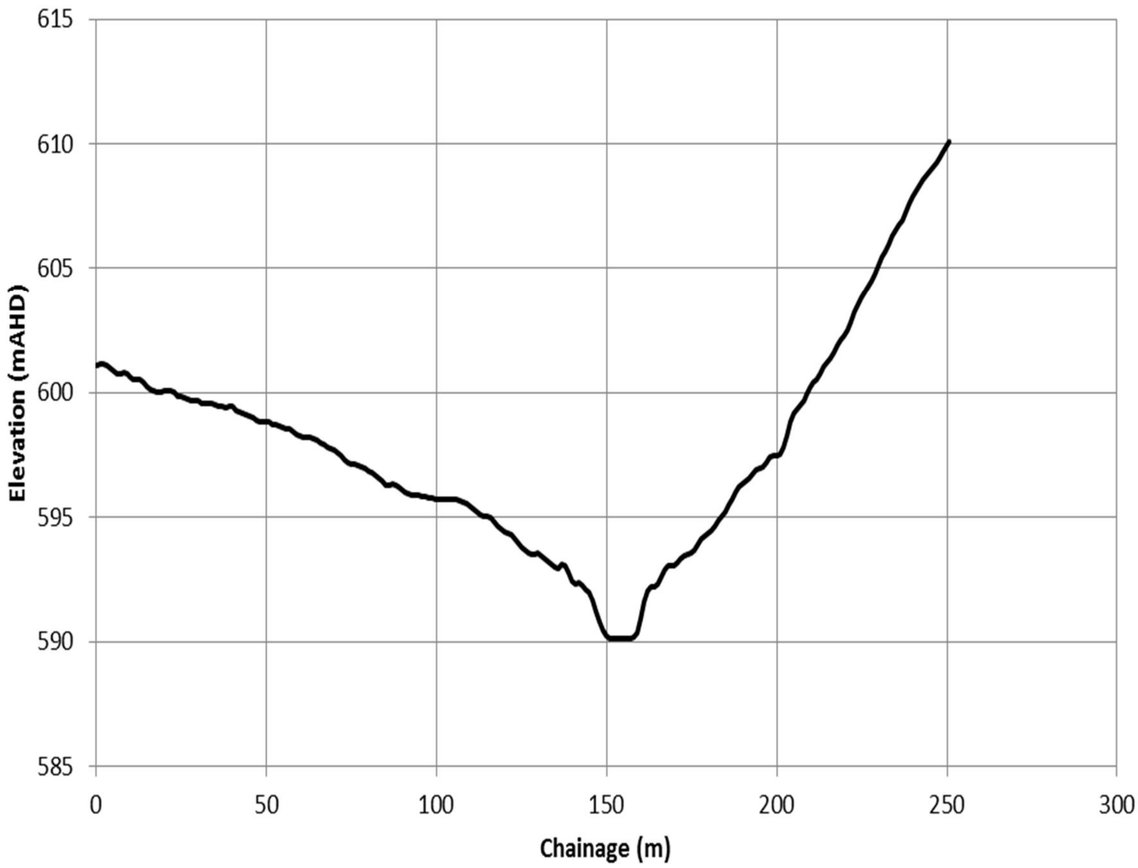


Figure 2-7 : Example of cross section extracted using LiDAR data

2.2.4 Gungahlin Spillway Modelling - Other Geometric Features and Model Parameters

Manning's roughness coefficients for the channel and banks at each reach were estimated using aerial imagery of the channels. The values used were adopted from Table 3.1 of the Hydraulic Reference Manual (US Army Corps of Engineers, 2010). Table 2-2 shows the Manning's values used in the main channels. A value of 0.1 was used on the left (east) overbank of the secondary channel due to the presence of housing along that side of the channel. In all other regions, the same Manning's coefficients were used on the overbank as in the channel.

Table 2-2 : Manning's Coefficients used in both channels

Channel	Manning's Coefficient
Primary	0.045
Secondary	0.03-0.04

The secondary spillway of Gungahlin dam is a grassed floodway that extends from the reservoir over a ridge and down into the secondary channel. This ridge was modelled in HEC-RAS as an inline (broad-crested) weir with a weir coefficient of 1.4.

The bench on the right (west) overbank of the secondary channel that helps define the upstream end of the channel and separates it from the primary channel, was modelled as a lateral (broad-crested) weir. The weir coefficient used was also 1.4. Additional elevation profile information from the LiDAR data in the GIS was used to define the physical characteristics and positioning of the bench within the HEC-RAS model.

A normal depth relationship was used to define the downstream boundary condition of the model.

2.2.5 Gungahlin Spillway Modelling – Run Model and Export Results

With all the geometric features, initial input flows, model parameters and boundary conditions in place, the model was run with a suite of input flows into the secondary channel. The flows modelled in the secondary channel ranged from 5 m^3/s to 6,000 m^3/s. The discharge at the confluence was simply the addition of the discharges from the primary and secondary channels.

The suite of input flows allowed a range of longitudinal water surface profile plots to be produced. An example of two of these plots along the secondary channel is shown below in Figure 2-8.

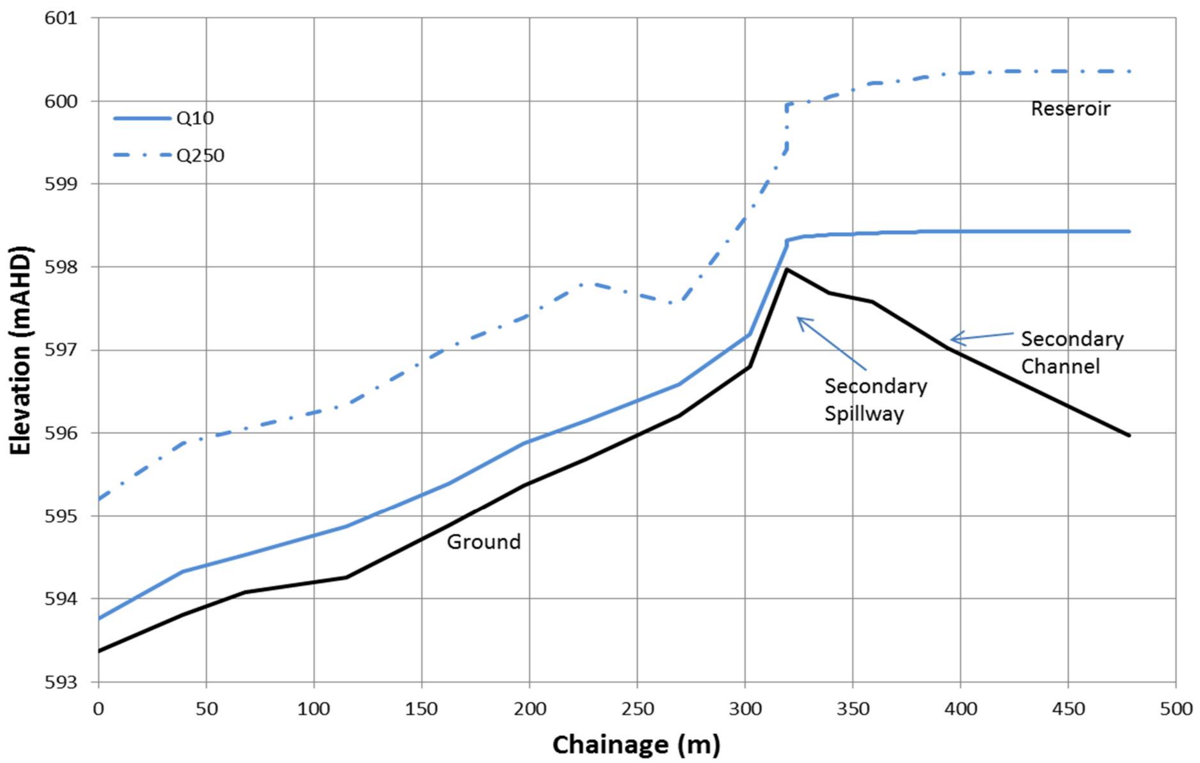


Figure 2-8 : Profile plots for secondary channel for input flows of 10 and 250 m^3/s

The resultant flows at the most upstream reach of the secondary channel (that contains the minimum energy grade slope) were extracted and plotted against energy grade elevation. This plot is shown in Figure 2-9.

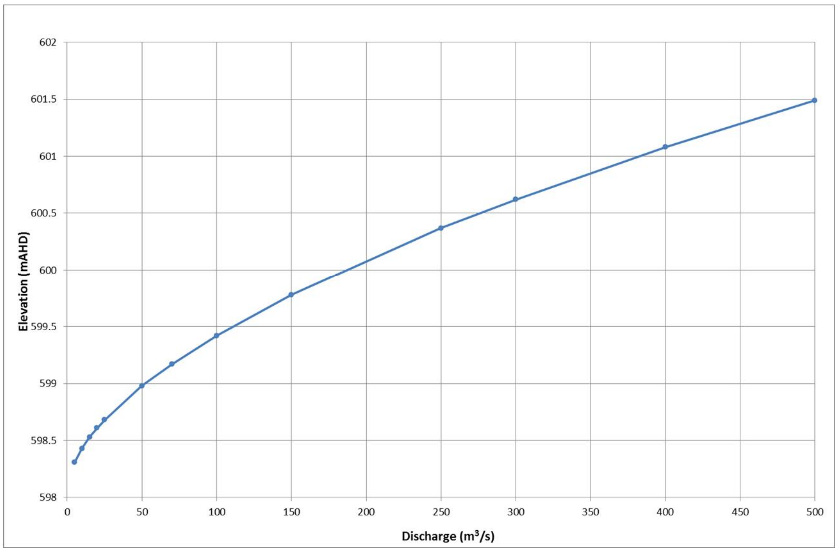


Figure 2-9 : Secondary Spillway Elevation-Discharge Relationship for Gungahlin Dam from HEC-RAS model

The total spillway relationship at Gungahlin Dam is comprised of the primary spillway (elevations up to 598.20m), the secondary spillway (elevations from 598 to 604m) and the embankment at elevations greater than 604m.

The relationship from (Ecowise, 1999) for the primary spillway, for the secondary spillway calculated by this current work along with the broad-crested weir calculations for flow over the embankment were summed at the appropriate elevations to arrive at a total elevation-discharge relationship for Gungahlin Dam spillway. This relationship is shown in Figure 2-10. The previous discharge relationship from ActewAGL (2010) is also shown for comparison.

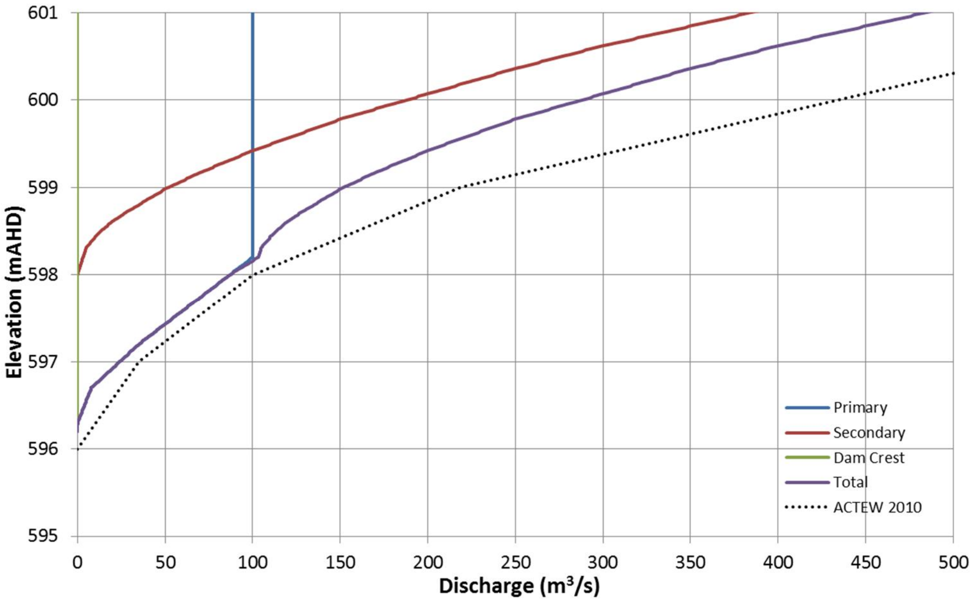


Figure 2-10 : Adopted elevation-discharge relationship for Gungahlin Dam (with ACTEW 2010 relationship shown for comparison)

2.2.6 Ginninderra Dam

Following a review of the documentation provided for this study, the elevation discharge relationship for Ginninderra Dam was identified in the following reference:

- Bill Guy and Partners (2003) Ginninderra Dam Augmentation Works (Appendix B: extract from PSP Design Report)
- This provides a detailed breakdown of the discharge relationship for the flow over the spillway, flow through the culverts and flow over the dam embankment. This relationship was comparable with that presented in Figure 2 in ActewAGL (2006). ActewAGL (2006) also provides data for the spillway discharge (Table 4 in that study)

Bill Guy and Partners (2003) also includes discussion of the culvert component of the above discharge relationship. They incorporate a number of alternative culvert discharge relationships, based on assumptions regarding the inlet and outlet control conditions. Specifically, since the spillway is on a steep slope, if flow submerges then they consider that it may be possible to change from an ogee weir control to inlet control condition. In this case, there would be a reduction in capacity of the spillway. Calculation of the spillway for culvert inlet control identified a reduction in spillway capacity of approximately 20%.

Guy and Partners (2003) considers it likely that there will be orifice control at the ogee weirs for the high surcharge/submergence flows. They therefore revisited the large culvert orifice calculation and provide a revised relationship for this component. This change in control is observed as a change in slope in the primary spillway relationship. However, the data presented by Guy and Partners (2003) includes an unexpected reduction in capacity above the embankment level.

Guy and Partners (2003) also recalculate the discharge over the dam embankment with the addition of the 1m parapet embankment and the installation of levees on the west side of the dam to protect Florey. Their overall recommended spillway relationship is presented in Figure 2-11. They do not provide any commentary on how this relationship was developed.

It is noted that this relationship makes no mention of the secondary spillway, although it is possible that this is represented in the relationship over the embankment. The NSW Public Works (2013) Lake Ginninderra Review of Crest Wall Operation provides an indicative discharge through the secondary spillway for the dam crest flood, but details on the discharge relationship are not provided. From that study, it is understood that overtopping of up to 0.71m is expected for the dam crest flood across a 73.5m length of the left hand side of the embankment.

Although there was some uncertainty in the derivation of the above total discharge-elevation relationship, it was considered acceptable for use.

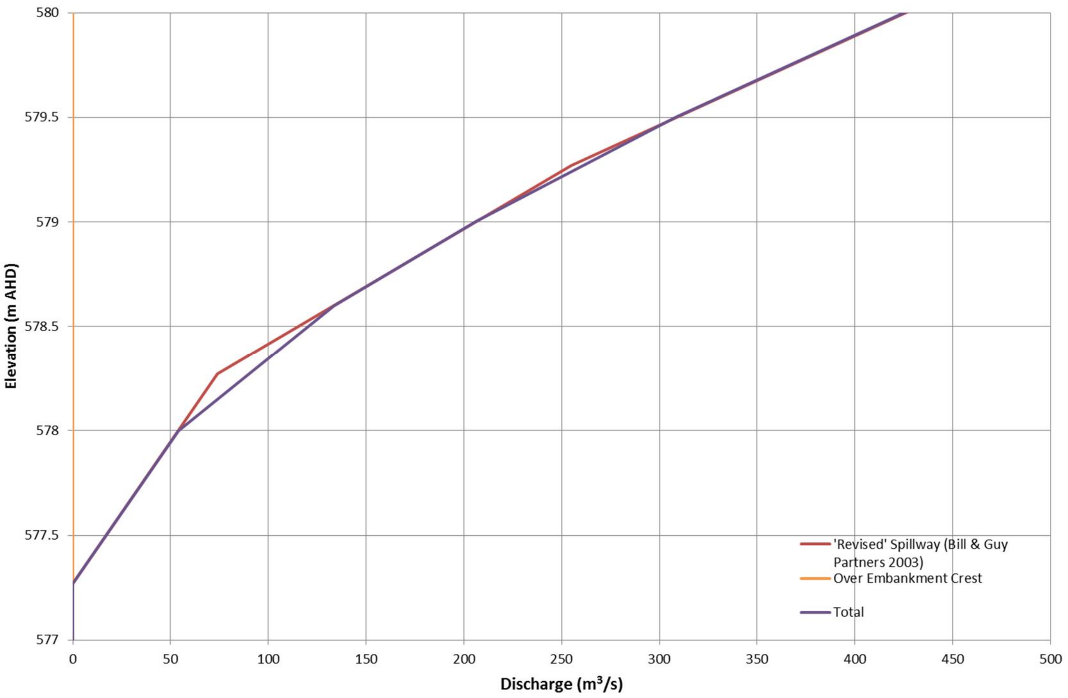


Figure 2-11 : Preliminary Ginninderra Elevation – Discharge relationship

3. Hydrologic Modelling

3.1 Approach

3.1.1 RORB Runoff Routing Model

RORB (Laurenson and Mein, 1995) is a general runoff and streamflow routing program that is used to calculate flood hydrographs from rainfall and other channel inputs. It subtracts losses from rainfall to determine rainfall excess and routes this through catchment storages to produce streamflow hydrographs at points of interest. The model is spatially distributed, non-linear, and applicable to both rural and urban catchments. It makes provision for both temporal and areal distribution of rainfall as well as losses, and can model flows at any number of points throughout a catchment (including upstream and downstream of reservoirs and retarding basins). RORB also has the capacity to use a Monte-Carlo approach to produce design flood estimates that incorporate the joint probability of several flood causing factors.

RORB models are based on catchment geometry and topographic data, and the two principal parameters are k_c and m. The parameter m describes the degree of non-linearity of the catchment's response to rainfall excess, while the parameter k_c describes the delay in the catchment's response to rainfall excess. An m value of 0.8 was adopted for this study. The remaining RORB model parameters relate to the representation of the rainfall losses. The RORB model can represent those losses either by an initial loss/continuing loss model, or by the initial loss/proportional loss (i.e. runoff coefficient) model. An initial loss/continuing loss model was adopted for this study.

3.1.2 Event based and Monte-Carlo approaches

Traditional practice for estimating design floods has typically been the "design event" approach, in which all model inputs other than rainfall are input as fixed, single values. This concept is illustrated in Figure 3-1 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

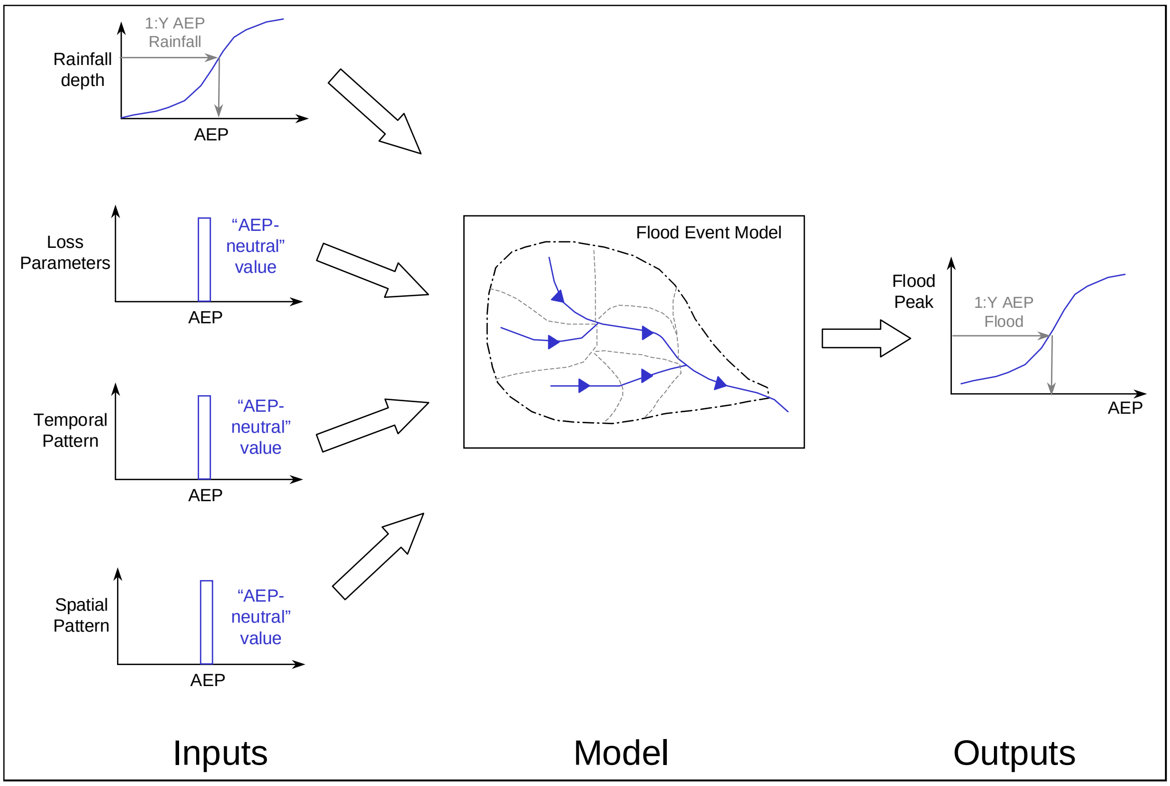


Figure 3-1 : Schematic illustration of the design event approach

Joint probability techniques offer an alternative to the 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 basin, or a large storm on a dry basin. 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 natural processes 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 3-2 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 following sections of outline the overall joint probability framework adopted in this study, and the evidence used to characterise the distribution of the inputs.

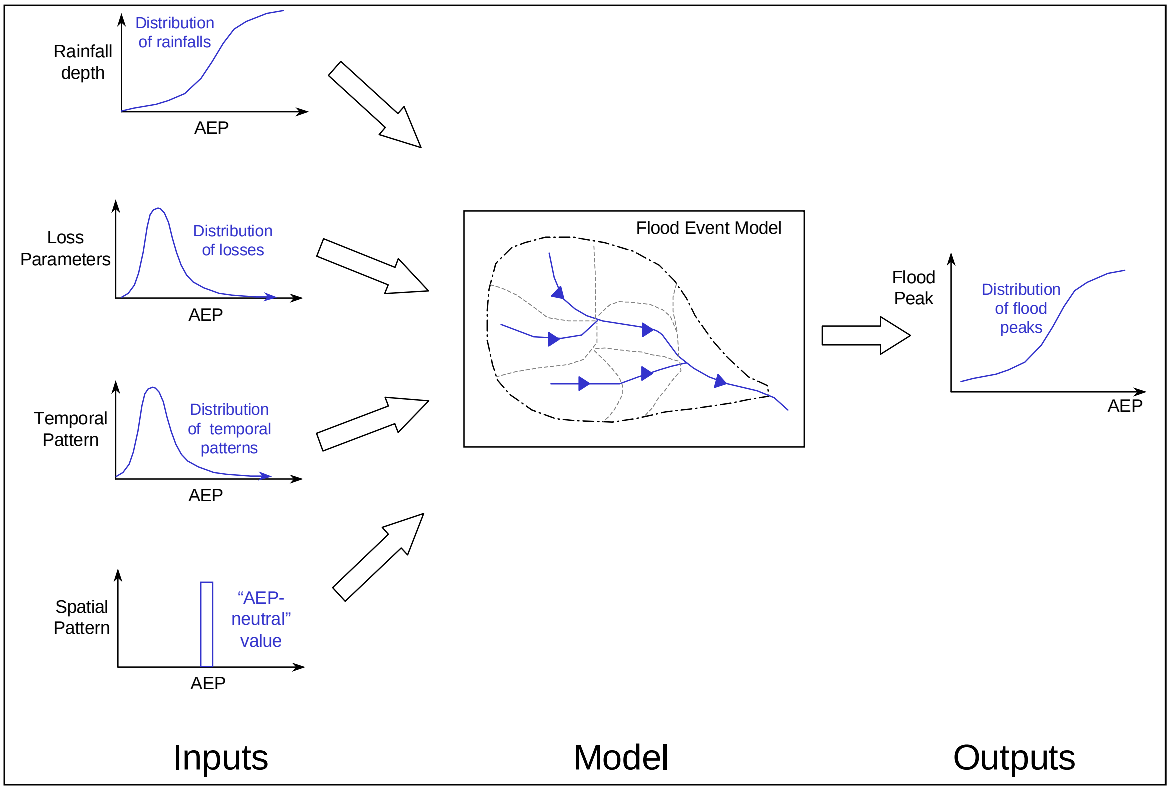


Figure 3-2 : Schematic illustration of the joint probability approach

3.1.3 Overview of adopted joint probability framework

In essence, the joint probability approach to design flood estimation involves undertaking numerous model simulations where the model inputs are varied in accordance with that observed in nature (Figure 3-3) (Nathan *et al*, 2002; 2003). The inputs are sampled from non-parametric distributions that are either based on readily available design information or the results of recent research.

In developing the joint probability framework for Ginninderra Creek catchment, particular attention was given to ensuring that the nature of the inputs and the manner in which they are incorporated are consistent with the philosophy detailed in Book VI of ARR. The following paragraphs briefly describe the main elements of the approach, and the manner in which they relate to established design information.

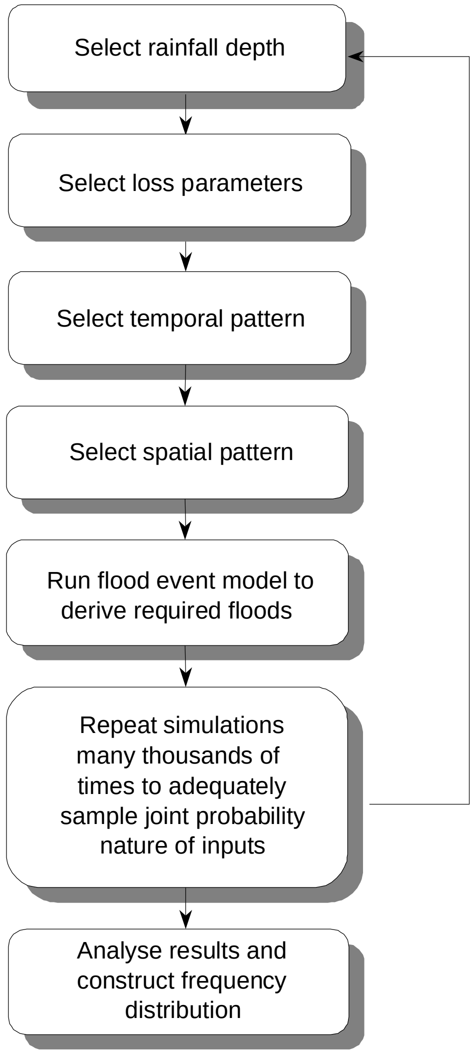


Figure 3-3 : Overview of adopted joint probability framework

a) *Select rainfall depth.* Rainfall depths are stochastically sampled from the cumulative distribution of rainfall depths. The relationship between burst depth and annual exceedance probabilities is based directly on Book VI, though additional information is obtained from *Australian Rainfall and Runoff* procedures to derive rainfalls down to AEPs as frequent as the 1 in 10 AEP event. In addition, approximate values for rainfalls more extreme than the PMP are derived by simple linear extrapolation in the logarithmic – normal probability domain. These extrapolated rainfalls represent burst depths to AEPs approximately one order of magnitude less frequent than that of the PMP, though these contribute little to the final results. Derivation of the design rainfall depths is discussed in more detail in Section 5.1.

b) *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 observations of large historic storms. For durations greater than 6 hours, the temporal patterns are from the same database used to construct the temporal patterns used in the design event approach. This is discussed in more detail in Section 5.3.

c) *Select storage drawdown.* Airspace in a storage can play an important role in reducing the peak flows downstream of the dam. Initial available volumes were set as zero for all model runs as the dams are usually existing at full supply level. Refer to Section 5.5 for further details.

d) *Select loss parameters:* Storm initial and continuing losses are stochastically sampled from a non-parametric distribution that was determined from the analysis of a large number of Victorian and ACT catchments (Hill *et al*, 1997). There is little information regarding the correlation between initial and continuing loss rates. Since antecedent conditions mostly influence initial loss rates, the continuing loss rates were held constant in this particular application. This is discussed in more detail in Section 5.6.

e) *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 in 10^6 annual probability of exceedance, a reduced number of simulations are undertaken over a specified number of probability intervals. The rainfall frequency curve was divided into 50 intervals uniformly spaced over the standardised normal probability domain, and at least 100 simulations were taken within each division.

f) *Spatial Patterns* were varied with burst rainfall duration so that they remained consistent with the approach used to derive the PMP rainfall depth. The design spatial patterns for the short and long duration events were based on the methods as outlined in the GSDM (BoM, 2003) and GSAM (BoM, 2006) respectively. A different spatial pattern, with the design storm centred on the relevant dam, was created for each of the three dams. Further detail is discussed in Section 5.3.

4. Model Structure

4.1 Catchment delineation

The Ginninderra Creek catchment, with catchment outlet at the Ginninderra Falls, was delineated based upon use of two digital terrain models (DTM). Higher resolution terrain data was provided by ACT as well as LiDAR data, provided by Canberra Airbourne Laser Scanning. A relatively lower resolution national DTM made available by Geosciences Australia was also utilised for areas within the Ginninderra Creek catchment that the high resolution data did not extend. The catchment area of Ginninderra Creek to the Ginninderra Falls is approximately 220 km^2.

This catchment was further divided into subareas based on drainage characteristics and to provide for areal variation of rainfall and losses. The catchment area upstream of Ginninderra, Gungahlin and Yerrabi Dams are 98 km^2, 49 km^2 and 19 km^2 respectively.

A series of nodes and reaches were also developed to represent the routing characteristics of the catchment.

Delineation of the RORB sub areas in the catchment files developed for this study is shown in Figure 4-1.

4.2 Model Calibration

The calibration process involves determining the combination of initial loss, continuing loss and k_c parameters that produce the best fit to observed streamflow hydrographs for chosen calibration events. When fitting the modelled hydrograph to the observed hydrograph a compromise is sometimes required between the reproduction of various attributes of the hydrograph. Greatest priority was given to matching the hydrograph peak, followed by the overall hydrograph shape. Calibration was undertaken for the Ginninderra Creek RORB model at the two streamflow gauges, and also (where available) the outflow locations at each dam in order to choose a k_c value for design.

The k_c parameters from calibration have also been compared to a regional equation to demonstrate that alternative estimates that would have been adopted if calibration data had not been available and to test the reasonableness of the adopted k_c parameters. See Section 4.2.7.

Calibrating the model allows information about the routing processes present within the catchment to be obtained. The calibrated loss values will be dependent on the antecedent conditions of each different calibration event and therefore may differ between different calibrations. These differences will be overcome during the verification process, which utilises the full suite of streamflow events over the entire period of record to determine loss values, rather than the three calibration events used during calibration.

4.2.1 Rainfall Data

Recorded rainfall data is required to estimate the spatial distribution and temporal distribution for each of the flood events selected for calibration. The rainfall gauges and pluviographs in the vicinity of the Ginninderra Creek catchment are shown in Figure 4-2. Where data was available for a given calibration event, these rainfall gauges were used to estimate the spatial distribution. Pluviographs were used to estimate the temporal pattern.

The spatial distribution for each event was derived using data from the daily rainfall and pluviograph stations in the vicinity of the Ginninderra Creek catchment. There was a reasonable coverage of daily rainfall gauges throughout most of the catchment, however the northernmost parts of the catchment were relatively sparsely populated. Rainfall gauges and pluviographs used to inform the spatial distribution of each event is shown in Appendix A.

To define the spatial distribution of rainfall for each event, isohyetal patterns were derived using the total rainfall depth from the daily rainfall and pluviograph stations over each event period (event periods shown in Table 4-1). Rainfall depths at the centroid of each RORB sub-area were then interpolated.

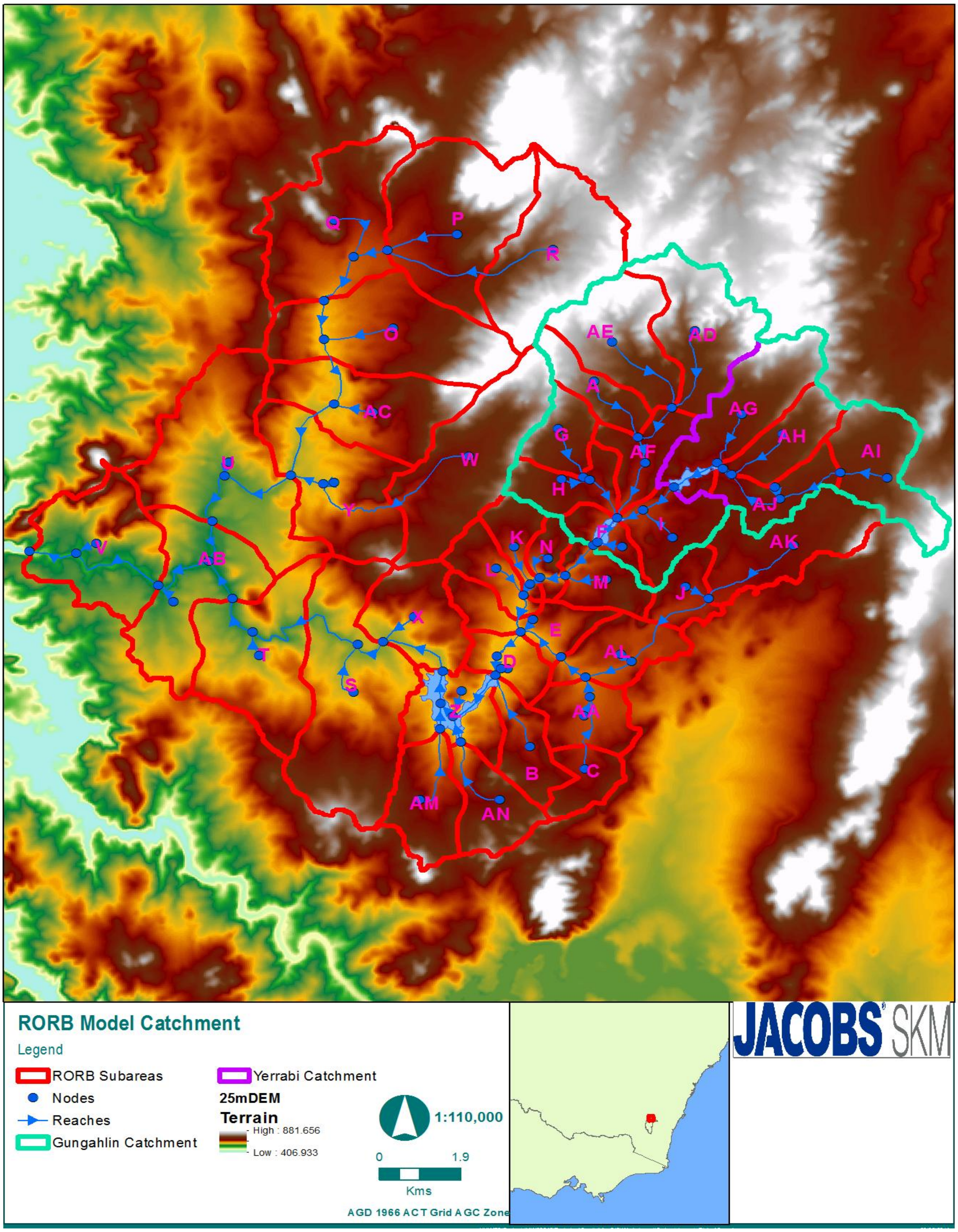


Figure 4-1 : RORB model layout for Ginninderra Creek catchment

4.2.2 Streamflow Data

The RORB model calibration was performed using flow records for three flood events (Table 4-1) at gauges 410750 (Ginninderra Ck U/S Charnwood Rd) and 410751 (Ginninderra Ck U/S Barton Hwy). The three events used for calibration were chosen as March 1989, January 1995 and March 2012. Continuously recorded reservoir water level data for each of the dams for the 2012 event was converted to a discharge and also used as calibration points. No reservoir water level data exists for Yerrabi Dam for the 1995 event, therefore only Ginninderra and Gungahlin outflows could be used for this event. Only the two gauges and the converted outflows from Ginninderra Dam were used for calibration of the 1989 event. Figure 4-2 displays the location of the two gauges used in this study.

The largest three flood events common to the two gauges were chosen as calibration events. The hydrographs of the three events were then plotted to ensure there were no unusual shapes encountered that would have rendered calibration difficult.

Table 4-1 : Event periods for three calibration events

Event	Date Range
1989	10/03/1989 9:00AM – 18/03/1989 9:00AM
1995	19/01/1995 9:00AM – 22/01/1995 9:00AM
2012	26/02/2012 9:00AM – 8/03/2012 9:00 AM

RORBWin was used to calibrate model parameters to these flow events at the points within the catchment mentioned above.

4.2.3 Impervious Areas

Ginninderra Creek catchment (upstream of Ginninderra Falls) has an area of approximately 220km^2. It contains a mixture of highly urbanised areas and relatively rural areas, with little to no development.

Current level of development impervious areas were estimated from aerial photography captured in 2012. The typical impervious fractions adopted were inferred by measuring the impervious areas of typical blocks from the aerial photography. Current levels of impervious development were used for the calibration of the RORB model for the most recent events.

The Effective Impervious Area (EIA) considers that portions of the Total Impervious Area (TIA) in an urban area will not be directly connected to the drainage network and therefore will not necessarily contribute to runoff. The EIA, which represents the portion of the TIA which will contribute to runoff, was investigated in ARR Project 6 Stage 2 – Losses for Design Flood Estimation (Cardno, 2013). This study estimated EIA/TIA proportions for a number of catchments around Australia. An EIA of 74% was used in all subareas.

The Ginninderra Creek catchment has undergone significant developmental changes between 1989 and the present. To represent this change, all subareas upstream of Gungahlin Dam were considered to have an impervious fraction of 0 (or close to 0) for the 1989 calibration event. In the absence of data, all subareas downstream of Gungahlin were considered to have not dramatically changed in terms of impervious fraction between 1989 and 2012. While there remains some uncertainty regarding how the effective impervious area has changed over time, the excellent calibration results achieved for all three calibration events afforded an acceptable degree of confidence to these effective impervious fraction figure used. The adopted total impervious area fractions are shown in Table 4-2.

Table 4-2 : Adopted total impervious area fractions for various land use types

Land Use Type	Total Impervious Area Fraction
Open space/pasture	0.0
Residential areas	0.6
Built up/commercial areas	0.9

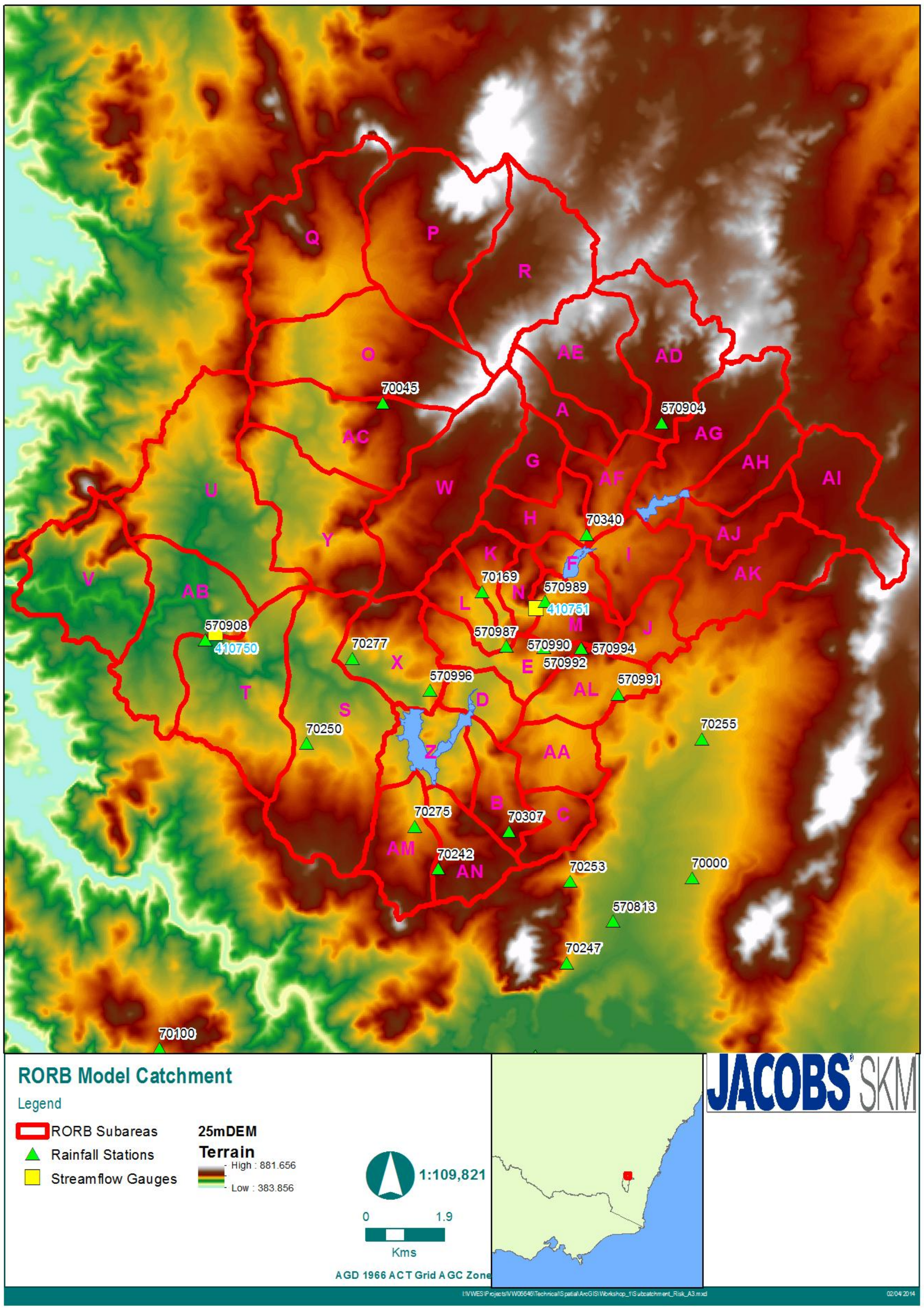


Figure 4-2 : Location of rainfall and streamflow gauges in Ginninderra Creek

4.2.4 1989 Event

Table 4-3 presents the calibrated model parameters using the 1989 event. At this time, the Ginninderra Creek catchment only contained the Ginninderra Dam. The Yerrabi and Gungahlin dams were not yet constructed. Figures Figure 4-3, Figure 4-5 and Figure 4-4 display the resultant calibration plots at gauging site 410751 (Ginninderra Ck U/S Barton Hwy), the outflow of Ginninderra Dam itself and gauging site 410750 (Ginninderra Ck U/S of Charnwood Rd) respectively.

Table 4-3 : Calibrated parameters for 1989 event

	k_c	m	Initial Loss (IL) (mm)	Continuing Loss (CL) (mm)
Gauge 410751	15.00	0.80	90	4.5
Ginninderra Outfow	16.00	0.80	80	5.5
Gauge 410750	10.00	0.80	100	5.0

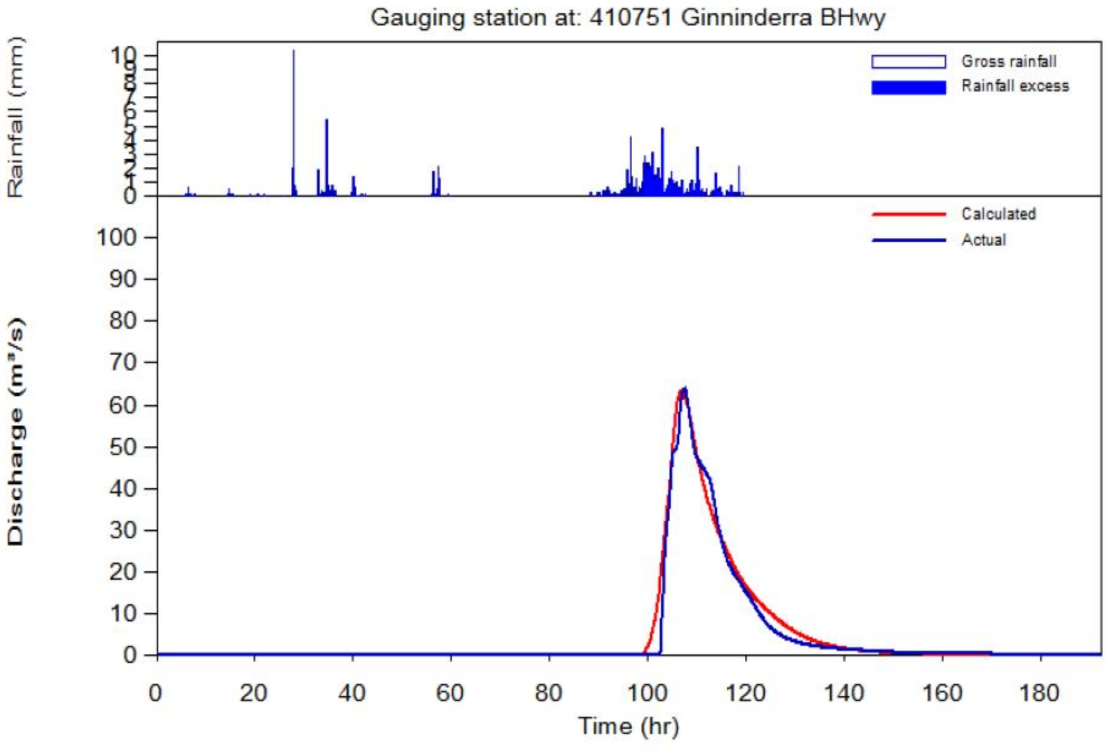


Figure 4-3 : 1989 Event Calibration plot at Gauge 410751

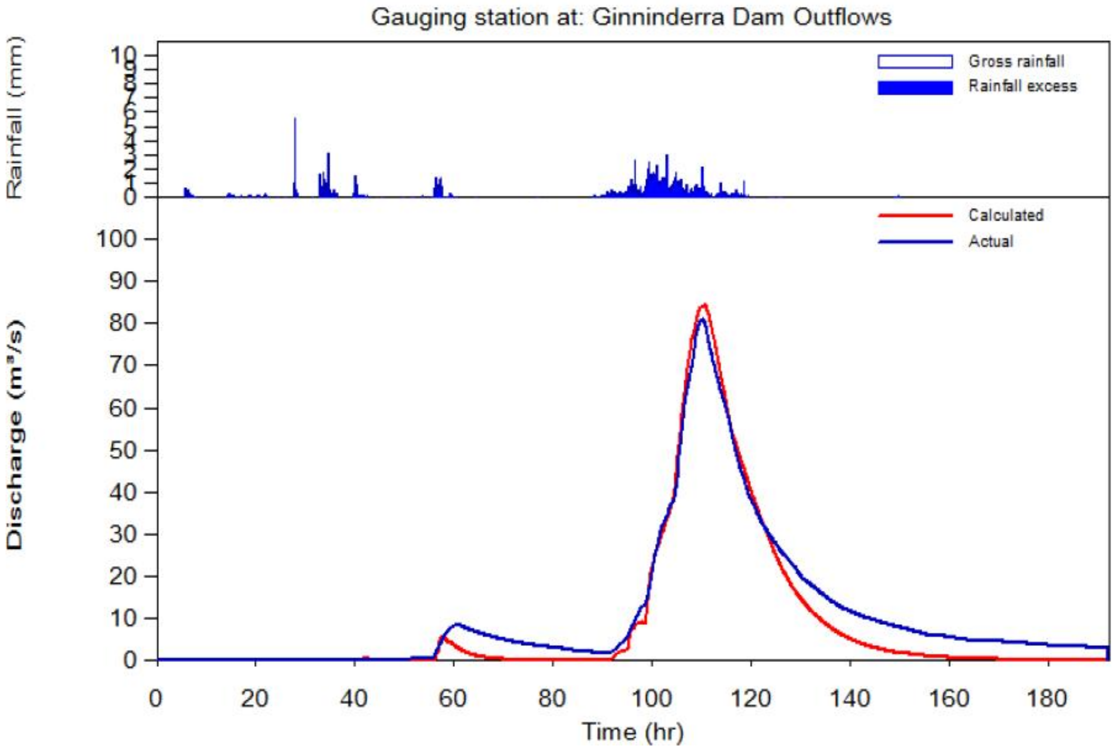


Figure 4-4 : 1989 Event Calibration plot at Ginninderra Outflow

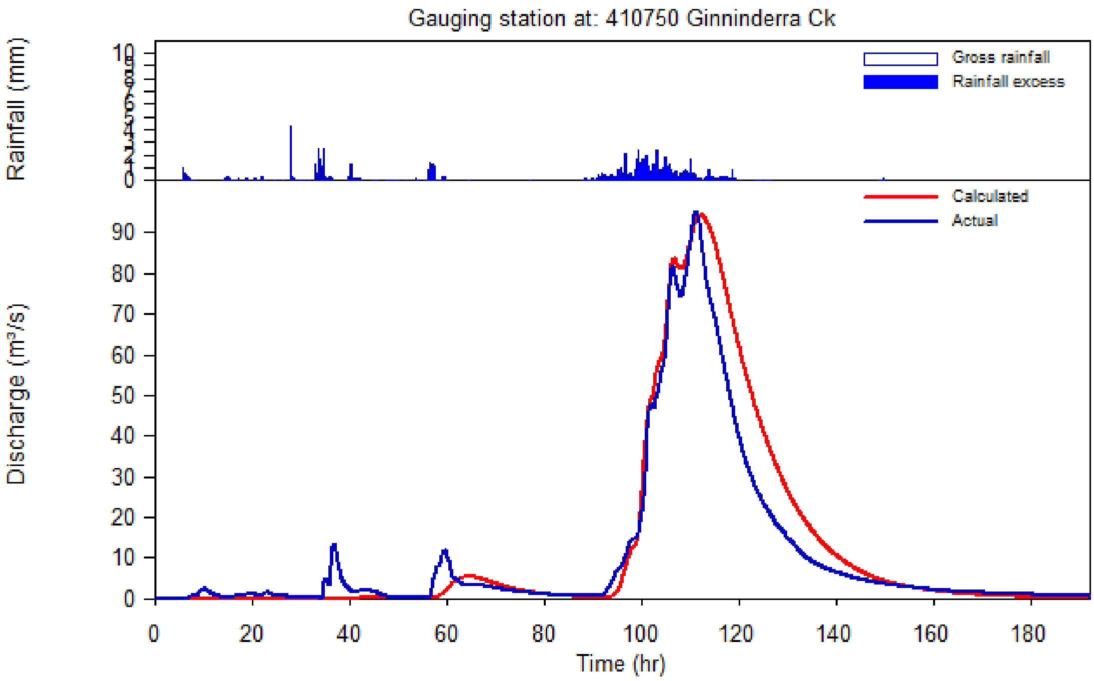


Figure 4-5 : 1989 Event Calibration plot at Gauge 410750

4.2.5 1995 Event

Table 4-4 presents the calibrated model parameters using the 1995 event. Figures 4.6 through 4.9 display the resultant calibration plots at the sites listed in Table 4-4. All dams were constructed during this event, however there was no reservoir level data for Yerrabi dam to calibrate to. Therefore, the Yerrabi outflow calibration site was omitted.

Table 4-4 : Calibrated parameters for 1995 event

Calibration site	k_c	m	Initial Loss (IL) (mm)	Continuing Loss (CL) (mm)
Gungahlin Outflow	9.90	0.80	45	5.0
Gauge 410751	0.50	0.80	5	5.0
Ginninderra Outflow	19.00	0.80	50	6.0
Gauge 410750	8.00	0.80	60	6.0

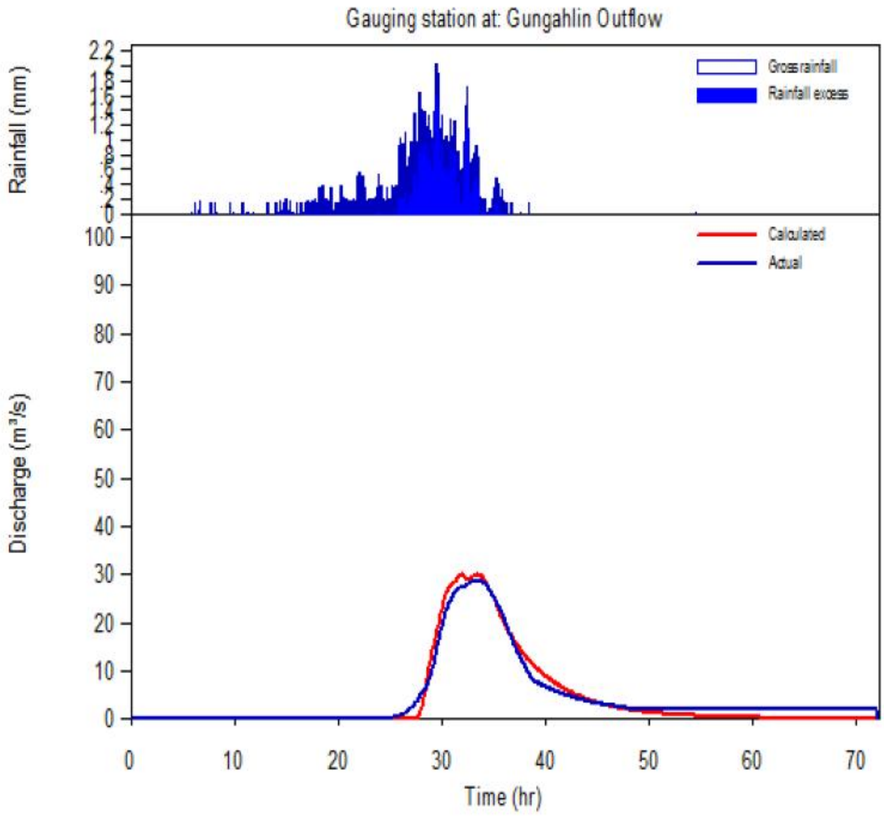


Figure 4-6 : 1995 Event Calibration plot at Gungahlin Outflow

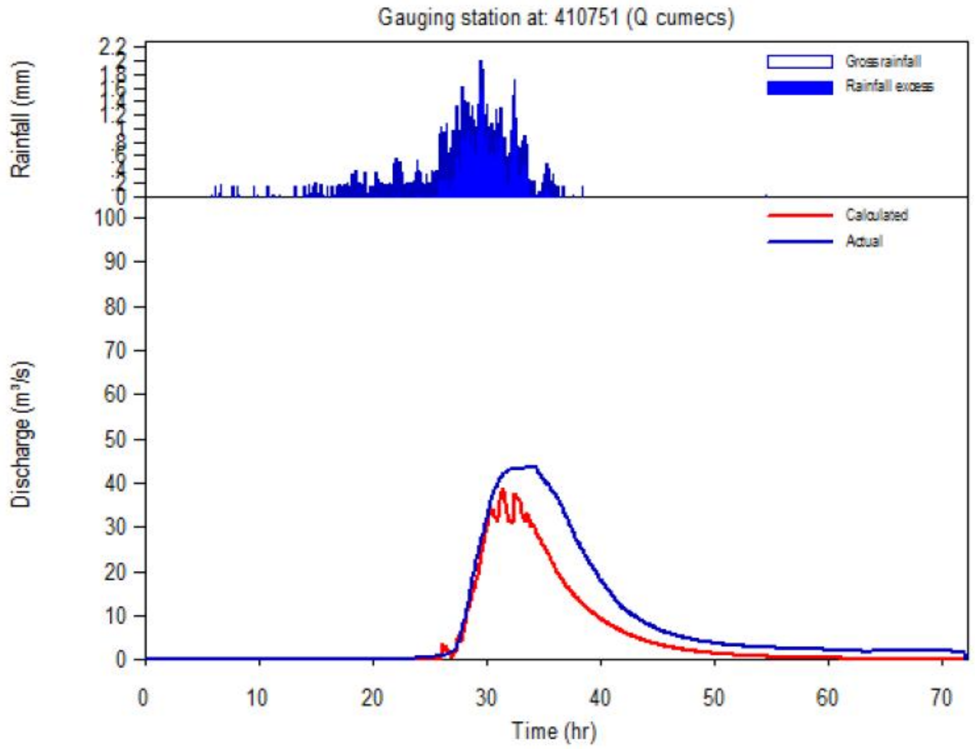


Figure 4-7 : 1995 Event Calibration plot at Gauge 410751

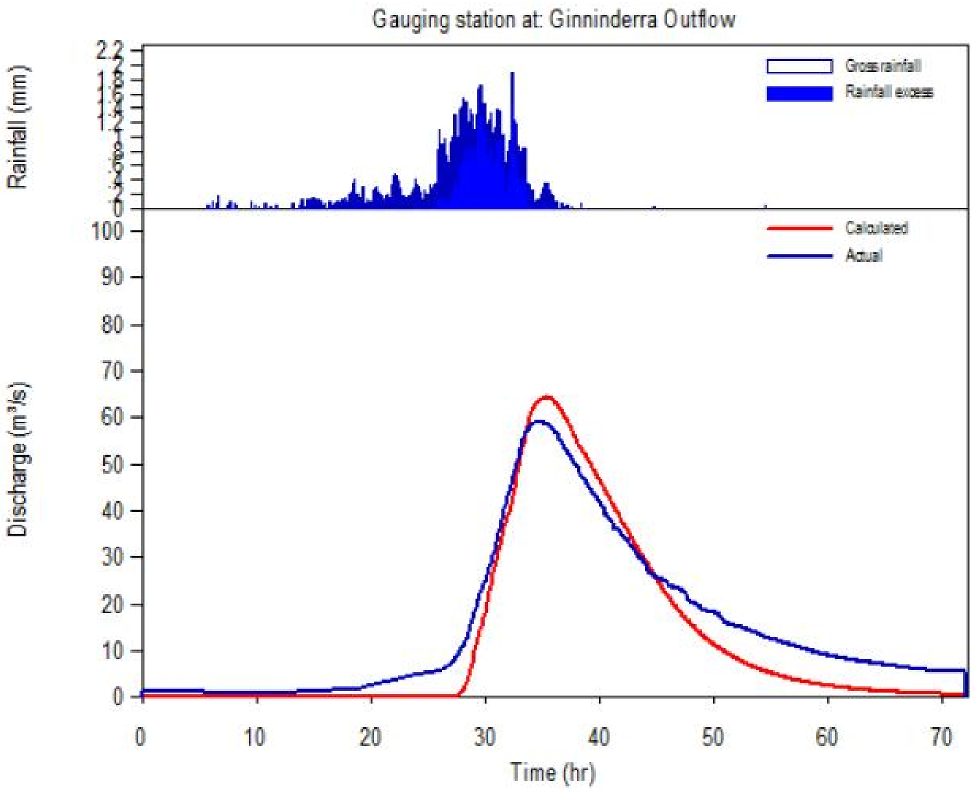


Figure 4-8 : 1995 Event Calibration plot at Gininderra Outflow

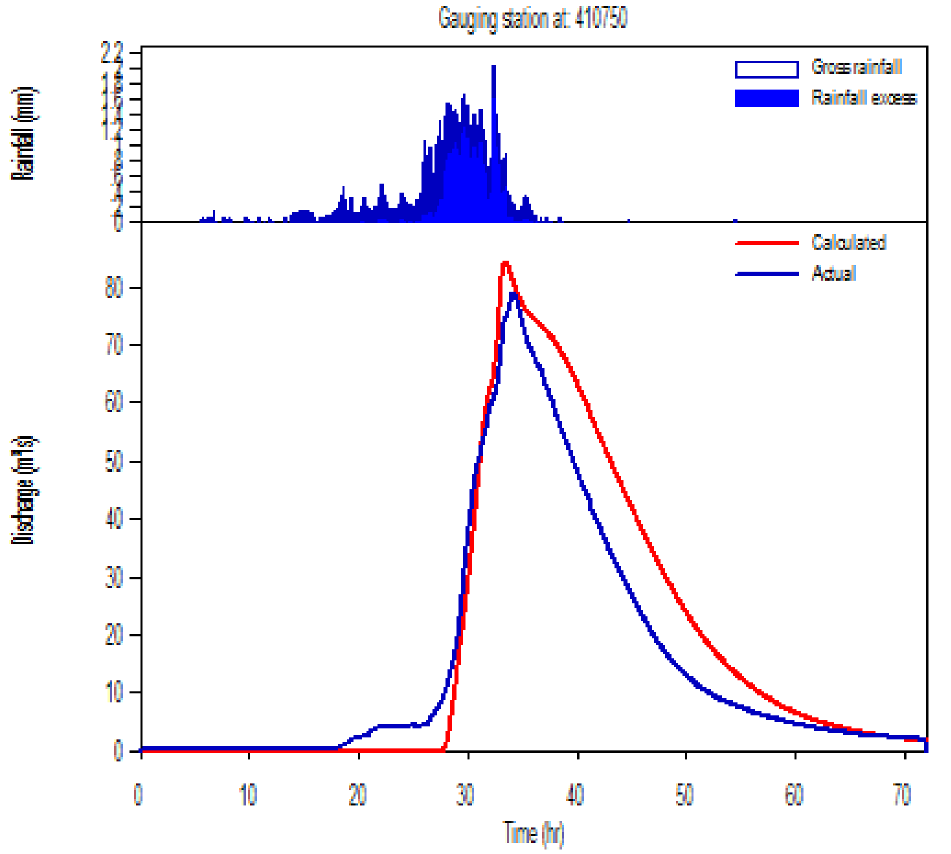


Figure 4-9 : 1995 Event Calibration plot at Gauge 410750

4.2.6 2012 Event

Table 4-5 presents the calibrated model parameters using the 2012 event. Figures 4.10 to 4.14 display the resultant calibration plots at the sites listed in Table 4-5. For this event, all dams were constructed and data was available for all dam outflows.

Table 4-5 : Calibrated parameters for 2012 Event

Calibration Site	**k_c**	**m**	**Initial Loss (IL) (mm)**	**Continuing Loss (CL) (mm)**
Yerrabi Outflow	9.00	0.80	10	3.8
Gungahlin Outflow	12.00	0.80	20	3.0
Gauge 410751	1.00	0.80	25	4.0
Ginninderra Outflow	11.00	0.80	10	4.9
Gauge 410750	10.00	0.80	80	4.5

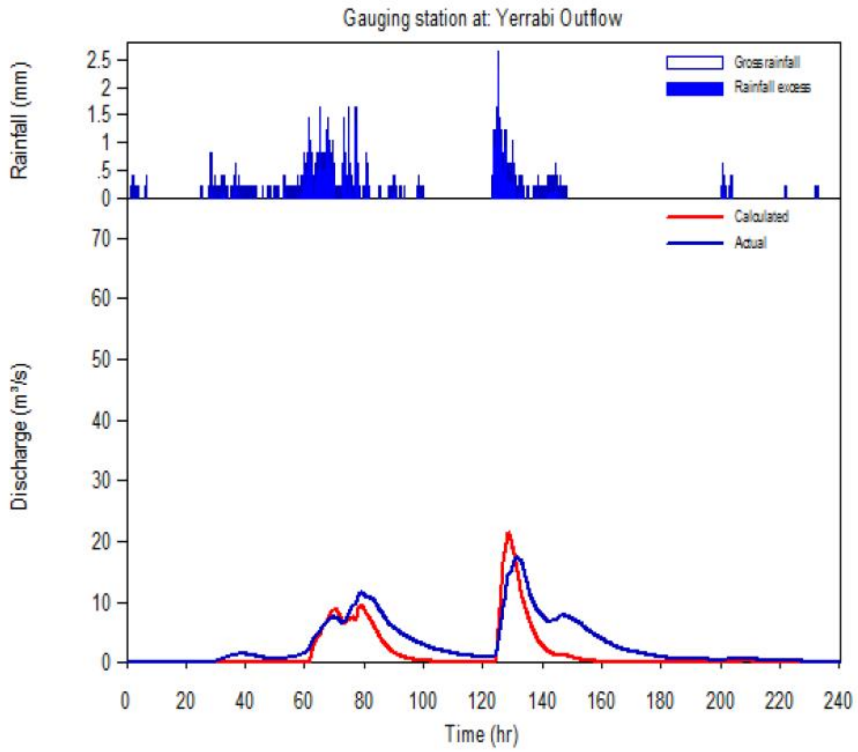


Figure 4-10 : 2012 Event Calibration plot at Yerrabi Outflow

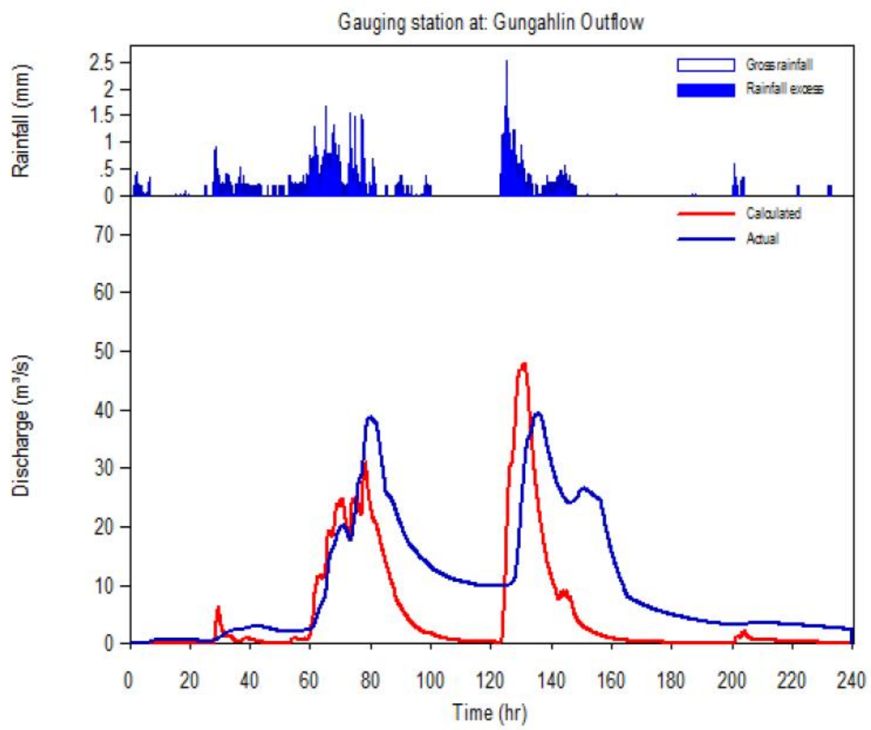


Figure 4-11 : 2012 Event Calibration plot at Gungahlin Outflow

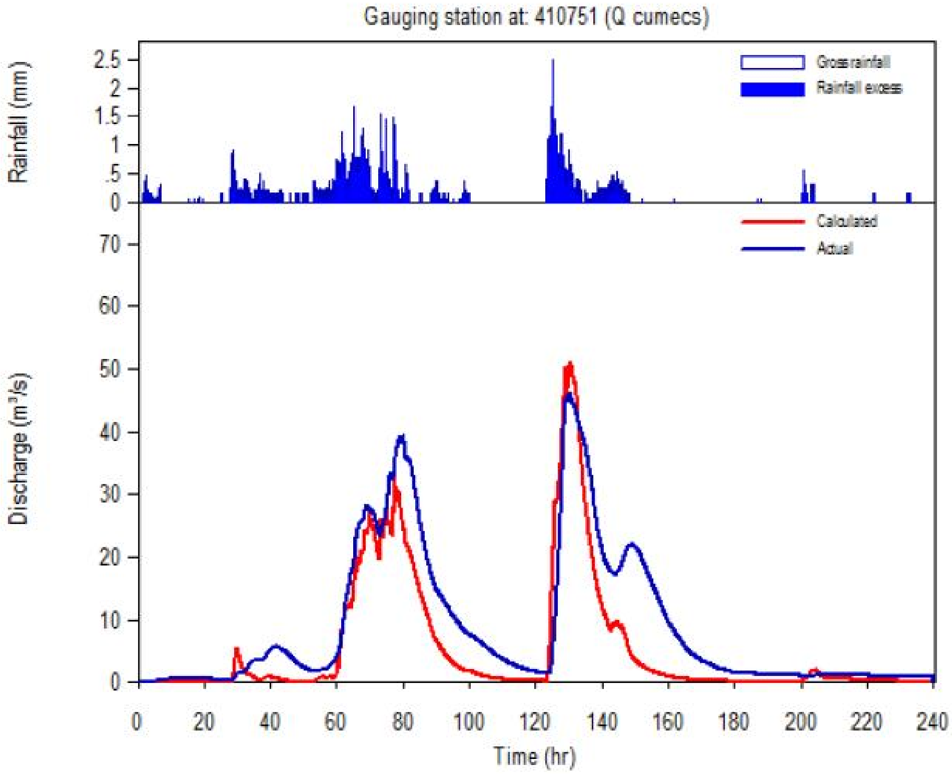


Figure 4-12 : 2012 Event Calibration plot at Gauge 410751

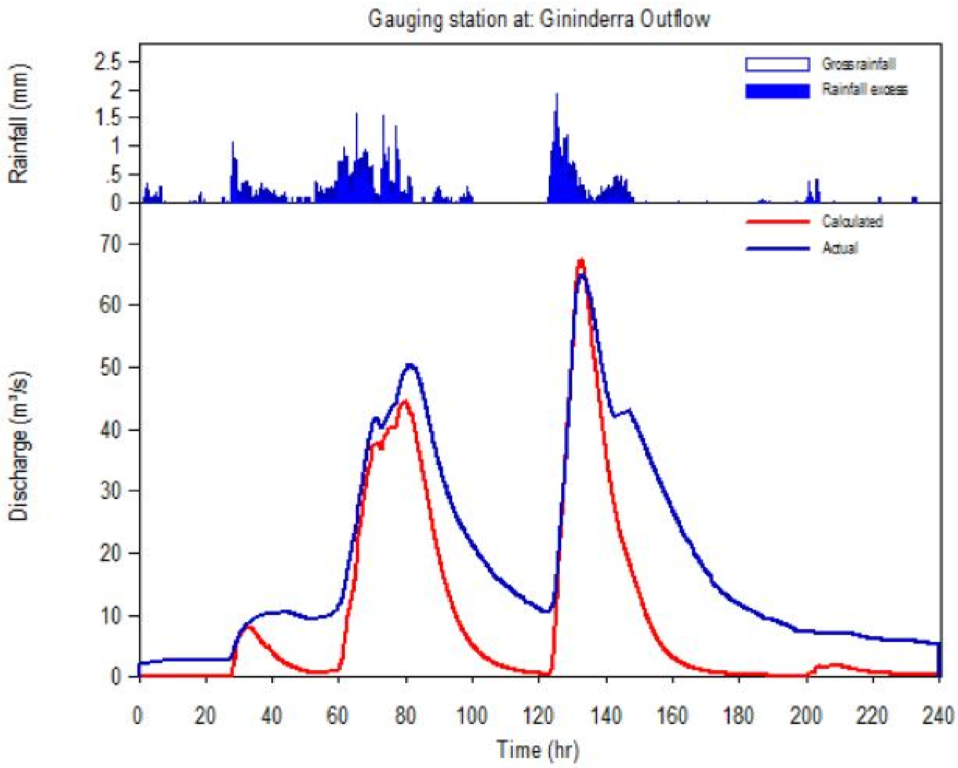


Figure 4-13 : 2012 Event Calibration plot at Ginninderra Outflow

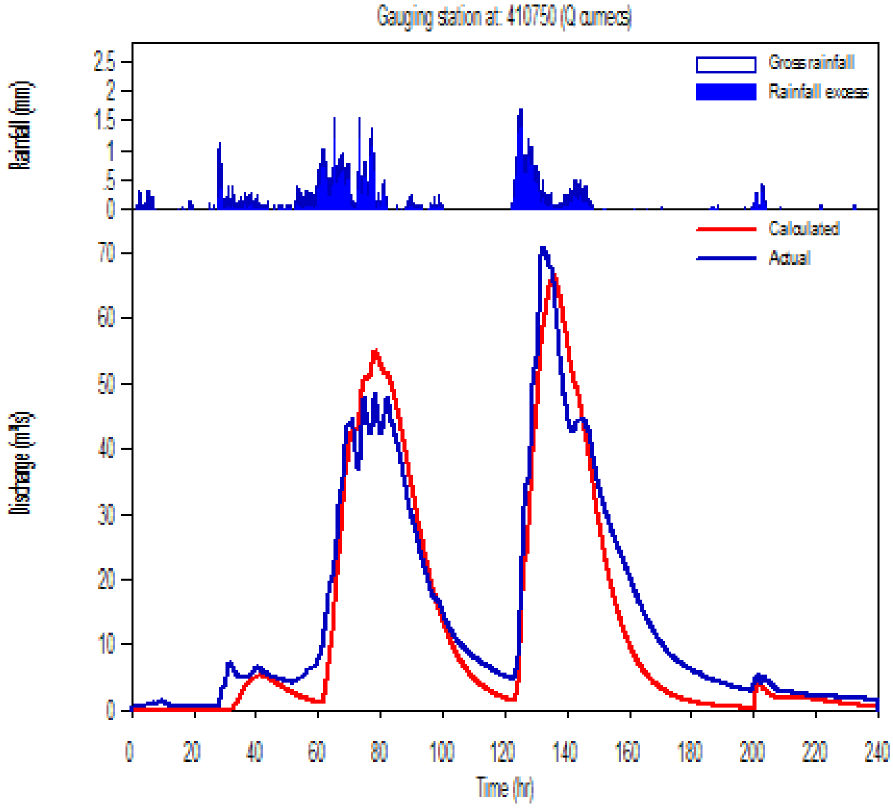


Figure 4-14 : 2012 Event Calibration plot at Gauge 410750

As can be seen from the calibration plots above, for the most part, the peak, shape and timing of the calibration curves seem to provide a reasonable fit.

4.2.7 Regional Equations

In order to test whether the k_c values obtained during calibration were reasonable, they were tested against the regional equation developed by McMahon and Muller (1983), which shows that k_c is directly proportional to d_{av}. This relationship is expressed as:

$$k_c = C_{0.8} d_{av}$$

Equation 4.1

Where:

- $C_{0.8}$ is a characteristic of the catchment that is independent of the scale or size of the catchment.
- d_{av} is the weighted average flow distance from all of the nodes within the catchment to the catchment outlet.

Pearse *et al.* (2002) analysed a large database of routing parameters collated by the CRC for Catchment Hydrology from 72 catchments from all states of Australia and the ACT. The expected value of $C_{0.8}$ was 1.14 with a range from 0.61 to 2.13. The calculated values of $C_{0.8}$ found in this study were between 0.99 and 2.57.

Values for k_c were obtained at each interstation area k_c at each event and an average was taken over the three events to arrive at an average k_c value. The interstation areas and calibrated k_c values are shown in Table 4-6.

Table 4-6 : Calibrated kc parameter values for each interstation area

Interstation Area	1989	1995	2012	Average k_c
1 (upstream of Yerrabi)	9.5	9.0	9.0	9.2
2 (Between Gungahlin and Yerrabi)	11.5	9.9	12.0	11.1
3 (Between gauge 410751 and Gungahlin)	2.5	1.0	1.0	1.5
4 (Between Ginninderra Dam and gauge 410751)	16.0	19.0	11.0	15.3
5 (Between gauge 410750 and Ginninderra Dam)	10.0	8.0	10.0	10.0
6 (Downstream of gauge 410750 to the Ginninderra Falls)	13.0	15.7	26.1	18.3

The adopted k_c parameters shown in Table 4-6 were then compared to k_c value ranges that were calculated using the region equation from Pearse *et al*. (2002). The comparisons of k_c values are shown in Table 4-7.

Table 4-7 : Comparison of calibrated kc parmeter with Pearse et al. (2002)

Interstation Area	Regional lower	Regional expected	Regional upper	Achieved from calibration
1 (upstream of Yerrabi)	2.4	4.4	8.2	9.2
2 (Between Gungahlin and Yerrabi)	2.8	5.3	9.9	11.1
3 (Between gauge 410751 and Gungahlin)	0.6	1.2	2.2	1.5
4 (Between Ginninderra Dam and gauge 410751)	3.9	7.3	13.7	15.3
5 (Between gauge 410750 and Ginninderra Dam)	3.0	5.6	10.5	10.0
6 (Downstream of gauge 410750 to the Ginninderra Falls)	7.8	14.7	27.4	18.3

4.3 Conclusion

Given the reasonable fits displayed in the calibration plots and the fact that the calibrated k_c values lay within or near the ranges predicted by Pearse *et al*. (2002), the model parameters discussed above were adopted for use in design.

The objective of the calibration process is to represent the catchment routing processes. In this process, the focus is on consistency of the k_c value from each of the calibration events. The loss values, and particularly the initial loss, will be a function of the antecedent condition for each particular event. Furthermore the selection of events has been based upon the largest flows (rather than rainfall) and this has the potential to bias the loss values towards wet antecedent conditions and hence underestimate the values of loss. For these reasons the loss values for application in design are determined through the verification process against the flood frequency analysis of the gauged records, which is outlined in Section 6.

5. Design Inputs

5.1 Design rainfall depths

A short description of how the design rainfalls were estimated for the range of AEPs is described below.

Intensity-Frequency-Duration (IFD) analysis for events up to the 1 in 50 AEP

The design rainfall depths were estimated from an IFD analysis using the procedure in *Book II of Australian Rainfall and Runoff* (I.E.Aust. 1998). Design rainfall depths were estimated for burst durations between 0.5 and 12 hours.

It should be noted that at the time of publication, revised rainfall IFD data were available from the Bureau of Meteorology. These revised IFD data were not adopted for this study primarily because they have yet to be formally released as a final version by the Bureau and this is consistent with a number of other concurrent flood studies undertaken in the ACT.

Growth curves for estimating depths between 1 in 100 and 1 in 2000 AEP

Short durations

A regional approach for estimating design rainfall depths developed by Jordan *et al* (2005) was adopted for durations between 0.5 and 9 hours (inclusive) and AEPs between 1 in 100 and 1 in 2000 (inclusive). Jordan *et al* (2005) obtained rainfall records from the Bureau of Meteorology for twelve continuously recording rain gauges located around Australia. These records were analysed using an approach similar to the CRC-FORGE method to estimate regional growth factors for rainfall depths between 1 in 100 and 1 in 2000 AEP.

Long durations
Design rainfalls for long durations were obtained directly from the results of the CRC-FORGE analysis. CRC-FORGE provides growth factors, which relate the 1 in 50 AEP rainfall estimate for a particular duration, to the 1 in 200, 500, 1000 and 2000 AEP. These growth factors were extracted from a gridded data set for standard durations of 24, 48, 72, 96 and 120 hours. The growth factors were used to generate rainfall depths between the 1 in 100 and 1 in 2000 AEP. For intermediate durations not provided by CRC-FORGE, growth factors were interpolated. For the 18 hour duration, the 24 hour CRC-FORGE growth factors were adopted.

Intersection of long and short durations
Design rainfall depths at the intersection of long and short durations were interpolated to ensure a smooth rainfall frequency curve.

Areal reduction factors

Point rainfall estimates were converted to catchment average values using areal reduction factors appropriate for the ACT region. Conceptually, this factor accounts for the fact that larger catchments are less likely to experience high intensity storms over the whole of the catchment. Areal reduction factors were obtained from SKM (2010).

Summary of adopted design rainfall depths

The design rainfall depth versus frequency curves for Yerrabi, Gungahlin and Ginninderra dam catchments are shown in Figure 5-1, Figure 5-2 and Figure 5-3.

The design rainfall depth tables for the catchments of each of the three dams in Ginninderra Creek catchment are shown in Appendix B.

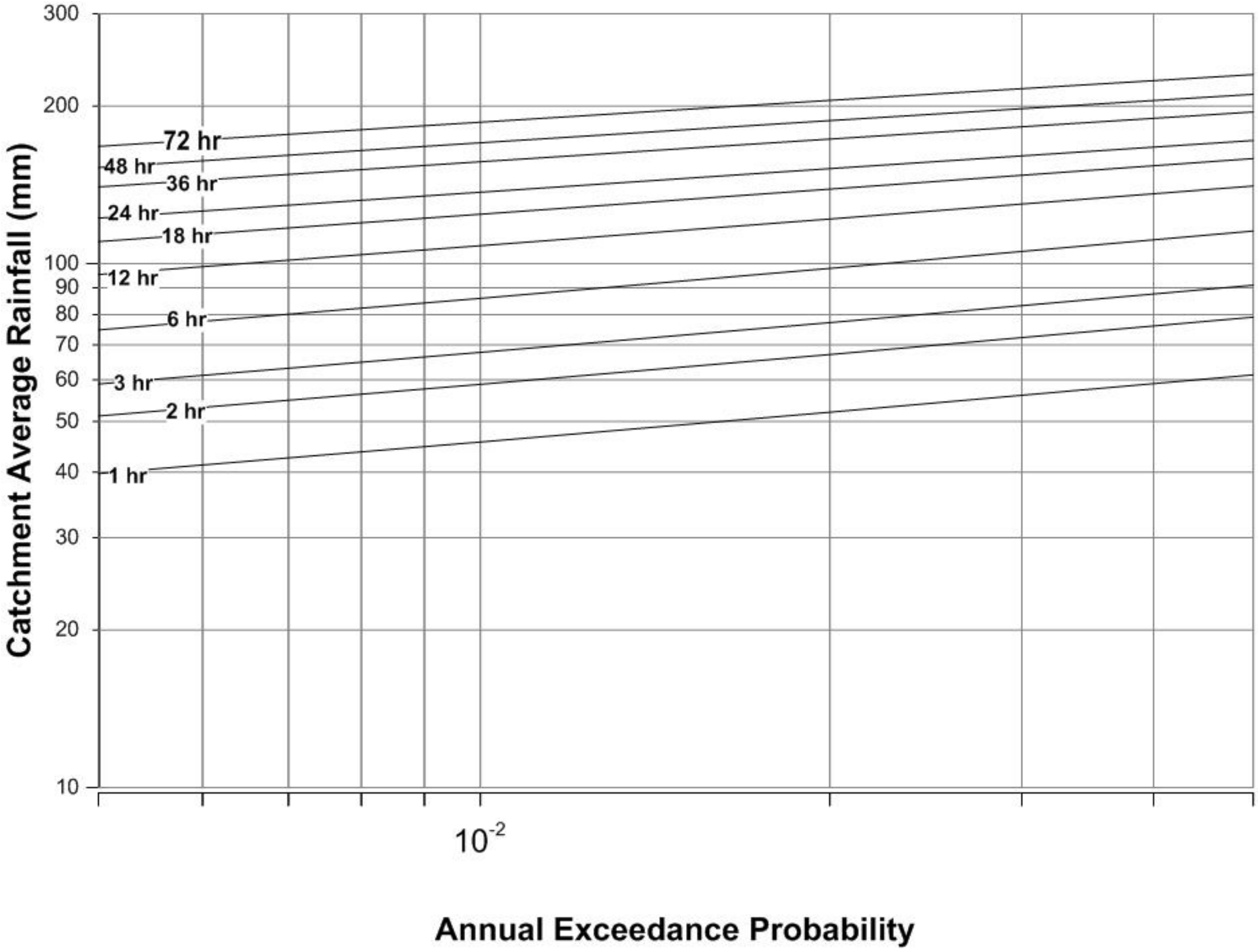


Figure 5-1 : Design rainfall depth versus frequency curves for Yerrabi Dam catchment

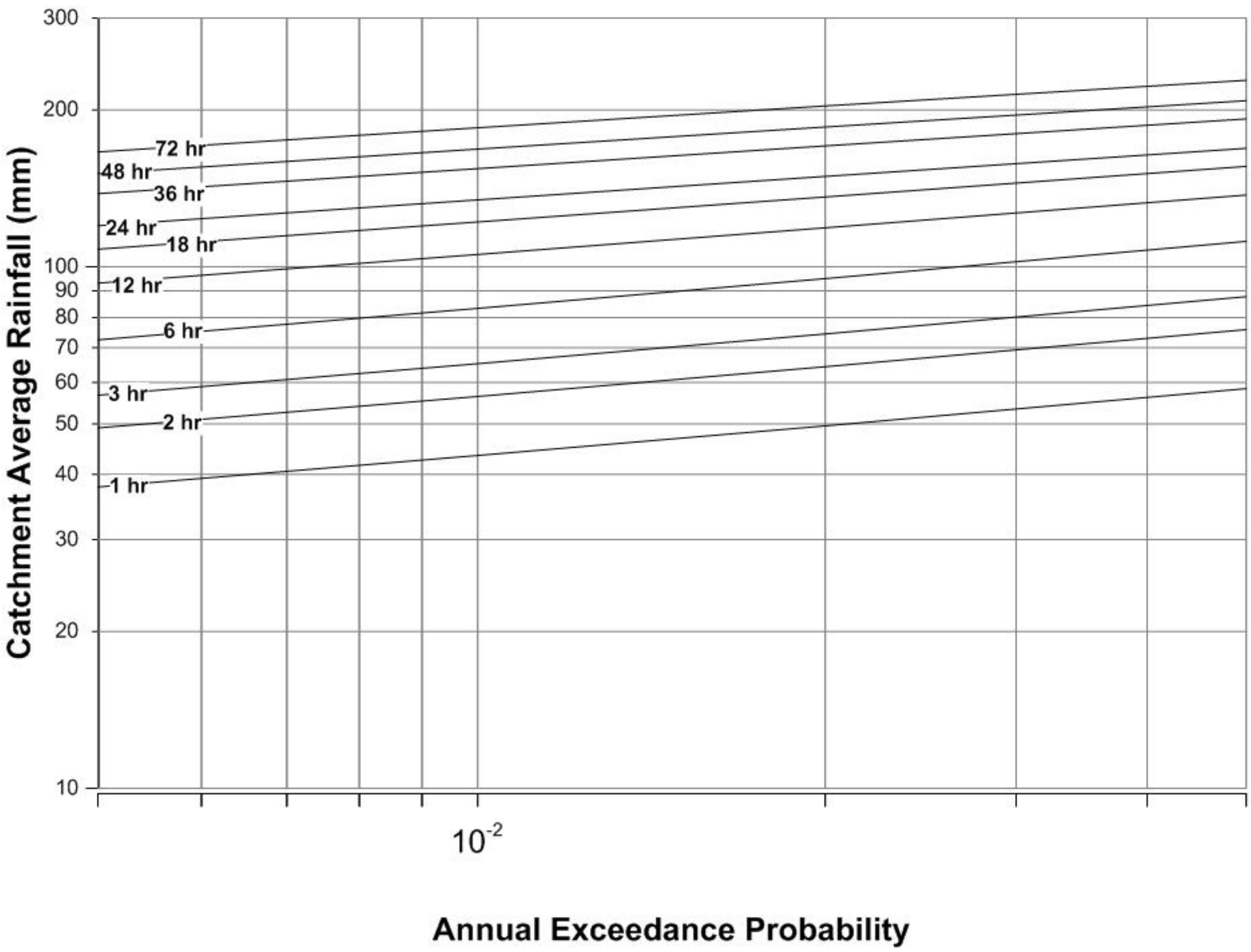


Figure 5-2 : Design rainfall depth versus frequency curves for Gungahlin Dam catchment

5.2 Pre-burst rainfall

The temporal pattern of rainfall antecedent to the main rainfall burst (pre-burst pattern) derived from Jordan *et al* (2005) was applied to durations of 12 hours or less, and the antecedent pattern derived from Minty *et al* (1996) was applied to burst events with durations of 18 hours or greater.

5.3 Design temporal patterns

The design temporal patterns varied with burst rainfall duration so that they remained consistent with the approach used to derive the PMP rainfall depth. For rainfall durations between 1 and 12 hours, the sample of 10 Monte-Carlo temporal patterns derived by Jordan *et al*, 2005 was applied. These temporal patterns were derived from analysis of the temporal patterns of convective thunderstorm events recorded by rainfall stations around Australia.

A sample of Monte-Carlo temporal patterns was also used for longer duration design storms. This Monte-Carlo sample came from the analysis of events used in the inland zone for the GSAM PMP (Minty *et al*, 1996). Temporal patterns are not available for 18 hour storms and thus patterns were derived from the cumulative patterns of 24 hour durations. For 12 hour duration events both GSDM (BoM, 2003) and GSAM (BoM, 2006) temporal patterns were run for the catchment and the mean peak flow estimate from each method was adopted.

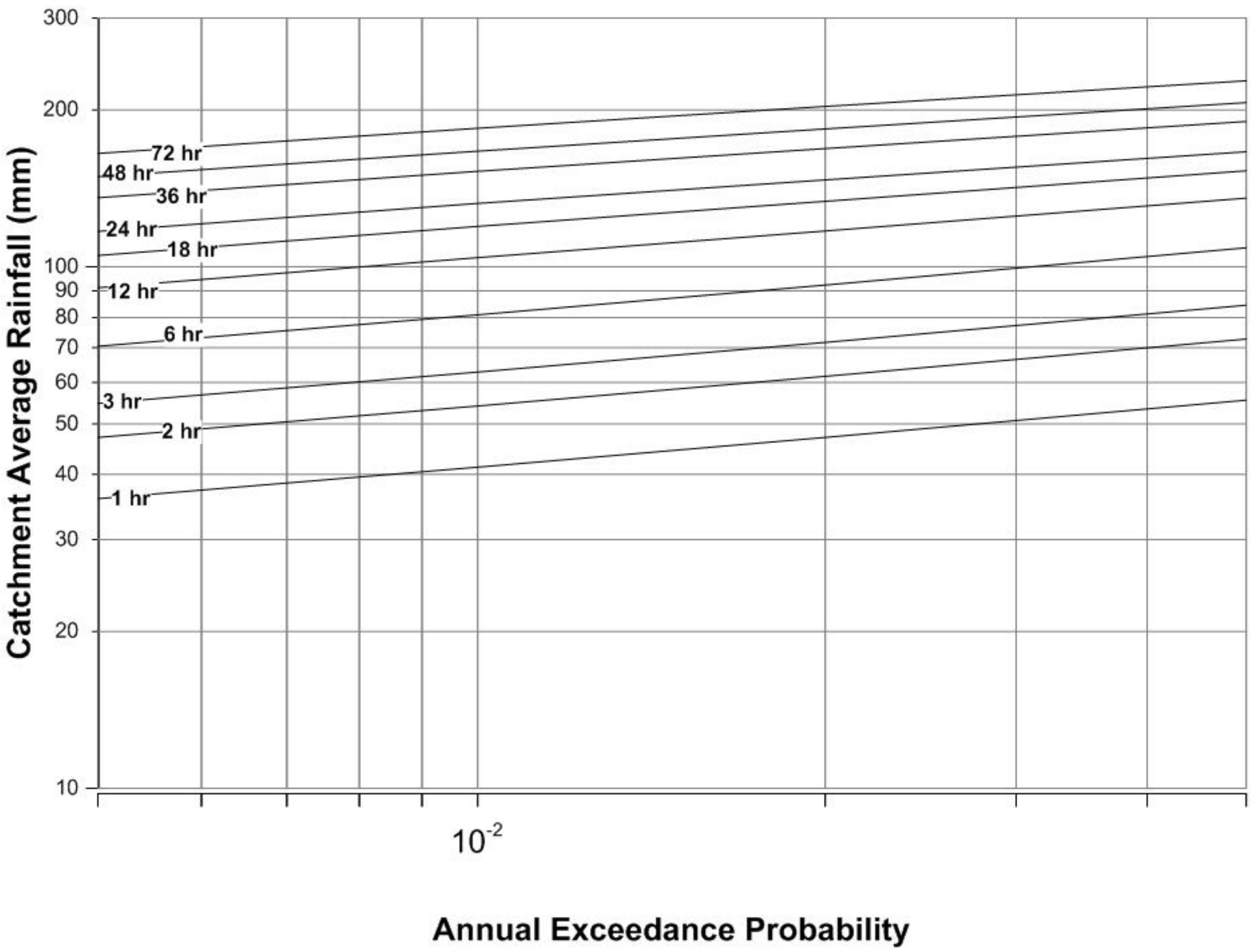


Figure 5-3 : Design rainfall depth versus frequency curves for Ginninderra Dam catchment

5.4 Design spatial patterns

The design spatial patterns were varied with burst rainfall duration so that they remained consistent with the approach used to derive the PMP rainfall depth. The design spatial patterns for the short and long duration events were based on the methods as outlined in the GSDM (BoM, 2003) and GSAM (BoM, 2006) respectively.

A different spatial pattern was derived for each of the three dams. Figure 5-4, Figure 5-5 and Figure 5-6 show the alignment of the GSDM spatial pattern and delineation of the Ginninderra Creek RORB sub-areas within the ellipses ‘centred’ over each of the three dams in the catchment. The spatial pattern applied in the modelling considered the proportion of each sub-area within each ellipse of the generalised storm pattern.

The long duration GSAM spatial pattern was determined using spatial data for the Topographic Adjustment Factor (TAF). The average TAF for each sub-area was extracted from a gridded data set and divided by the catchment average TAF value to generate the distribution for modelling.

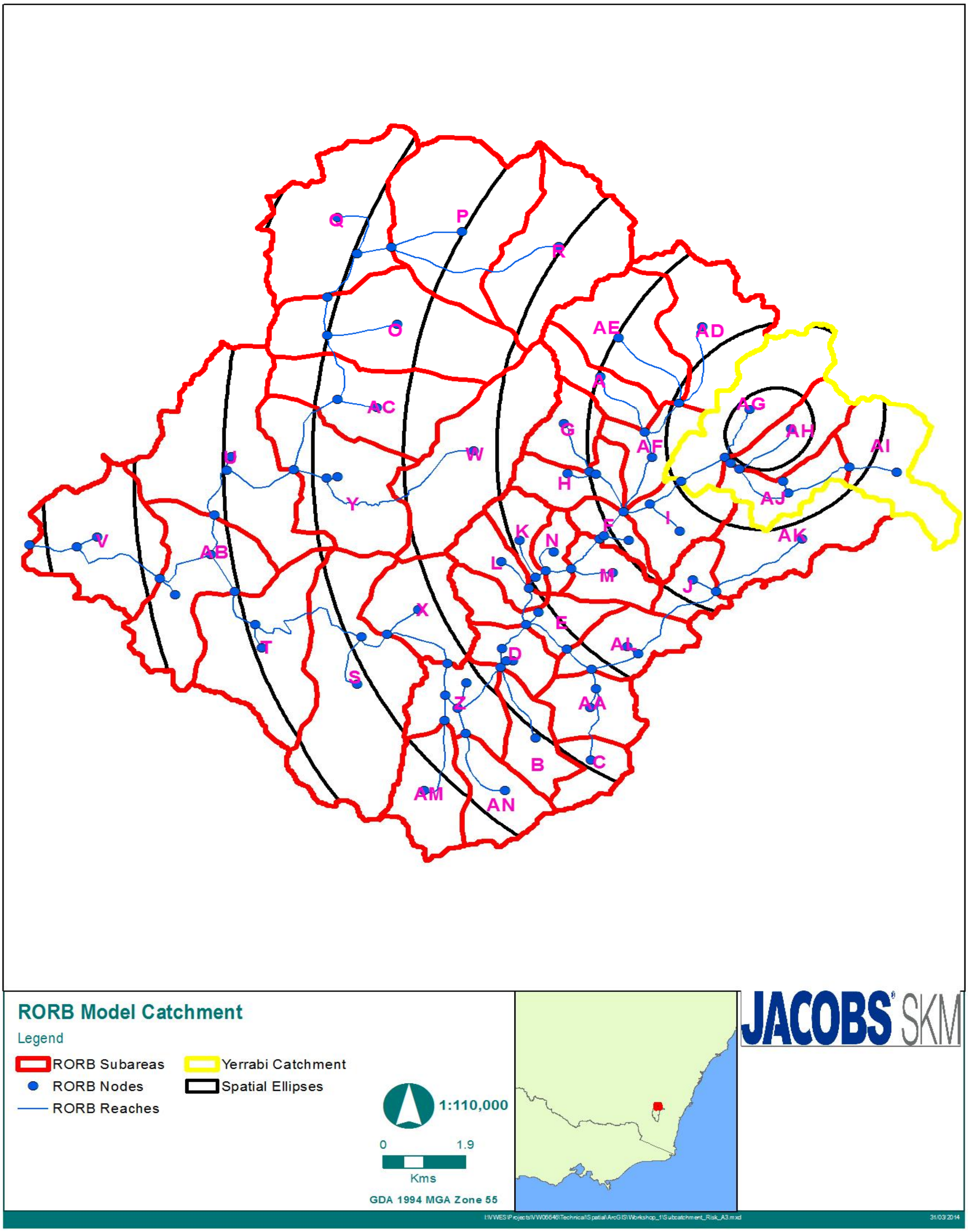


Figure 5-4 : GSDM spatial pattern alignment over Yerrabi Dam catchment using all RORB sub-areas

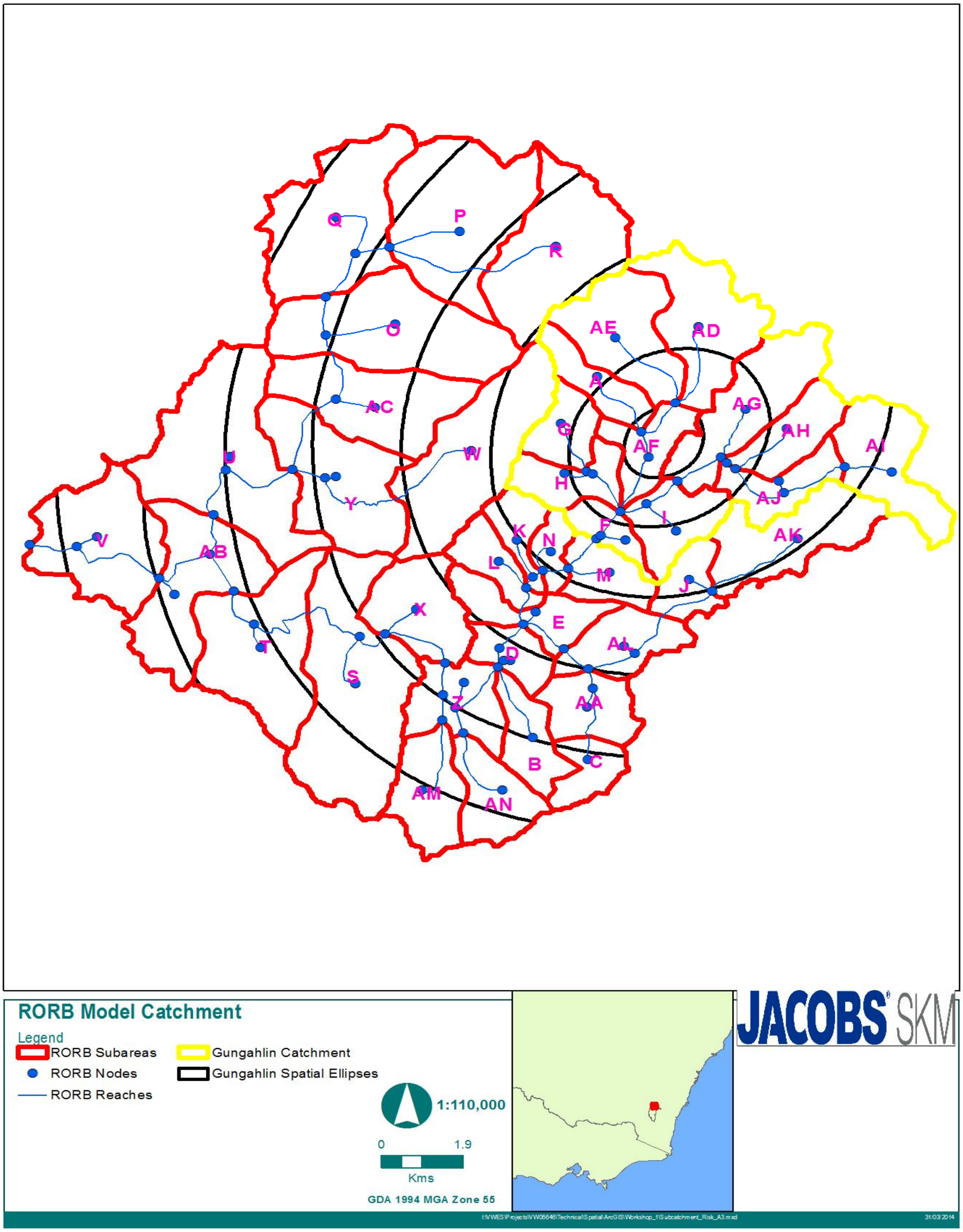


Figure 5-5 : GSDM spatial pattern alignment over Gungahlin Dam catchment using all RORB sub-areas

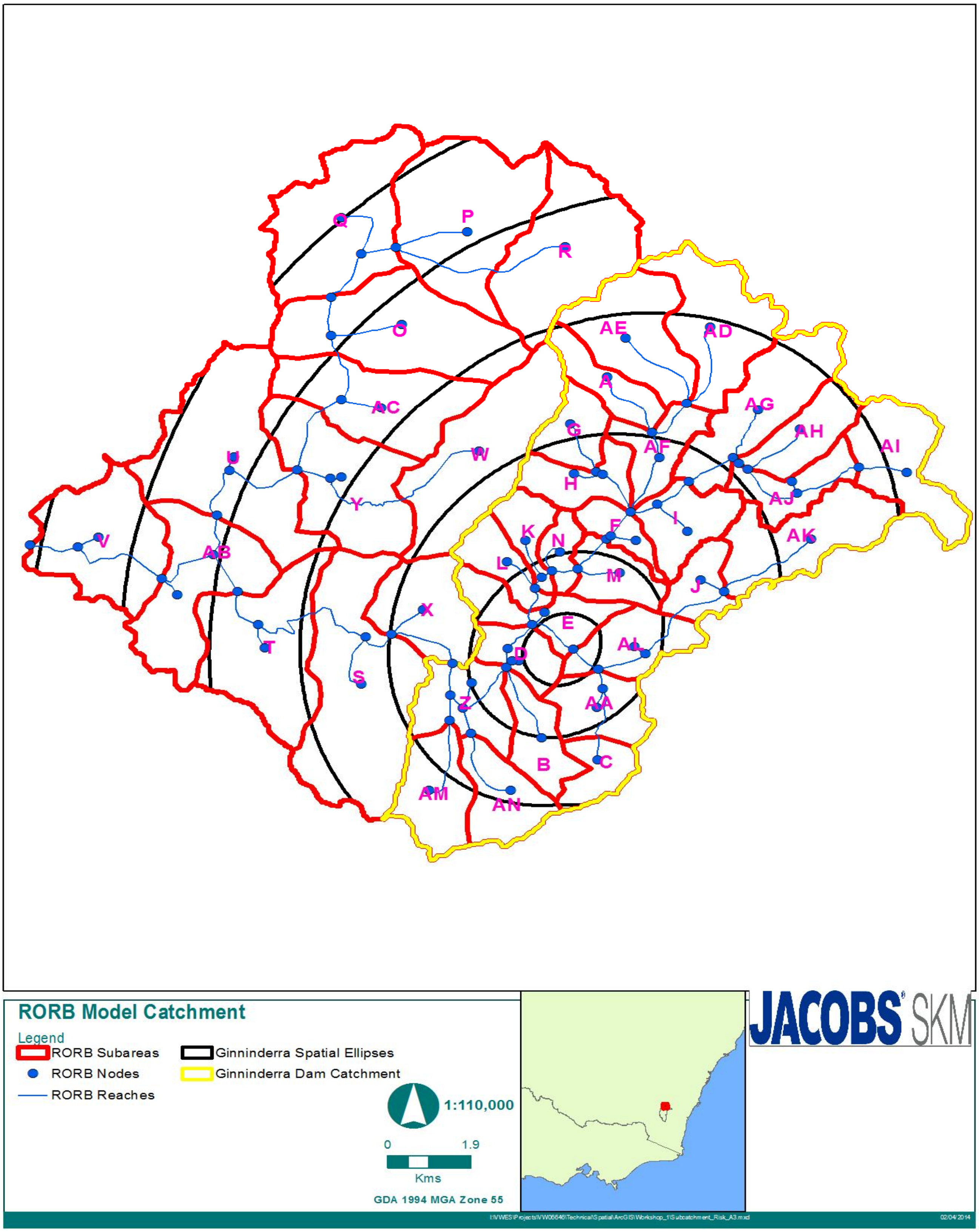


Figure 5-6 : GSDM spatial pattern alignment over Ginninderra Dam catchment using all RORB sub-areas

5.5 Reservoir drawdowns

Yerrabi, Gungahlin and Ginninderra dams were constructed primarily as a means of controlling the water quality in Ginninderra Creek and to also serve as places of recreation and aesthetic value (ACTEW, 2010 and SMEC, 2011). As opposed to larger dams that are operated for water supply purposes, the three dams within Ginninderra Creek remain at full supply level (FSL) most of the time. In order to accurately represent the storage levels within the model, it is necessary to ensure the correct initial airspace available below the spillway (drawdown) is entered as an input for each dam.

The main source of inflows to the water storages is provided by natural rainfall. The Ginninderra Creek catchment underwent a major drought event stretching from the late 1990s until about 2009. The change in climatic conditions over Yerrabi Dam is displayed in Figure 5-7 with water levels from about 2009 onwards being at approximately FSL.

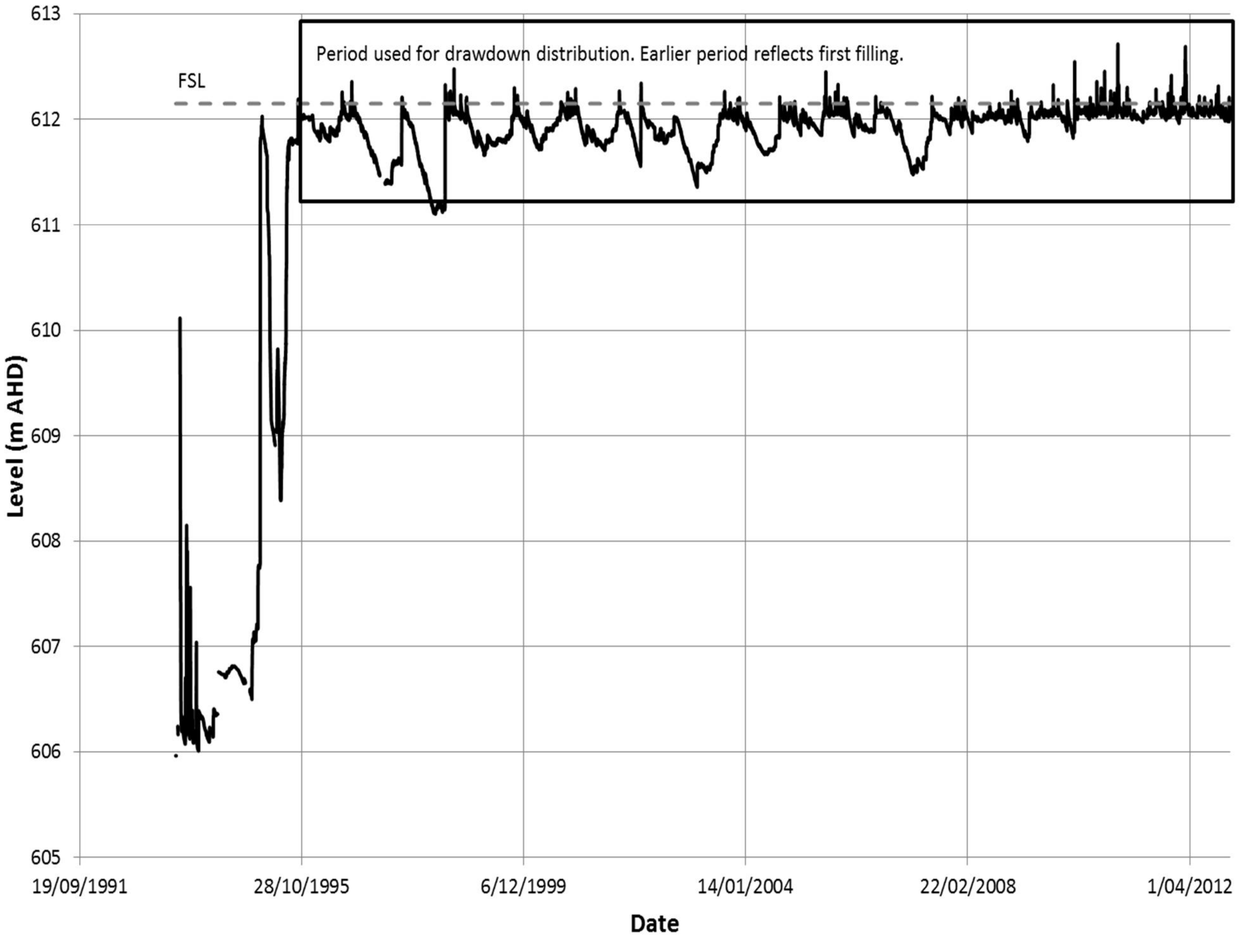


Figure 5-7 : Time series of water level in Yerrabi Dam

Given the fact that for over half the period of record of levels in Yerrabi dam, the catchment has experienced a major drought, it would be expected that the proportion of time these dams have operated at below FSL would be relatively high. The distribution curve that shows the proportion of time exceeded for each level in Yerrabi Dam is displayed in Figure 5-8. It does display a significant proportion of the water level time series being below FSL. However, given the fact the last time there was significant available airspace in Yerrabi dam was 2007, it was considered reasonable to input zero available airspace for Yerrabi Dam into the modelling.

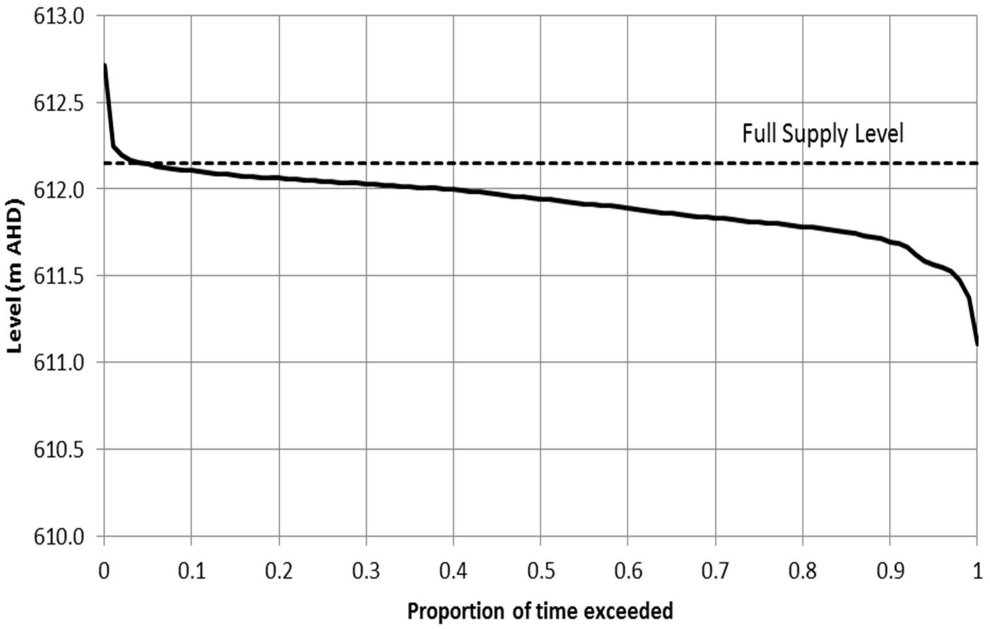


Figure 5-8 : Drawdown Distribution - Yerrabi Dam with FSL

A similar distribution of water levels for Gungahlin Dam is presented in Figure 5-9. It can be seen from this curve that even though there does appear to be some free airspace available for a small proportion of the time series, this volume is small enough to be neglected for the purposes of modelling floods through the storage. Therefore, the initial airspace available for Gungahlin Dam was set to zero.

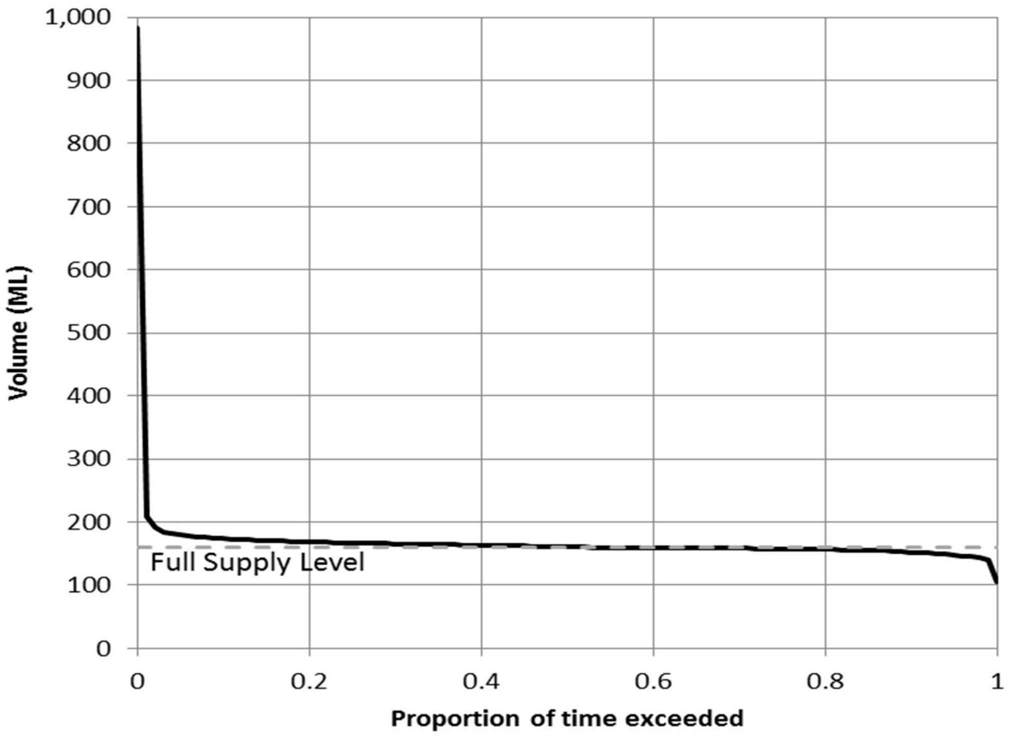


Figure 5-9 : Drawdown distribution – Gungahlin Dam

A somewhat more pronounced available airspace distribution curve was produced for the Ginninderra Dam (Figure 5-10). While it appears that there is a significant proportion of the time series where the levels in the dam are below FSL, upon further inspection it is revealed that only 8% of the time Ginninderra dam is operating at only 5% below FSL. The combination of this relatively low overall volume and proportion of time the dam is at that volume and its impact on flood attenuation led to this low proportion of available airspace not being specifically modelled.

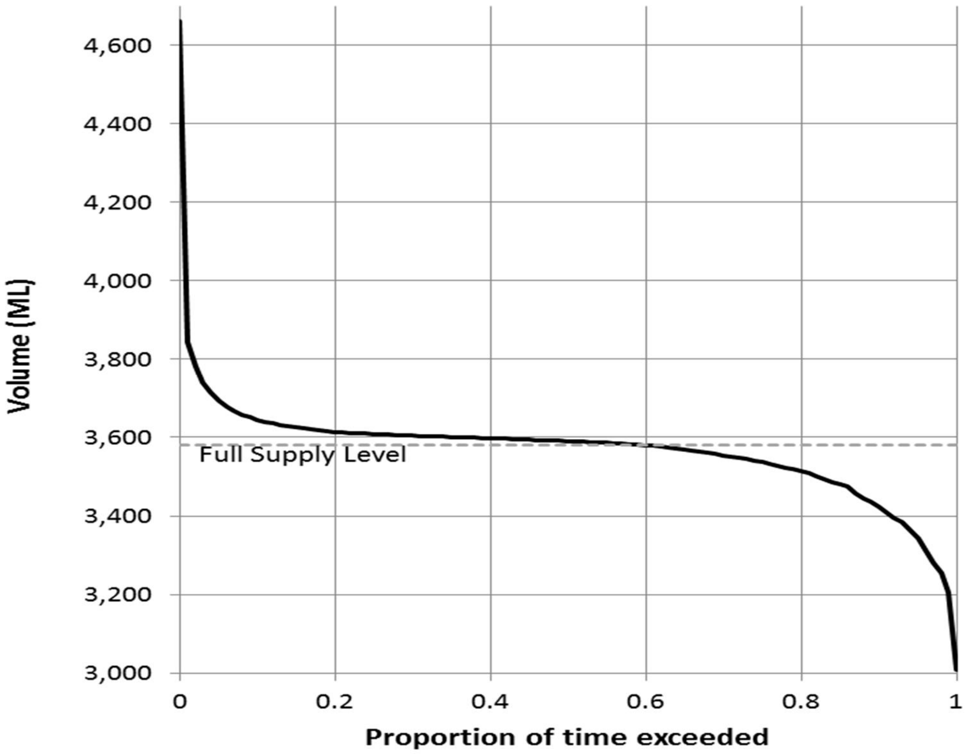
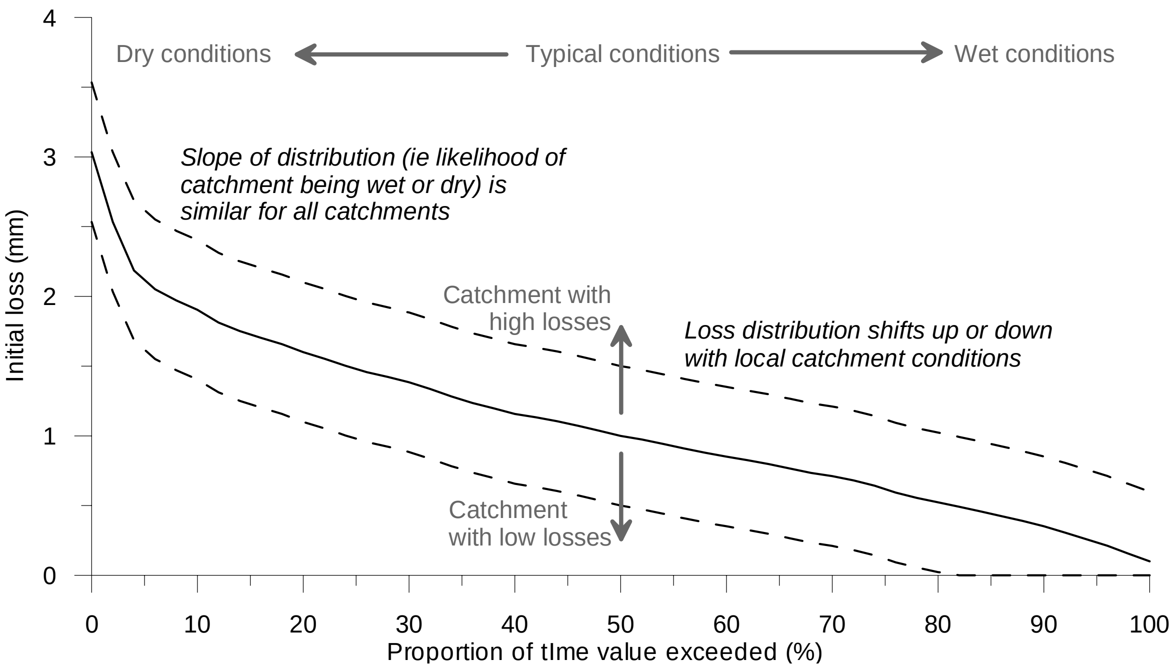


Figure 5-10 : Drawdown Distribution - Ginninderra Dam

5.6 Design losses

Design losses were stochastically sampled from non-parametric probability distributions. The shapes of the distributions of initial loss and continuing loss rate were derived by Hill *et al* (1997), using results from analysis of a large number of catchments in south-eastern Australia, including the ACT. Hill *et al* (1997) examined results from catchments with more than 15 recorded flood events to derive non-dimensional distributions of loss values. Losses obtained for each catchment were standardised by representing each value as a proportion of the median loss. This allowed the distributions of losses across different catchments to be directly compared.

The standardised distributions of losses exhibited a remarkable degree of consistency, and the results clearly support the assumption that, while the *magnitude* of losses may vary between different catchments, the *shape* of the distribution does not. In other words, 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 schematically illustrated in Figure 5-11. Values for the median initial loss and median continuing loss rate for the Ginninderra Creek catchment were estimated by verifying the flood quantiles produced by RORB to regional flood frequency analyses. Details of the verification process and the median adopted initial and continuing loss parameters are given in Section 6.2



- Figure 5-11: Schematic illustration of variation in location but not shape of initial loss distribution.

5.7 Impervious areas

Future levels of development were estimated by drawing upon a number of data sources, including ACT Mapi Territory Plan data layers for future urban areas and the Gungahlin Strategic Assessment. These sources identified future development around the suburbs of Harrison, Palmerston, Lawson and Jacka. Within these regions, the identified urban areas were considered to represent areas for development whereas offset areas identified in the Strategic Assessment were assumed to remain as open space, To supplement these planning data sets, the 2012 aerial imagery was reviewed to identify urban developments in progress at the time of the imagery capture. It was assumed that these development areas would be completed into the future.

In all areas of expected future development, it was assumed that the future level of urban density would be consistent with that currently found within surrounding fully developed suburbs.

In the absence of any specific information on the development of commercial and industrial areas, it was assumed that all future development could be reasonably represented by urban residential levels of imperviousness.

6. Model verification

6.1 Approach

The value of losses obtained from calibration to large events are likely to be biased towards low loss rates, as large floods are more likely to occur on catchments with wet antecedent conditions. The manner in which loss values vary with rainfall depends on chance, although it would be expected that some systematic variation occurs with season. Thus, while calibrating the model to a small sample of historic events should provide useful data for the selection of the routing parameter (k_c), these few events provide less information about the appropriate values of loss to be used in design flood estimation. Suitable values of loss are usually determined through the process of verification, where the estimated peak flows for given AEPs are compared to an at-site frequency analysis of recorded peak flows.

Verification of design losses was only undertaken for the catchment upstream of gauge 410750. This catchment has an area of 134 km^2 and the gauge is located approximately 6 km downstream of the outlet of Ginninderra Dam.

6.1.1 Site flood frequency analysis

Being downstream of all three dams, streamflow data at gauge 410750 (Ginninderra Ck U/S Charnwood Rd – see Figure 4-2 for location, 35 year period of record) has been impacted both by ongoing urban development of the upstream catchment (refer to Section 4.2.3) as well as construction of the Yerrabi and Gungahlin dams (the gauge was installed after the construction of Ginninderra Dam). As such, the gauge data is non-stationary and cannot be used for flood frequency analysis without adjustment for these factors. Therefore, there would have been a change in the streamflow patterns at the gauge site between the years preconstruction of the dams and postconstruction of the dams due to the interception capacity of the newly constructed dams.

In order to capture this change and to produce a single timeseries for the purpose of generating a flood frequency analysis, the preconstruction time series of streamflow was scaled to bring it in line with the postconstruction flows. Table 6-1 shows the dates used in this report to define 'preconstruction' and 'postconstruction' in this context. There was also a 'buffer' applied to either side of the construction dates list above of 6 months to a year whereby streamflow data was excluded from the frequency analysis in order to account for first filling and design-stage clearing of the area etc.

Table 6-1 : Pre and Post-construction dates for scaled time series of gauge 410750

Periods for new scaled time series at gauge 410750 – refers to construction of Yerrabi and Gungahlin Dams		
	Start Date	**End Date**
Pre-construction	9/12/1978	31/07/1991
Post-construction	1/01/1996	1/11/2013

The scaling factor was calculated by plotting the scatter of the 1989 and 2012 peak 18 and 24 hour duration flows as outputted from RORB for AEPs ranging from 1 in 5 to 1 in 100 inclusive. The slope of the line of best fit through the scatter provided the scaling factor to be used. The scatter plot is shown in Figure 6-1 below and the scaling factor achieved was 0.62.

There are two main factors that contribute to the scaling factor achieved. On one hand, the construction of the dams upstream of the gauge would attenuate the flows and hence the scaling factor. Conversely, given the urbanisation that has occurred throughout the period of record at the gauge, the impervious area of the catchment would have increased. This increase in impervious area would cause an associated increase in the streamflow recorded at the gauge. The scaling factor of 0.62 indicates that the effect of the dams intercepting runoff is outweighing the increase in impervious area over time.

The scaling factor was then applied to the maximum daily peak flows for all *preconstruction* streamflow time series data points. Postconstruction data points were not altered. The resultant 'scaled-flow' time series is shown in Figure 6-2.

The flood frequency analysis was performed using the scaled streamflow data from gauge 410750. Annual maximum instantaneous flow was extracted from the recorded streamflow data and a Generalised Extreme Value (GEV) distribution, fitted using LH-Moments with a shift of two was applied (Figure 6-3). Flood quantiles were extracted for the 1 in 20, 1 in 50 and 1 in 100 AEP. These are shown in Table 6-2.

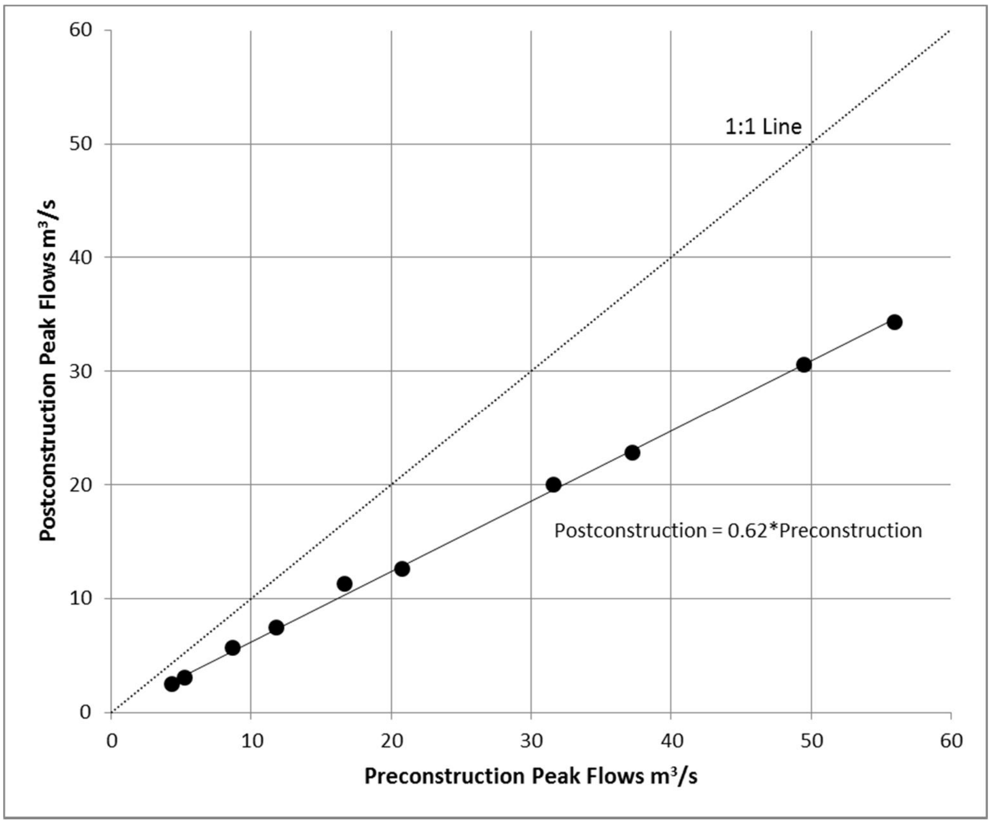


Figure 6-1 : Pre and postconstruction scatter of peak flows used to determine scaling factor

Table 6-2 : Peak scaled flows for gauge 410750 for different AEPs

Frequency	410750 Peak Scaled Flow (m^3/s)
1 in 20 AEP	70
1 in 50 AEP	57
1 in 100 AEP	80

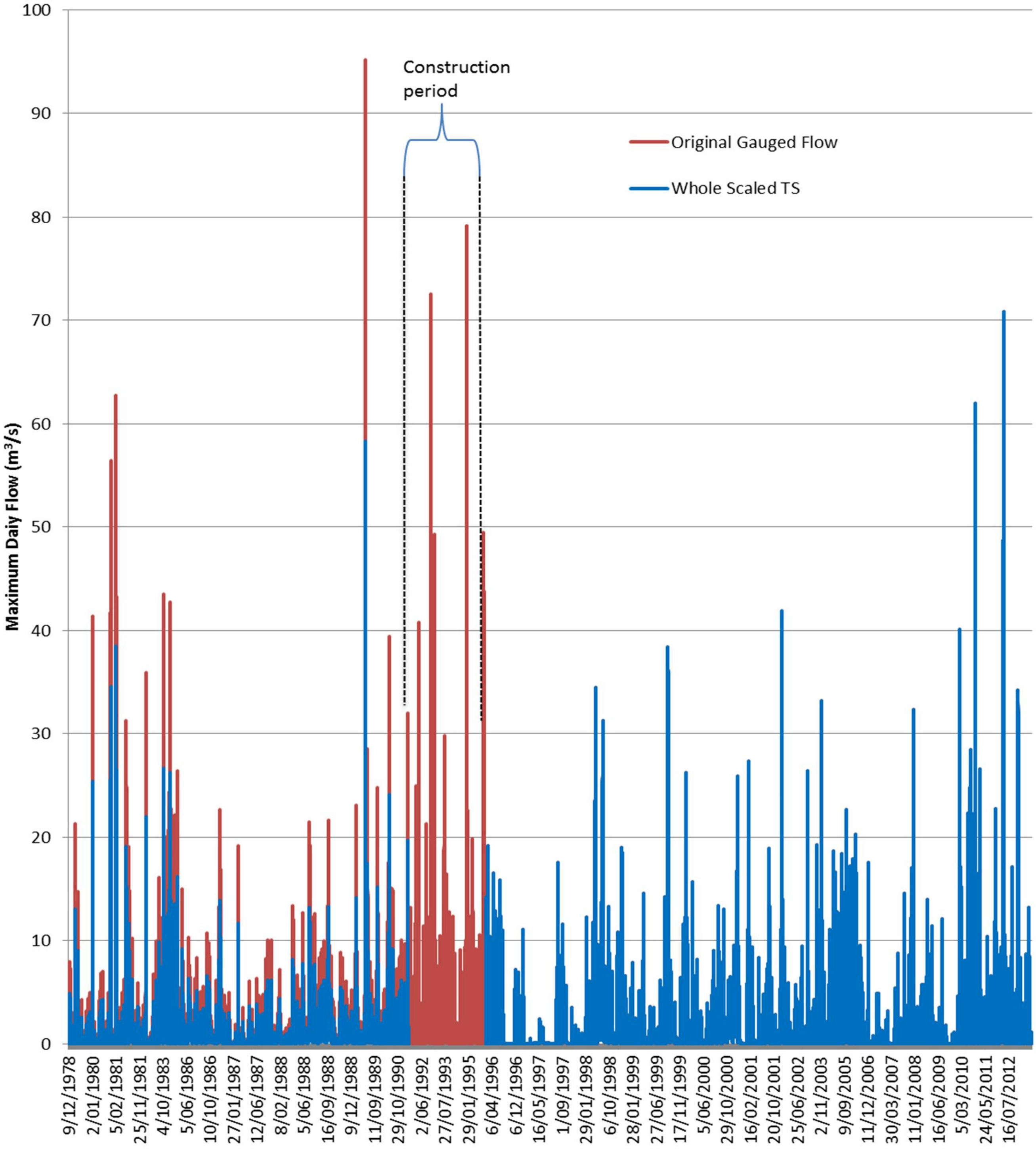


Figure 6-2 : Original and scaled streamflow time series from gauge 410750

Figure 6-3 displays the flood frequency analysis up to a frequency of a 1 in 100 year (0.01 AEP) associated with scaled time series shown in Figure 6-2. This frequency analysis does not include the years marked as within the construction period.

6.2 Verification results

The RORB model of the catchment of gauge 410750 (Ginninderra Ck U/S of Charnwood Rd) was run in a Monte-Carlo framework with different combinations of median initial loss (*IL*) and continuing loss (*CL*), and the resulting flood quantiles were compared with results from the at-site flood frequency analysis (Figure 6-3).

The results of the at-site analysis are presented with 95% confidence intervals based on the distribution fit. These confidence intervals are wide, representative of the level of uncertainty associated with the flood frequency analysis on such a short period of record.

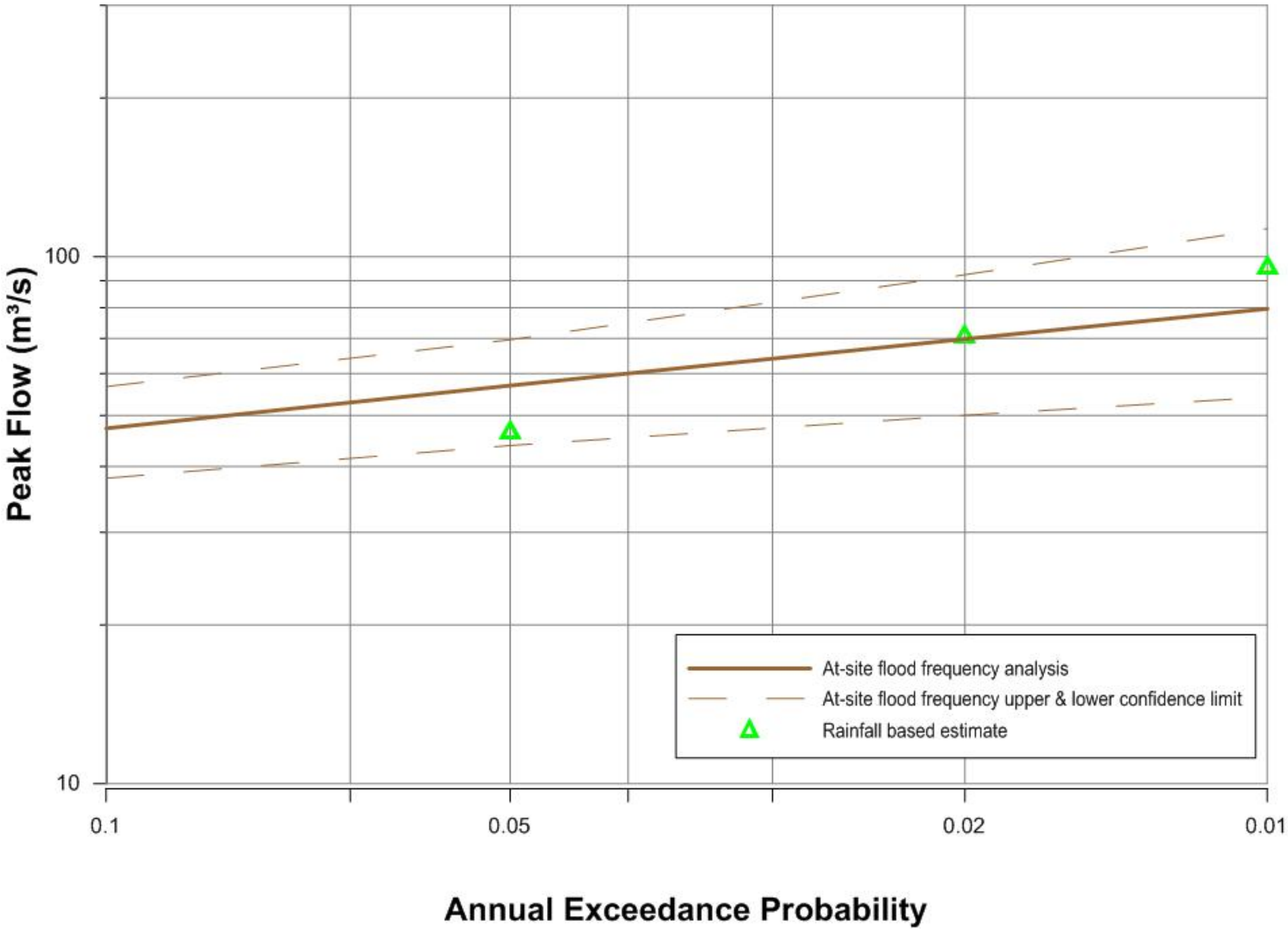


Figure 6-3 : Gauge 401750 at-site frequency quantiles based upon adjusted 'current development' annual maxima and verification fit

As can be seen from Figure 6-3, the adopted model parameters produce a relatively good fit when compared to the at-site flood frequency quantiles. All three peak flows are well within the 90% confidence limits of the at-site flood frequency curve. Table 6-3 shows that an initial loss (IL) value of 30mm was used throughout the catchment, whereas the continuing loss (CL) and k_c values varied with interstation area.

Table 6-3 : Adopted design model parameters

Interstation area	k_c	m	Initial Loss (IL) (mm)	Continuing Loss (CL) (mm)
Yerrabi Outflow	9.2	0.8	30	4.0
Gungahlin Outflow	11.1	0.8	30	3.0
Gauge 410751	1.5	0.8	30	4.0
Ginninderra Outflow	15.3	0.8	30	5.0
Gauge 410750	10.0	0.8	30	4.0

7. Flood Hydrology Results

7.1 Design flood frequency curves

The RORB model described in Section 4 was run in the joint probability framework described in Section 3.1.3, with design rainfall inputs described in Section 5.1 and design model parameters described in Section 6.2. The RORB model was run for the three different spatial patterns described in Section 5.3.

The results for lower AEPs (rarer events) are presented in Jacobs (2014).

7.1.1 Yerrabi Dam

Figure 7-1 presents the peak inflows into and outflows out of Yerrabi Dam, which are also tabulated in Table 7-1. The peak inflows displayed here were calculated independent of the associated inflows.

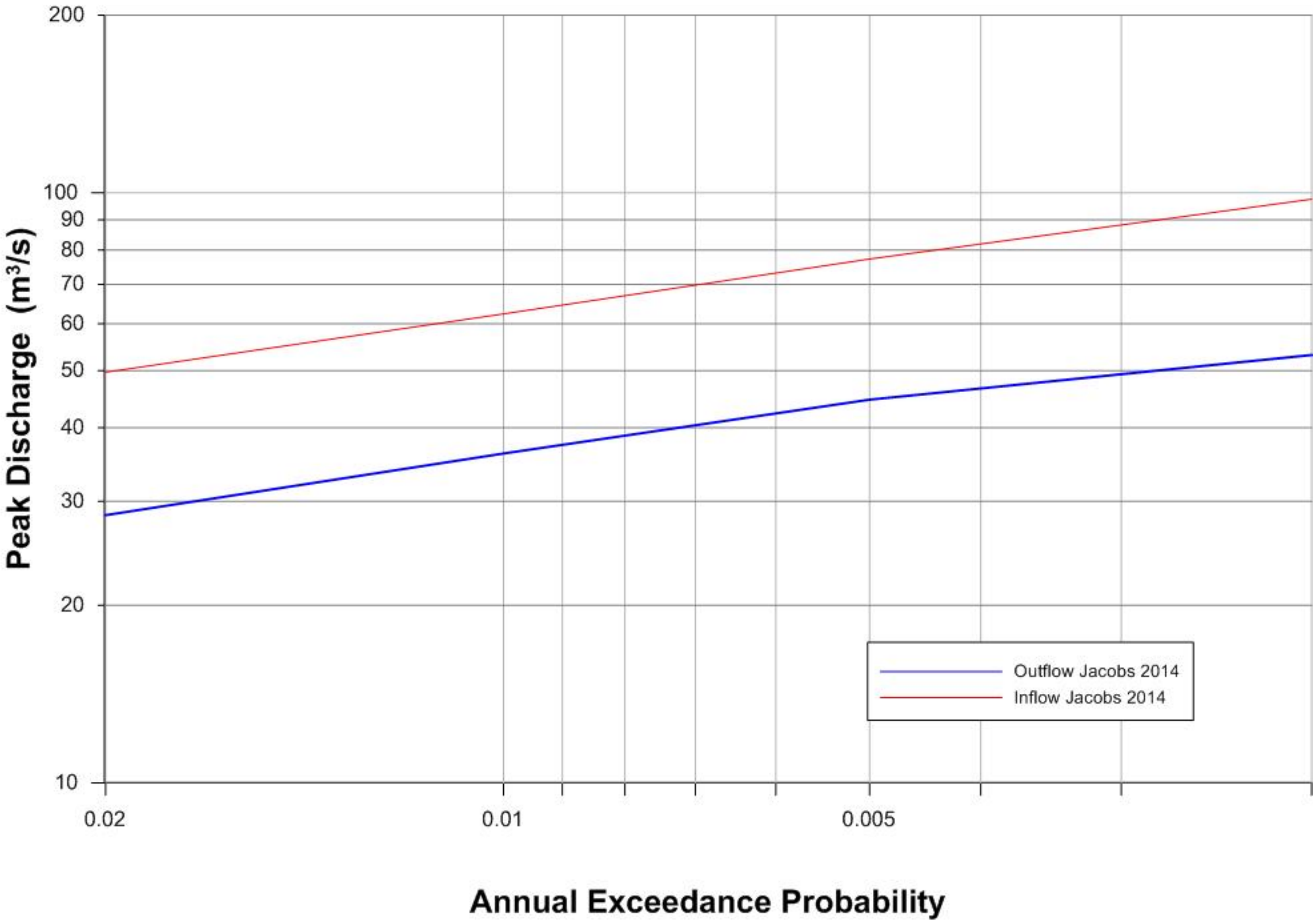


Figure 7-1 : Peak flow frequency curves for Yerrabi Dam

The estimated 1 in 50 AEP flow was 28 m^3/s and the 1 in 100 AEP outflow was estimated to be 36 m^3/s. Corresponding peak levels and associated AEP frequencies for Yerrabi dam are shown in Figure 7-2.

Table 7-1 : Summary table of design peak outlfow results for Yerrabi Dam

AEP (1 in Y)	Outflow (m^3/s)	Critical Duration (hrs)	Peak Level (mAHD)
50	28	6 hr	612.94
100	36	6 hr	613.12

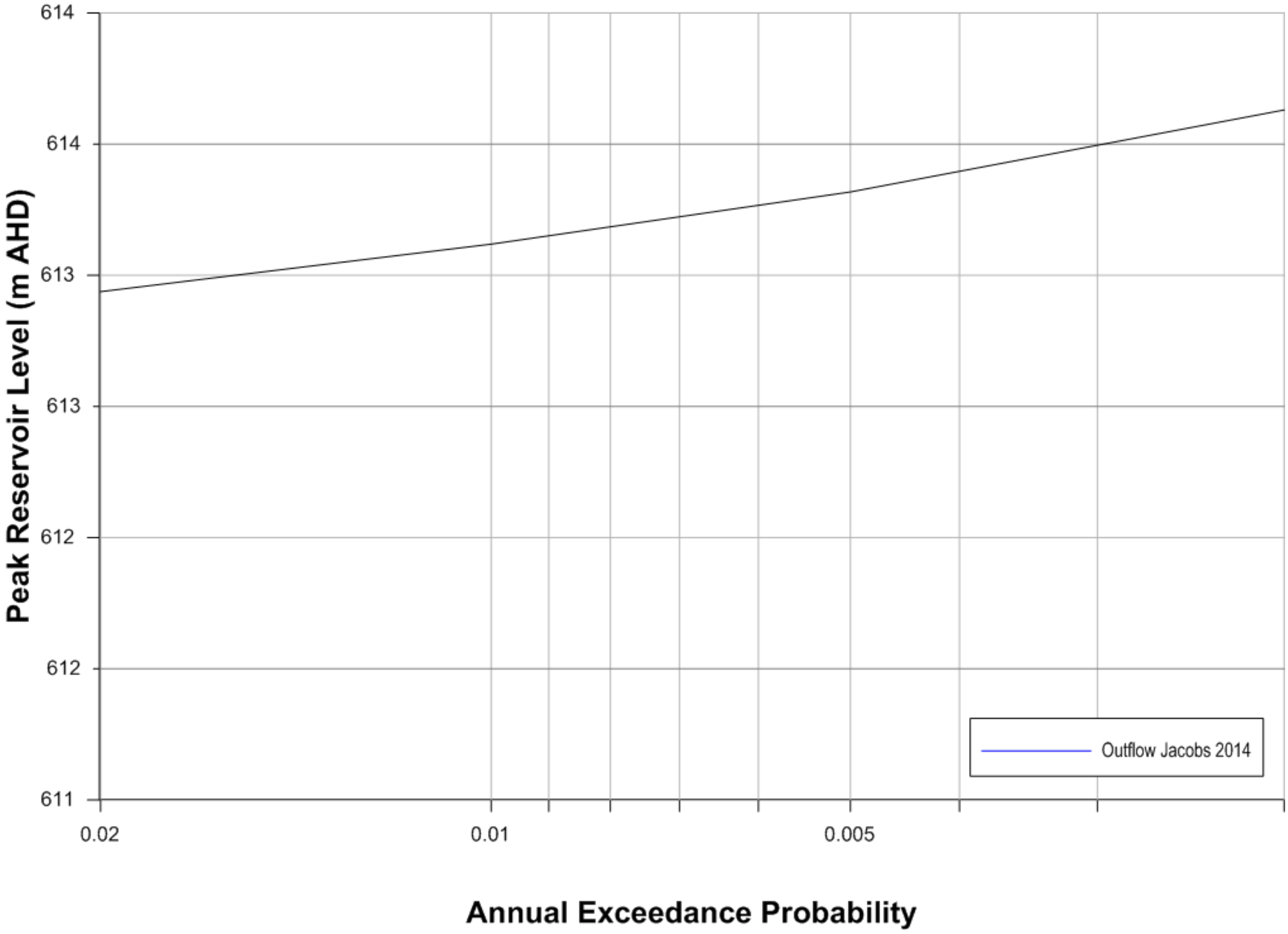


Figure 7-2 : Peak reservoir level frequency curve for Yerrabi Dam

7.1.2 Gungahlin Dam

Figure 7-3 presents the peak discharges for Gungahlin Dam, which are also tabulated in Table 7-2.

The current estimate of peak outflow for Gungahlin dam for the 1 in 50 AEP design flood event was estimated to be 58 m^3/s and 72 m^3/s for the 1 in 100 AEP event.

Figure 7-4 displays the estimates of reservoir levels resulting from peak outflows.

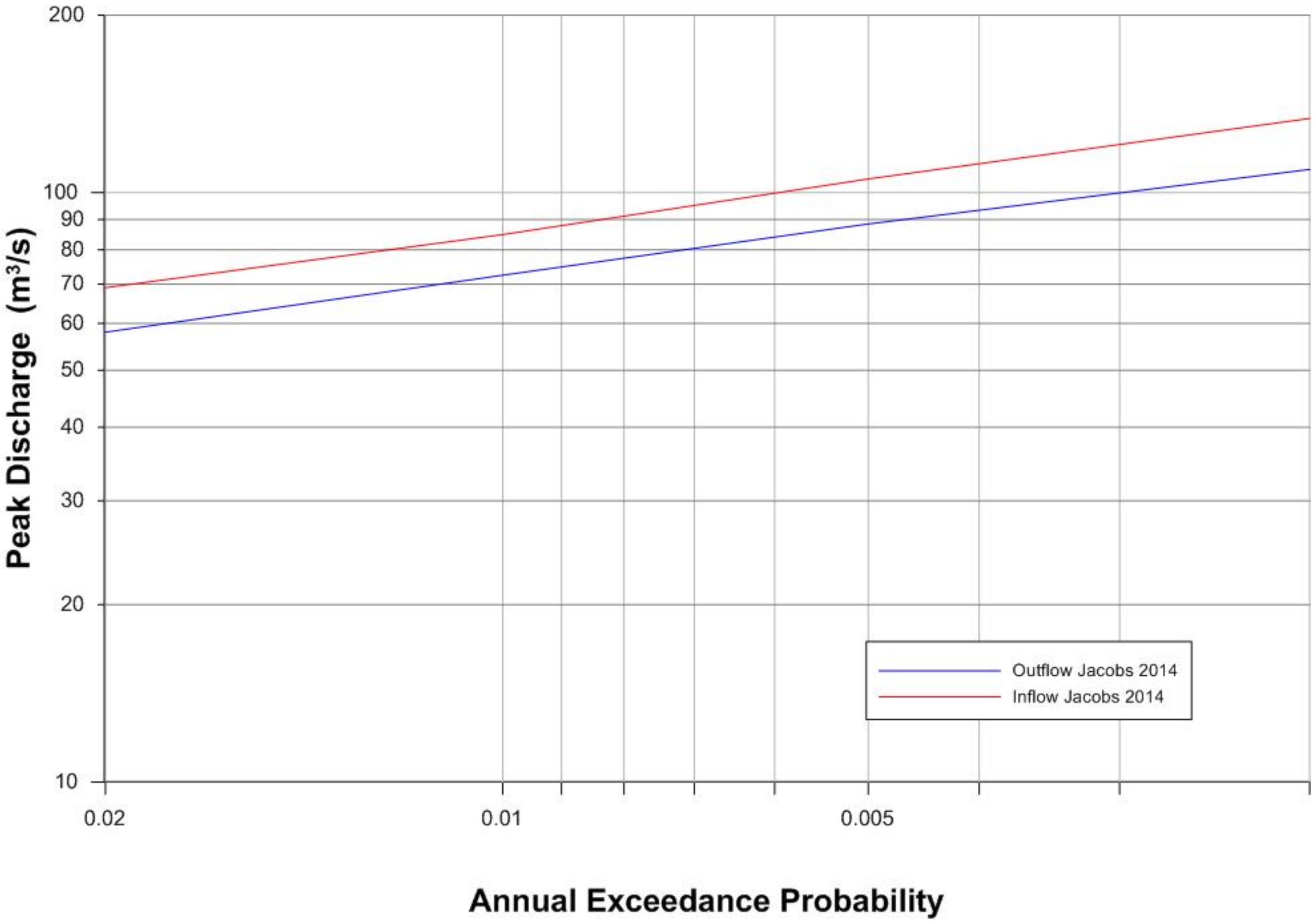


Figure 7-3 : Peak frequency curves for Gungahlin Dam\

Table 7-2 : Summary table of design peak outlfow results for Gungahlin Dam

AEP (1 in Y)	Outflow (m^3/s)	Critical Duration (hrs)	Peak level (mAHD)
50	58	12 hr	597.57
100	72	12 hr	597.79

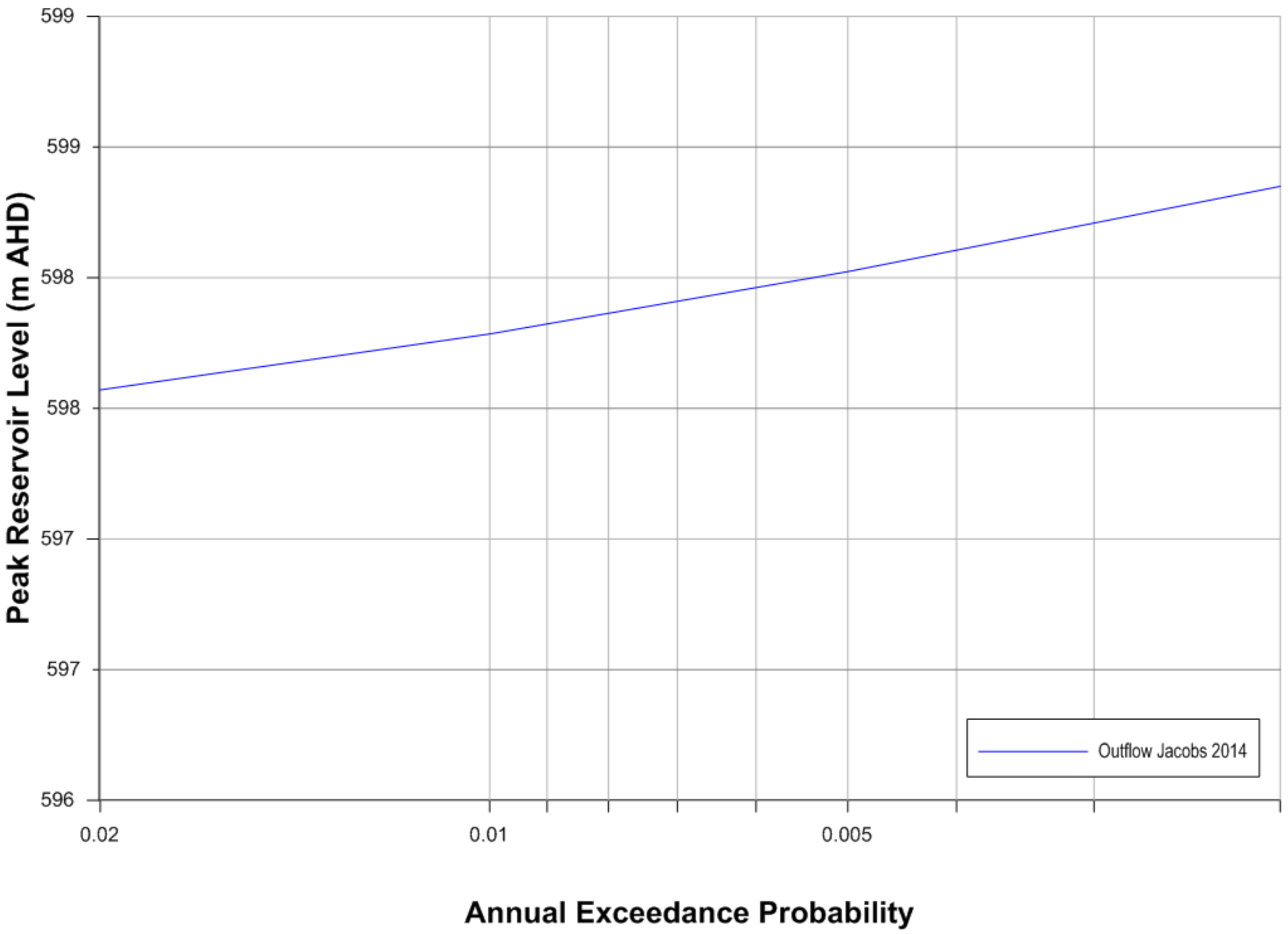


Figure 7-4 : Peak reservoir level frequency curves for Gungahlin Dam

7.1.3 Ginninderra Dam

Figure 7-5 (tabulated in Table 7-3) presents the resultant outflows for Ginninderra Dam. The 1 in 50 AEP estimate of peak outflow from Ginninderra Dam is 76 m^3/s and the 1 in 100 AEP estimate is 99 m^3/s.

Given the results achieved through the calibration (Section 4.2) of the carefully selected model parameters used in this current report confidence is given to the estimates of peak discharge for Ginninderra (and the other dams) at these higher frequency design events. Further confidence is afforded to the current results given the verification of design losses achieved also (as shown in Figure 6-3 in Section 6.2).

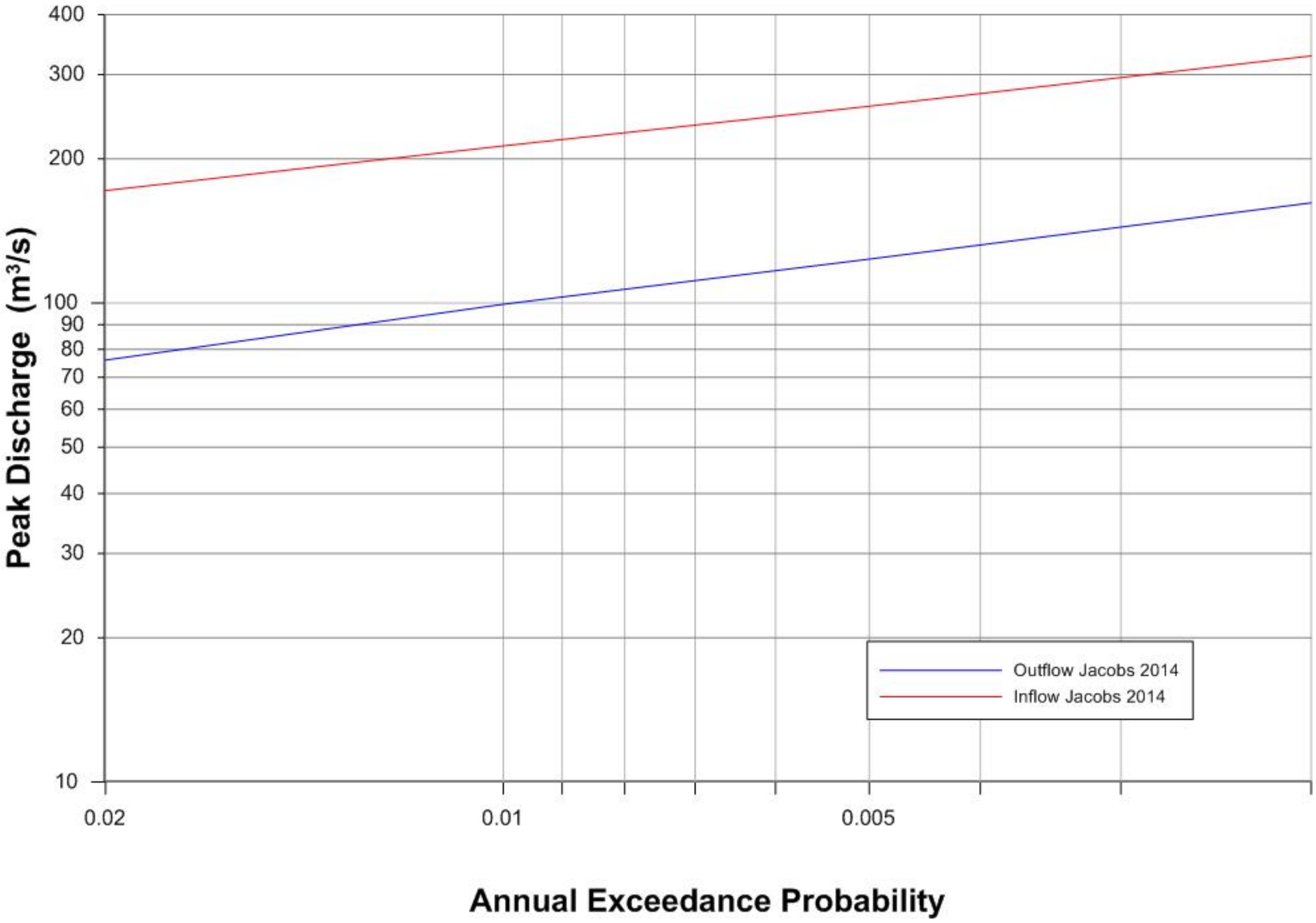


Figure 7-5 : Peak frequency curves for Ginninderra Dam

Table 7-3 : Summary table of design peak outlfow results for Ginninderra Dam

AEP (1 in Y)	Outflow (m^3/s)	Critical Duration (hrs)	Peak level (mAHD)
50	76	12 hr	578.17
100	99	12 hr	578.34

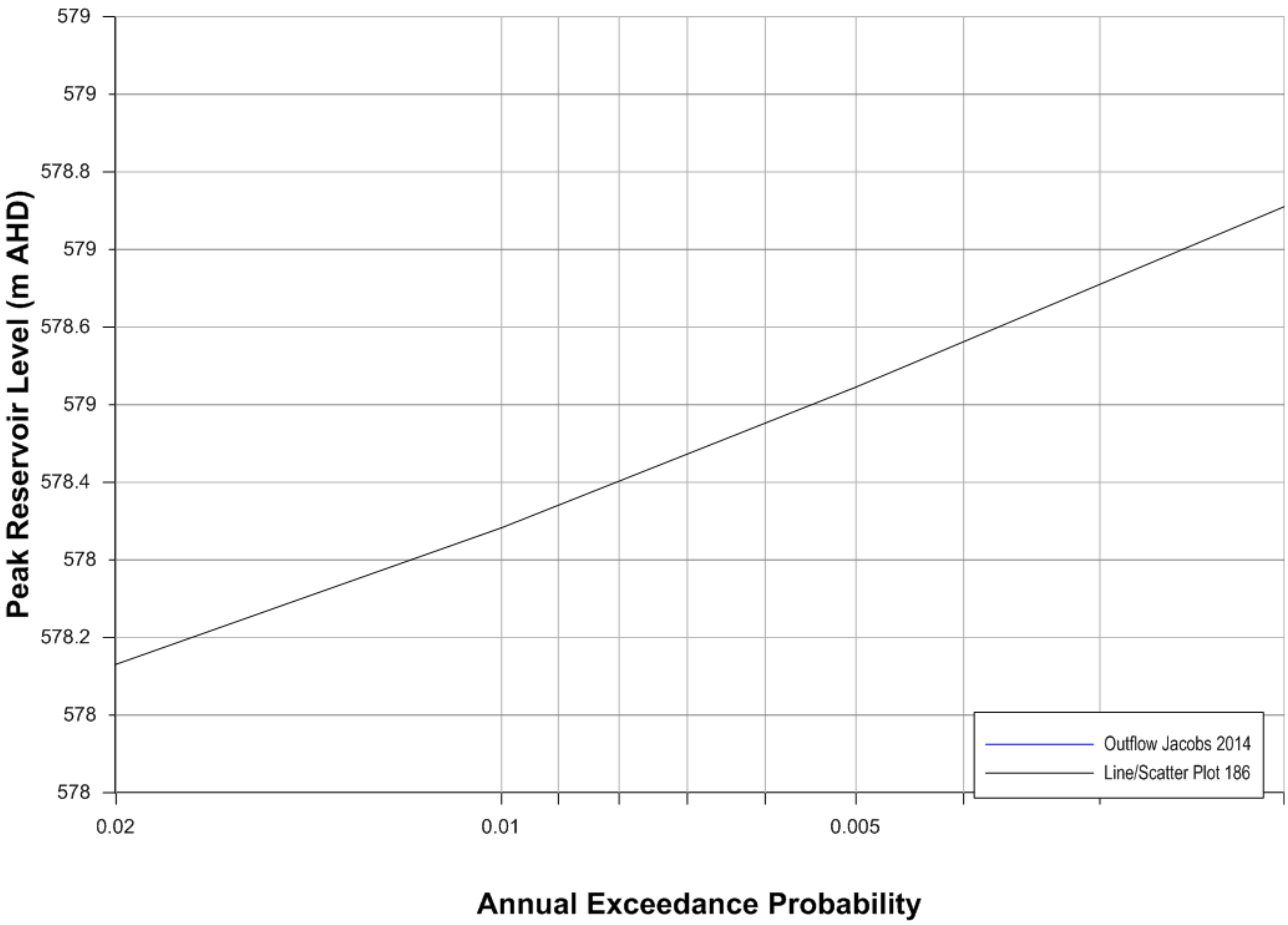


Figure 7.6 : Peak reservoir level frequency curves for Ginninderra Dam

8. Hydraulic Modelling

This section of the report describes the development of the hydraulic model developed for the purposes of assessing the constructing inundation maps for the Ginninderra Creek catchment downstream of Yerrabi Dam. The decisions and assumptions made during the model development process are outlined along with the uncertainties of the work undertaken as part of this study.

To fulfil the requirements of other components of the wider dambreak modelling project associated with Ginninderra Creek, a range of dambreak scenarios were modelled. Since three dams are situated on Ginninderra Creek in series, these scenarios included a range of events and the possibility of downstream cascade failure.

The scenarios modelled involved investigating the resultant floods generated by both very infrequent events, such as the Probable Maximum Flood (PMF), failures not resulting from a flood event (sunny day failure) and the more frequent events that are the focus of this current report. In general, consistent methods were applied to develop these different events. This approach is described here with a focus on the 1 in 50 and 1 in 100 AEP events.

Detailed descriptions of the PMF and dam failure scenarios are not provided in this report, but can be found in the more detailed report on the full study (Jacobs,2014).

8.1 Model approach

The most recent detailed hydraulic modelling for the Ginninderra Creek catchment and associated dams was completed in 2002 by Ecowise. That work was undertaken using a model originally developed in 1999, which was enhanced by incorporating additional survey data collected in 2001 at 13 locations within the catchment.

Since that previous hydraulic modelling was undertaken, significant catchment development has occurred particularly in the reaches downstream of Lake Ginninderra that should now be represented in the model. There is also an area of proposed development around the West Belconnen area. In addition, LiDAR data is available over much of the catchment which provides high quality information on the catchment topography that can be directly used to develop the hydraulic model. This provides an alternative approach that does not involve field based survey campaigns, yet provides high resolution data across a large area (not limited to the surveyed cross sections). As such, a new hydraulic model was developed for this study.

The MIKE modelling software suite developed by DHI was used for this project, specifically the MIKE 11 component. This 1-dimensional river model was used to represent the main Ginninderra Creek channel. The key assumption for a 1-dimensional hydraulic model is that flow occurs predominantly in one defined direction along the channel, and that the effect of lateral flow can be neglected. Given the topography of the catchment, this was considered a reasonable approach as the channel and flood path is well defined.

The focus of the hydraulic modelling was on the main reach of the river and therefore the drainage network has not been explicitly modelled.

High resolution LiDAR data is available over most, but not all, of the catchment area. This LiDAR was used to generate a 1m resolution Digital Elevation Model (DEM) covering Yerrabi Dam downstream to Gooromon Ponds Creek. The readily available national 1 second DEM was used to provide information on the reaches downstream of Gooromon Ponds to Ginninderra Falls. This coarser data was applied to a reach length of approximately 3.5 km of the total 20 km of Ginninderra Creek represented in the hydraulic model. Cross sections were developed for the MIKE 11 model using these available DEM data sets. The cross sections were spaced at appropriate distances along the creek to ensure key topographic and flow path characteristics were represented. This resulted in cross section intervals between 300 m and 700 m throughout the catchment.

8.2 Hydrologic inputs to hydraulic model

Inflows into the hydraulic model for the 1in 50 and 1 in 100 AEPs for Ginninderra Dam were derived through the hydrologic component of this study. The approach to derive these inputs is described in detail in Sections 3 to 7 of this document. This section of the report describes the modelling of concurrent inflows into Ginninderra Creek downstream of each of the storages.

The RORB model applied in this study to undertake the hydrologic modelling for each of the storages was developed to capture the entire Ginninderra Creek catchment to Ginninderra Falls. This includes representation of all major tributaries and areas downstream of the dams, as shown in Figure 4-1.

The RORB model and associated inputs (as described earlier) were used to simulate the coincident flooding in all downstream reaches and tributaries, with hydrographs extracted at relevant locations, such as Gungaderra Creek, Gooromon Ponds Creek and other small tributaries, to provide information for the dambreak modelling. This approach ensures that consistent model inputs (such as rainfall depth, spatial and temporal patterns) associated with each of the design events are used to generate the coincident flooding hydrographs. The spatial patterns associated with each of the Yerrabi, Gungahlin and Ginninderra events are shown in Figure 5-4, Figure 5-5 and Figure 5-6. As shown in these figures, the patterns were extended over the downstream catchments. This information was used as direct input into the RORB model to generate runoff hydrographs for each of the individual design rainfall events to represent the downstream catchments.

8.3 Hydraulic roughness

The Manning's n roughness coefficient reflects the influence of bank and bed materials, channel obstructions, irregularity of the riverbanks and vegetation. Typically for hydraulic applications, Manning's n roughness coefficients are determined through model calibration to known water levels. When this is not possible, values are selected from available references.

Manning's n roughness coefficients have mostly been developed through observing floods originating from rainfall. As any flood progresses, the hydraulic roughness may vary. The Manning's n values have been chosen to provide a reasonable estimate of roughness and thus water level at the peak of the event, based on the different land uses within the catchment. Given that the focus of the original study was on extreme events, the model was not calibrated. A Manning's n of 0.06 has been applied to the creek and floodplain, while the urban areas are represented with a greater roughness n value of 0.1. These values are consistent with those suggested by US Army Corps of Engineers (2010).

8.4 Boundary conditions

At the downstream end of the model, Ginninderra Creek flows over Ginninderra falls. This was taken as the downstream boundary of the model, represented with a fixed water level at the downstream face of the falls. As the falls are located downstream of all other areas of interest, the boundary conditions do not significantly impact on the model results at upstream locations.

8.5 Representation of reservoirs and dams

There are three main aspects of the dams that need to be represented in the hydraulic model: the dam failure events (not covered in this report), the outflow through the spillways, and the storage capacity of the reservoirs.

The outflows through the spillways were represented as tabulated structures in MIKE 11. A tabulated structure is a user-defined relationship between upstream water level, downstream water level and discharge. The rating used in the tabulated structure for each storage is consistent with that applied in the RORB hydrologic model as described in Section 2.

The storage capacities of the three reservoirs were modelled slightly differently. For Yerrabi Dam, the reservoir capacity was represented in a single MIKE 11 cross section. For Gungahlin and Ginninderra Dams, the effect of flood routing through the storage must be simulated in the hydraulic model. Therefore, the capacity of these storages was represented by a series of MIKE 11 cross sections along the length of the reservoir. The additional storage area functionality was used to correct the total storage to the known reservoir volume.

Yerrabi, Gungahlin and Ginninderra Dams were all assumed to be at FSL at the start of each scenario, consistent with the operational management of these storages.

8.6 Representation of bridges

Several bridges are located along Ginninderra Creek and tributaries. Where these were considered to have the potential to influence on a dambreak flood they were explicitly modelled in MIKE 11. The following bridge structures were incorporated in the model:

- Mirrabei Drive
- Gungahlin Drive bridges over Gininderra Creek and tributaries
- Barton Highway bridges (north and southbound)
- William Slim Drive
- Baldwin Drive
- Four crossings of Ginninderra Drive (north and southbound)
- Copland Drive
- Kingsford Smith Drive (north and southbound)
- Florey Drive
- Osburn Drive

8.7 Hydraulic modelling results

The hydraulic modelling results are summarised in the following sections for the two scenarios. Note that the values are representative of peak conditions in the channel. Peak depths are reported relative to the invert level. The time to peak is relative to the commencement of flooding.

8.7.1 Composite 1 in 50 AEP event

A nominal 1 in 50 AEP event has been generated using a composite of the 1 in 50 AEP results for each of the Yerrabi, Gungahlin and Ginninderra dam model scenarios. This approach utilises the appropriate design inputs for the relevant reaches of the waterway, maintaining consistency with the design hydrology. This ensures that the unique assumptions around storm location for each of the three dam catchments are incorporated. However, it should be noted that the individual 1 in 50 AEP events were of different critical durations for the three storages. It should also important to be aware that the hydrologic and hydraulic inputs to the models were derived with a focus on larger, more extreme floods as part of a dambreak analysis. Therefore the hydraulic model is structured to capture these larger events accurately and consequently, there may be limitations in using this type of model for more frequent flood events such as the 1 in 50 AEP. As such, this event should be represented as indicative.

The longitudinal section showing peak water depth for the composite 1 in 50 AEP event is presented in Figure 8-1. Table 8-1 presents the model results for this scenario, showing the peak water level, depth, discharge, velocity and time to peak. For this scenario, the time to peak is reported relative to the start of the inflows to the dam of the relevant scenario.

The 1 in 50 AEP event is expected to overtop the spillway for each of the three dams. At Yerrabi and Ginninderra, it is estimated that the peak water level would be more than 5 m below the embankment, at Gungahlin the peak water level is more than 6 m below the crest.

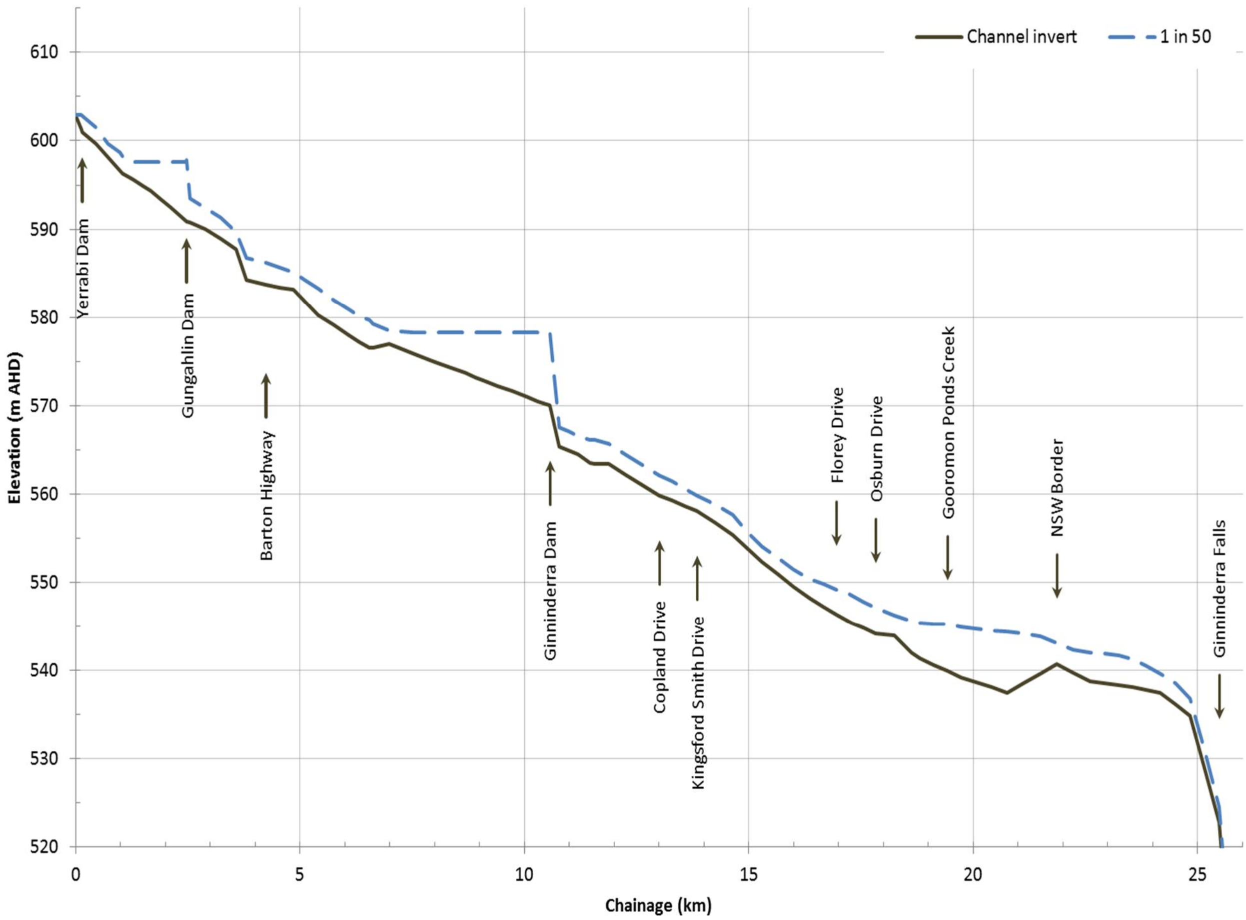


Figure 8-1 : Peak water levels downstream of Yerrabi Dam associated with the composite 1 in 50 event

Table 8-1 : Composite 1 in 50 AEP flood modelling results

Location	Distance from Yerrabi Dam (km)	Peak water level (m AHD)	Peak depth (m)	Peak discharge (m³/s)	Peak mean channel velocity (m²/s)	Time to peak (h)
Yerrabi Dam	0.0	612.9	0.7 [1]	30	0.0 [2]	3:29
Gungahlin Dam	2.5	597.8	1.6 [1]	70	0.9 [2]	9:19
Barton Highway	4.2	586.2	2.5	80	1.0	9:19
Ginninderra Dam	10.6	578.2	0.9 [1]	90	0.8[2]	10:55
Copland Drive	13.0	562.1	2.3	80	0.8	15:29
Kingsford Smith Drive	13.8	559.8	1.7	80	0.8	13:59
Florey Drive	17.0	549.2	2.9	100	0.9	13:55

Location	Distance from Yerrabi Dam (km)	Peak water level (m AHD)	Peak depth (m)	Peak discharge (m^3/s)	Peak mean channel velocity (m^2/s)	Time to peak (h)
Osburn Drive	17.8	547.2	3.0	100	1.1	12:55
Gooromon Ponds Creek	19.4	545.3	5.3	180	0.8	14:05
NSW border	21.9	543.2	2.4	340	1.3	15:05
Ginninderra Falls	25.5	524.5	1.7	340	4.5	15:35

[1, 2] Depth and velocity over spillway.

8.7.2 Composite 1 in 100 AEP event

As described above for the 1 in 50 AEP results, a nominal 1 in 100 AEP event has been generated using a composite of the 1 in 100 AEP results for each of the Yerrabi, Gungahlin and Ginninderra dam model scenarios. This approach utilises the appropriate design inputs for the relevant reaches of the waterway, maintaining consistency with the design hydrology. This ensures that the unique assumptions around storm location for each of the three dam catchments are incorporated. However, it should be noted that the individual 1 in 100 AEP events were of different critical durations for the three storages. As with the 1 in 50 AEP event, care should be taken in interpreting the results of the 1 in 100 AEP event. The hydrologic and hydraulic inputs and structure of the model were derived using methods that are focussed on understanding and assessing the consequences of much larger, more extreme floods. As such, the results of this event should be represented as indicative

The longitudinal section showing peak water depth for the composite 1 in 100 AEP event is presented in Figure 8-2. Table 8-2 presents the model results for this scenario, showing the peak water level, depth, discharge, velocity and time to peak. For this scenario, the time to peak is reported relative to the start of the inflows to the dam of the relevant scenario.

The 1 in 100 AEP event is expected to overtop the spillway for each of the three dams. At Yerrabi, it is estimated that the peak water level would be more than 4.7 m below the embankment, at Gungahlin the peak water level is 6 m below the crest and at Ginninderra Dam the peak water level is 5.5m below the crest.

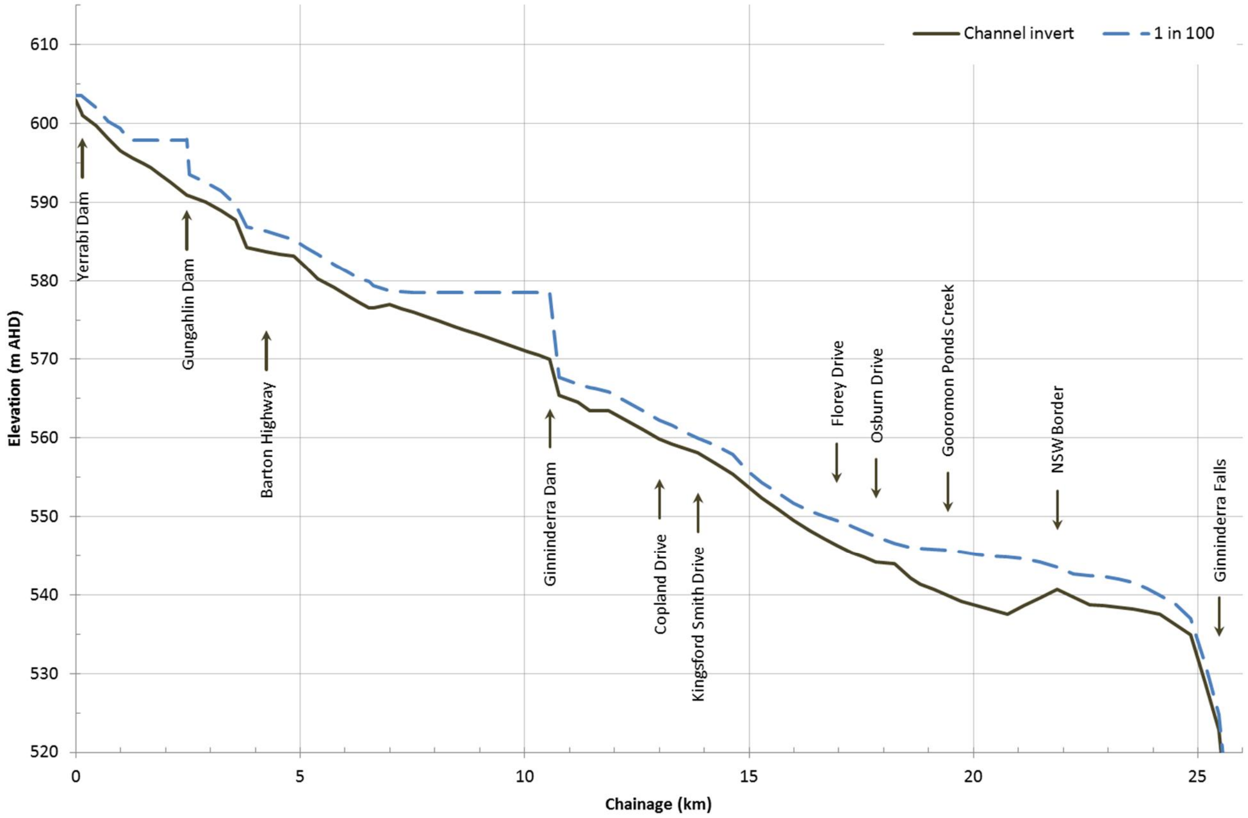


Figure 8-2 : Peak water levels downstream of Yerrabi Dam associated with the composite 1 in 100 AEP event

Table 8-2 : Ginninderra Dam 1 in 100 AEP flood modelling results

Location	Distance from Yerrabi Dam (km)	Peak water level (m AHD)	Peak depth (m)	Peak discharge (m^3/s)	Peak mean channel velocity (m^2/s)	Time to peak (h)
Yerrabi Dam	0.0	613.5	1.3 [1]	50	0.1 [2]	4:15
Gungahlin Dam	2.5	598.0	1.8 [1]	90	0.9 [2]	9:19
Barton Highway	4.2	586.3	2.6	90	1.0	9:15
Ginninderra Dam	10.6	578.3	1.1 [1]	110	0.8 [2]	10:30
Copland Drive	13.0	562.3	2.5	100	0.8	14:29
Kingsford Smith Drive	13.8	560.0	1.9	100	0.8	13:29
Florey Drive	17.0	549.5	3.2	130	0.9	13:25
Osburn Drive	17.8	547.5	3.3	130	1.1	13:14
Gooromon Ponds Creek	19.4	545.7	5.8	230	0.7	14:10
NSW border	21.9	543.5	2.7	430	1.3	14:59
Ginninderra Falls	25.5	524.7	1.9	430	4.3	15:25

[1, 2] Depth and velocity over spillway.

8.8 Inundation maps

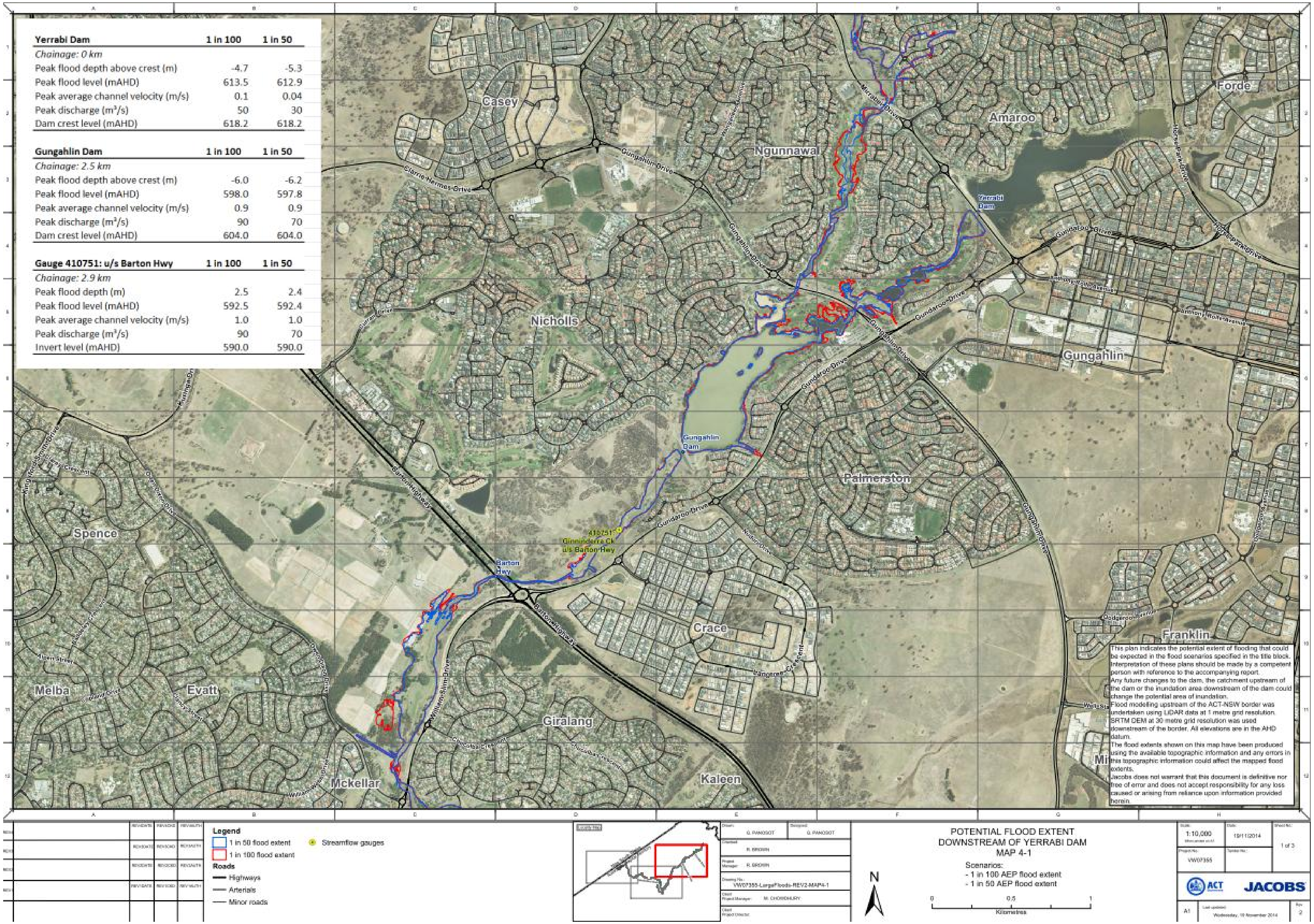
Inundation maps were generated for the 1 in 50 and 1 in 100 AEP events using the hydraulic model outputs. Figure 8.3 presents these inundation maps for the reaches from Yerrabi Dam to Ginninderra Falls.

JACOBS

Yerrabi Dam	1 in 100	1 in 50
Chainage: 0 km		
Peak flood depth above crest (m)	-4.7	-5.3
Peak flood level (mAHD)	613.5	612.9
Peak average channel velocity (m/s)	0.1	0.04
Peak discharge (m^3/s)	50	30
Dam crest level (mAHD)	618.2	618.2

Gungahlin Dam	1 in 100	1 in 50
Chainage: 2.5 km		
Peak flood depth above crest (m)	-6.0	-6.2
Peak flood level (mAHD)	598.0	597.8
Peak average channel velocity (m/s)	0.9	0.9
Peak discharge (m^3/s)	90	70
Dam crest level (mAHD)	604.0	604.0

Gauge 410751: u/s Barton Hwy	1 in 100	1 in 50
Chainage: 2.9 km		
Peak flood depth (m)	2.5	2.4
Peak flood level (mAHD)	592.5	592.4
Peak average channel velocity (m/s)	1.0	1.0
Peak discharge (m^3/s)	90	70
Invert level (mAHD)	590.0	590.0

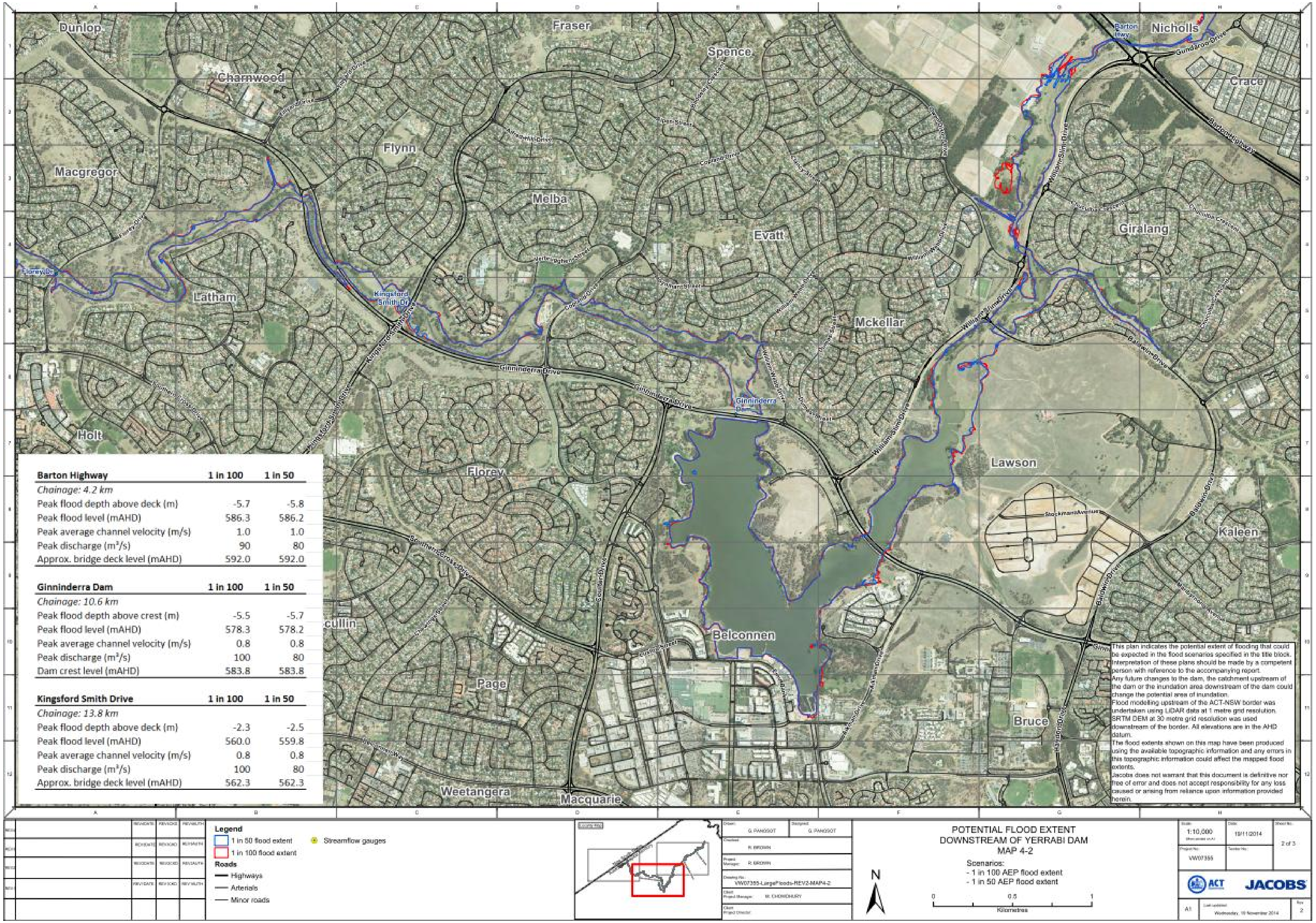


This plan indicates the potential extent of flooding that could be expected in the flood scenarios specified in the title block. Interpretation of these plans should be made by a competent person with reference to the accompanying report.
Any future changes to the dam, the catchment upstream of the dam or the inundation area downstream of the dam could change the potential area of inundation.
Flood modelling upstream of the ACT-NSW border was undertaken using LIDAR data at 1 metre grid resolution. SRTM DEM at 30 metre grid resolution was used downstream of the border. All elevations are in the AHD datum.
The flood extents shown on this map have been produced using the available topographic information and any errors in this topographic information could affect the mapped flood extents.
Jacobs does not warrant that this document is definitive nor free of error and does not accept responsibility for any loss caused or arising from reliance upon information provided herein.

POTENTIAL FLOOD EXTENT
DOWNSTREAM OF YERRABI DAM
MAP 4-1

Scenarios:
- 1 in 100 AEP flood extent
- 1 in 50 AEP flood extent

Drawn: G. PANOSOT | Designed: G. PANOSOT | Checked: R. BROWN | Project Manager: R. BROWN
Drawing No.: VW07355-LargeFloods-REV2-MAP4-1 | Client Project Manager: M. CHOWDHURY
Scale: 1:10,000 | Date: 19/11/2014 | Sheet No.: 1 of 3 | Project No.: VW07355
ACT JACOBS
A1 | Last updated: Wednesday, 19 November 2014 | Rev: 2



Barton Highway	1 in 100	1 in 50
Chainage: 4.2 km		
Peak flood depth above deck (m)	-5.7	-5.8
Peak flood level (mAHD)	586.3	586.2
Peak average channel velocity (m/s)	1.0	1.0
Peak discharge (m³/s)	90	80
Approx. bridge deck level (mAHD)	592.0	592.0

Ginninderra Dam	1 in 100	1 in 50
Chainage: 10.6 km		
Peak flood depth above crest (m)	-5.5	-5.7
Peak flood level (mAHD)	578.3	578.2
Peak average channel velocity (m/s)	0.8	0.8
Peak discharge (m³/s)	100	80
Dam crest level (mAHD)	583.8	583.8

Kingsford Smith Drive	1 in 100	1 in 50
Chainage: 13.8 km		
Peak flood depth above deck (m)	-2.3	-2.5
Peak flood level (mAHD)	560.0	559.8
Peak average channel velocity (m/s)	0.8	0.8
Peak discharge (m³/s)	100	80
Approx. bridge deck level (mAHD)	562.3	562.3

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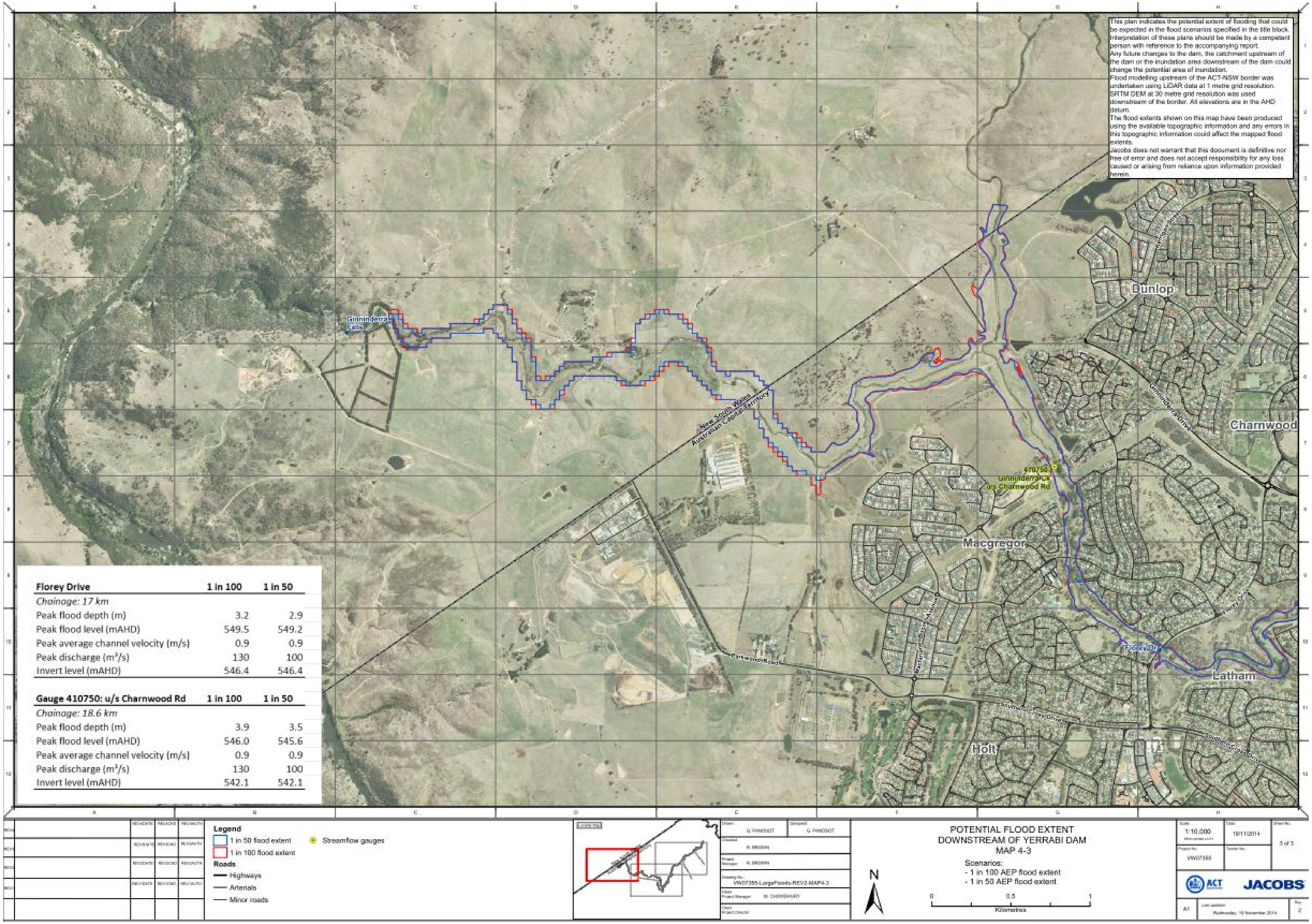

Florey Drive	1 in 100	1 in 50
Chainage: 17 km		
Peak flood depth (m)	3.2	2.9
Peak flood level (mAHD)	549.5	549.2
Peak average channel velocity (m/s)	0.9	0.9
Peak discharge (m^3/s)	130	100
Invert level (mAHD)	546.4	546.4

Gauge 410750: u/s Charnwood Rd	1 in 100	1 in 50
Chainage: 18.6 km		
Peak flood depth (m)	3.9	3.5
Peak flood level (mAHD)	546.0	545.6
Peak average channel velocity (m/s)	0.9	0.9
Peak discharge (m^3/s)	130	100
Invert level (mAHD)	542.1	542.1

9. Flood Risk Management in NSW

The NSW Government's Flood Policy is directed at providing solutions to existing flooding problems in developed areas and to ensuring that new development is compatible with the flood hazard and does not create additional flooding problems in other areas. Policy and practice are defined in the Government's Floodplain Development Manual (2005).

Under the Policy the management of flood liable land remains the responsibility of Local Government. The NSW Government subsidises flood mitigation works to alleviate existing problems and provides specialist technical advice to assist Councils in the discharge of their floodplain management responsibilities.

The policy provides for a floodplain management system comprising the following five sequential stages:

1. Data Collection	Involves compilation of existing data and collection of additional data
2. Flood Study	Determines the nature and extent of the flood problem
3. Floodplain Risk Management Study	Evaluates management options in consideration of social, ecological and economic factors relating to flood risk with respect to both existing and future development
4. Floodplain Risk Management Plan	Involves formal adoption by Council of a plan of management for the floodplain
5. Implementation of the Plan	Implementation of flood, response and property modification measures (including mitigation works, planning controls, flood warnings, flood preparedness, environmental rehabilitation, ongoing data collection and monitoring by Council

This report describes the first two stages of the floodplain risk management process for Ginninderra Creek which includes data collection and flood study. In particular, this document focusses on the data collected and the flood study outcomes for the 1 in 50 and 1 in 100 AEP events for Ginninderra Creek. It is to be noted however, that these results were generated through a more comprehensive assessment of the full range of flood events, up to and including the PMF. The flood impacts associated with potential dambreak events for each of Yerrabi, Gungahlin and Ginninderra Dams were also assessed. The full details of the results from the flood hydrology, dambreak and consequence assessments are reported in a separate report (Jacobs, 2014).

The outputs reported in this document provide the necessary information for primary end users to support the floodplain risk management process. This report provides the information required to enable council to consider the hydraulic and hazard categorisation for the events of interest, and provides flood level information in various formats. In addition, this report provides a summary of the flood behaviour along Ginninderra Creek, flood levels and flood extent maps to support emergency planning by the SES. These activities will be undertaken by the relevant specialist practitioners in consultation with Council as a part of subsequent steps in the planning process.

10. Conclusions

This project documents the work that has been undertaken to estimate the 1 in 50 and 1 in 100 AEP flood extents for Ginninderra Creek. The inputs to the hydraulic model and the parameters used to model the two scenarios examined this report were derived as part of the study to update the hydrology, dambreak analysis and consequence assessment of the three dams along Ginninderra Creek.

The results from the dambreak modelling have been used as the basis for calculating the flood extents for the 1 in 50 and 1 in 100 AEP flood events. Given that the inputs to the model and parameters that were used in the hydraulic modelling for the main channel of Ginninderra Creek were derived using methods that were focussed on understanding the nature of more extreme events, the results of the 1 in 50 and 1 in 100 AEP events should be viewed as indicative.

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Appendix A. **Rainfall and Streamflow Gauges**

Table A.11-1 : Rainfall and Streamflow gauges used for three calibration events

Daily Gauge Number	**Daily Gauge Name**	**Pluvio Number**	**Pluvio Name**
570813	Turner at Barry Drive	570904	Horse Park at Gundaroo Rd
570987	Stormwater Drain at Giralang	570987	Giralang
570989	Ginninderra Trib. at Gungahlin Catchment	570989	Gungahlin Catchment Outlet
570990	Gungahlin Catchment East	570990	Gungahlin East
570991	Barton Highway	570991	Barton Highway
7000	Ainslie Tyson St	570992	Gungahlin West
70045	Hall (Lochleigh)	570994	Giralang East
70169	Ginninderra CSIRO	570908	Ginninderra Ck at Charnwood
70242	Aranda (Bindaga St)	570996	Lake Ginninderra Dam
70247	Canberra Botanic Gardens	**Streamflow Gauge Number**	**Streamflow Gauge Name**
70250	Scullin (Broadsmith Street)	410750	Ginninderra Ck U/S Charnwood Rd
70253	O'Connor (Belconnen Way)	410751	Ginninderra Ck U/S Barton Hwy
70255	Mitchell (Exhibition Park)		
70275	Macquarie Bennelong Crescent		
70277	Melba Verbrugghen Street		
70307	Bruce (Australian Institute of Sport)		
70340	Gungahlin Lakes		

Appendix B. **Design Rainfall Depths**

Table B.1 : Yerrabi Dam catchment average design rainfall depths (in mm) for varying event durations and AEP

AEP (1 in Y)	1 hour	2 hour	3 hour	6 hour	12 hour	18 hour	24 hour	36 hour	48 hour	72 hour
20	33.1	42.8	49.3	62.8	80.2	92.5	102.4	117.0	127.2	139.6
50	39.8	51.2	59.0	74.8	95.4	110.2	122.3	140.2	152.6	167.5
100	45.7	58.9	67.7	85.9	108.3	124.3	137.0	156.6	170.2	186.4
200	52.1	67.1	77.2	98.0	121.8	138.9	151.9	173.0	187.6	205.1

Table B.2: Gungahlin Dam catchment average design rainfall depths (in mm) for varying event durations and AEP

AEP (1 in Y)	1 hour	2 hour	3 hour	6 hour	12 hour	18 hour	24 hour	36 hour	48 hour	72 hour
20	31.4	41.0	47.5	60.9	78.3	90.7	100.5	115.3	125.7	138.5
50	37.8	49.1	56.8	72.5	93.1	108.1	120.0	138.2	150.9	166.1
100	43.5	56.5	65.2	83.3	107.0	121.2	134.5	154.4	168.3	184.9
200	49.6	64.4	74.4	95.0	122.0	134.4	149.1	170.7	185.6	203.5

Table B.3: Ginninderra Dam catchment average design rainfall depths (in mm) for varying event durations and AEP

AEP (1 in Y)	1 hour	2 hour	3 hour	6 hour	12 hour	18 hour	24 hour	36 hour	48 hour	72 hour
20	29.9	39.3	45.8	59.1	76.6	88.5	98.1	113.4	124.1	136.6
50	36.0	47.1	54.7	70.5	91.2	105.2	117.0	135.7	149.0	165.1
100	41.3	54.1	62.9	81.0	104.8	119.2	132.4	152.5	166.8	184.5
200	47.1	61.7	71.7	92.3	119.5	132.3	146.8	168.6	183.9	203.1

Appendix C. Summary of Modelling Parameters

C.1 RORB model parameters

Table C.1 : Adopted design flood model parameters

Interstation area	k_c	m	Initial Loss (IL) (mm)	Continuing Loss (CL) (mm)
Yerrabi Outflow	9.2	0.8	30.0	4.0
Gungahlin Outflow	11.1	0.8	30.0	3.0
Gauge 410751	1.5	0.8	30.0	4.0
Ginninderra Outflow	15.3	0.8	30.0	5.0
Gauge 410750	10.0	0.8	30.0	4.0

Table C.2 : Adopted PMF model parameters

Interstation area	k_c	m	Initial Loss (IL) (mm)	Continuing Loss (CL) (mm)
Yerrabi Outflow	9.2	0.8	0	1
Gungahlin Outflow	11.1	0.8	0	1
Gauge 410751	1.5	0.8	0	1
Ginninderra Outflow	15.3	0.8	0	1
Gauge 410750	10.0	0.8	0	1

C.2 Hydraulic model parameters

C.2.1 Hydraulic roughness parameter

A Manning's n of 0.06 has been applied to the creek and floodplain, while the urban areas are represented with a greater roughness n value of 0.1.

C.2.2 Boundary Conditions

Ginninderra falls was taken as the downstream boundary of the model, represented with a fixed water level at the downstream face of the falls. As the falls are located downstream of all other areas of interest, the boundary conditions do not significantly impact on the model results at upstream locations.

C.2.3 Representation of reservoirs

The storage capacities of the three reservoirs were modelled slightly differently. For Yerrabi Dam, the reservoir capacity was represented in a single MIKE 11 cross section. For Gungahlin and Ginninderra Dams, the effect of flood routing through the storage must be simulated in the hydraulic model. Therefore, the capacity of these storages was represented by a series of MIKE 11 cross sections along the length of the reservoir. The additional storage area functionality was used to correct the total storage to the known reservoir volume.

Yerrabi, Gungahlin and Ginninderra Dams were all assumed to be at FSL at the start of each scenario, consistent with the operational management of these storages.