



# Design Flood Hydrographs for the Goulburn and Broken River Catchments

Goulburn-Broken Catchment Management Authority

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### Document history and status

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**Appendix A. Additional Information**

## 1. Introduction

Over the last few years there have been a number of detailed flood studies being undertaken for locations throughout Victoria. Whilst these studies provide detailed information for planning and emergency response purposes in and around towns, they typically do not extend significantly beyond the urban areas. As such, there are large parts of the state, particularly in rural areas, where flood information is poor or non-existent.

The Goulburn-Broken Catchment Management Authority (GBCMA) is currently undertaking flood modelling to help infill flood information in some of these rural areas. This programme is largely being undertaken using in-house hydraulic TUFLOW modelling. However, in some areas there is a need to also provide hydrologic inflows to calibrate and undertake design runs for the hydraulic models. The priority areas next to be modelled by GBCMA include:

- Big River upstream of Eildon Dam tailwater;
- Upper Goulburn River upstream of Eildon Dam tailwater (particularly the settlements of Woods Point, Kevington and Jamieson);
- Howqua River upstream of Eildon Dam tailwater;
- Delatite River upstream of Eildon Dam tailwater (particularly the settlements of Sawmill and Merrijig);
- Ford Creek upstream of Eildon Dam tailwater (note that a detailed flood study is available for Mansfield);
- Goulburn River downstream of Eildon Dam; and
- Upper Broken River upstream of Lake Nillahcootie.

Design flood hydrology for the majority of these catchments was completed by Jacobs for Goulburn-Murray Water (G-MW) as part of G-MW's ongoing dam safety programme. Jacobs developed detailed RORB hydrological models for both Eildon Dam and Lake Nillahcootie.

In this project, the RORB hydrological models were used to determine suitable inflows for use in hydraulic modelling. The layout of the existing models is shown in Figure 1-1 and Figure 1-2. These models were adjusted and then used to provide critical duration inflow hydrographs at key locations for the catchments nominated above. The inflow hydrographs are provided for AEPs of 1 in 10, 20, 50, 100, 200 and 500. Historic flood inflows for the September 1998 and September 2010 floods, where available were also provided to enable the TUFLOW models to be calibrated. The general approach adopted for the models was to:

- 1) Modify the existing RORB models to incorporate sufficient subdivision of subcatchment areas to generate hydrographs that are representative of incremental inflows along the domain of the hydraulic models. This involved subdividing existing subcatchment areas in the RORB models into smaller subcatchments, minor adjustment of subcatchment boundaries for consistency with contemporary digital terrain data and making the appropriate adjustments to the reach and node network in the RORB models;
- 2) Re-calibrate the RORB models to the May 1974, September 1975, September 1998, September 2010 and December 2010 flood events to gauged flows for these events, where gauged data is available in each catchment;
- 3) Fit at-site flood frequency curves to flood peaks at gauge sites within each subcatchment;
- 4) Use a Monte-Carlo joint probability approach to simulate the design flood frequency curve at each gauge site and then adjust the parameters of the RORB model (typically the loss parameters only – median initial loss, continuing loss rate or runoff coefficient) to verify to the flood frequency curves fitted to the gauged peaks;
- 5) Run the RORB model with a single temporal pattern to generate hydrographs for all nominated inflow locations to the hydraulic model for the 10% AEP (1 in 10 year) event, selecting loss parameters so that the hydrograph at the subcatchment outlet matches the 10% AEP flood quantile from the Monte-Carlo runs;
- 6) Repeat step 5 for flood events with AEP of 5% (1 in 20), 2% (1 in 50), 1% (1 in 100), 0.5% (1 in 200) and 0.2% (1 in 500).

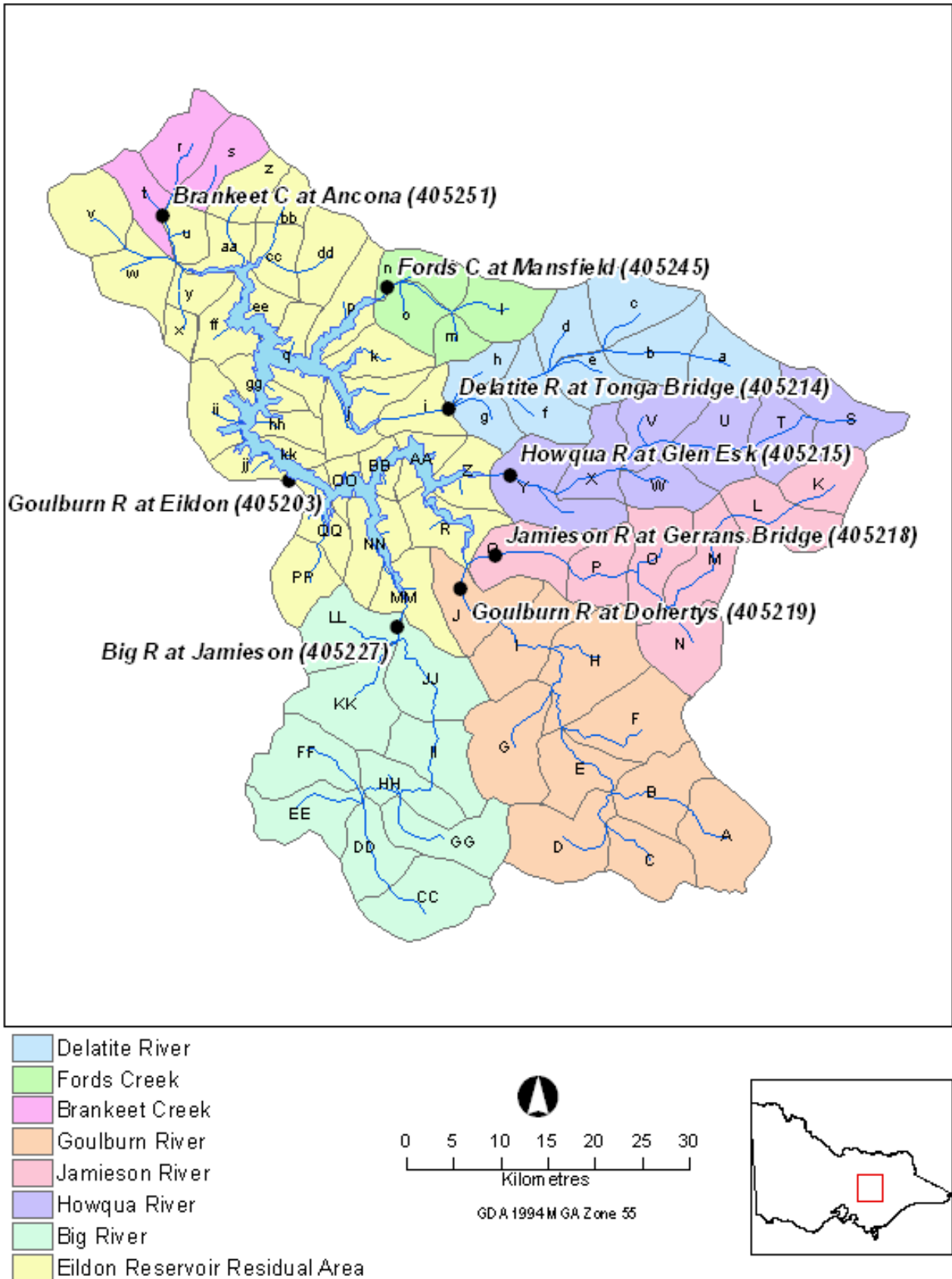


Figure 1-1: Eildon Dam RORB model layout (SKM, 2013)

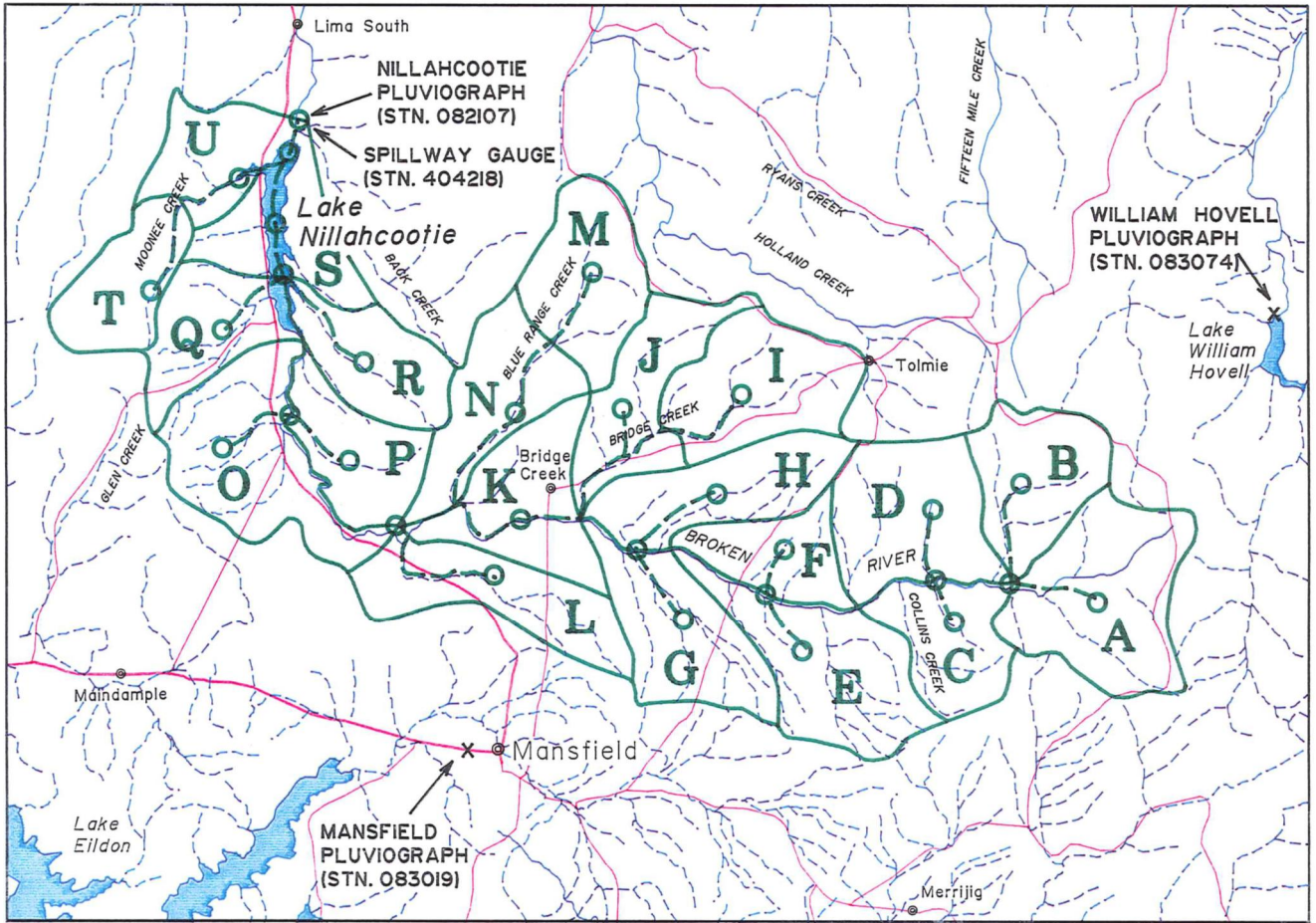


Figure 1-2: Lake Nillahcootie RORB model layout (Hydrotechnology, 1994)

## **2. Streamflow data available for calibration, flood frequency analysis and verification**

Recorded streamflow data was available from a number of flow gauges in each of the subcatchments that were modelled. The streamflow data was applied to several purposes, namely:

- Checking the calibration of RORB model parameters for specific historical events, and hence also providing inflow hydrographs for the TUFLOW models for these calibration events;
- Flood frequency analysis of annual maxima gauged flow peaks; and
- Verification of design flood estimates at the flow gauge against the flood frequency analysis.

Calibration was only performed for some of flood events at some gauges, due to missing flow data and/or limited pluviograph data and/or the particular flood event being relatively small at that particular gauge.

Table 2-1 lists the gauges that were used and the purposes that the data was applied to from each gauge in this study. Although there was sufficient data to undertake flood frequency analysis for the Big River downstream of Frenchmans Creek gauge (405264), the area of interest for hydraulic modelling was well downstream of this gauge and therefore verification of the RORB model parameters was only applied for the Big River at Jamieson gauge (405227). Across the Goulburn, Jamieson, Big and Howqua subcatchments (represented in the Southern Eildon RORB model), the flood peaks in the September 2010 event exceeded those in December 2010 and therefore for these subcatchments calibration was not undertaken for the December 2010 flood event. There was also generally insufficient flow gauging and pluviograph data to undertake calibration to the September 1975 event in the same subcatchments.

Table 2-1 : Streamflow gauges applied to calibration of RORB model parameters for particular events and flood frequency analysis and verification of RORB model parameters

Site Number	Site Name	Catchment Area (km <sup>2</sup> )	Calibration Events					Flood Frequency Analysis and Verification	
			1974 May	1975 Sep	1998 Sep	2010 Sep	2010 Dec		Data Period
405245	Fords Creek at Mansfield	117		✓	✓	✓	✓	✓	1971-2014
405214	Delatite River at Tonga Bridge	368		✓	✓	✓	✓	✓	1957-2014
405263	Goulburn River upstream of Snake Creek	327			✓	✓		✓	1976-2011
405219	Goulburn River at Dohertys	702			✓	✓		✓	1968-2014
405218	Jamieson River at Gerrang Bridge	362			✓	✓		✓	1960-2014
405264	Big River downstream of Frenchmans Creek	331			✓	✓			
405227	Big River at Jamieson	627			✓	✓		✓	1971-2014
405215	Howqua River at Glen Esk	368			✓	✓		✓	1975-2014
404218	Broken River at Lake Nillahcootie Head Gauge	416	✓		✓	✓	✓	✓	1993-2014

## 3. Hydrological Modelling Approach

### 3.1 Runoff routing model

The runoff-routing model RORB (Laurenson and Mein, 1995, Nathan *et al.*, 2006) was used to estimate the design floods for the Goulburn and Broken River catchments. RORB is a general runoff and streamflow routing program that is used to calculate flood hydrographs from rainfall and other catchment and channel inputs. The model subtracts losses from rainfall to determine rainfall excess and routes this through catchment storages to produce streamflow hydrographs at points of interest. RORB is a spatially distributed, non-linear model that is applicable to both urban and rural catchments. The model can account for both temporal and spatial distribution of rainfall and losses.

The model is based on catchment geometry and topographic data, and the two principal parameters are  $k_c$  and  $m$ . The adopted  $k_c$  value is dependent on the value of  $m$ . The  $m$  value describes the degree of non-linearity of the catchment's response to rainfall excess, and Book VI of ARR (I.E.Aust., 1998) recommends that this be kept at a constant value of 0.8 when modelling large to extreme events. Keeping  $m$  constant, the  $k_c$  parameter describes the catchment's response to rainfall excess with a lower  $k_c$  resulting in a "peaky" hydrograph, and a higher  $k_c$  resulting in a flatter hydrograph. The remaining parameters relate to the model representation of the rainfall losses which affects the volume of the hydrograph. The RORB model can represent those losses either by an initial loss/continuing loss model, or by the initial loss/proportional loss model. An initial loss/proportional loss model was adopted for this study.

Appropriate values of the model and loss parameters are selected through the processes of calibration and verification. Calibration involves a trial and error process of altering the model parameters to obtain the best possible fit with observed hydrographs. Verification, in the this instance, is the process of determining suitable losses for design flood estimation by comparing the estimated peak flows for a given AEP with results of an at-site flood frequency analysis of recorded peak flows.

### 3.2 Event-based and Monte Carlo approaches

Traditional practice for estimation of design floods has typically been based on the "design event" approach, in which all parameters other than rainfall are input as fixed, single values. This concept is illustrated in Figure 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.

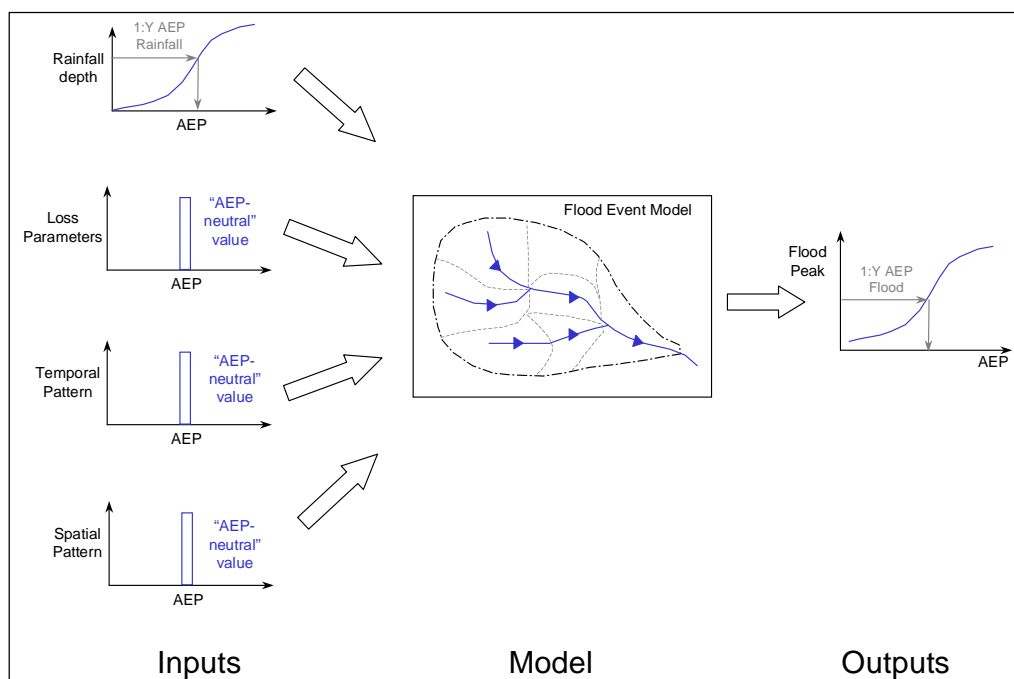


Figure 3-1: Schematic illustration of the design event approach

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; and,
- the treatment of more complex interactions (such as the seasonal variation of inputs) becomes rapidly more complex and less easy to defend.

Joint probability techniques offer an 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% AEP flood could be the result of a 2% AEP rainfall on a very wet catchment, or a 0.5% AEP rainfall on a dry catchment. Joint probability approaches attempt to mimic “mother nature” in that the influence of all probability distributed inputs are explicitly considered, thereby providing a more realistic representation of the flood generation processes (Nathan *et al.*, 2002).

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. Initial reservoir drawdown has also been included in the Monte Carlo framework for this catchment.

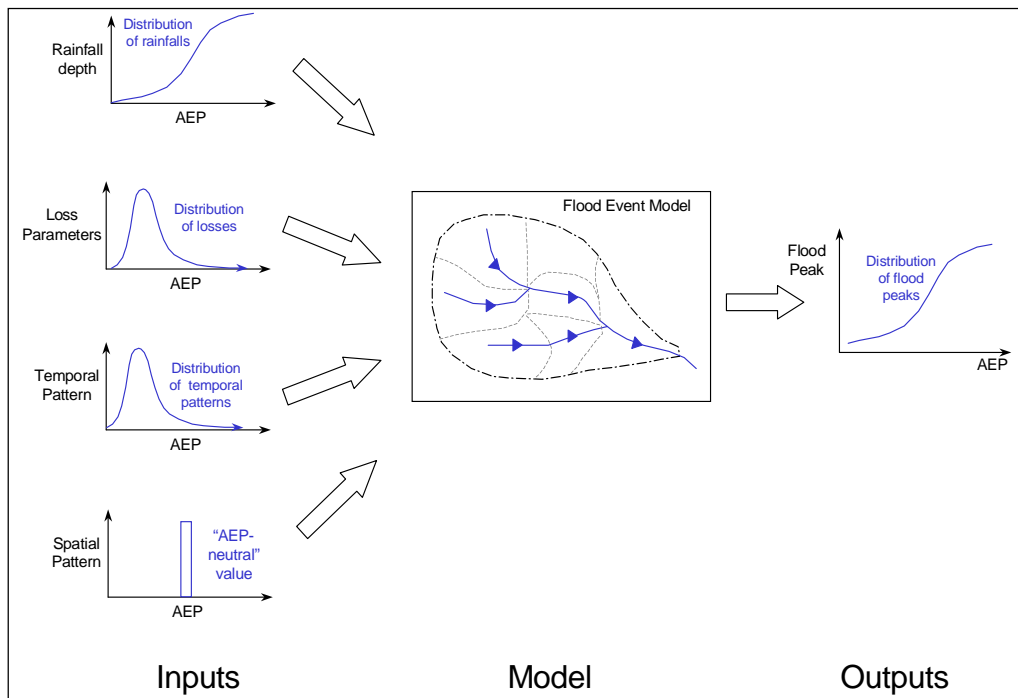


Figure 3-2: Schematic illustration of the joint probability approach

The application of Monte Carlo joint probability approaches to flood estimation has received some attention in the scientific literature over the past 20 years, but it is only recently that these techniques have been used in Australian design practice (Nathan *et al.* 2002; 2003). However, given advantages of Monte Carlo approach the most recent edition of Australian Rainfall and Runoff (Ball *et al.*, 2016) recommends the use of this technique for the calculation of design hydrology.

The following sections outline the overall framework adopted, and the nature of the evidence used to characterise the distribution of the inputs.

### 3.3 Overview of adopted joint probability framework

An overview of the joint probability framework adopted is illustrated in Figure 3-3. In essence the approach involves the undertaking of numerous model simulations where the model inputs are varied in accordance with that observed in nature. The inputs are sampled from non-parametric distributions that are either based on readily available design information or else on the results of recent research.

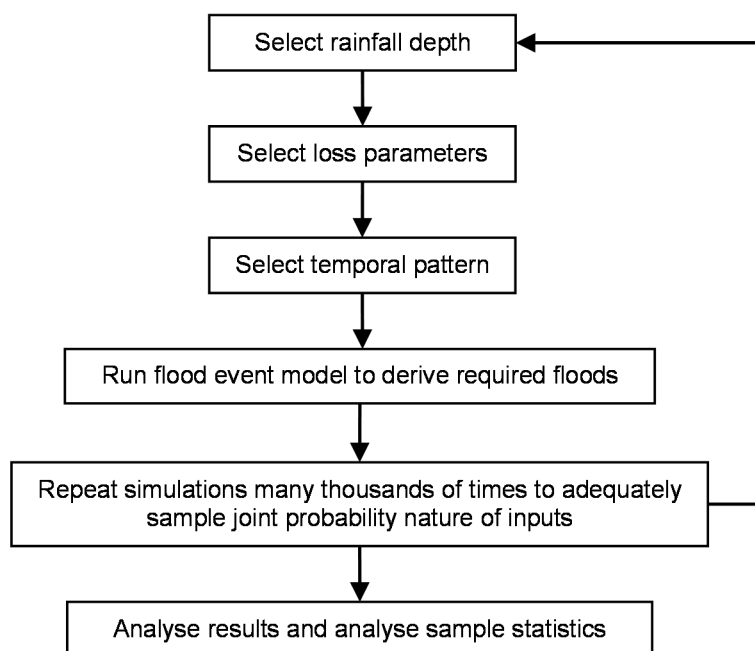


Figure 3-3: Overview of adopted joint probability framework

In developing the joint probability framework 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 Australian Rainfall and Runoff (I.E.Aust. 1998). The following briefly describes the main elements of the approach, and the manner in which they relate to established design information.

*Select rainfall depth.* Rainfall depths are stochastically sampled from the cumulative distribution of rainfall depths. Rainfall depths were derived for this study using methods detailed in Section 7. Rainfall depths are sampled in the range between 1 in 5 and 1 in 10,000 AEP (in order to derive the design flood quantiles within the more narrow range of 1 in 10 to 1 in 500 AEP for this study).

*Select storm losses:* Storm initial and continuing losses are stochastically sampled from a non-parametric distribution that was determined from the analysis of a large number of catchments from south-eastern Australia (Hill *et al.* 2013).

*Select Temporal Pattern.* Temporal patterns are randomly selected from a sample of temporal patterns relevant to the catchment area and duration of the storm. The temporal patterns are derived from large historic storms that have been observed in the region, and are from the same database used to construct the design patterns used in the current design event approach.

*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  $10^6$  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 200 simulations were taken within each division. Thus, a total of 10,000 simulations were undertaken to derive the frequency curve corresponding to each of the storm durations considered.

## **4. Adjustments to RORB model subarea and reach layout**

### **4.1 Purpose of adjustment of RORB model subareas**

RORB is a non-linear rainfall-runoff routing model, which represents the routing effect of flows as they pass along reaches of streams within a catchment using a non-linear relationship between the flow through a reach and the volume of water “stored” in the reach during the model time-step. The RORB models used in this study adopted either initial loss-continuing loss or initial loss-proportional loss models, which are both conventional in Australian hydrological practice. The runoff hydrograph generated from any single subcatchment before it is routed by the model therefore is often a “blocky” representation of the rainfall excess, or the rainfall hyetograph on the subarea minus the removal of the losses. Due to the often “blocky” nature of the rainfall excess hydrograph generated from any individual subarea, it is accepted practice with the use of RORB that there should be at least four subareas upstream of any location where a hydrograph is extracted from the model, to allow for sufficient reaches in the model to represent the non-linear routing effects on flows to that location.

The RORB models that existed prior to this project of the Goulburn River to Eildon Dam and the Broken River to Nillahcootie Dam were created to simulate design inflow and outflow floods from the two dams. They therefore incorporated sufficient subdivision to represent floods for the dams. This current report discusses generation of hydrographs that extend up each of the catchments, as inputs to hydraulic models that run along the tributaries well upstream of the reservoirs of each dam. The existing RORB models were subdivided into a much larger number of subcatchments, to incorporate sufficient subdivision of subcatchment areas (normally at least four subareas of each input location) to generate hydrographs that are representative of incremental inflows along the domain of the hydraulic models. The previous versions of the RORB models were generated using hard copy topographic maps. In re-defining the subcatchment boundaries of the models, they were generated using digital terrain data from a 10 m resolution digital terrain model. The reach lengths and topology were re-defined once the new subcatchment boundaries were created.

### **4.2 Results: revised RORB models and inflow locations**

Revised RORB model and hydrograph input locations are shown in Figure 4-1 to Figure 4-13 on the following pages.

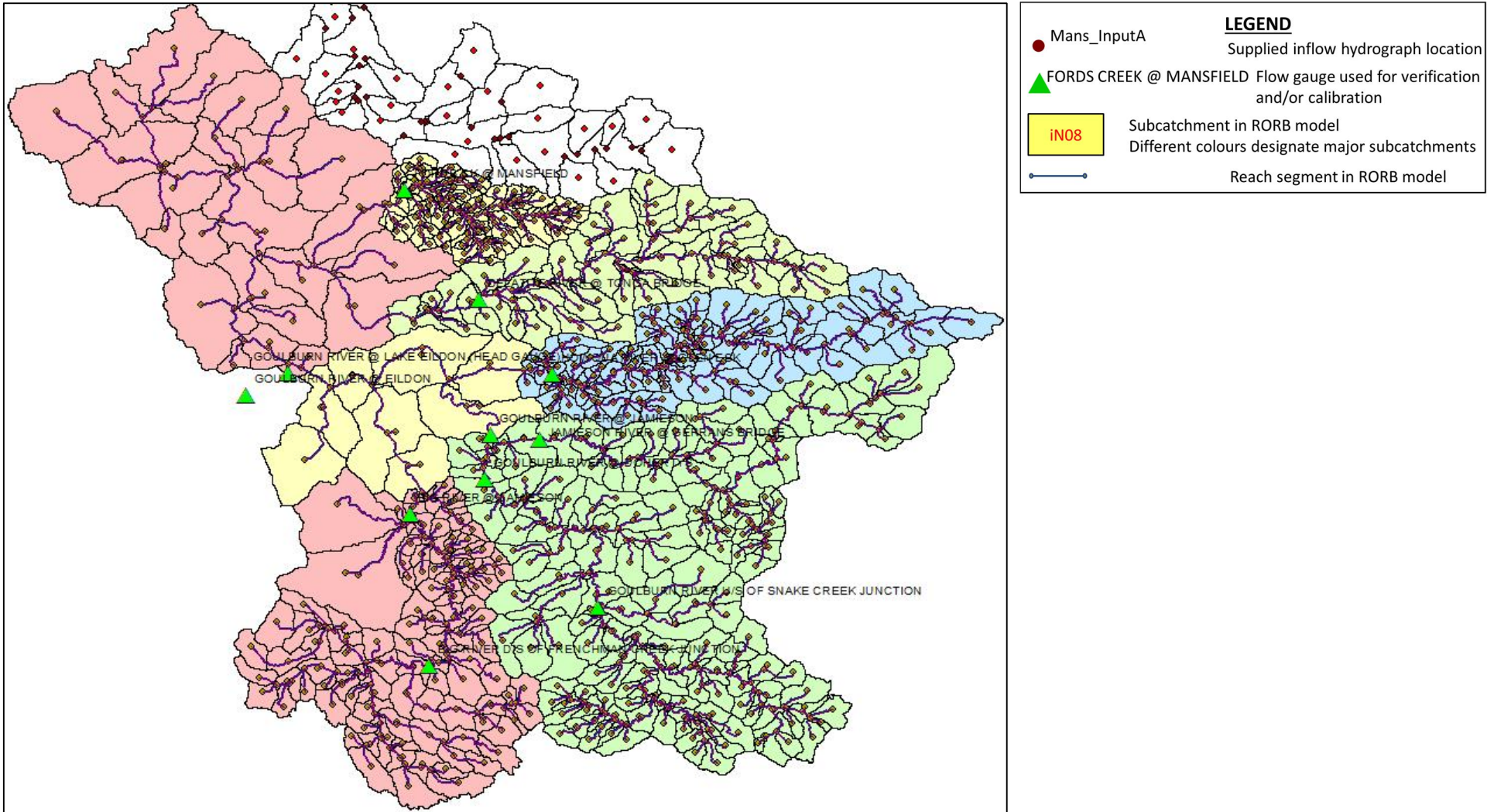


Figure 4-1 Revised RORB model layout for the whole Eildon catchment RORB model

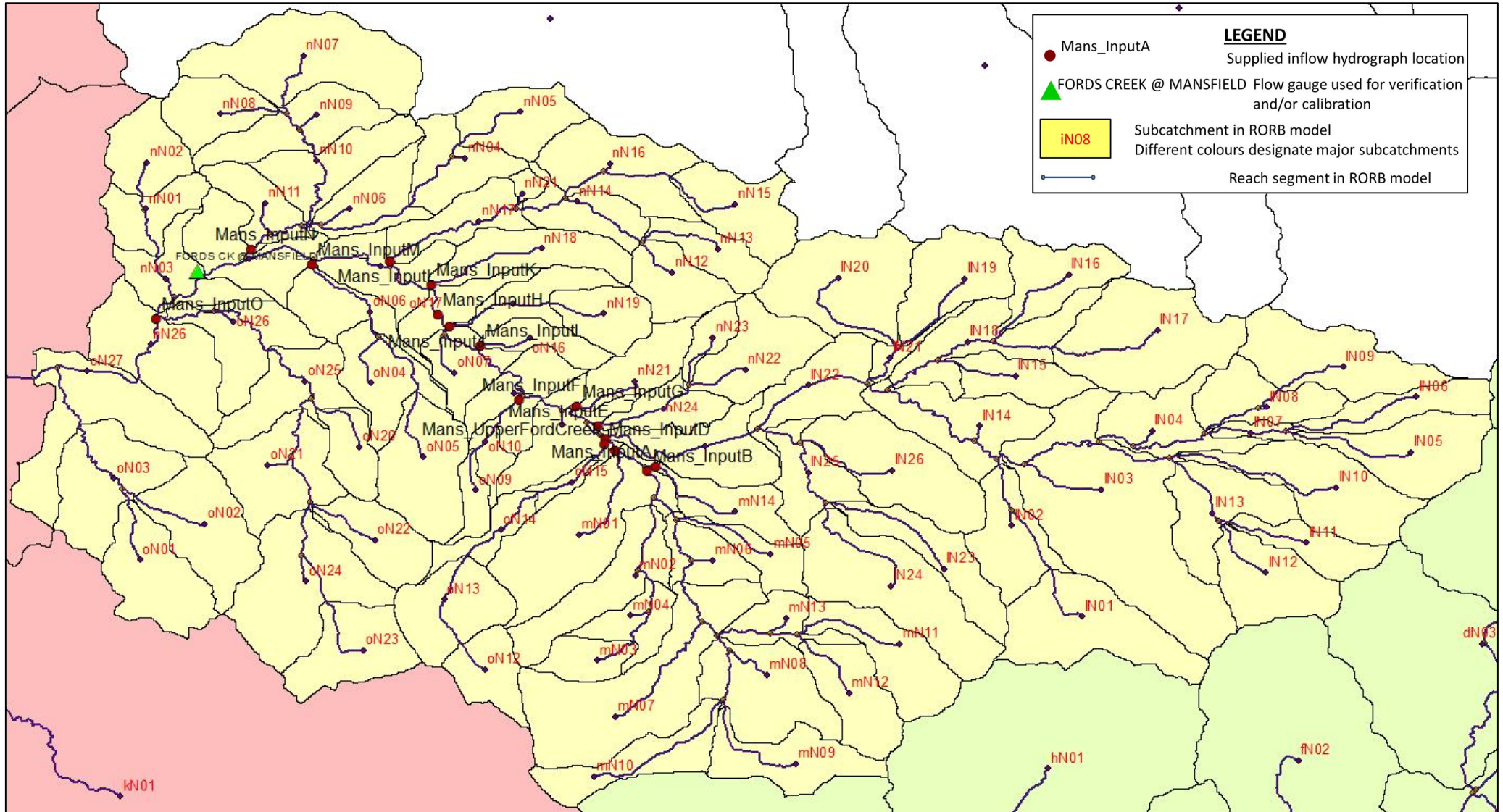


Figure 4-2 Revised RORB model layout for the Fords Creek subcatchment of the Eildon Northern Area RORB model, showing hydrograph input locations to the TUFLOW model

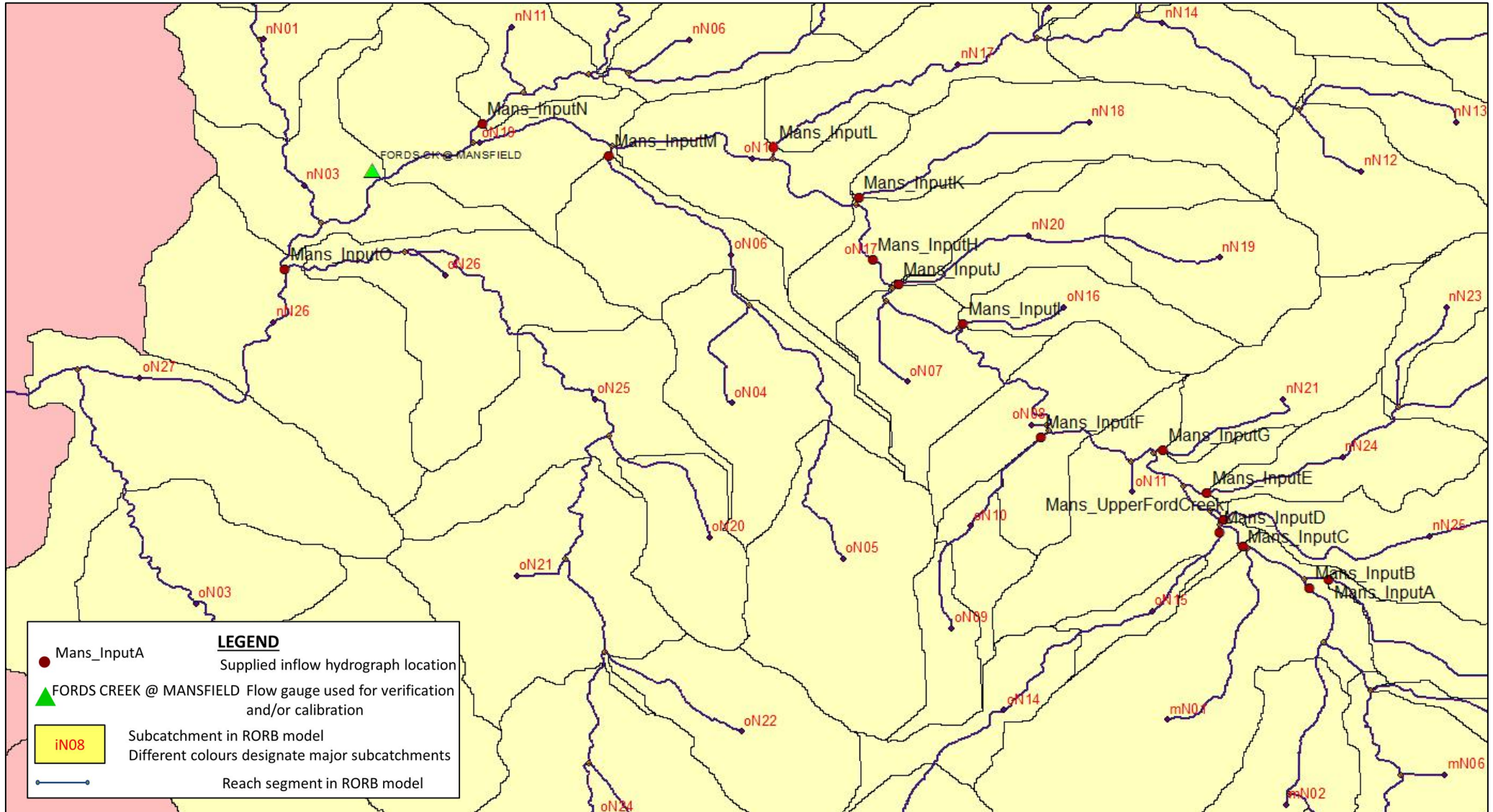


Figure 4-3 Hydrograph input locations for TUFLOW model from the Fords Creek portion of the RORB model

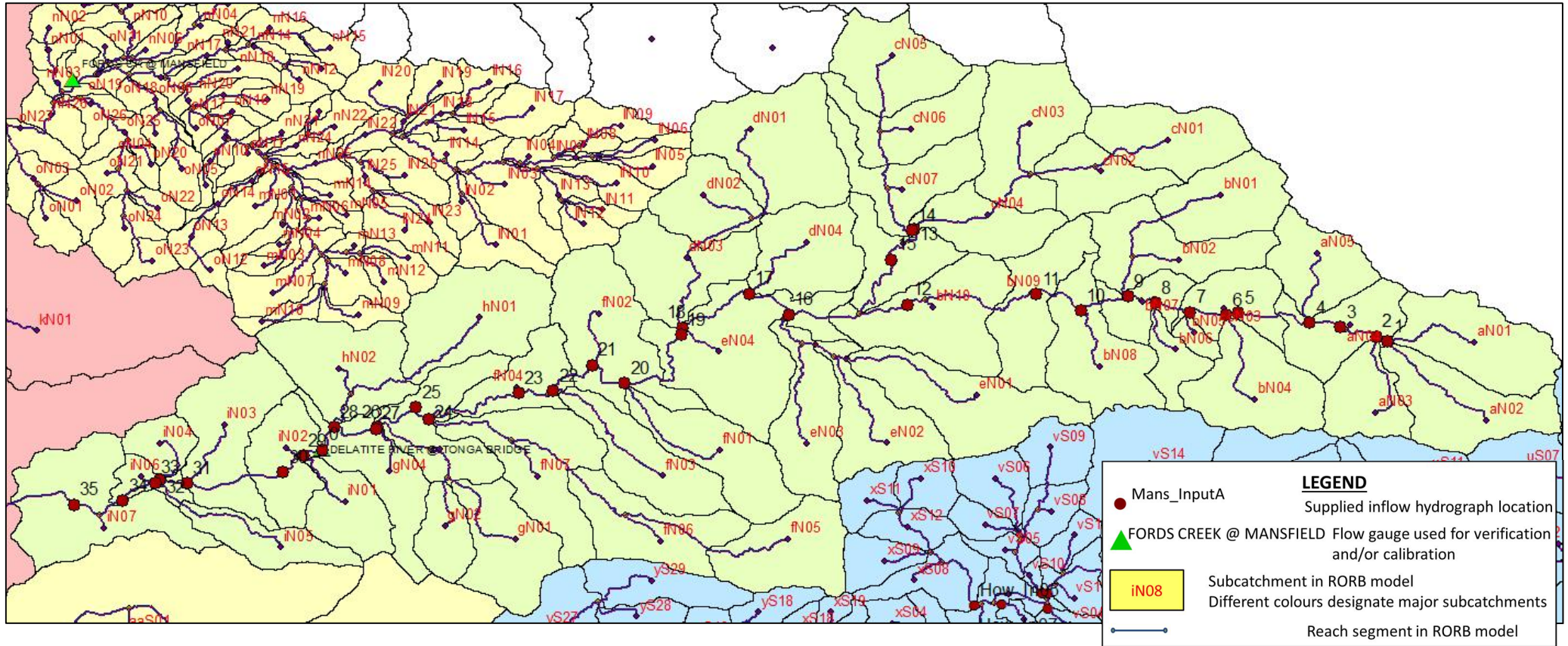


Figure 4-4 Revised RORB model layout for the Delatite River subcatchment of the Eildon Northern Area RORB model, showing hydrograph input locations to the TUFLOW model

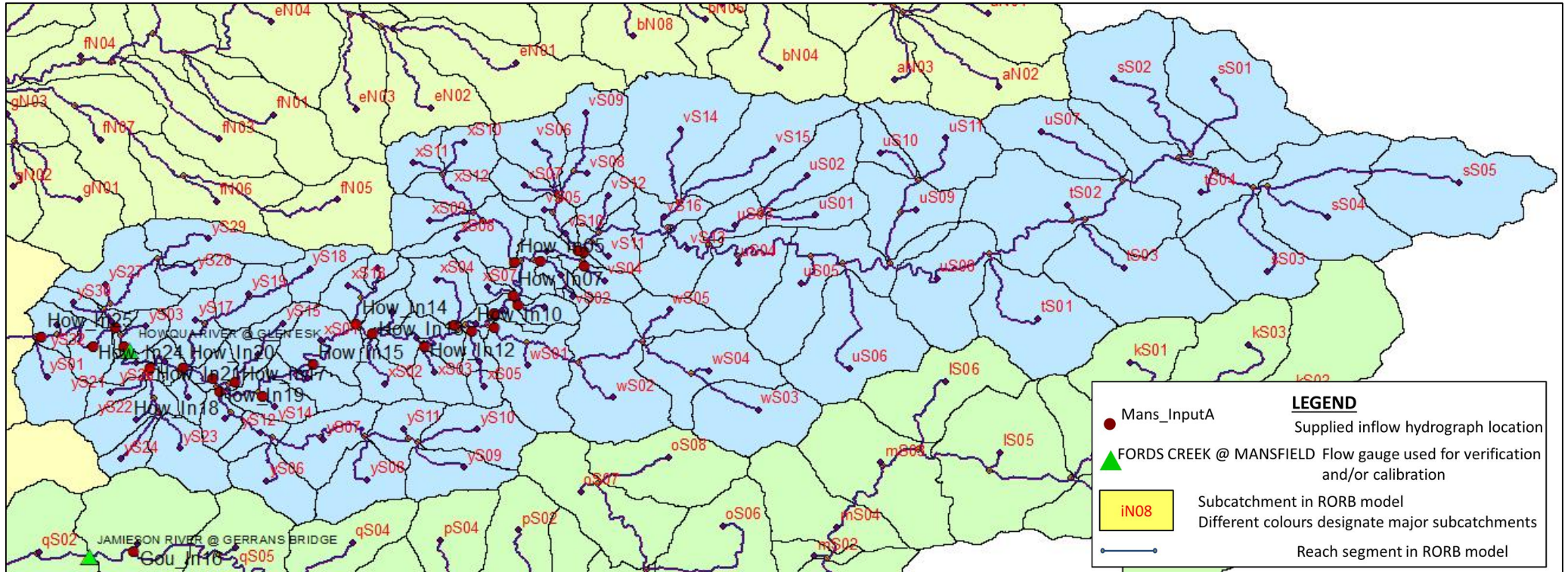


Figure 4-5 Revised RORB model layout for the Howqua River subcatchment of the Eildon Southern Area RORB model, showing hydrograph input locations to the TUFLOW model

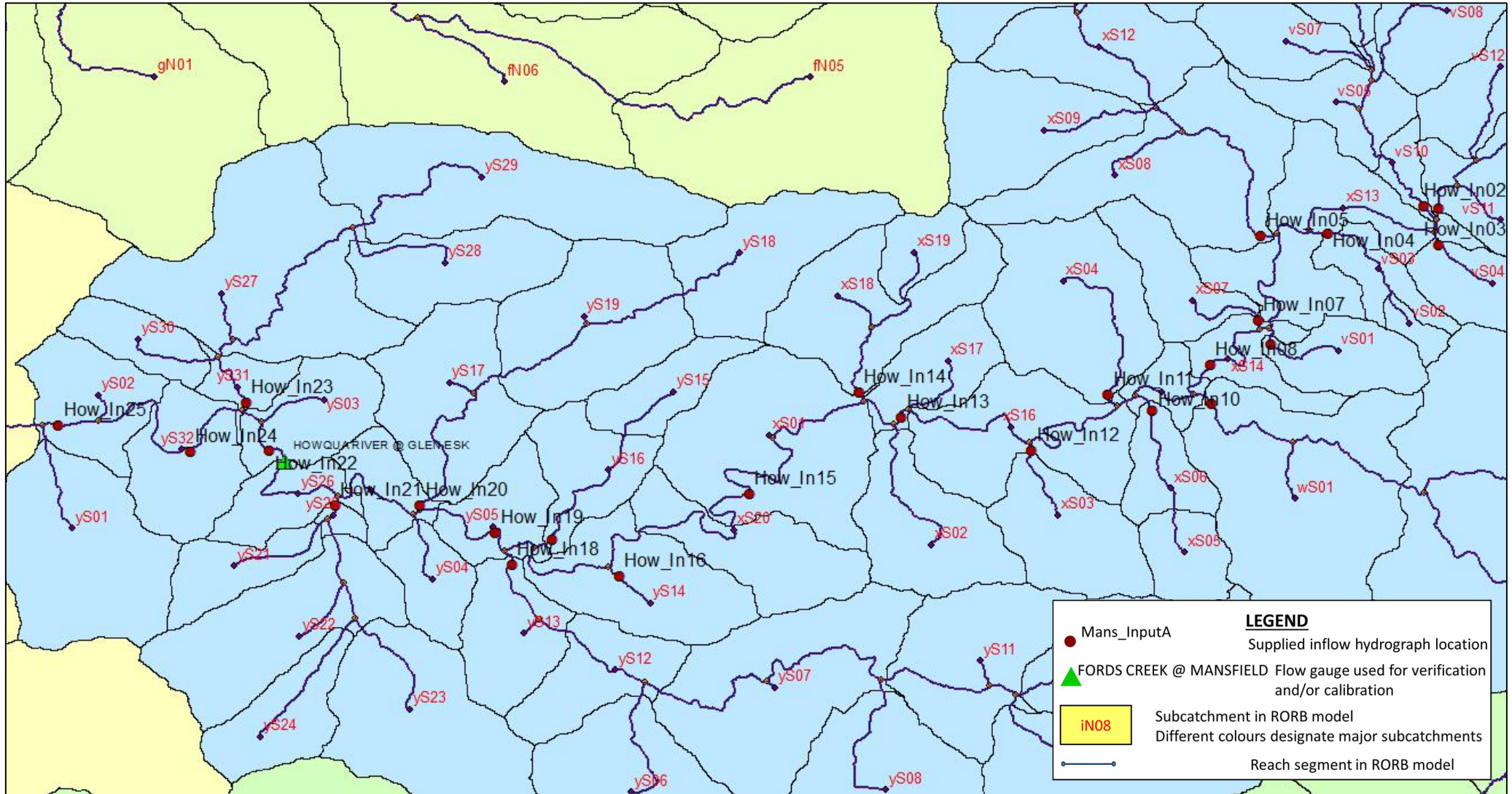


Figure 4-6 Hydrograph input locations from the Howqua River subcatchment of the Southern Area RORB model

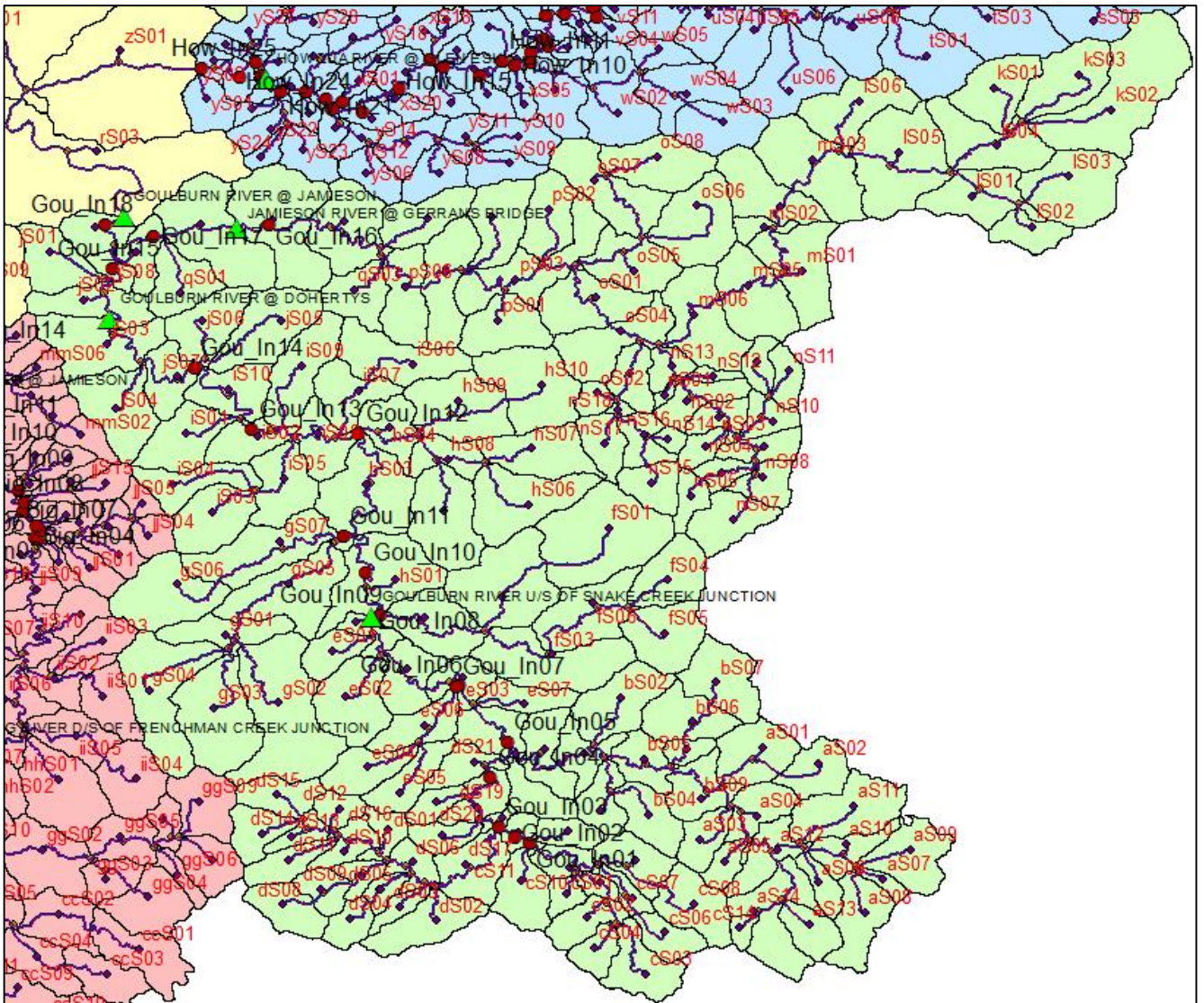
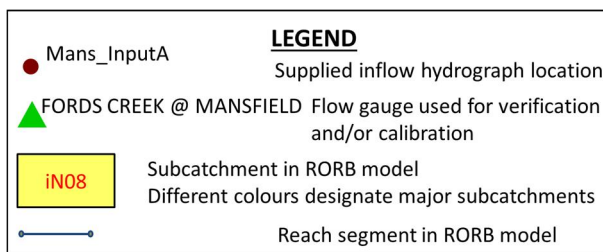


Figure 4-7 Revised RORB model layout for the Upper Goulburn and Jamieson River subcatchment of the Eildon Southern Area RORB model, showing hydrograph input locations to the TUFLOW model



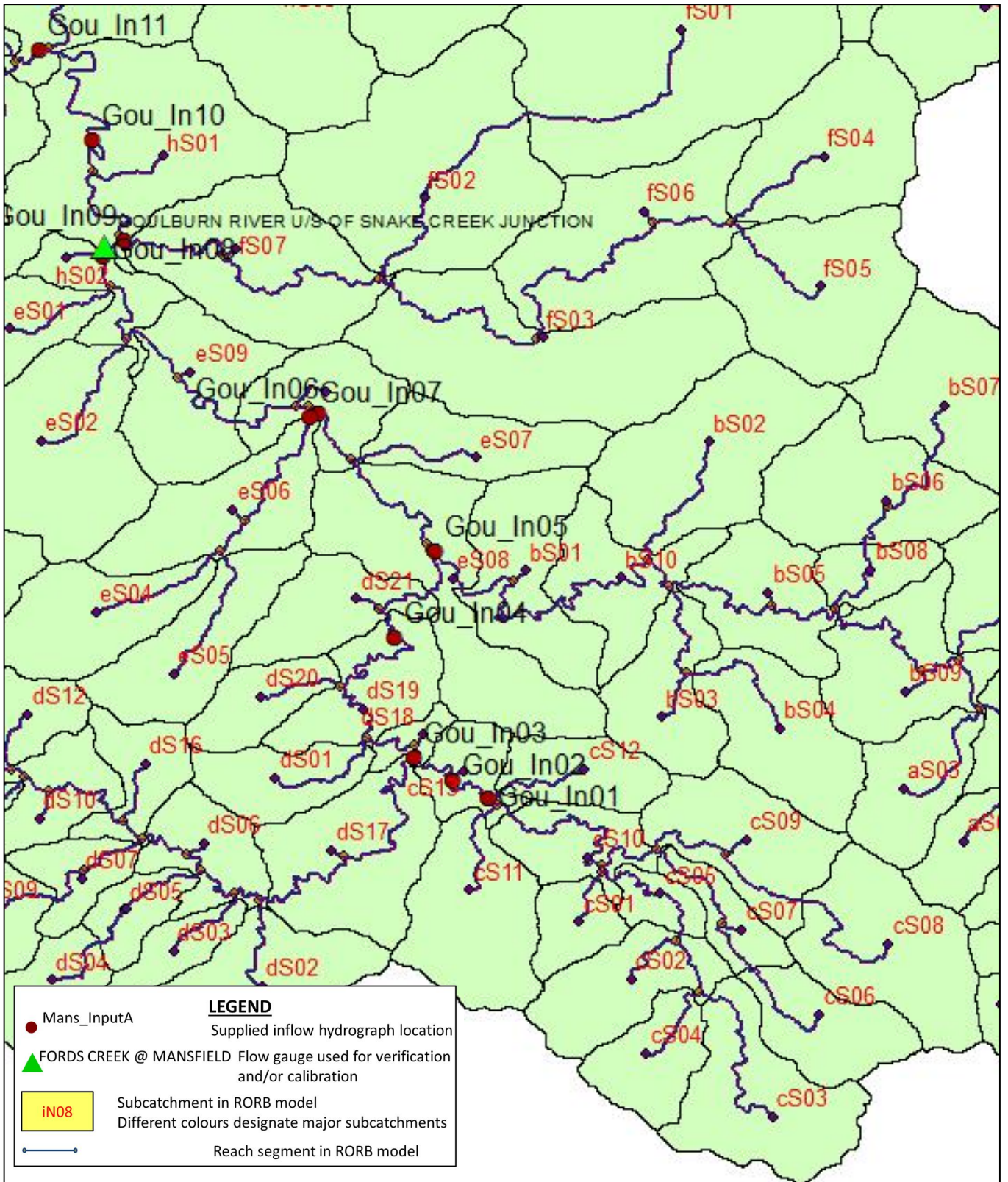


Figure 4-8 Hydrograph input locations from the upper portion of the Goulburn River subcatchment of the RORB model

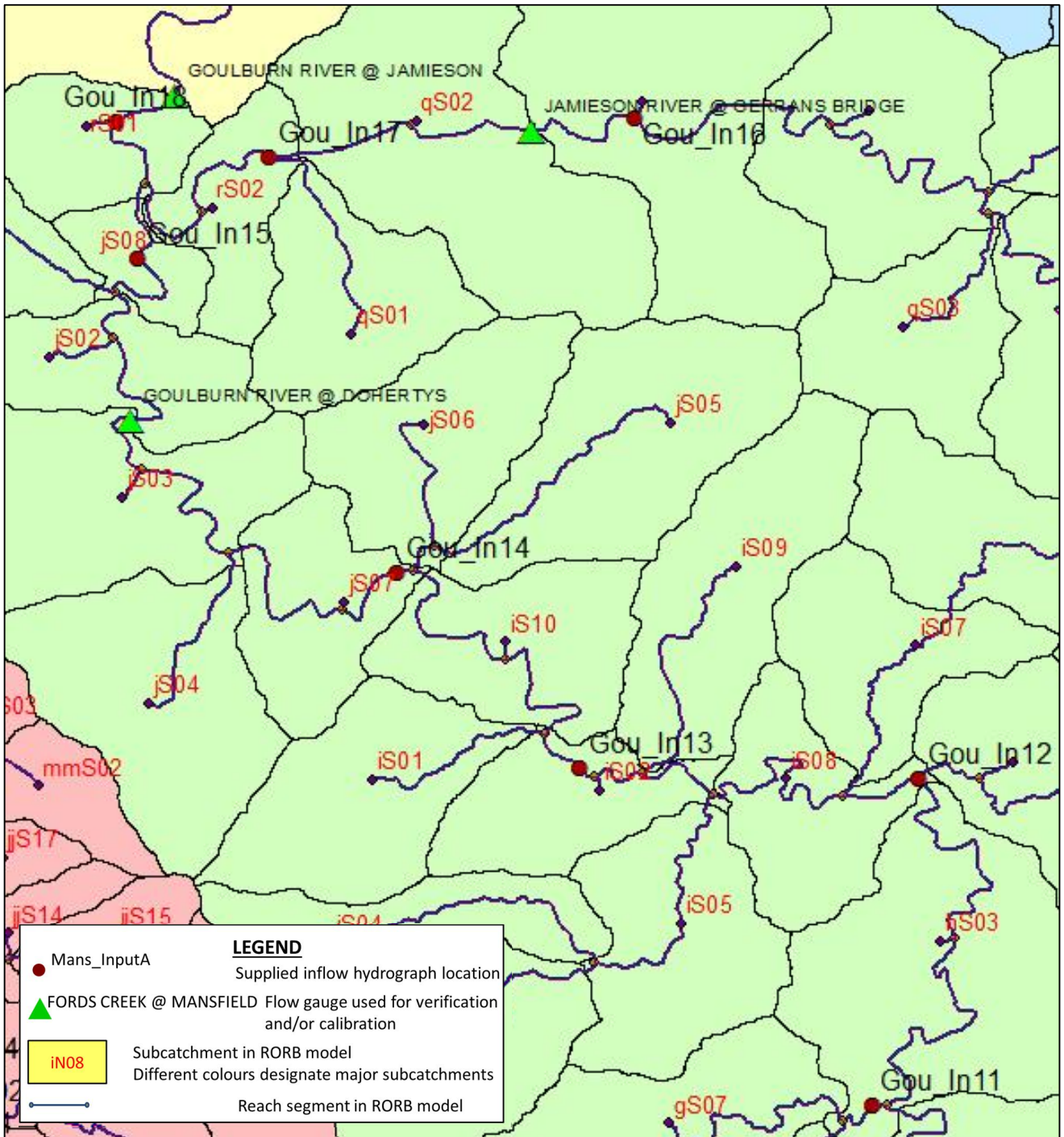


Figure 4-9 Hydrograph input locations from the lower portion of the Goulburn and Jamieson River subcatchments of the RORB model

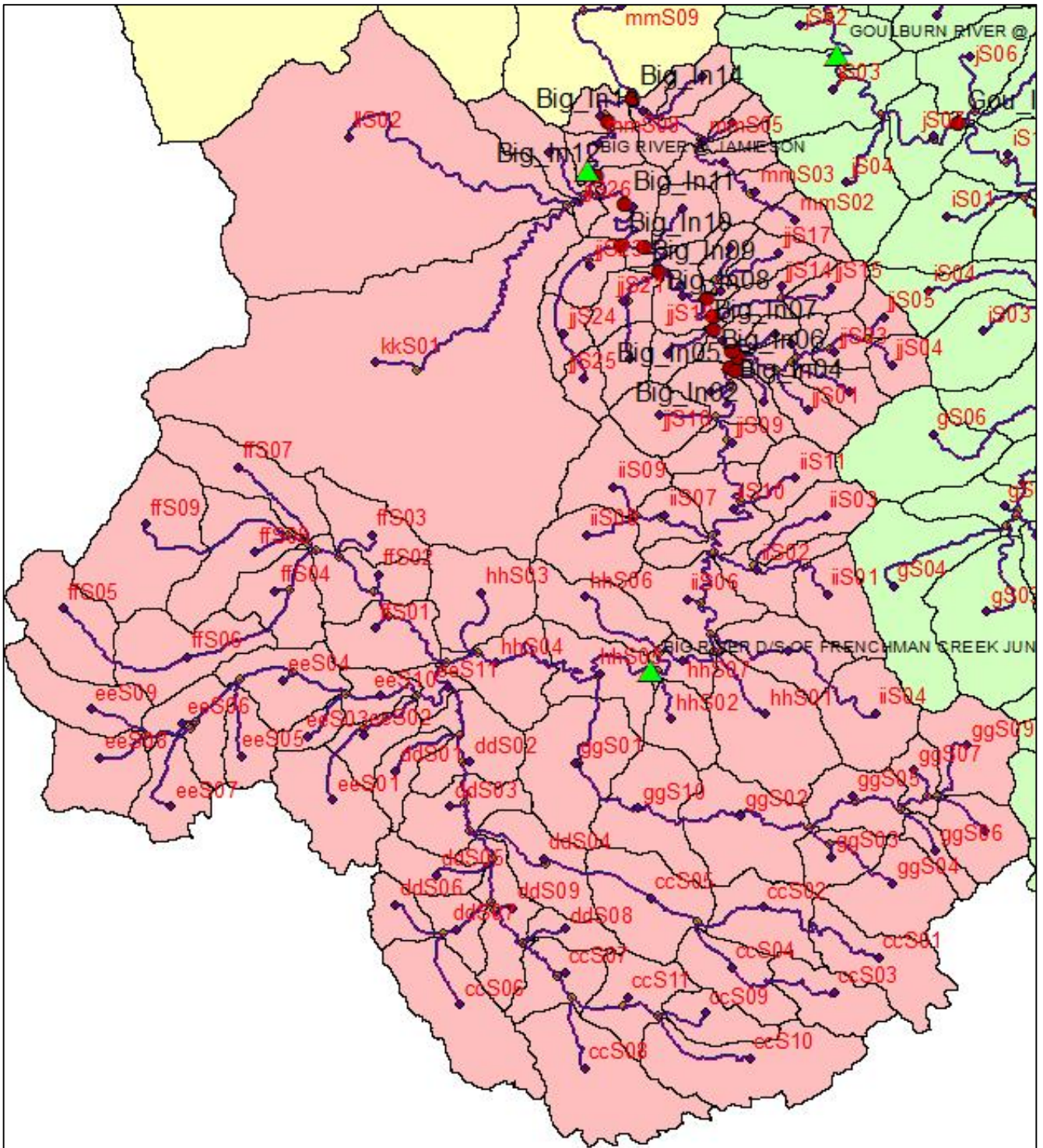
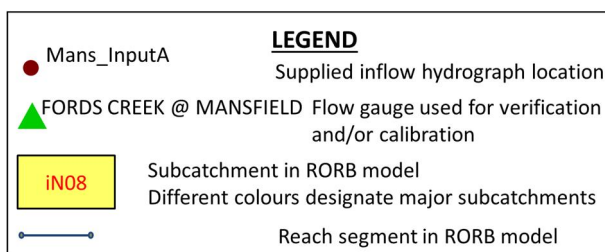


Figure 4-10 Revised RORB model layout for the Big River subcatchment of the Eildon Southern Area RORB model, showing hydrograph input locations to the TUFLOW model



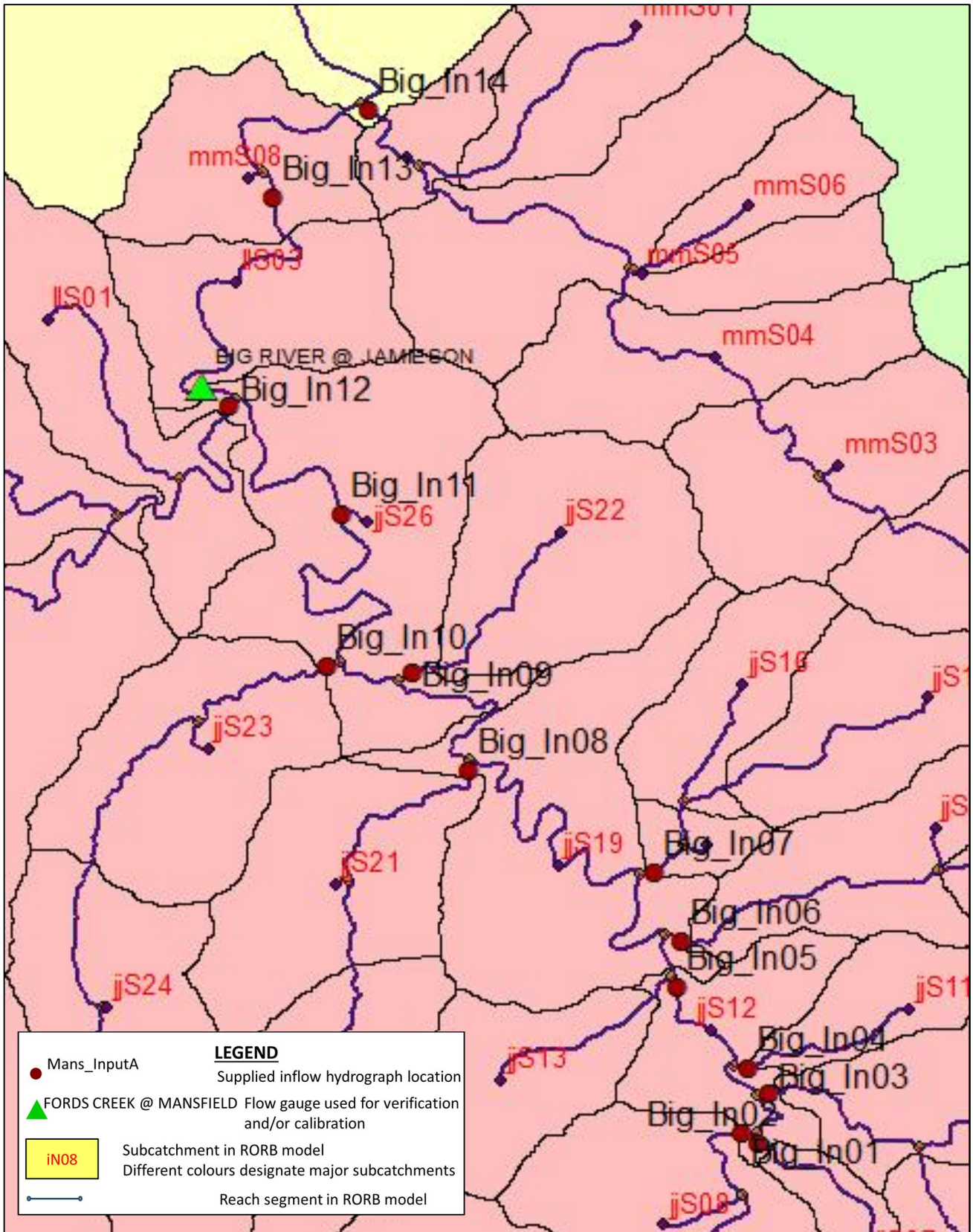


Figure 4-11 Hydrograph input locations from the Big River subcatchment of the RORB model

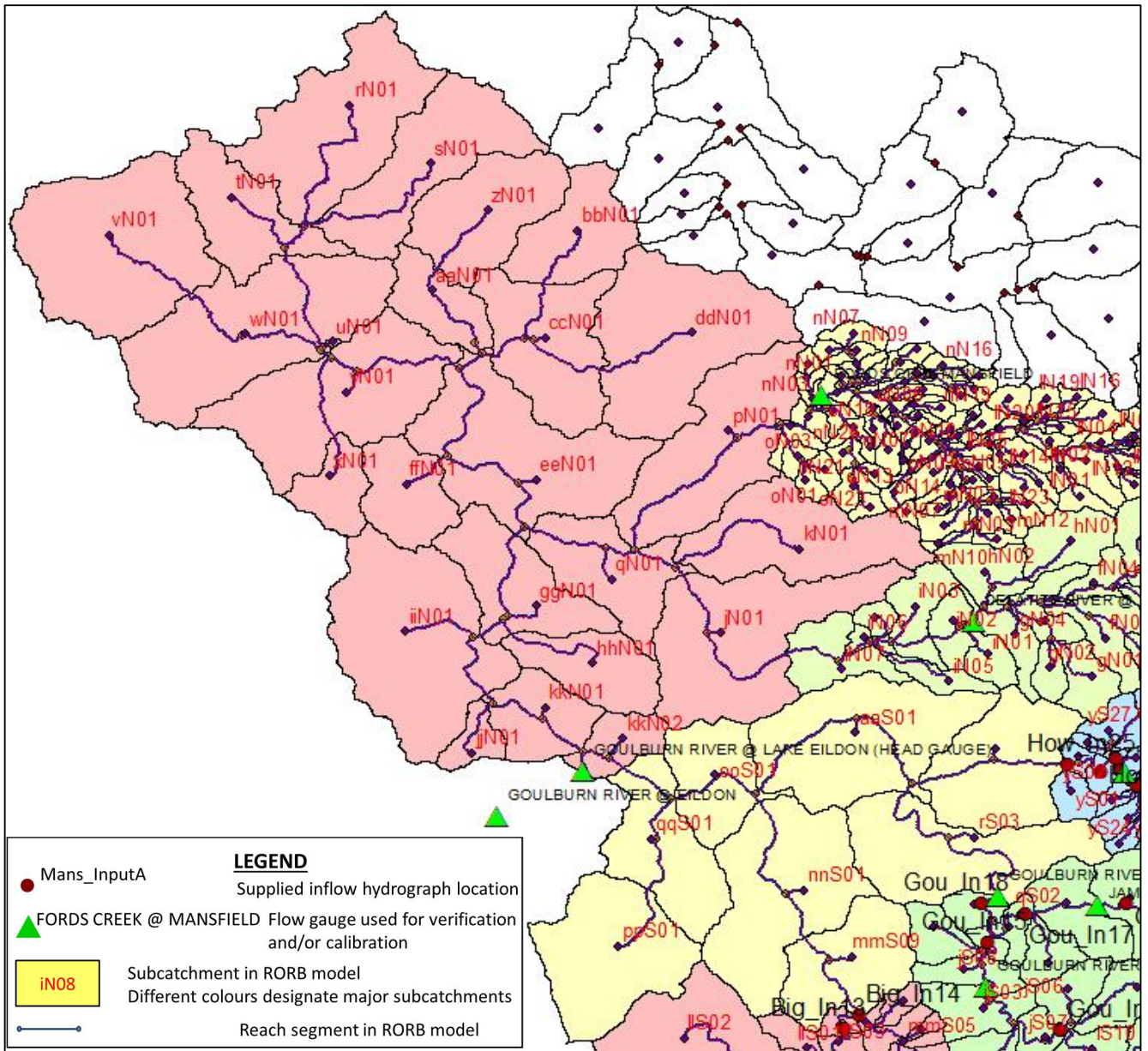


Figure 4-12 Revised RORB model layout for the residual areas of the Northern and Southern areas of the Eildon RORB model

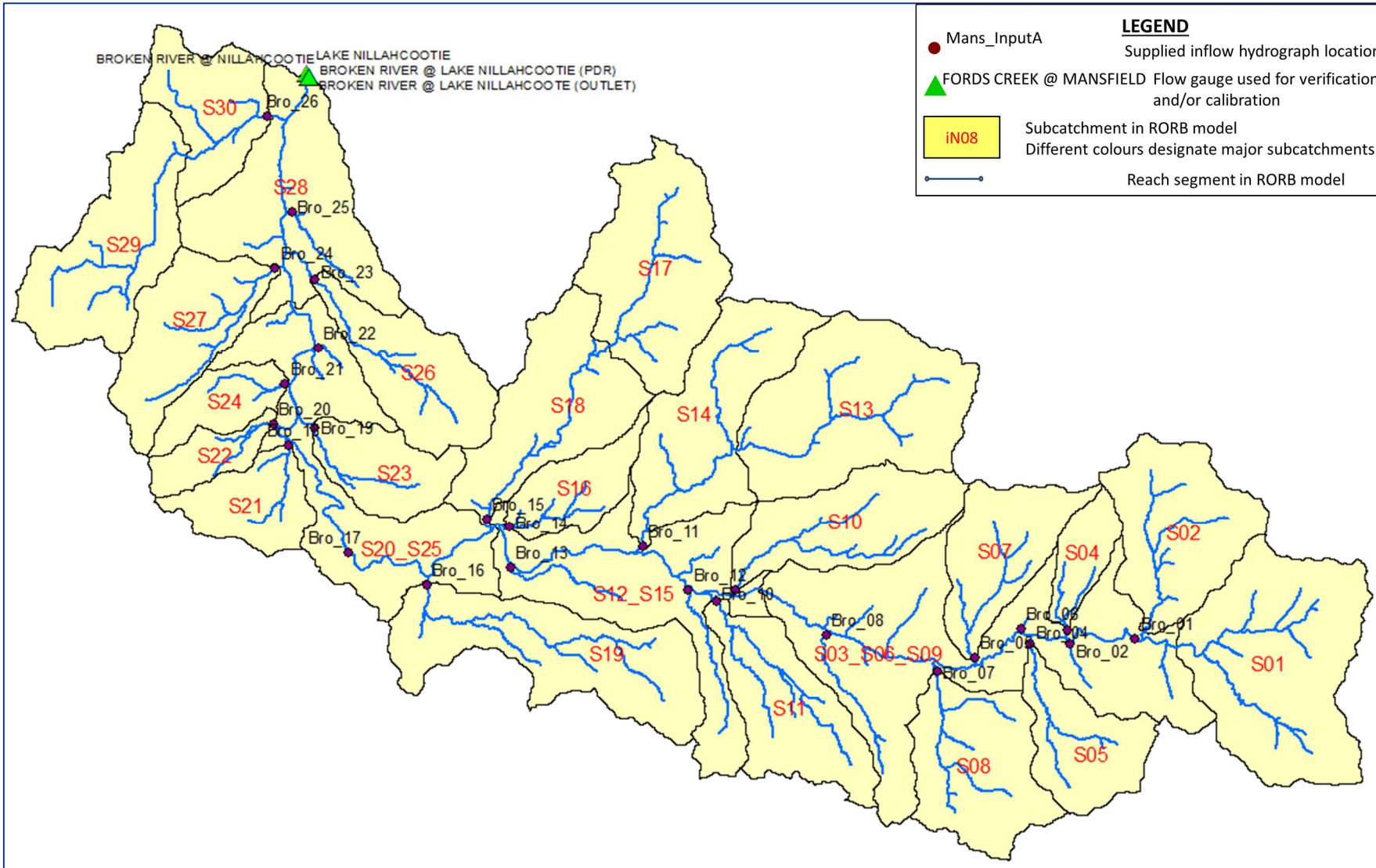


Figure 4-13 Revised RORB model layout for the Broken River to Nillahcootie Model, showing hydrograph input locations to the TUFLOW model

## 5. Review of calibration of RORB model to particular historical flood events

### 5.1 Purpose and approach to review of RORB model calibration

Flood hydrographs were produced from the RORB model for a number of calibration events at the inflow locations. The main purpose of the calibration was to produce inflow flood hydrographs that could be applied in calibration of the hydraulic (TUFLOW) models that will subsequently be developed by Goulburn Broken CMA to the same calibration events. A secondary purpose of the calibration was to estimate parameters of the RORB model for each subcatchment for a number of events, which provided a starting point for verification of the RORB models to flood frequency analysis, as discussed in Section 8.

The Eildon RORB models were last calibrated to historical events by Sinclair Knight Merz (1999). The adjustments to the RORB model subareas could result in changes to the routing characteristics from the existing RORB model. The routing response was therefore checked by re-calibrating each subcatchment of the RORB models to gauged flows. The re-calibration also provided the opportunity to calibrate the models to well gauged large floods in September 1975, September 1998, September 2010 and December 2010. Not all floods were used for calibration in all subcatchments because for some floods there was missing streamflow and/or pluviograph data and in other cases the particular flood was relatively minor at the particular streamflow gauge as shown in Table 2-1.

### 5.2 Delatite River subcatchment

The RORB model with revised catchment subdivision was calibrated to streamflow data at the Delatite River at Tonga Bridge gauge for the September 1998, September 2010 and December 2010 flood events. For all events reasonable calibrations were achieved, as shown in Figure 5-1, Figure 5-2 and Figure 5-3. The calibrated parameter values are shown in Table 5-1.

Table 5-1 Parameters of Delatite River RORB Model fitted to calibration events

Parameter	September 1998	September 2010	December 2010	Adopted Routing Parameters
Routing parameter ( $k_c$ ) to streamflow gauge at Tonga Bridge	28	27	27	27
Non-linearity parameter ( $m$ )	0.8	0.8	0.8	0.8
Event Initial loss ( $IL$ )	25 mm	10 mm	15 mm	
Event Continuing loss ( $CL$ )	2.5 mm/h	1.5 mm/h	2.5 mm/h	

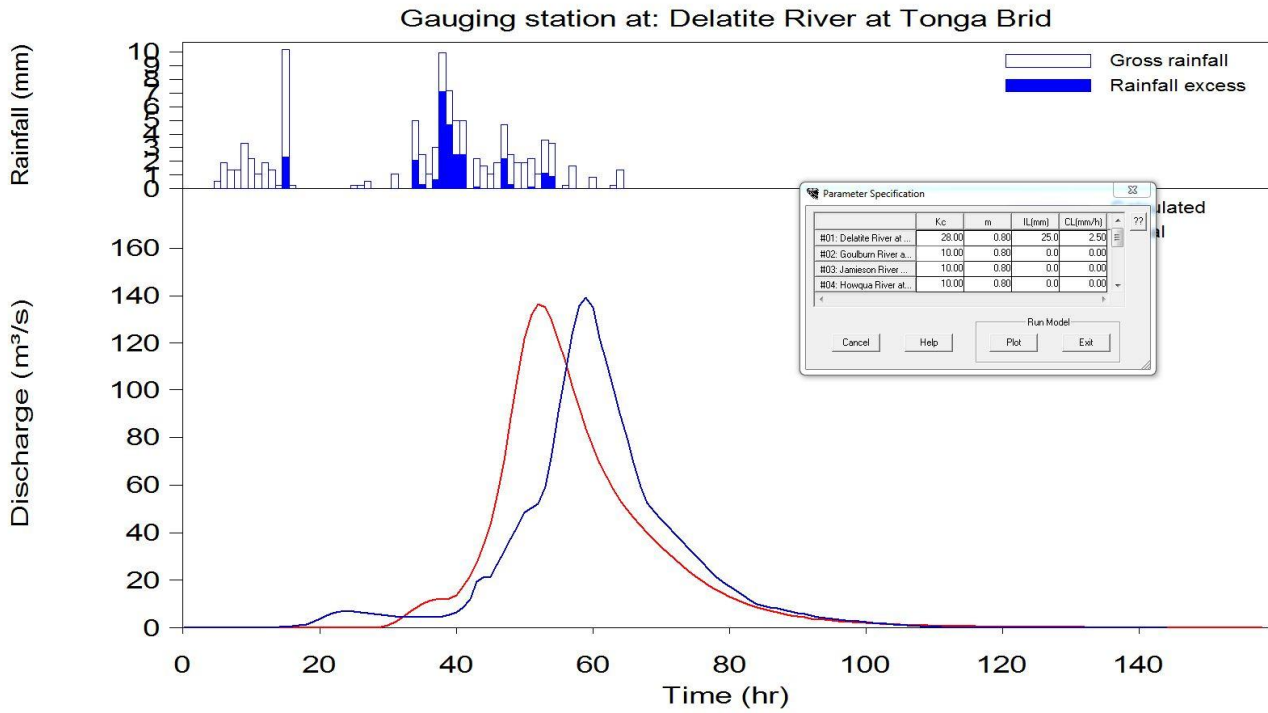


Figure 5-1 RORB Model Calibration – Delatite River at Tonga Bridge for September 1998 flood event

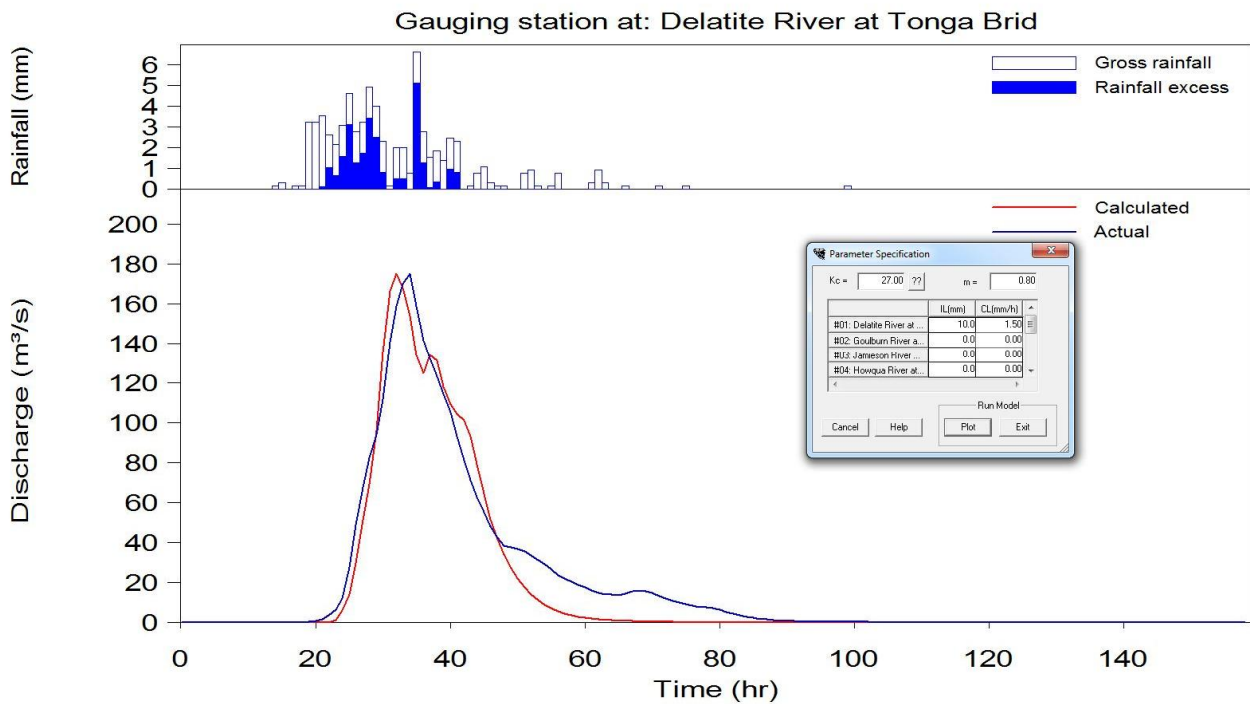


Figure 5-2 RORB Model Calibration – Delatite River at Tonga Bridge for September 2010 flood event

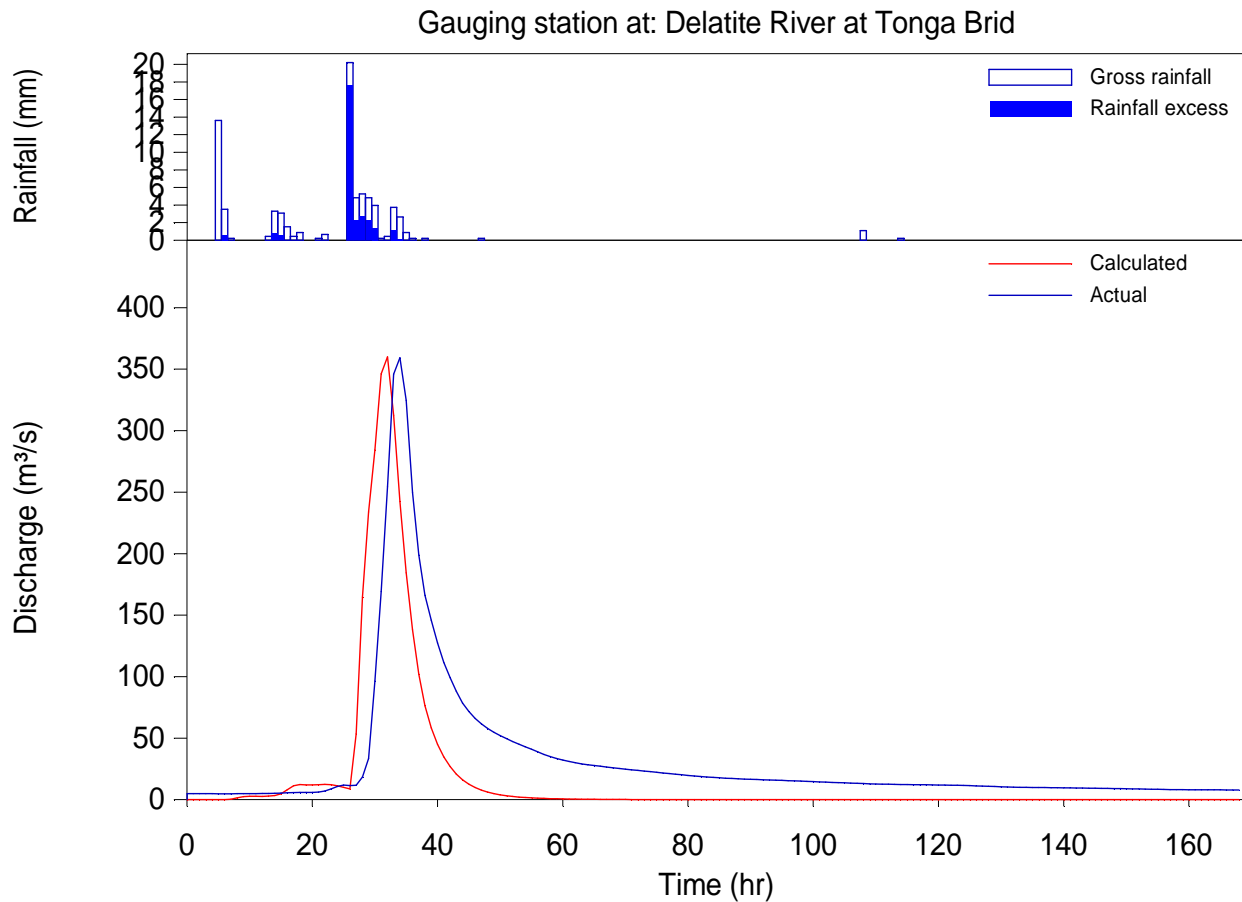


Figure 5-3 RORB Model Calibration – Delatite River at Tonga Bridge for December 2010 flood event

### 5.3 Fords Creek subcatchment

The RORB model with revised catchment subdivision was calibrated to streamflow data at the Fords Creek at Mansfield gauge for the September 1998, September 2010 and December 2010 flood events. For all events reasonable calibrations were achieved, as shown in Figure 5-4 to Figure 5-7. The calibrated parameter values are shown in Table 5-2.

Table 5-2 Parameters of Fords Creek RORB Model fitted to calibration events

Parameter	September 1975	September 1998	September 2010	December 2010	Adopted Routing Parameters
Routing parameter ( $k_c$ ) to streamflow gauge at Mansfield	5.0	11.0	6.0	5.8	5.5
Non-linearity parameter ( $m$ )	0.8	0.8	0.8	0.8	0.8
Event Initial loss ( $IL$ )	17 mm	48 mm	9 mm	19 mm	
Event Continuing loss ( $CL$ )	0.2 mm/h	1.3 mm/h	0.2 mm/h	1.5 mm/h	

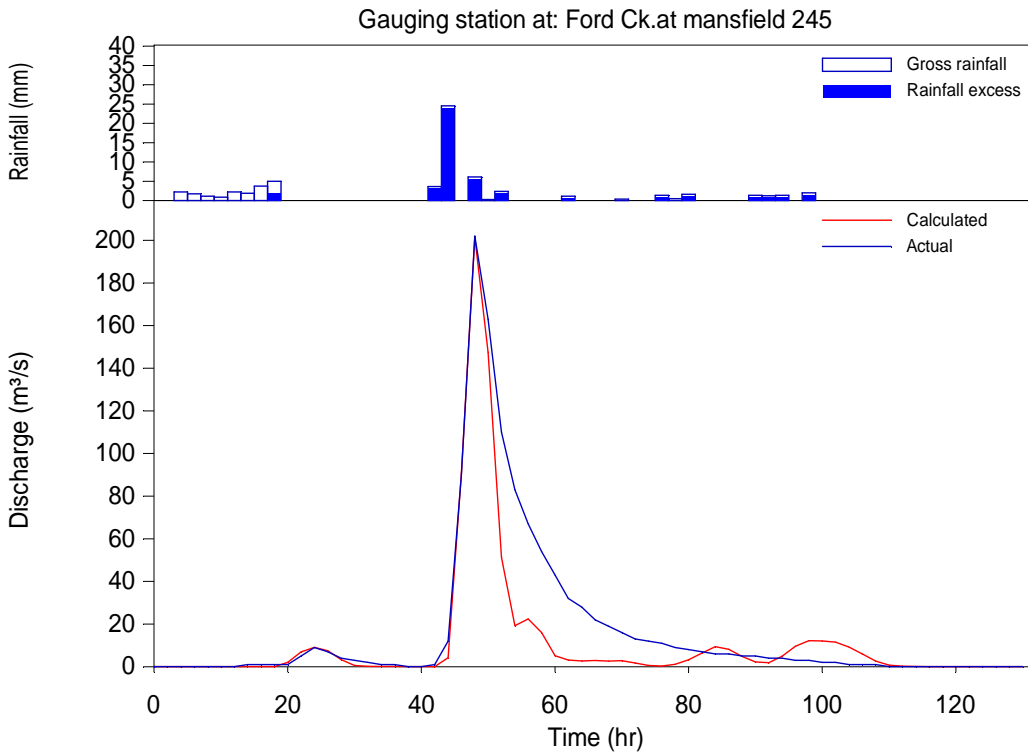


Figure 5-4 RORB Model Calibration – Fords Creek at Mansfield for September 1975 flood event

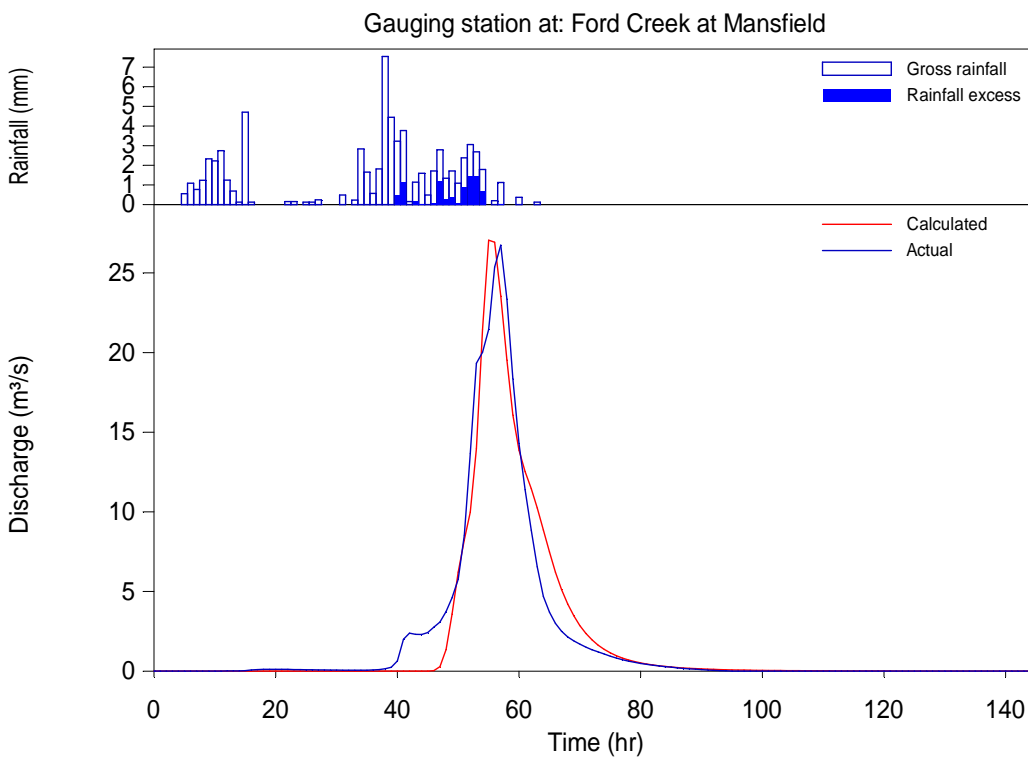


Figure 5-5 RORB Model Calibration – Fords Creek at Mansfield for September 1998 flood event

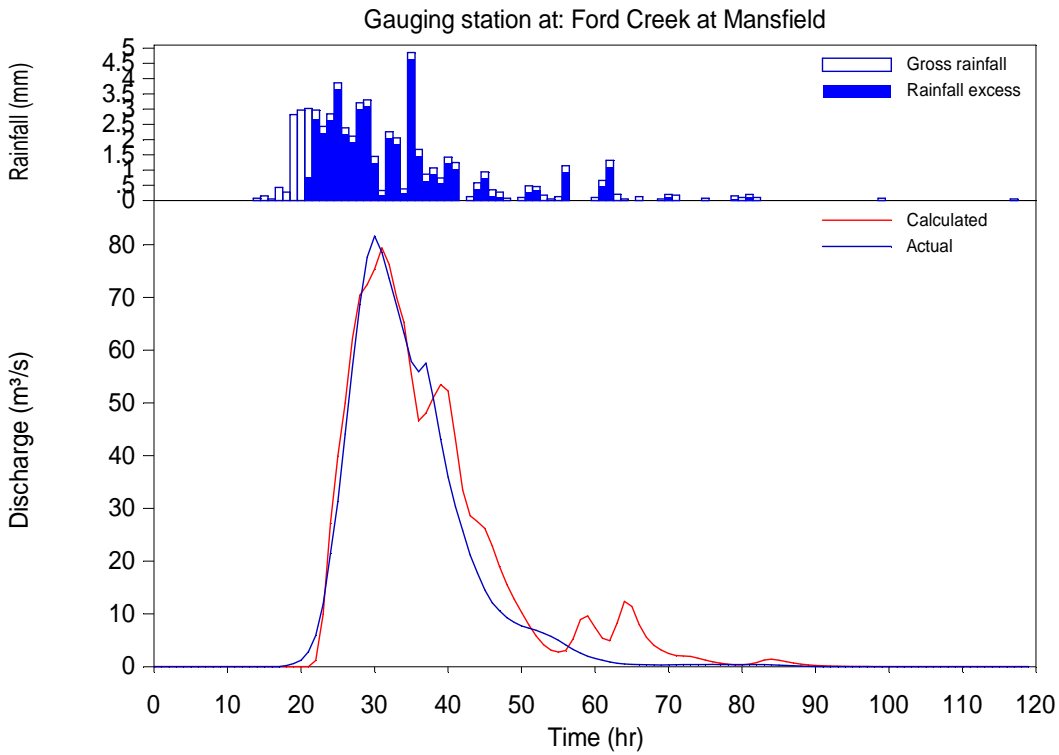


Figure 5-6 RORB Model Calibration – Fords Creek at Mansfield for September 2010 flood event

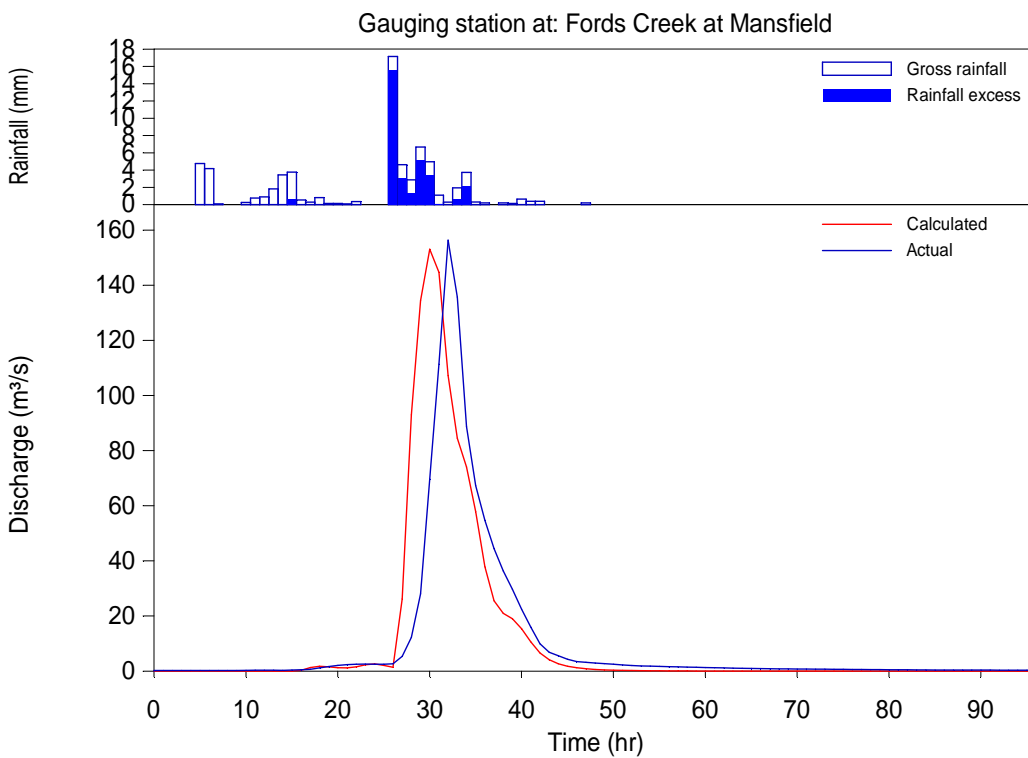


Figure 5-7 RORB Model Calibration – Fords Creek at Mansfield for December 2010 flood event

### 5.4 Big River subcatchment

The RORB model with revised catchment subdivision was calibrated to streamflow data at the Big River at downstream of Frenchmans Creek (405264) and Jamieson (405227) streamflow gauges for the September 1998 and September 2010 flood events. For both events only fair calibrations were achieved, as shown in Figure 5-8 to Figure 5-11. The rainfall coverage of this catchment was relatively poor, leading to the relatively poor calibration performance in this subcatchment. The calibrated parameter values are shown in Table 5-1.

As discussed in Section 8, in the verification process it was identified that the Eildon Southern Area RORB model subcatchments, including the Big River, were better modelled using an initial loss runoff coefficient model and the events were therefore also calibrated using an initial loss runoff coefficient model.

Table 5-3 Parameters of Big River RORB Model fitted to calibration events

Parameter	September 1998		September 2010		Adopted Routing Parameters	
	Downstream Frenchmans Creek gauge	Jamieson gauge	Downstream Frenchmans Creek gauge	Jamieson gauge	Downstream Frenchmans Creek gauge	Jamieson gauge
Routing parameter ( $k_c$ )	30	37	30	16	32	28
Non-linearity parameter ( $m$ )	0.8	0.8	0.8	0.8	0.8	0.8
Event Initial Loss ( $IL$ ) (mm)	35	32	5	18		
Event Runoff Coefficient	90%	90%	70%	95%		

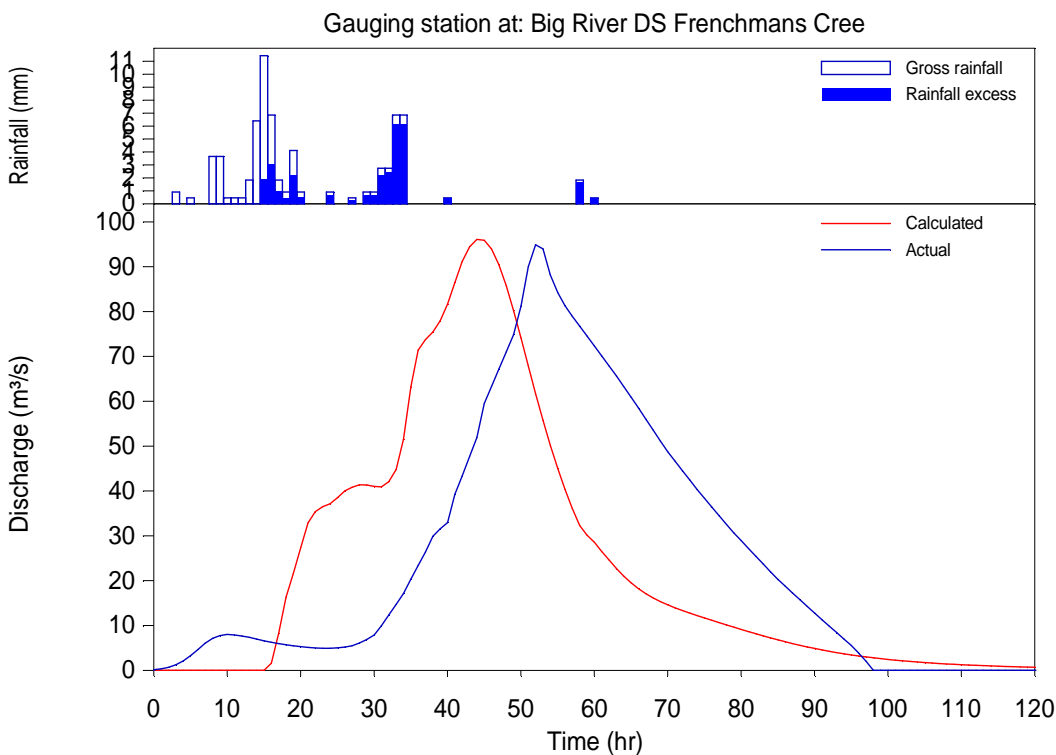


Figure 5-8 RORB Model Calibration – Big River at downstream of Frenchmans Creek gauge for September 1998 flood event

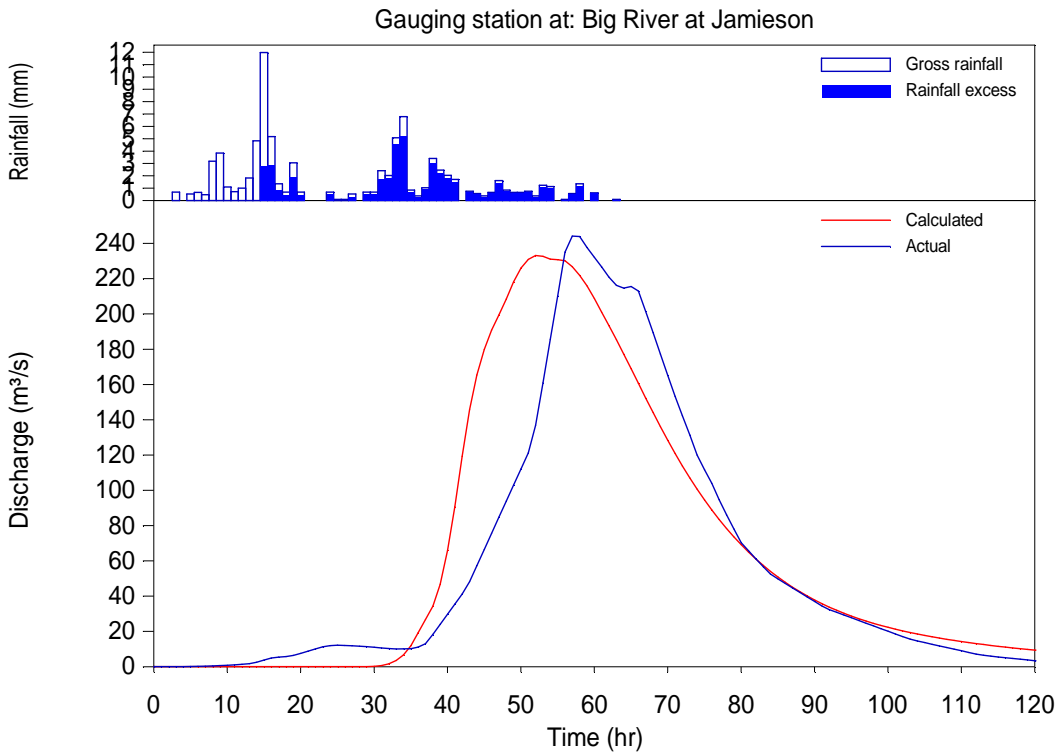


Figure 5-9 RORB Model Calibration – Big River at Jamieson gauge for September 1998 flood event

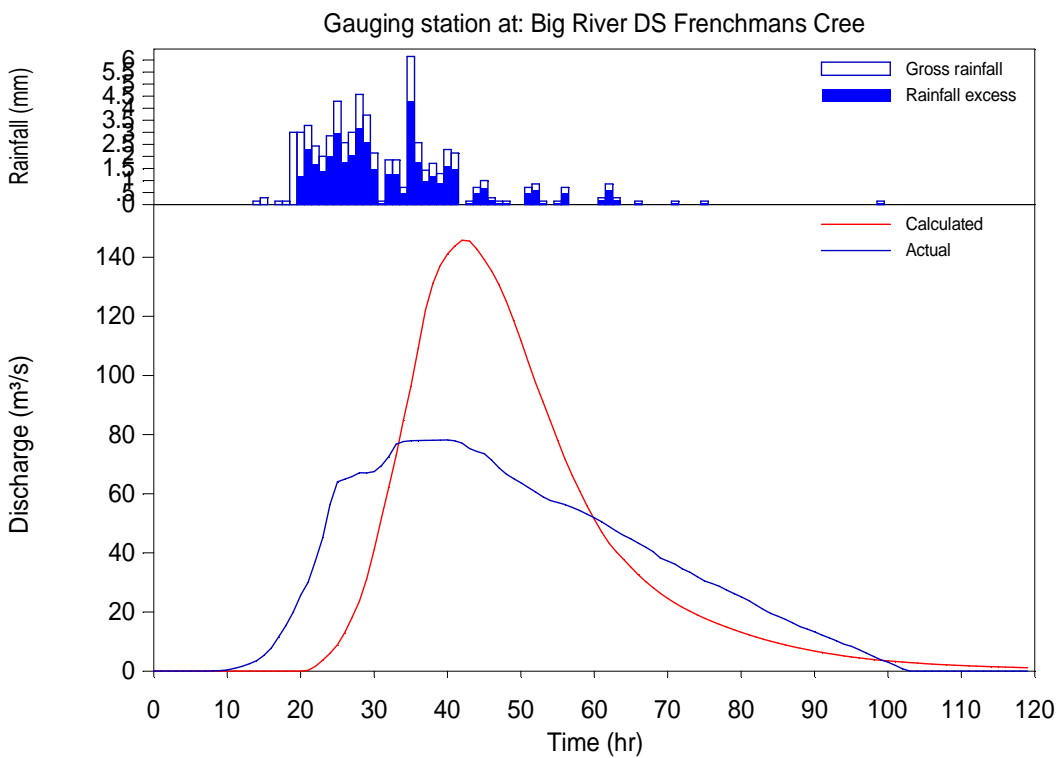


Figure 5-10 RORB Model Calibration – Big River at downstream of Frenchmans Creek gauge for September 2010 flood event

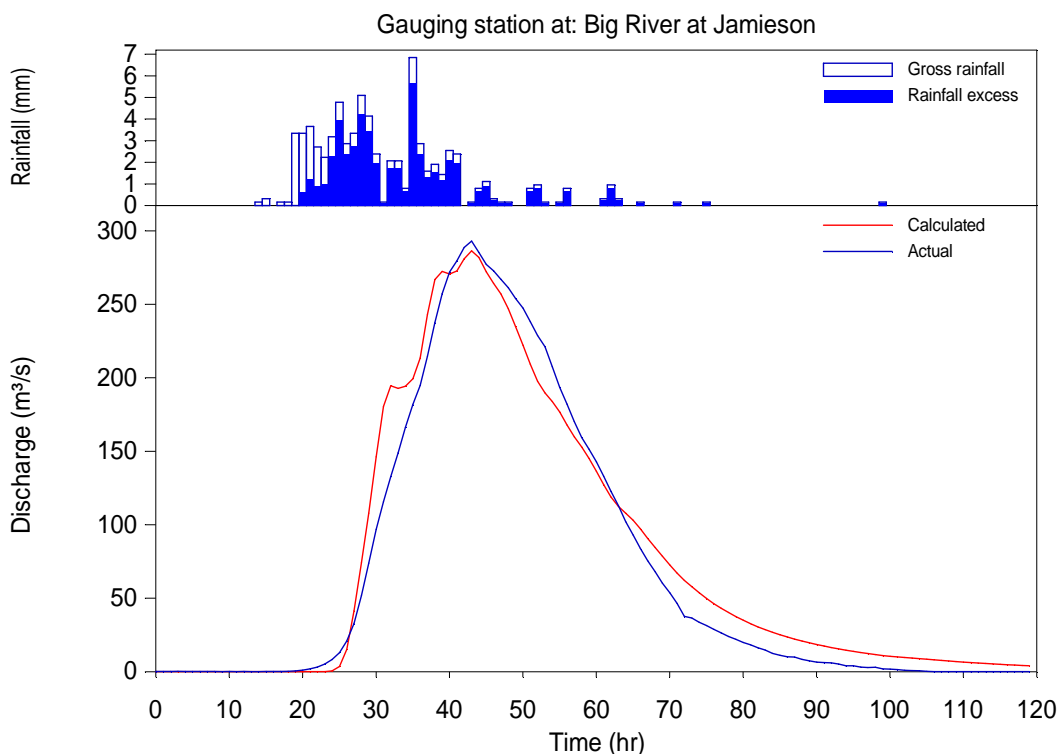


Figure 5-11 RORB Model Calibration – Big River at Jamieson gauge for September 2010 flood event

## 5.5 Upper Goulburn River subcatchment

The RORB model with revised catchment subdivision was calibrated to streamflow data at the Goulburn River at upstream of Snake Creek (405263) and Dohertys (405219) streamflow gauges for the September 1998 and September 2010 flood events. For both events reasonable calibrations were achieved, as shown in Figure 5-12 to Figure 5-15. The calibrated parameter values are shown in Table 5-4.

As discussed in Section 8, in the verification process it was identified that the Eildon Southern Area RORB model subcatchments, including the Goulburn River, were better modelled using an initial loss runoff coefficient model and the events were therefore also calibrated using an initial loss runoff coefficient model.

Table 5-4 Parameters of Goulburn River RORB Model fitted to calibration events

Parameter	September 1998		September 2010		Adopted Routing Parameters	
	Upstream of Snake Creek gauge	Dohertys gauge	Upstream of Snake Creek gauge	Dohertys gauge	Upstream of Snake Creek gauge	Dohertys gauge
Routing parameter ( $k_c$ )	44	48	50	27	47	39
Non-linearity parameter ( $m$ )	0.8	0.8	0.8	0.8	0.8	0.8
Event Initial Loss ( $IL$ ) (mm)	88	55	0	22		
Event Runoff Coefficient	95%	90%	70%	26%		

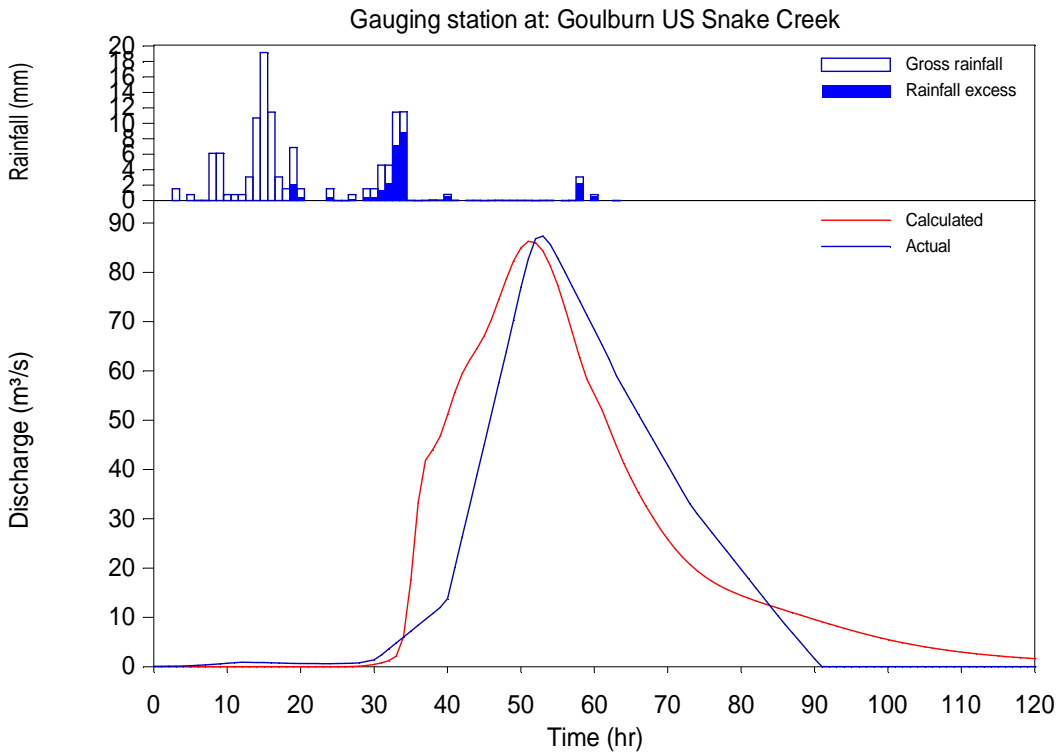


Figure 5-12 RORB Model Calibration – Goulburn River at upstream of Snake Creek gauge for September 1998 flood event

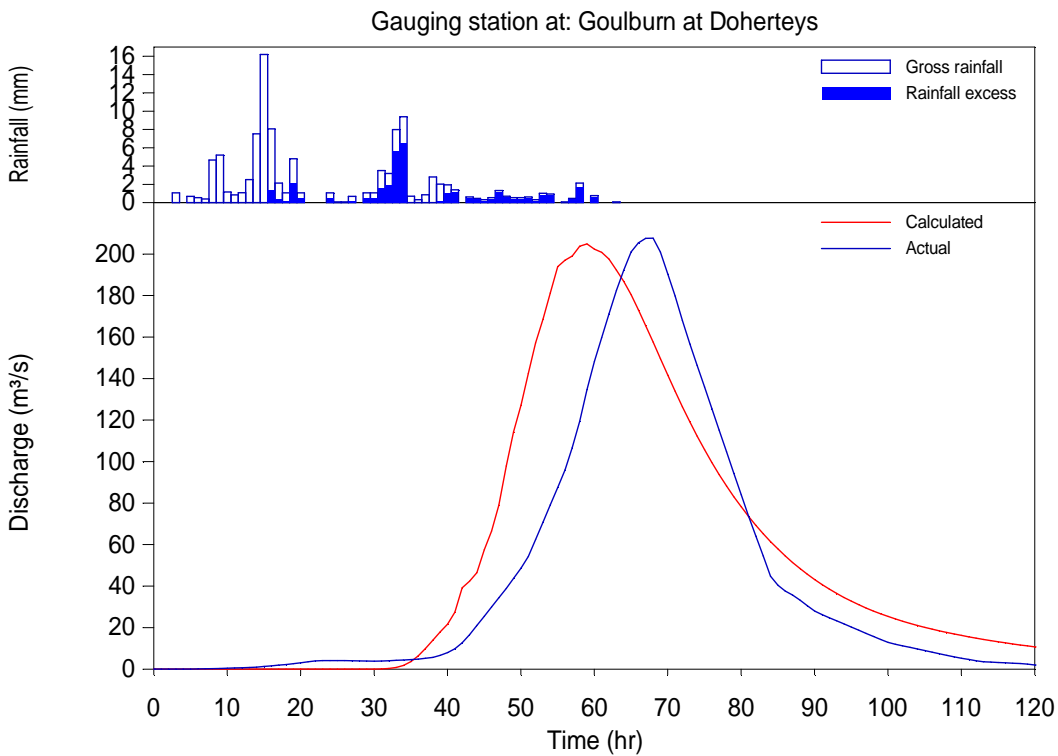


Figure 5-13 RORB Model Calibration – Goulburn River at Doherteys gauge for September 1998 flood event

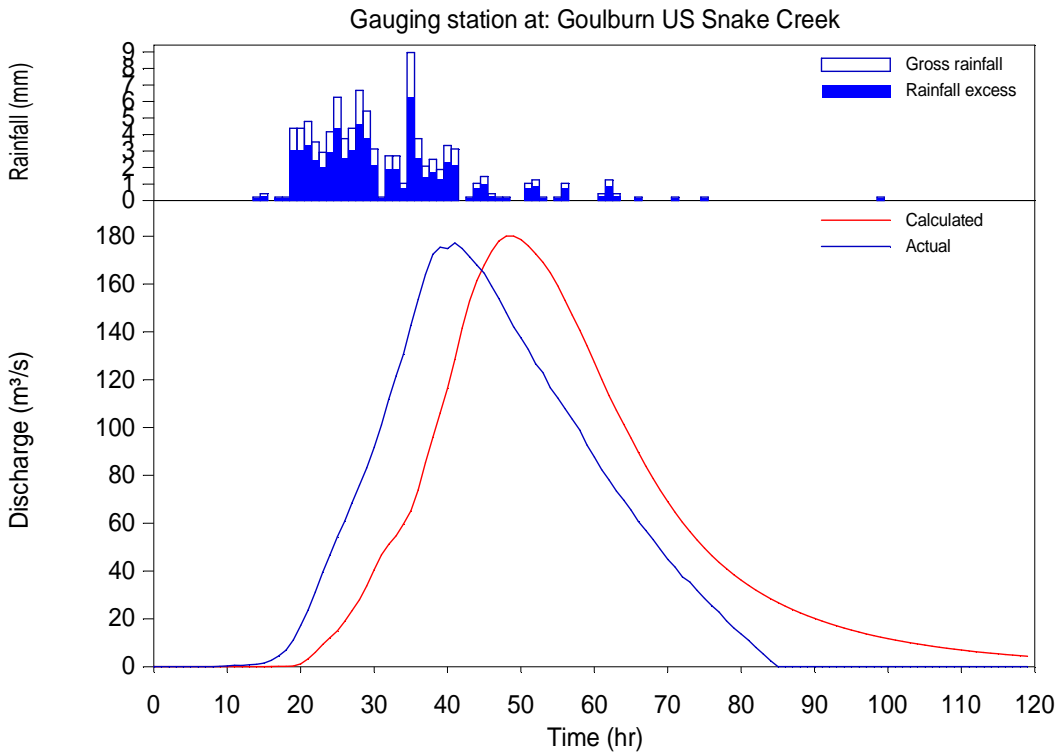


Figure 5-14 RORB Model Calibration – Goulburn River at upstream of Snake Creek gauge for September 2010 flood event

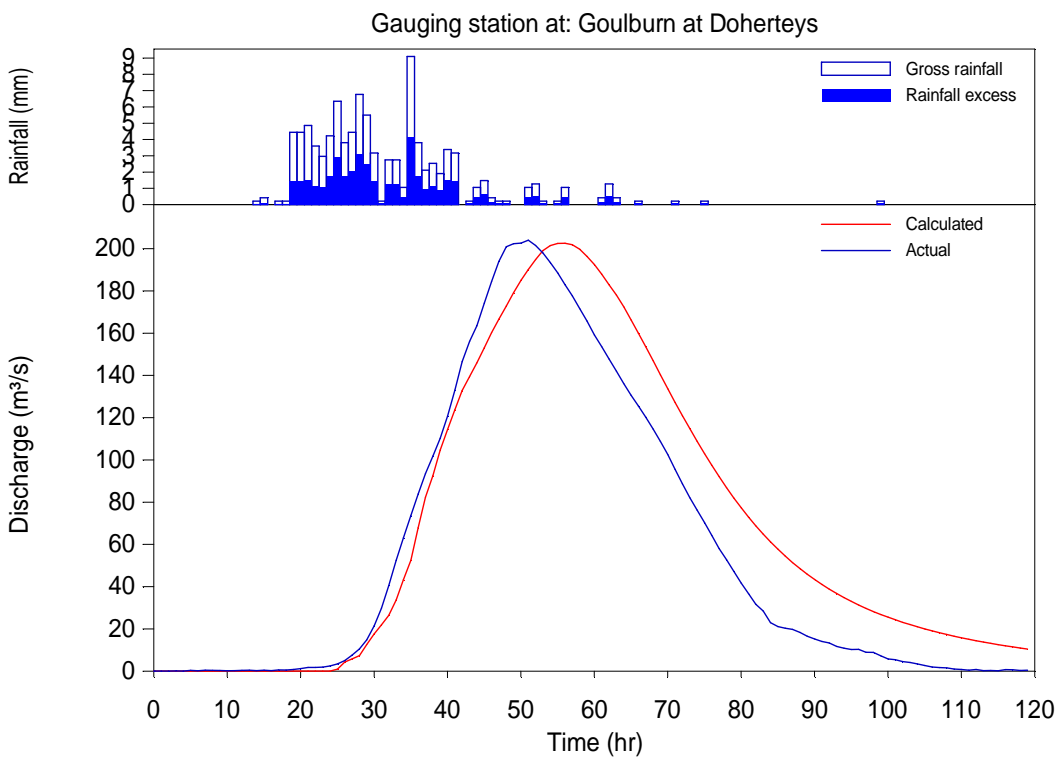


Figure 5-15 RORB Model Calibration – Goulburn River at Doherteys gauge for September 2010 flood event

### 5.6 Jamieson River subcatchment

The RORB model with revised catchment subdivision was calibrated to streamflow data at the Jamieson River at Gerrang Bridge (405218) streamflow gauge for the September 1998 and September 2010 flood events. For both events reasonable calibrations were achieved, as shown in Figure 5-16 and Figure 5-17. The calibrated parameter values are shown in Table 5-5.

As discussed in Section 8, in the verification process it was identified that the Eildon Southern Area RORB model subcatchments, including the Jamieson River, were better modelled using an initial loss runoff coefficient model and the events were therefore also calibrated using an initial loss runoff coefficient model.

Table 5-5 Parameters of Jamieson River RORB Model fitted to calibration events

Parameter	September 1998	September 2010	Adopted Routing Parameters
Routing parameter ( $k_c$ ) to streamflow gauge at Gerrang Bridge	30	50	39
Non-linearity parameter ( $m$ )	0.8	0.8	0.8
Event Initial loss ( $IL$ )	30 mm	0 mm	
Runoff Coefficient ( $RC$ )	90%	69%	

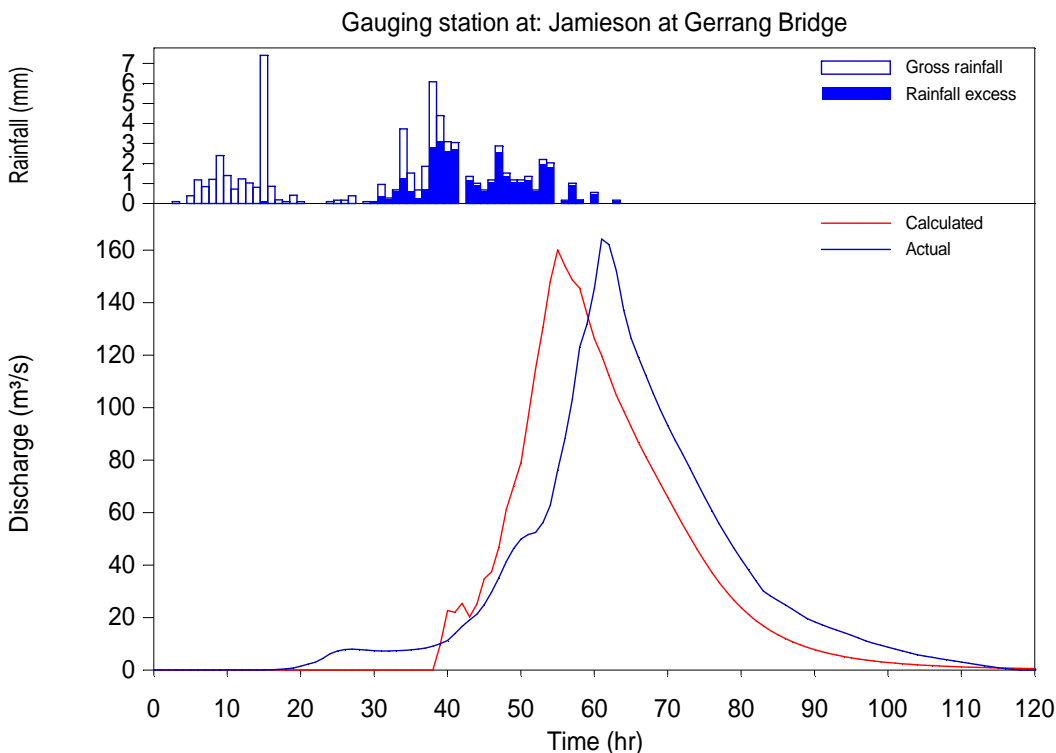


Figure 5-16 RORB Model Calibration – Jamieson River at Gerrang Bridge gauge for September 1998 flood event

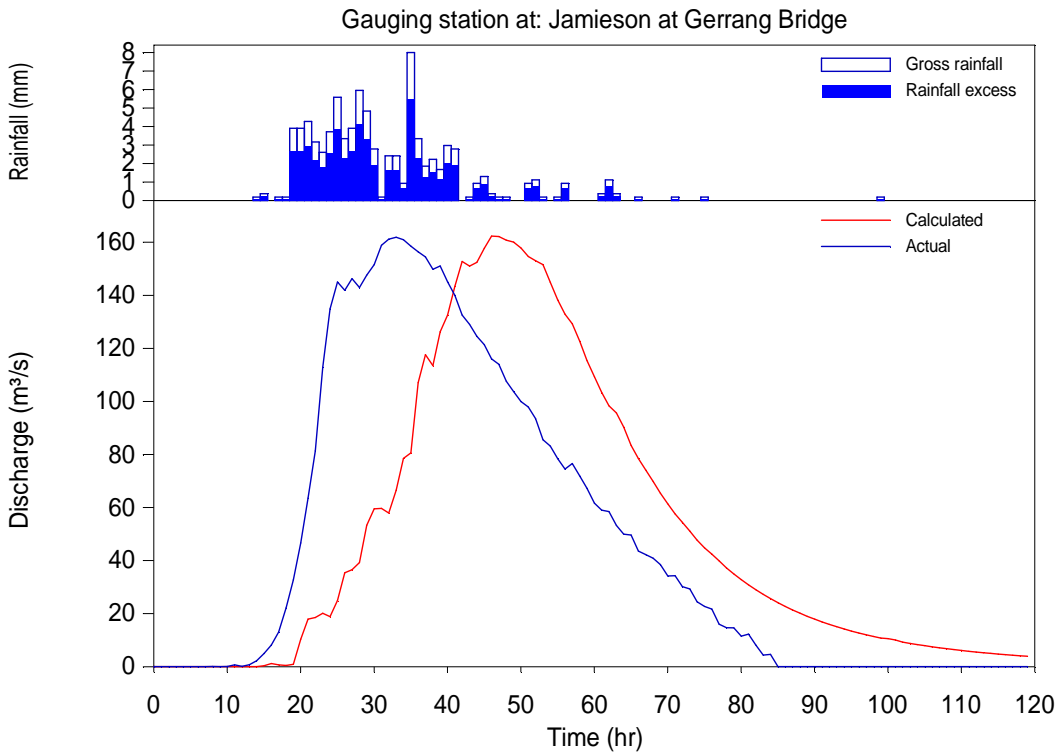


Figure 5-17 RORB Model Calibration – Jamieson River at Gerrang Bridge gauge for September 2010 flood event

## 5.7 Howqua River subcatchment

The RORB model with revised catchment subdivision was calibrated to streamflow data at the Howqua River at Glen Esk (405215) streamflow gauge for the September 1998 and September 2010 flood events. For both events reasonable calibrations were achieved, as shown in Figure 5-18 and Figure 5-19. The calibrated parameter values are shown in Table 5-6.

As discussed in Section 8, in the verification process it was identified that the Eildon Southern Area RORB model subcatchments, including the Howqua River, were better modelled using an initial loss runoff coefficient model and the events were therefore also calibrated using an initial loss runoff coefficient model.

Table 5-6 Parameters of Howqua River RORB Model fitted to calibration events

Parameter	September 1998	September 2010	Adopted Routing Parameters
Routing parameter ( $k_c$ ) to streamflow gauge at Glen Esk	40	38	39
Non-linearity parameter ( $m$ )	0.8	0.8	0.8
Event Initial loss ( $IL$ )	10 mm	10 mm	
Runoff Coefficient ( $RC$ )	33%	42%	

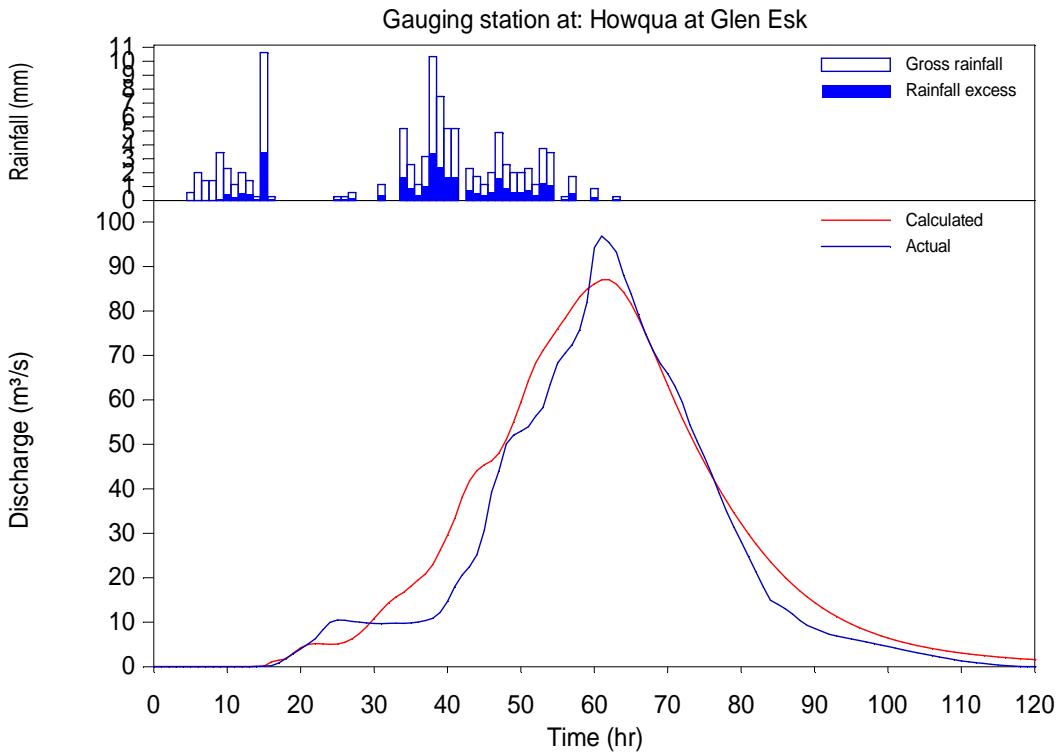


Figure 5-18 RORB Model Calibration – Howqua River at Glen Esk gauge for September 1998 flood event

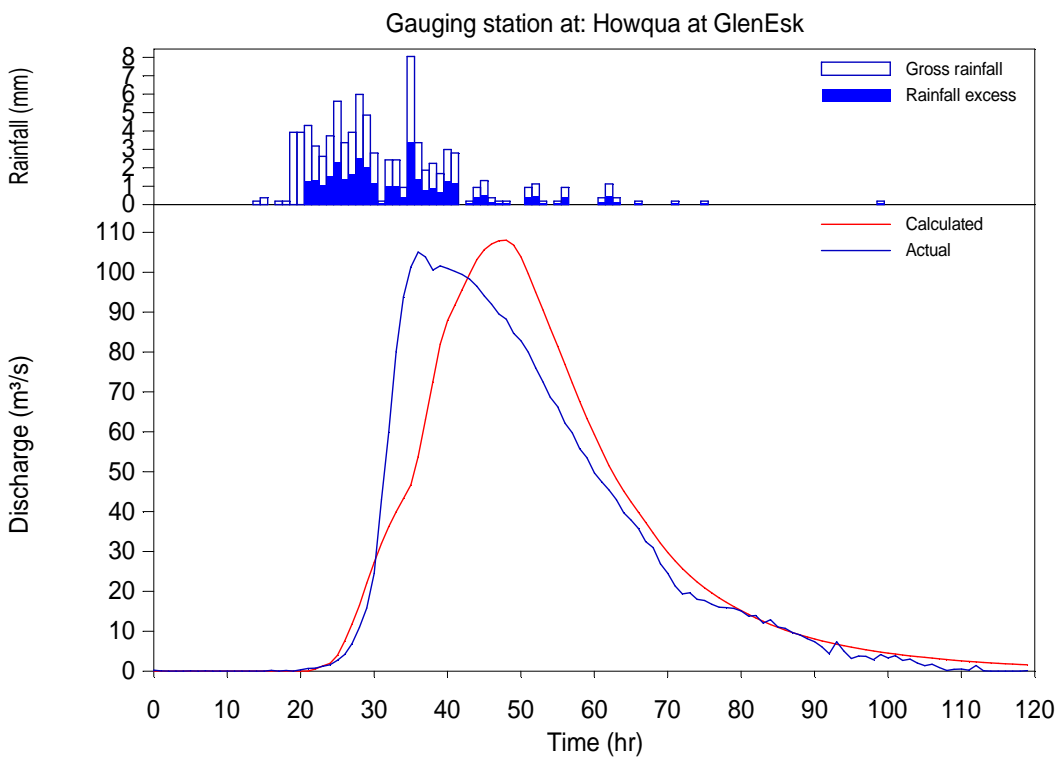


Figure 5-19 RORB Model Calibration – Howqua River at Glen Esk gauge for September 2010 flood event

## 5.8 Broken River catchment

The only streamflow gauge in the Broken River catchment to Lake Nillahcootie is the head gauge within the Lake Nillahcootie reservoir. Recorded water levels in Lake Nillahcootie were used with the rating curve for the Lake Nillahcootie spillway from the existing RORB model (Sinclair Knight Merz, 1998) to provide gauged hydrographs for the outflows from Lake Nillahcootie. This provided a less than ideal basis for calibration of the RORB model for the catchment upstream of the dam, as the routing effect of the dam and reservoir appreciably attenuates the shape of the inflow to the outflow hydrograph. The initial catchment loss may be difficult to estimate for each event, as some of the apparent initial loss may represent filling of initial drawdown of the reservoir before each event.

The RORB model with revised catchment subdivision was calibrated to the estimated outflow hydrograph from Lake Nillahcootie for the May 1974, September 1993, October 2010 and December 2010 flood events. For all events reasonable calibrations were achieved, as shown in Figure 5-20 to Figure 5-27. The calibrated parameter values are shown in Table 5-7.

Table 5-7 Parameters of Broken River to Lake Nillahcootie RORB Model fitted to calibration events

Parameter	May 1974	October 1993	September 2010	December 2010	Adopted Routing Parameters
Routing parameter ( $k_c$ ) for catchment	35	25	22	32	29
Non-linearity parameter ( $m$ )	0.8	0.8	0.8	0.8	0.8
Event Initial loss ( $IL$ )	50 mm	25 mm	25 mm	45 mm	
Event Continuing loss ( $CL$ )	0.68 mm/h	1.60 mm/h	0.60 mm/h	0.68 mm/h	

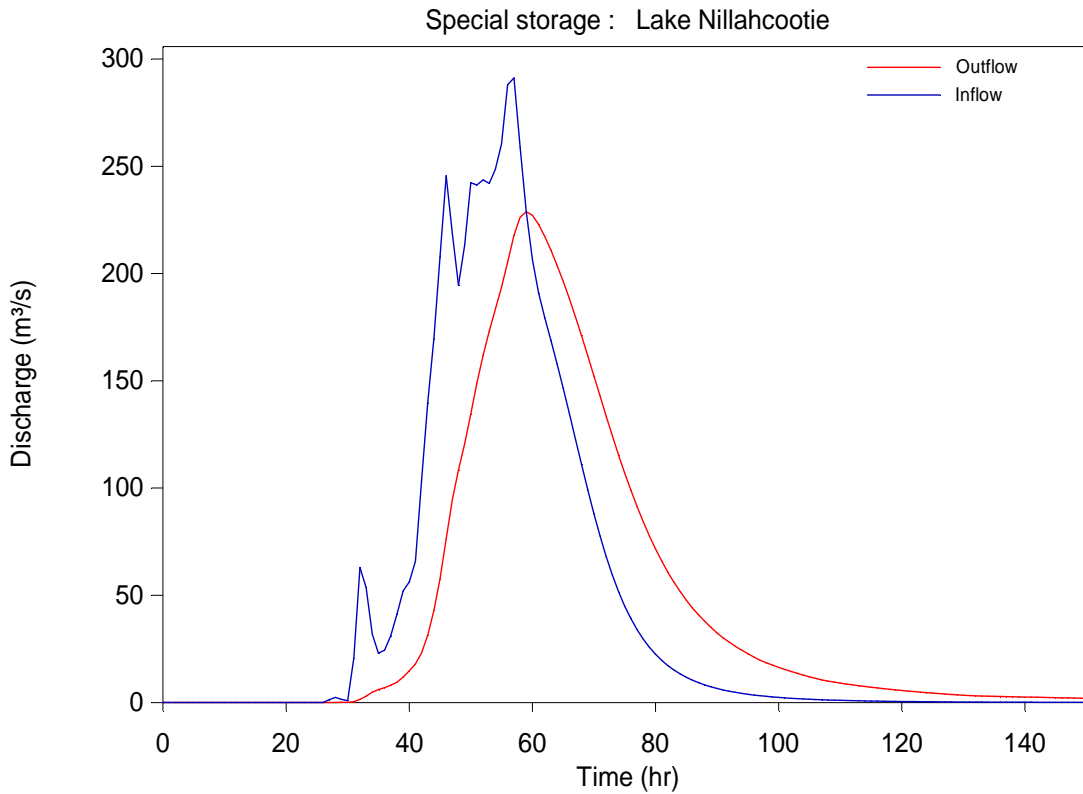


Figure 5-20 RORB Model Calibration – Inflows and outflows from Lake Nillahcootie for the May 1974 event

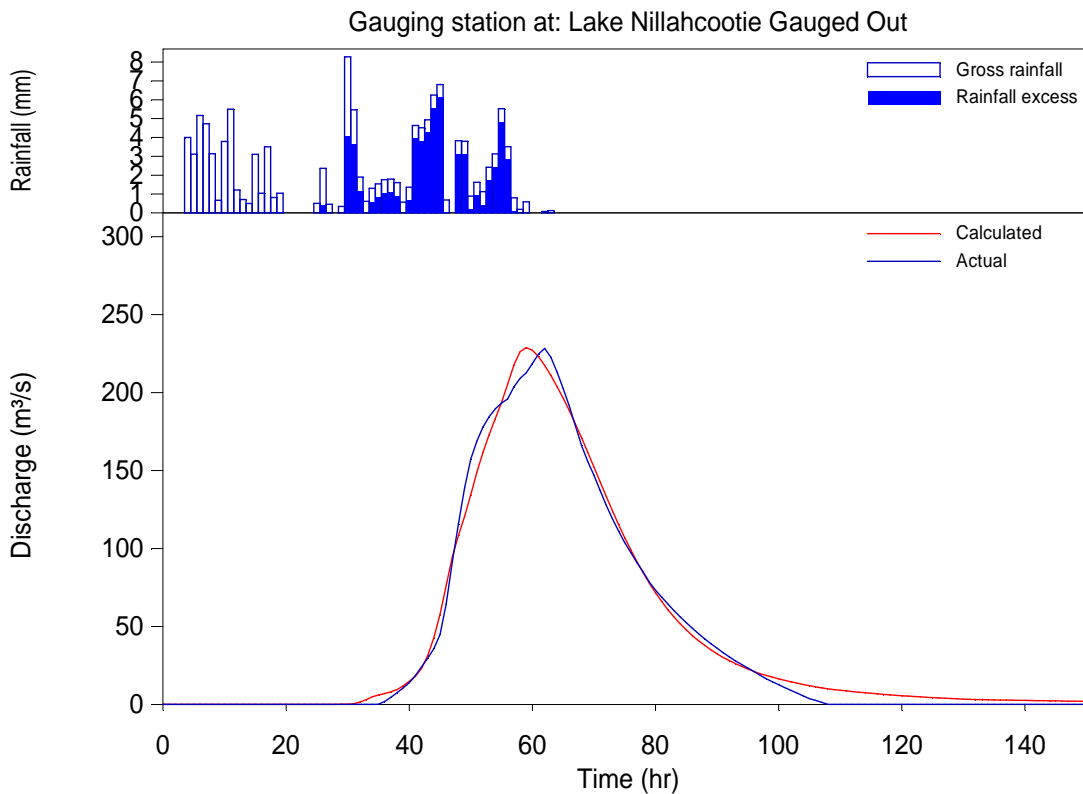


Figure 5-21 RORB Model Calibration – Gauged and modelled outflows from Lake Nillahcootie for the May 1974 event

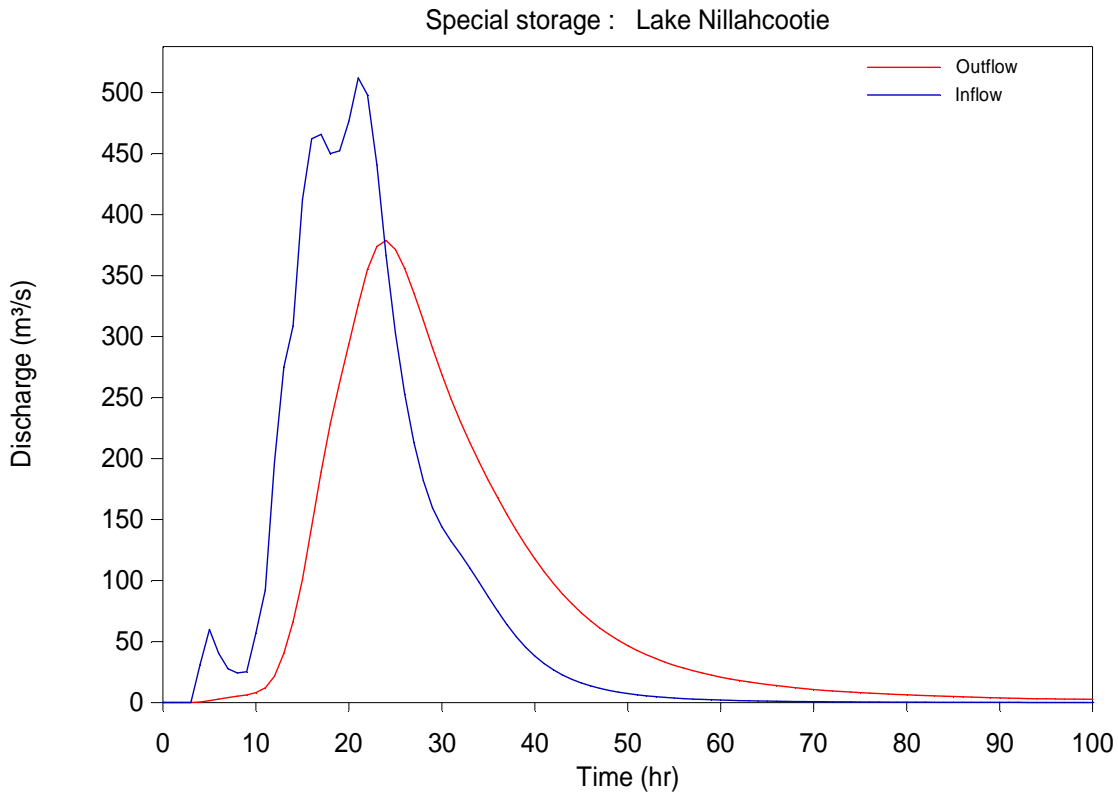


Figure 5-22 RORB Model Calibration – Inflows and outflows from Lake Nillahcootie for the October 1993 event

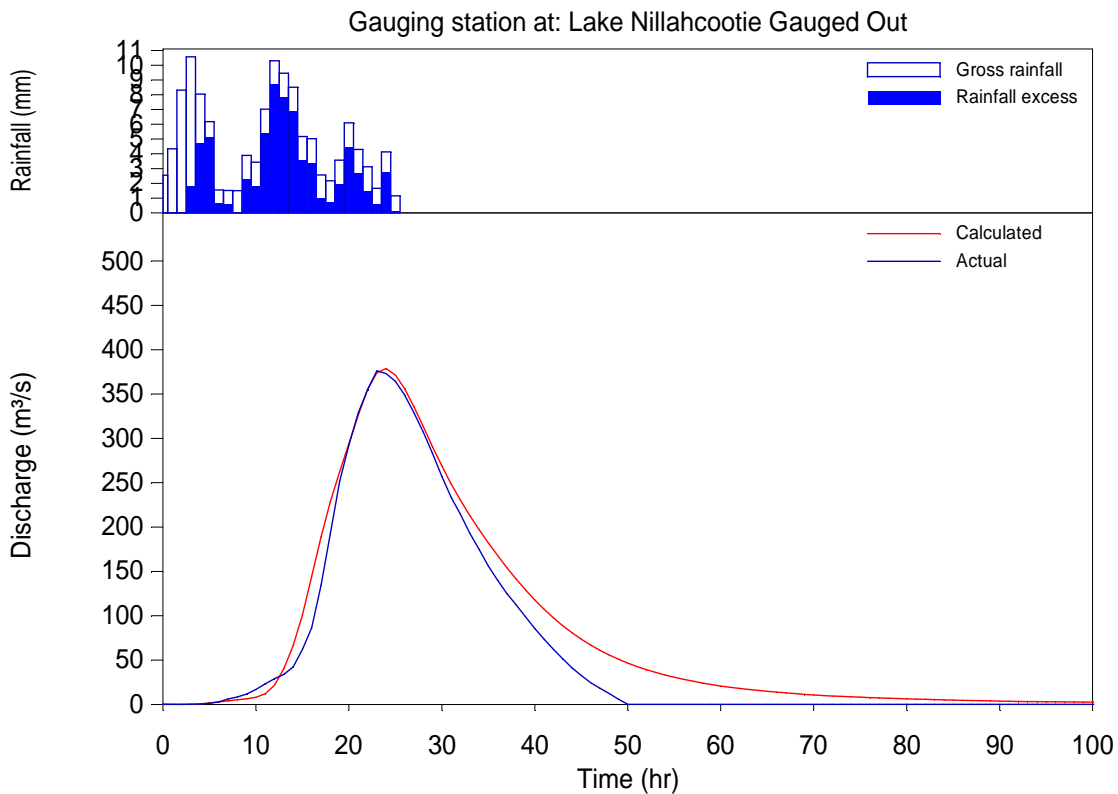


Figure 5-23 RORB Model Calibration – Gauged and modelled outflows from Lake Nillahcootie for the October 1993 event

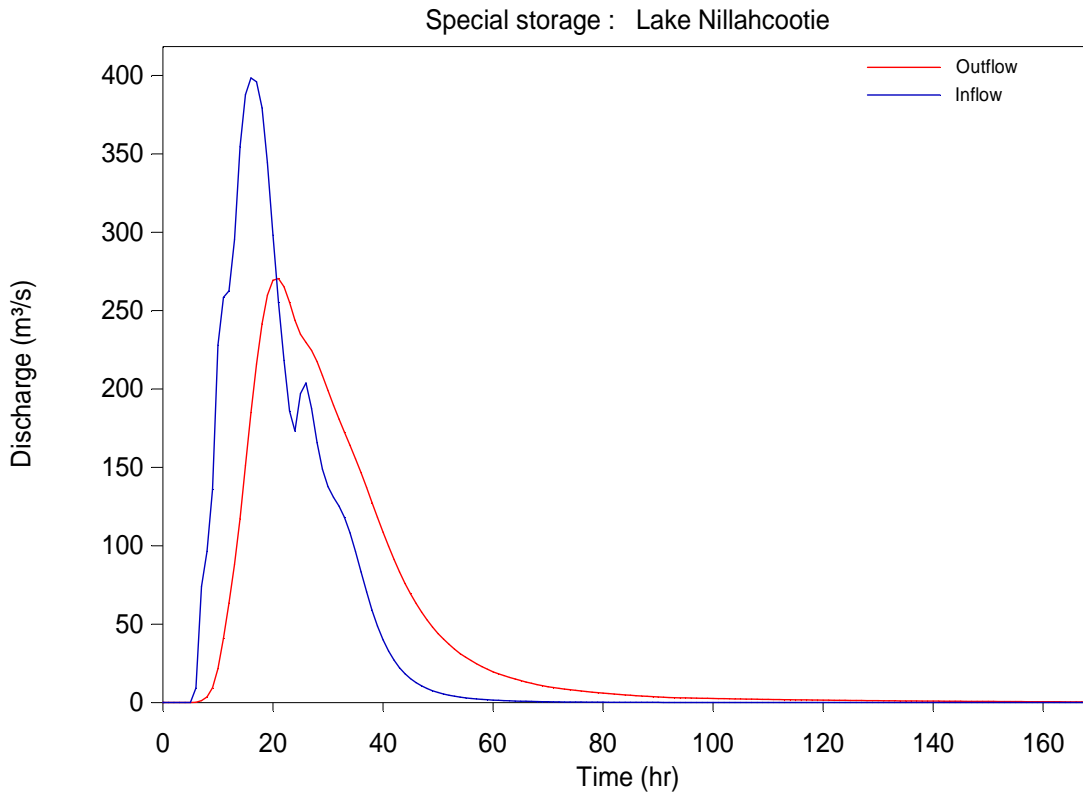


Figure 5-24 RORB Model Calibration – Inflows and outflows from Lake Nillahcootie for the September 2010 event

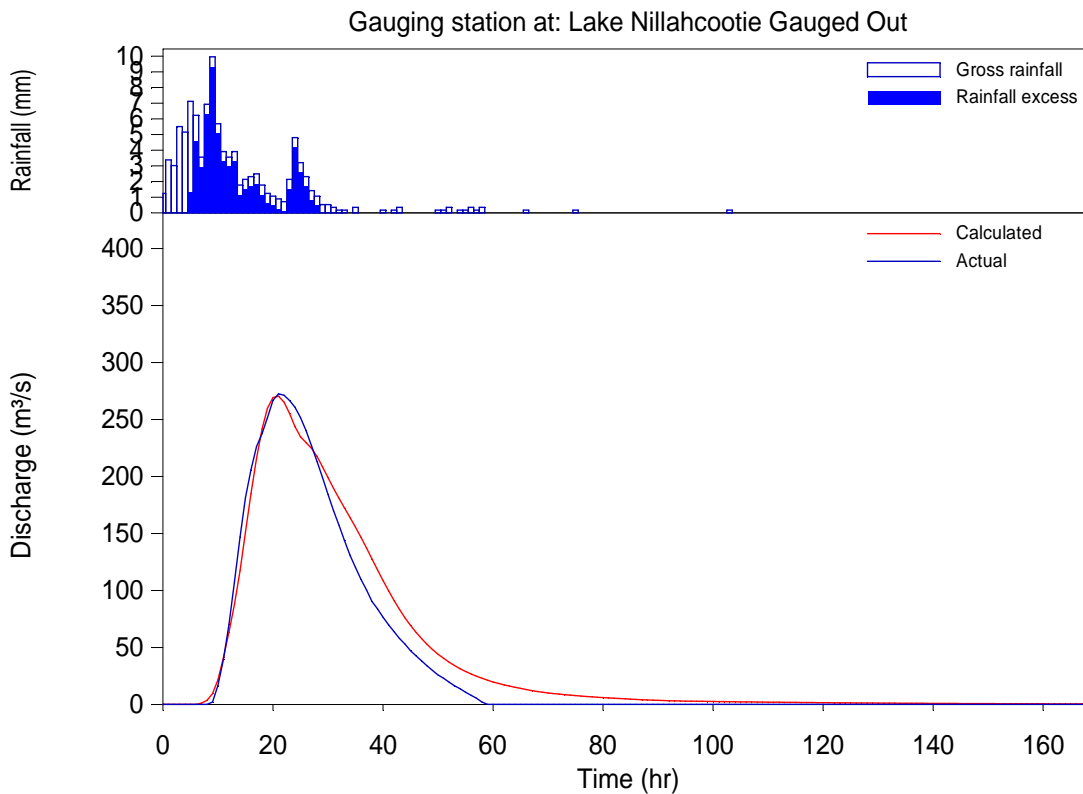


Figure 5-25 RORB Model Calibration – Gauged and modelled outflows from Lake Nillahcootie for the September 2010 event

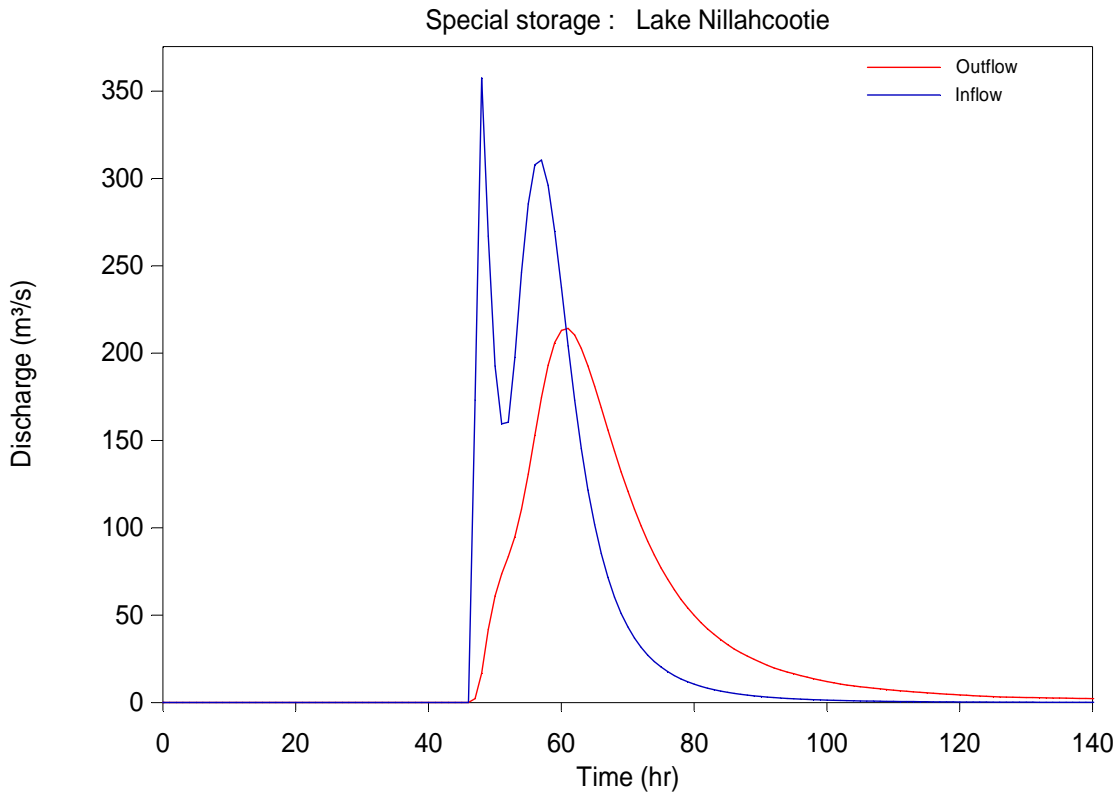


Figure 5-26 RORB Model Calibration – Inflows and outflows from Lake Nillahcootie for the December 2010 event

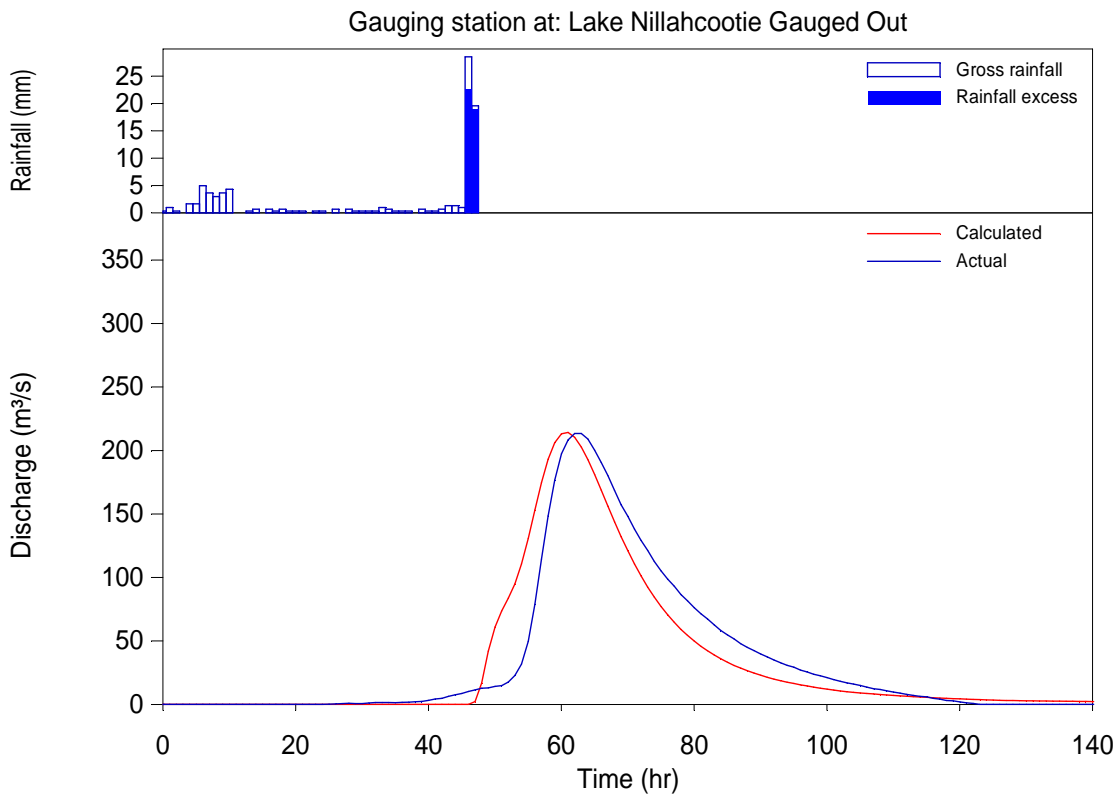


Figure 5-27 RORB Model Calibration – Gauged and modelled outflows from Lake Nillahcootie for the December 2010 event

## 6. Flood frequency analysis

### 6.1 Approach to flood frequency analysis

Flood frequency analysis is used to verify the parameter values adopted in the Monte-Carlo RORB model simulations. An at-site flood frequency analysis was conducted using gauged annual maxima flows extracted from each calendar year (i.e. water year runs January to December). Annual maxima series of inflows for Eildon Dam were derived from a series discussed in Section 6.3.

Flood frequency curves were fitted at each site using the FLIKE flood frequency analysis software (BMT WBM, 2015). FLIKE is the flood frequency analysis software that is recommended in the revised 2015 draft of *Australian Rainfall and Runoff* (Engineers Australia, 2015). The best fit to the observed annual maxima was achieved by fitting a Generalised Extreme Value (GEV) distribution using the Bayesian fitting approach. All annual maxima were included in the flood frequency analysis and censoring was only performed on the outflows from Lake Nillahcootie. The peaks were fitted with the assumption of no rating error in the estimation of the peak flows for the observed events from gauged water level peak heights. FLIKE provided the best fit flood frequency curve to the gauged annual maxima and also 5% and 95% confidence limits on the fitted flood frequency curves.

### 6.2 Annual maxima gauged flows

Annual maxima of gauged flows that were used in the flood frequency analysis are listed in Table 6-1.

Table 6-1 : List of annual maxima of gauged flows at gauges used for flood frequency analysis

	405245	405214	405263	405219	405218	405227	405215
Year	Fords Creek at Mansfield	Delatite River at Tonga Bridge	Goulburn River upstream of Snake Creek	Goulburn River at Doherteys	Jamieson River at Gerrang Bridge	Big River at Jamieson	Howqua River at Glen Esk
1957	-	49.5	-	-	-	-	-
1958	-	145.2	-	-	-	-	-
1959	-	59.4	-	-	-	-	-
1960	-	137.9	-	-	52.3	-	-
1961	-	29.5	-	-	22.6	-	-
1962	-	61.5	-	-	70.8	-	-
1963	-	75.3	-	-	54.7	-	-
1964	-	171.6	-	-	129.5	-	-
1965	-	77.5	-	-	51.4	-	-
1966	-	55.7	-	-	104.0	-	-
1967	-	18.0	-	-	25.6	-	-
1968	-	75.2	-	178.0	149.0	-	-
1969	-	50.2	-	43.5	66.2	-	-
1970	-	164.5	-	134.9	97.8	-	-
1971	34.2	90.2	-	137.5	83.7	102.7	-
1972	6.0	16.2	-	52.6	48.2	40.6	-
1973	58.7	118.7	-	116.2	72.5	95.7	-
1974	45.9	97.7	-	164.1	133.6	124.7	-
1975	234.9	468.7	-	143.2	142.6	89.4	83.9
1976	17.7	57.4	32.3	57.0	23.6	35.5	19.2
1977	9.5	38.6	31.9	73.6	51.7	60.3	38.8

	405245	405214	405263	405219	405218	405227	405215
Year	Fords Creek at Mansfield	Delatite River at Tonga Bridge	Goulburn River upstream of Snake Creek	Goulburn River at Dohertys	Jamieson River at Gerrang Bridge	Big River at Jamieson	Howqua River at Glen Esk
1978	71.1	153.2	39.7	101.3	62.2	69.4	69.3
1979	17.4	53.4	57.1	141.0	98.6	115.0	79.9
1980	32.7	69.2	34.2	79.0	40.5	61.7	39.8
1981	45.9	81.8	53.7	126.9	93.8	121.1	73.8
1982	0.5	13.4	10.4	14.1	13.4	14.6	10.7
1983	30.1	85.8	23.6	68.5	51.9	66.1	44.2
1984	32.8	96.8	43.5	99.2	64.0	108.6	62.6
1985	31.1	76.0	45.3	112.7	76.4	105.5	67.6
1986	39.0	98.2	59.2	163.6	119.4	114.9	98.7
1987	47.5	78.2	31.2	63.5	40.1	53.9	35.3
1988	39.0	83.1	41.0	115.2	89.1	113.4	48.4
1989	41.2	88.2	28.7	70.6	49.1	54.9	38.6
1990	72.5	123.1	41.1	96.0	85.9	101.7	56.9
1991	40.3	88.2	59.0	130.3	74.7	105.4	76.4
1992	80.4	226.9	30.8	72.7	64.7	62.2	46.5
1993	167.1	277.4	114.3	179.0	119.4	92.0	83.4
1994	0.9	9.8	15.2	34.8	49.0	29.8	22.5
1995	81.6	145.7	53.0	150.8	140.5	133.0	124.3
1996	68.2	186.4	64.8	171.1	143.0	181.1	72.7
1997	19.5	24.6	13.8	39.5	36.9	32.8	23.0
1998	27.4	142.6	97.0	223.0	177.1	257.3	105.2
1999	37.7	77.1	29.5	69.0	40.1	47.9	26.9
2000	61.3	73.3	38.3	101.9	76.4	97.2	54.7
2001	3.6	38.1	29.3	70.3	49.4	46.1	29.5
2002	0.8	9.7	18.7	34.6	42.3	22.0	21.6
2003	48.7	111.1	30.2	85.5	52.0	85.1	47.8
2004	24.0	70.0	25.8	68.0	33.9	49.2	31.3
2005	68.9	125.4	74.5	167.0	72.5	154.0	44.2
2006	0.2	2.7	3.3	10.4	16.6	10.4	9.6
2007	4.9	27.5	126.0	185.9	137.1	33.3	54.8
2008	1.4	7.4	12.0	34.8	21.0	23.7	15.6
2009	2.5	24.8	38.8	86.7	53.6	64.4	30.9
2010	175.6	359.3	198.9	246.8	193.9	323.7	-
2011	19.7	83.0	26.3	75.0	65.5	81.3	-
2012	31.5	35.9	-	155.4	62.7	89.8	30.7
2013	32.8	43.2	-	105.1	90.9	95.4	63.4
2014	15.3	3.3	-	55.9	63.6	42.7	39.0

### **6.3 Annual Maxima of Inflows for Eildon Dam**

Sinclair Knight Merz (2004) examined historical floods at Eildon Dam for the period between 1881 and 2003. The analysis of historical floods in the Eildon catchment is complicated by the fact that there was only a short period of record in the early 20<sup>th</sup> century where direct observations of floods were unaffected by impoundments. Between the construction of Sugarloaf Dam and its subsequent augmentation, streamflow observations were only available for a minor proportion of the upstream catchment. However, since around 1970, streamflow gauging was available on all significant inflow tributaries to Eildon Dam. Sinclair Knight Merz (2004) therefore used data from a number of sources and adjusted for the effect of the impoundments to derive a complete series of annual maxima daily inflow floods to Lake Eildon for the period between 1881 and 2003 inclusive.

Table 6-2 : List of annual maxima of estimated daily inflows to Lake Eildon for 1881 to 2003, from Sinclair Knight Merz (2004)

Year	Annual Maximum Daily Inflow (m <sup>3</sup> /s)	Year	Annual Maximum Daily Inflow (m <sup>3</sup> /s)	Year	Annual Maximum Daily Inflow (m <sup>3</sup> /s)
1882	312	1883	608	1884	176
1885	559	1886	301	1887	797
1888	293	1889	763	1890	407
1891	652	1892	369	1893	576
1894	671	1895	449	1896	284
1897	584	1898	317	1899	608
1900	494	1901	358	1902	162
1903	364	1904	424	1905	534
1906	821	1907	220	1908	302
1909	675	1910	580	1911	738
1912	883	1913	229	1914	122
1915	665	1916	1770	1917	1741
1918	748	1919	232	1920	646
1921	1091	1922	195	1923	885
1924	675	1925	188	1926	361
1927	246	1928	620	1929	218
1930	495	1931	562	1932	654
1933	736	1934	1275	1935	463
1936	636	1937	199	1938	46
1939	971	1940	69	1941	204
1942	671	1943	248	1944	127
1945	175	1946	358	1947	304
1948	247	1949	524	1950	581
1951	589	1952	768	1953	663
1954	435	1955	514	1956	746
1957	183	1958	746	1959	278
1960	320	1961	92	1962	202
1963	282	1964	550	1965	234
1966	404	1967	86	1968	694
1969	227	1970	515	1971	514
1972	178	1973	440	1974	686
1975	810	1976	143	1977	259
1978	346	1979	464	1980	256
1981	513	1982	55	1983	292
1984	385	1985	406	1986	532
1987	227	1988	384	1989	292
1990	391	1991	462	1992	305
1993	759	1994	131	1995	626
1996	815	1997	159	1998	707
1999	210	2000	370	2001	237
2002	121				

## 6.4 Annual Maxima of Outflows from Lake Nillahcootie

Gauged water level data was available for the head gauge at Lake Nillahcootie (404218) for the period between 1973 and 2015. However there were large periods of missing data prior to 1993, so a continuous series was only available for the 22 year period between 1993 and 2014 inclusive. The rating table for Lake Nillahcootie from the Sinclair Knight Merz (1998) RORB model was used to convert gauged levels in Lake Nillahcootie Reservoir to estimated outflows from the dam. Annual maxima were then extracted from this series. There were several years during this period when the dam did not spill at all. There were also several years when the annual maxima spill was relatively small. The resulting series that was therefore used for annual maxima included 11 years when the annual maxima outflow was greater than 10 m<sup>3</sup>/s and 11 years when the annual maxima was less than the 10 m<sup>3</sup>/s threshold. The censored fitting procedure was therefore adopted in FLIKE whereby the actual values of the gauged annual maxima from the largest 11 years (as listed in Table 6-3) were used along with 11 censored years when the annual maxima was unknown but less than 10 m<sup>3</sup>/s.

Table 6-3 : List of annual maxima of outflows from Lake Nillahcootie, estimated from recorded maximum water levels at the head gauge (404218) and the spillway rating curve adopted from the Sinclair Knight Merz (1998) RORB model

Year	Maximum reservoir level (m AHD)	Maximum outflow (m <sup>3</sup> /s)
1993	266.45	382.8
2010	266.07	272.5
1996	265.50	132.8
2005	265.37	107.0
2000	265.12	62.5
2011	265.02	47.5
2012	264.98	41.8
1995	264.84	24.3
2003	264.82	22.2
2013	264.80	19.1
1998	264.75	14.7
11 years between 1993-2014		Less than 10

## 6.5 Flood Frequency Analysis Results

Results from the flood frequency analysis to annual maxima gauged flows for each of the gauges are shown in Figure 6-1 to Figure 6-9 below.

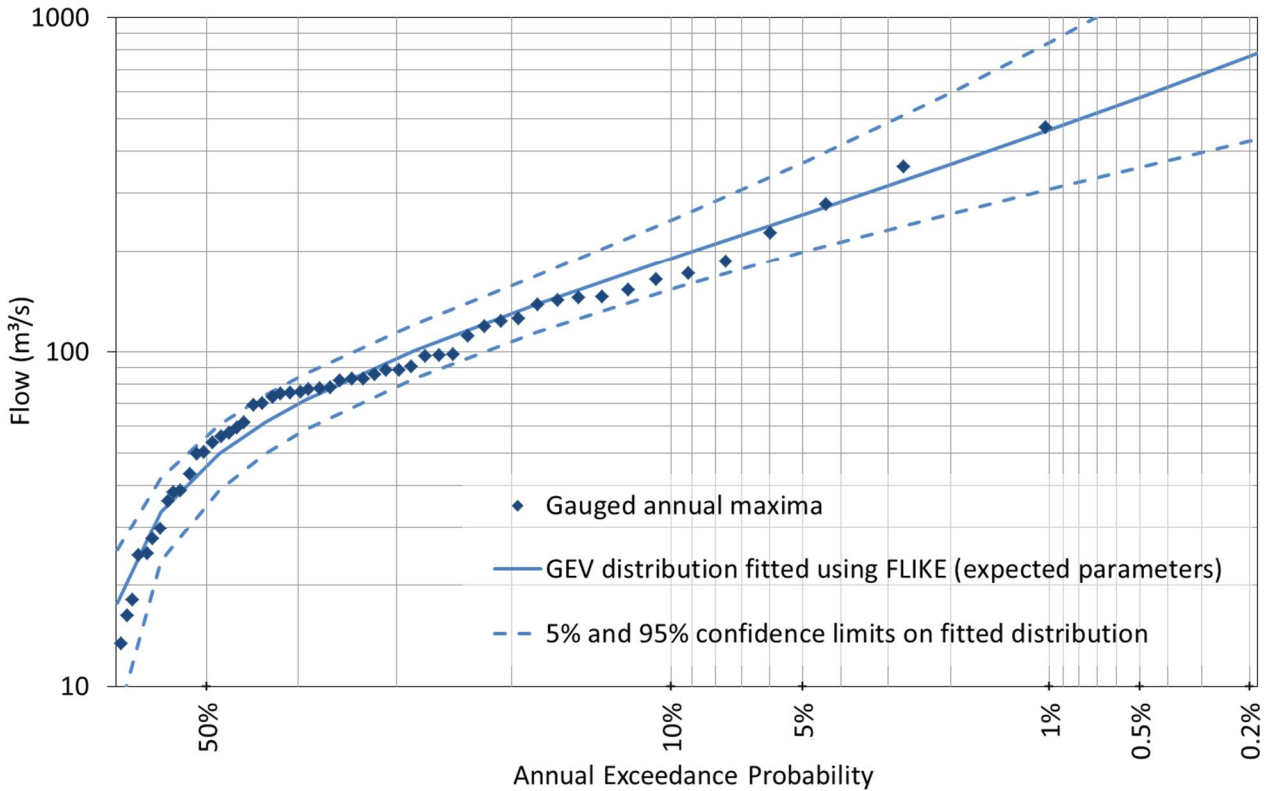


Figure 6-1 Flood frequency analysis fitted to annual maxima gauged data for the Delatite River at Tonga Bridge gauge (405214)

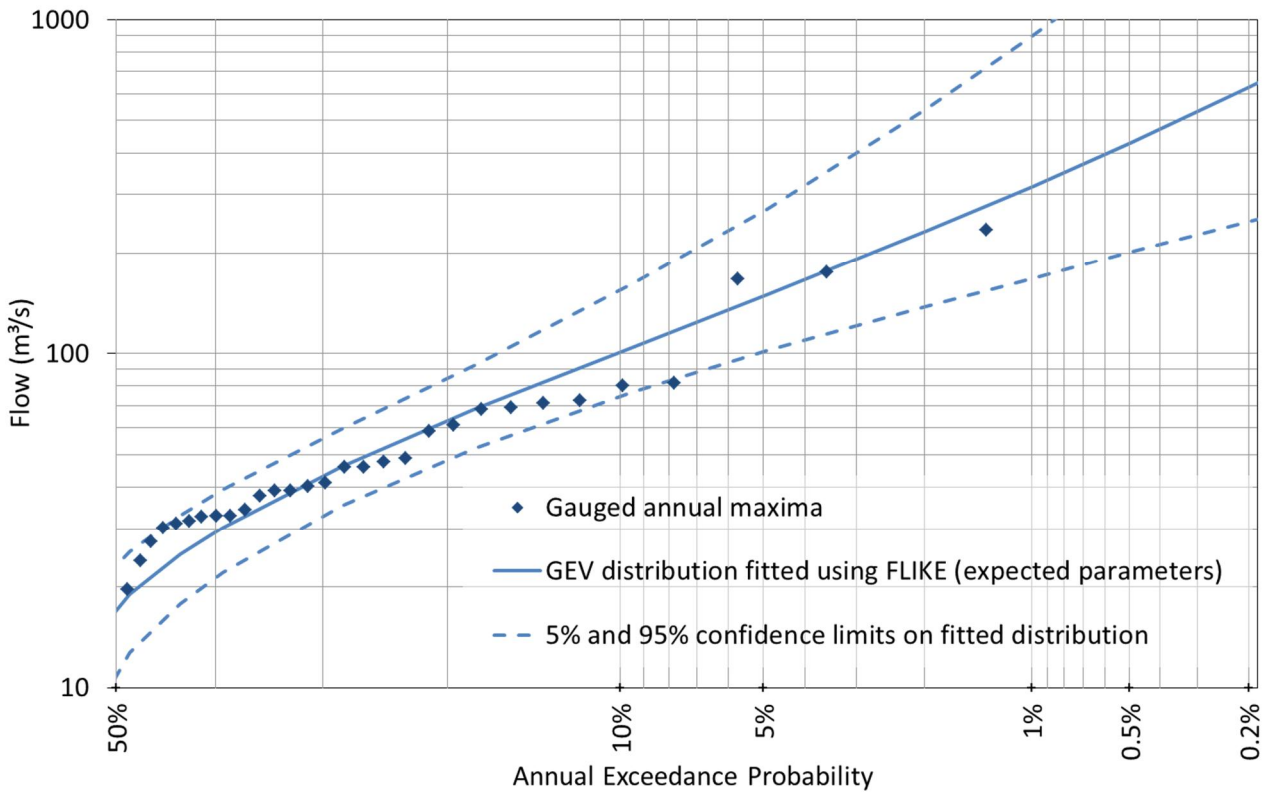


Figure 6-2 Flood frequency analysis fitted to annual maxima gauged data for the Fords Creek at Mansfield gauge (405245)

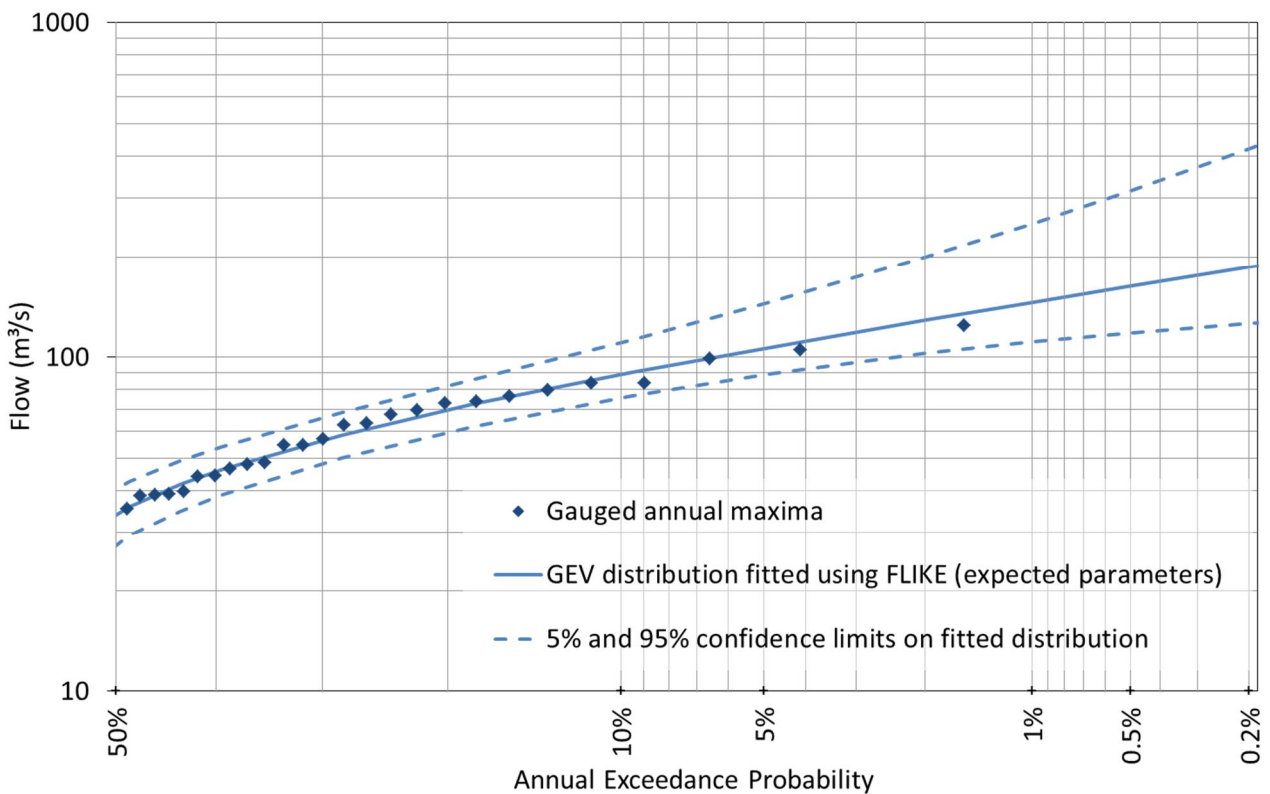


Figure 6-3 Flood frequency analysis fitted to annual maxima gauged data for the Howqua River at Glen Esk gauge (405215)

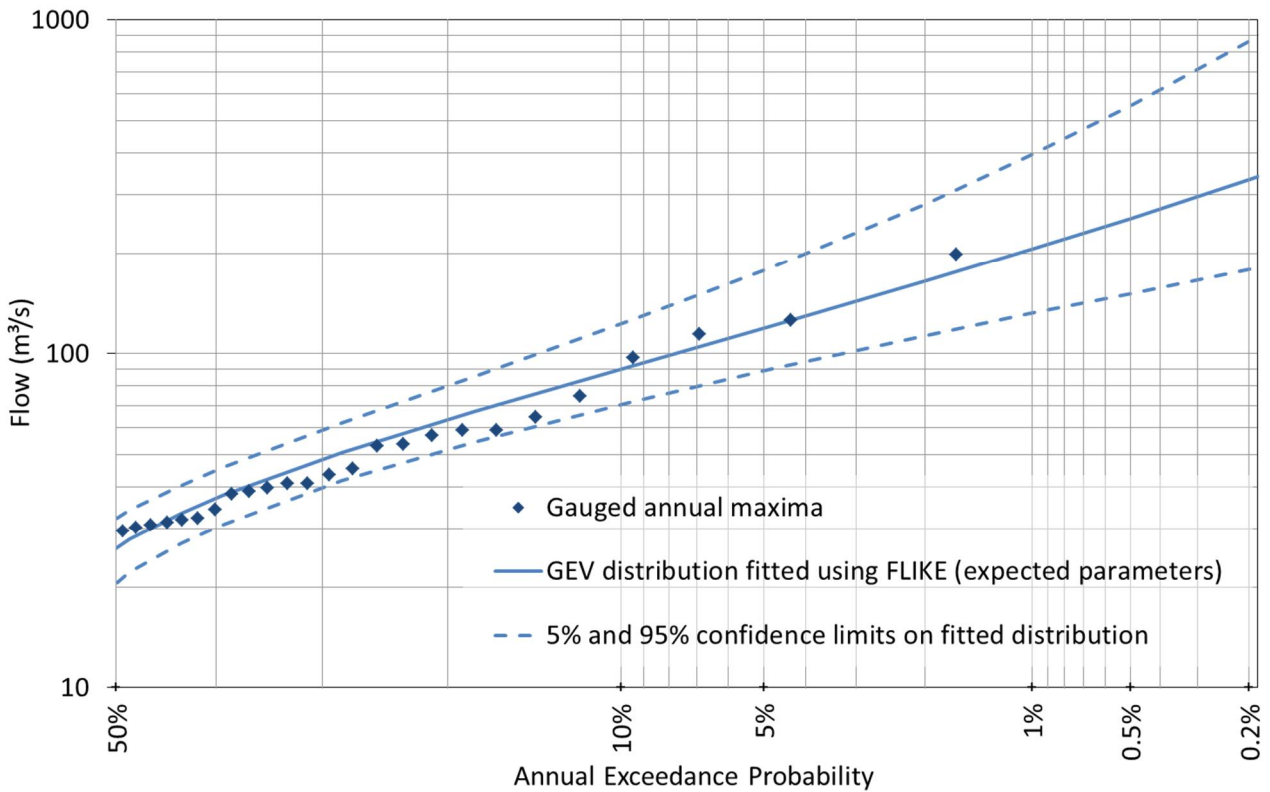


Figure 6-4 Flood frequency analysis fitted to annual maxima gauged data for the Goulburn River upstream of Snake Creek gauge (405263)

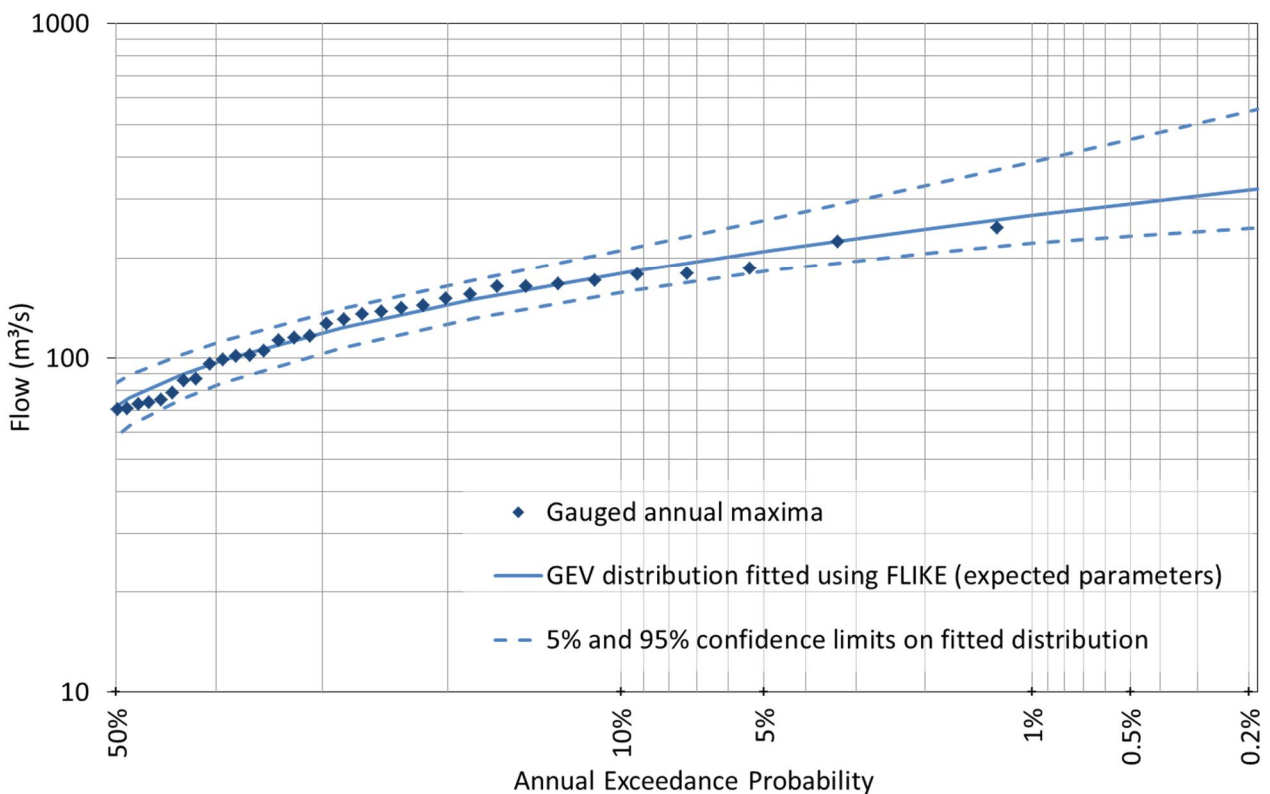


Figure 6-5 Flood frequency analysis fitted to annual maxima gauged data for the Goulburn River at Dohertys gauge (405219)

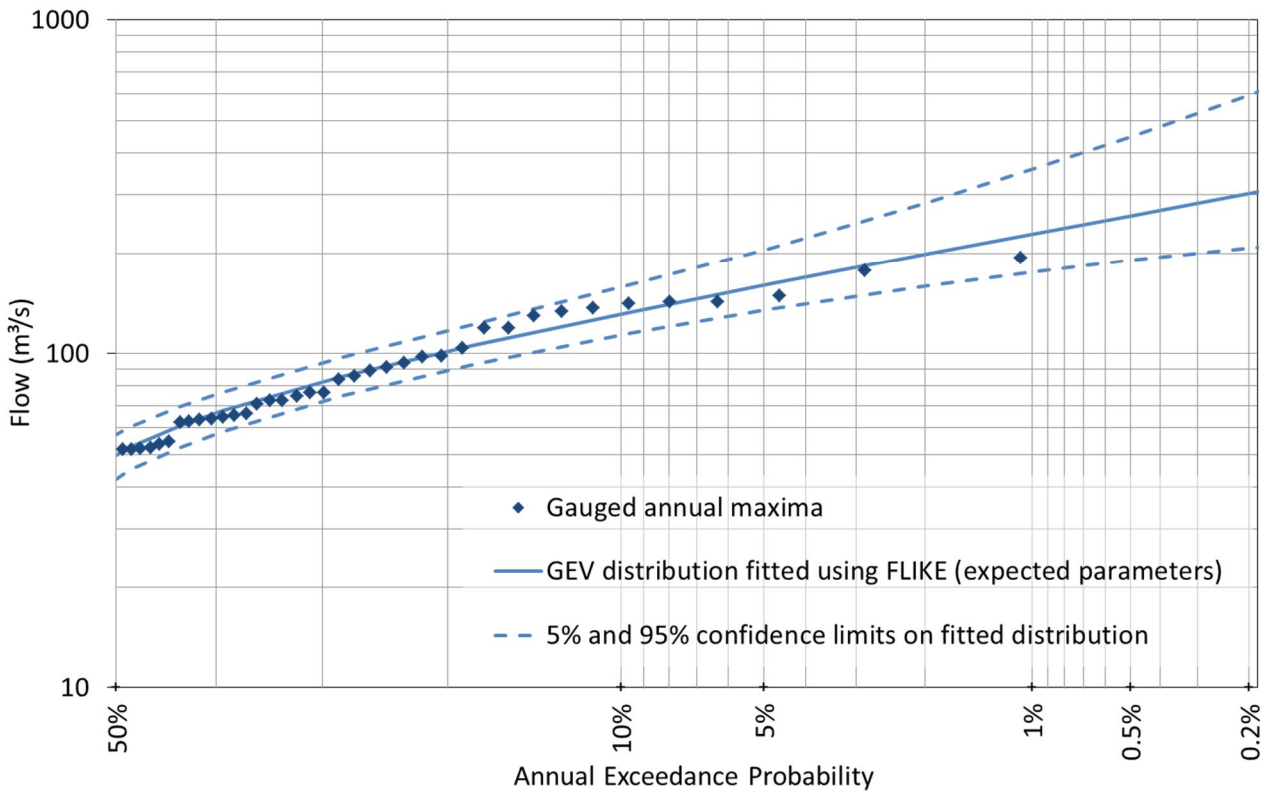


Figure 6-6 Flood frequency analysis fitted to annual maxima gauged data for the Jamieson River at Gerrang Bridge gauge (405218)

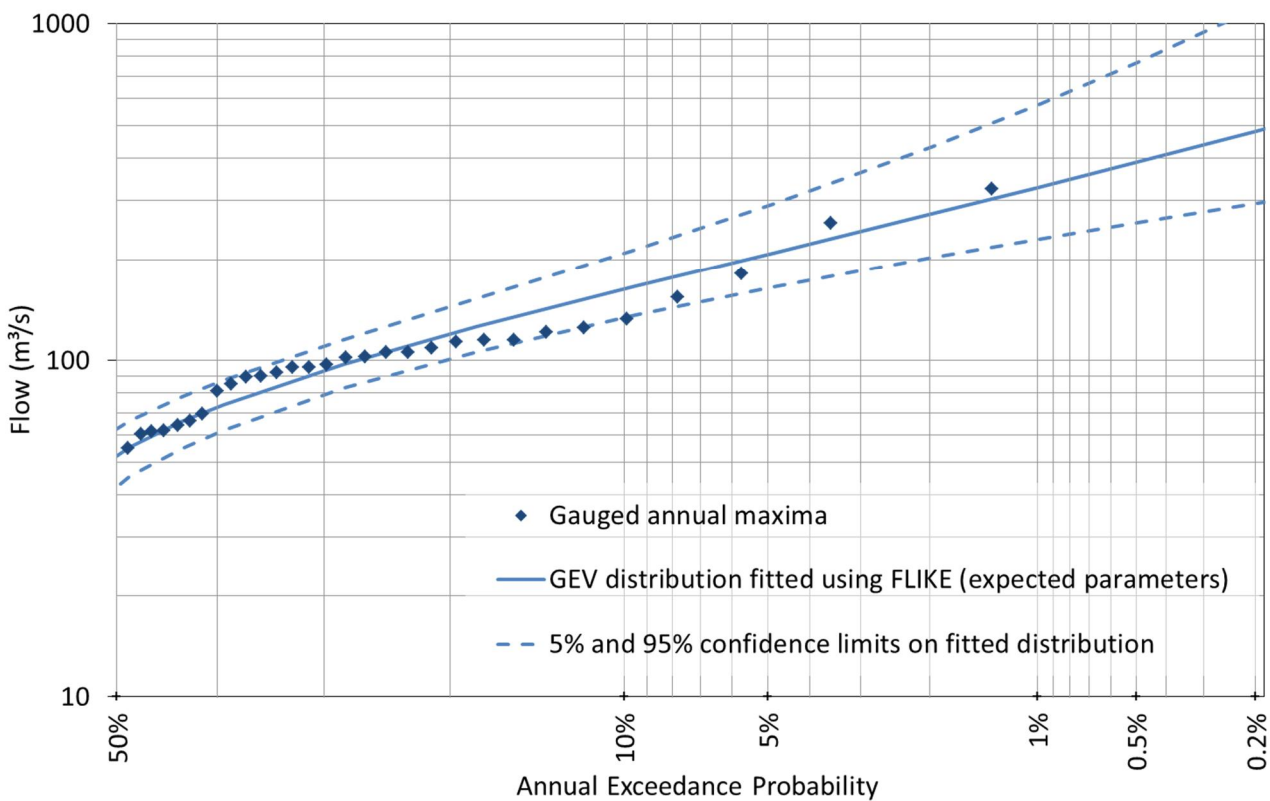


Figure 6-7 Flood frequency analysis fitted to annual maxima gauged data for the Big River at Jamieson gauge (405227)

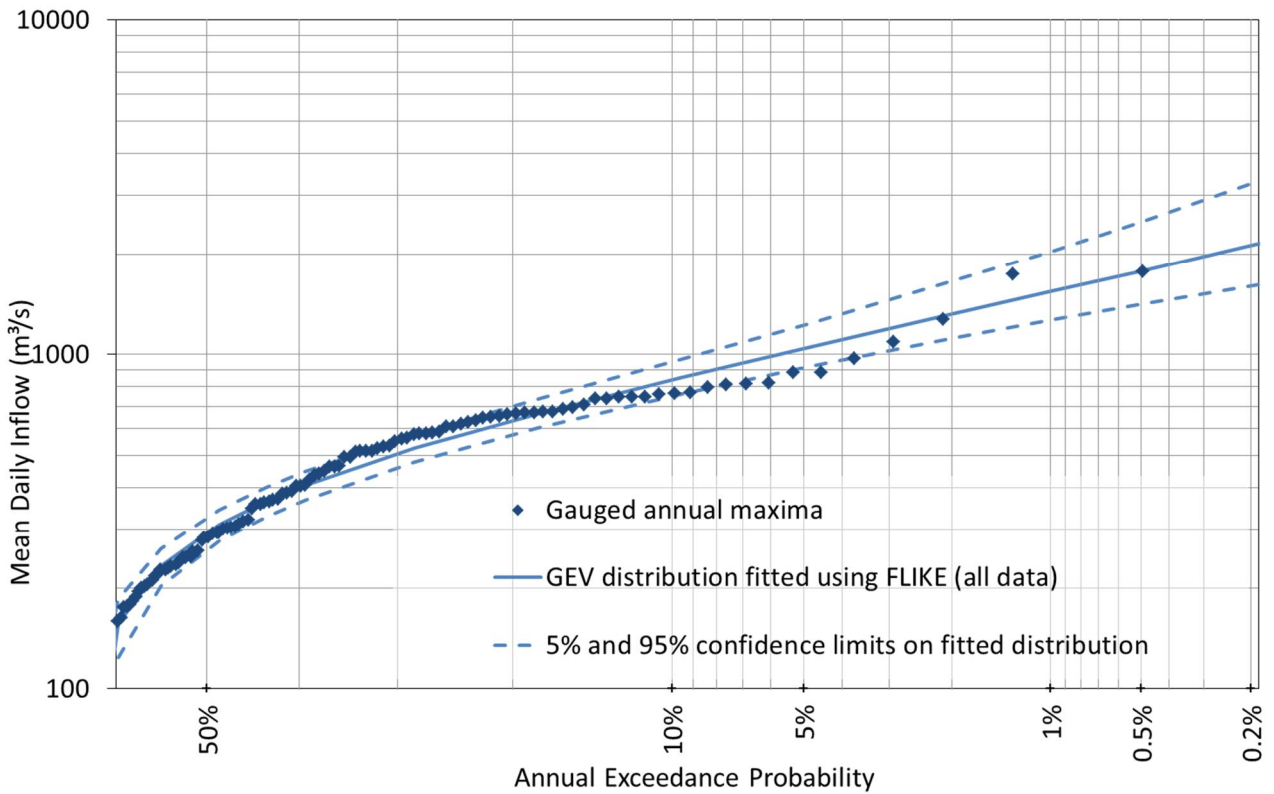


Figure 6-8 Flood frequency analysis fitted to annual maxima of estimated historical inflows to Lake Eildon

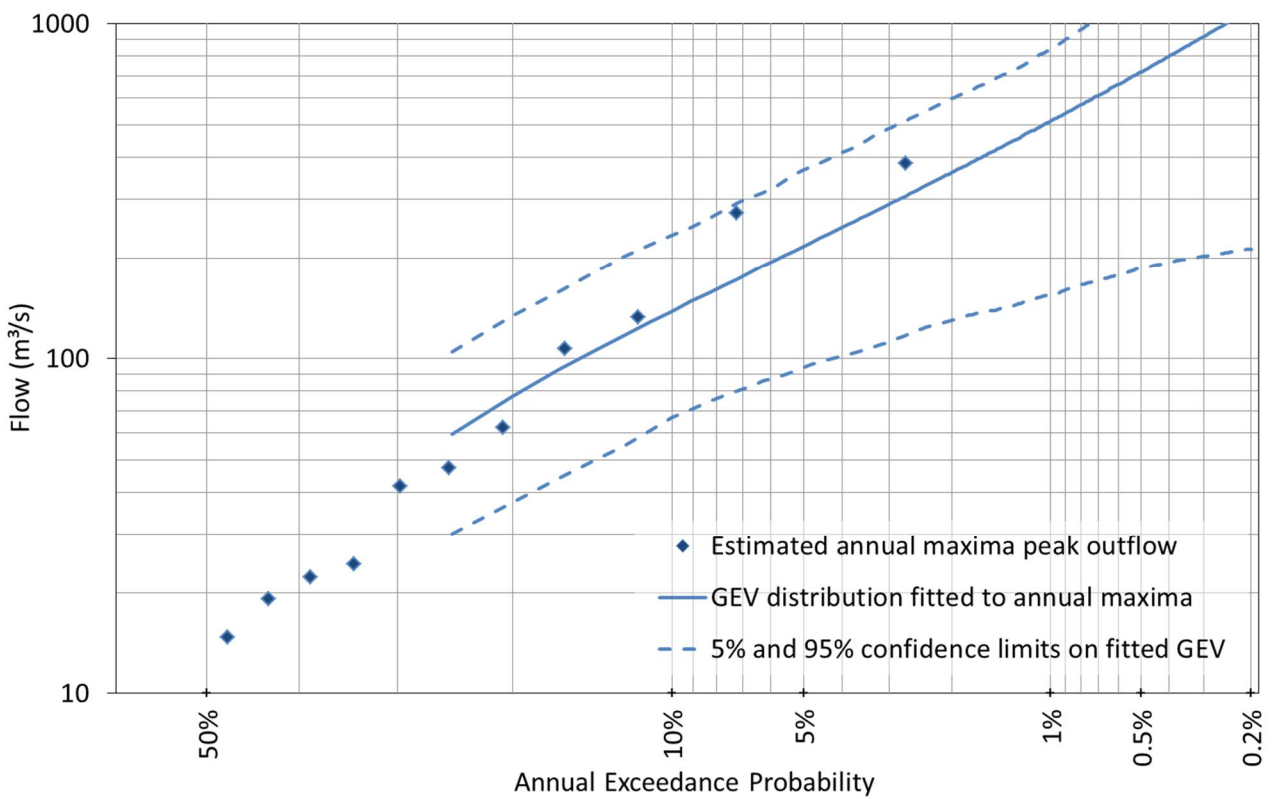


Figure 6-9 Flood frequency analysis fitted to annual maxima gauged outflows from the Broken River at Lake Nillahcootie outflow (404218)

## 7. Design rainfall inputs and reservoir drawdown distribution

### 7.1 General Approach

Design rainfall inputs are used to generate the design event hydrographs. They are also used in the Monte-Carlo joint probability simulation approach to demonstrate that the flood frequency curves produced from the RORB model verify to flood frequency curves at the flow gauges.

#### 7.1.1 Intensity-Frequency-Duration Curves for Design Rainfall

##### *1 in 2 to 1 in 100 AEP*

Design rainfall depths for AEP between 50% and 1% (1 in 100 AEP) and for all durations (between 1 and 72 hours) were estimated using the revised rainfall IFD data that were released by the Bureau of Meteorology in 2013 (Green *et al.*, 2012).

##### *1 in 100 to 1 in 2000 AEP*

For long duration events (between 18 and 120 hours duration), growth factors for the relevant catchments were obtained from the CRC-FORGE rainfall database for Victoria (Nandakumar *et al.* 2000). These growth factors were multiplied by the 1% (1 in 100) AEP design rainfall depths for the catchment to derive the design rainfall intensities.

For the 12 hour duration event, a regional approach for estimating design rainfall depths developed by Jordan *et al.* (2005) was adopted for AEPs between 1 in 200 and 1 in 500. The Jordan *et al.* (2005) study obtained rainfall records from the Bureau of Meteorology for twelve continuously recording rain gauges located around Australia to estimate regional growth factors for rainfall depths between 1 in 200 and 1 in 500 AEP and durations up to 12 hours.

##### *Areal Reduction Factors*

Point rainfall estimates were converted to catchment average values using areal reduction factors appropriate for Victoria. 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 Jordan *et al.* (2013), which references Siriwardena and Weinmann (1996).

In many cases, the most downstream flow gauge used for verification of RORB model parameters is located somewhat upstream of the outlet of the subcatchment (which is usually where the subcatchment meets the full supply level of Lake Eildon or Lake Nillahcootie). Separate areal reduction factors and IFD curve inputs were therefore generated for the catchment upstream of each flow gauge and for the whole of the model subcatchment (to the reservoir tailwater).

#### 7.1.2 Pre-burst rainfall

The temporal pattern of rainfall antecedent to the main rainfall burst (pre-burst pattern) was applied to the large events in this study using the average patterns derived from the storms in the GSAM PMP catalogue (Minty & Meighen 1999).

#### 7.1.3 Design temporal patterns

For the Monte-Carlo simulations, temporal patterns were sampled from a set of possible patterns for each of the durations (12, 18, 24, 36, 48 and 72 hours) from the GSAM PMP catalogue (Minty *et al.* 1996).

For the single event simulations, the unsmoothed GSAM PMP temporal pattern (Minty *et al.* 1996) was adopted, which is consistent with the recommendations of Nathan and Weinmann (2000).

### 7.1.4 Design spatial patterns

All of the simulations for this project adopted uniform spatial patterns of design rainfall across the subcatchments of the model. Whilst Nathan and Weinmann (2000) would recommend adopting a non-uniform spatial pattern for estimation of large and extreme floods, a uniform spatial pattern was adopted (for most catchments) in this case as the intention is to derive design flood inundation levels from the hydraulic model along relatively long distances of river within each catchment and adopting a uniform spatial pattern allowed for more consistency in estimating flood levels along these reaches. There were two exceptions where a non-uniform spatial pattern was applied: (1) in the Howqua River subcatchment, where a adopting a non-uniform spatial pattern appreciably improved the verification to flood quantiles at the gauge in comparison to the uniform spatial pattern and (2) for the inflows and outflows for the whole catchment to Eildon Dam, where only the inflow and outflow hydrographs are provided at the dam wall and a non-uniform spatial pattern was required to represent the spatial pattern of rainfall and hence the spatial pattern of contribution of runoff across the substantial catchment area upstream of Eildon Dam.

### 7.2 Design rainfall frequency curves for each catchment

Design rainfall intensity-frequency-duration tables and curves were developed for each the catchment areas upstream of each location of interest. The locations of interest included the streamflow gauge locations that were used for verification to flood frequency analysis and also locations of interest for subsequent modelling of flood influences that would have catchment areas that were appreciably different to any of the flow gauges that were used for verification. Hence, design rainfall inputs were also produced for the Delatite River at Eildon dam tailwater, Upper Goulburn River at Eildon Dam tailwater and Upper Goulburn River at Woods Point.

The curves produced are shown in Figure 7-1 to Figure 7-12 below.

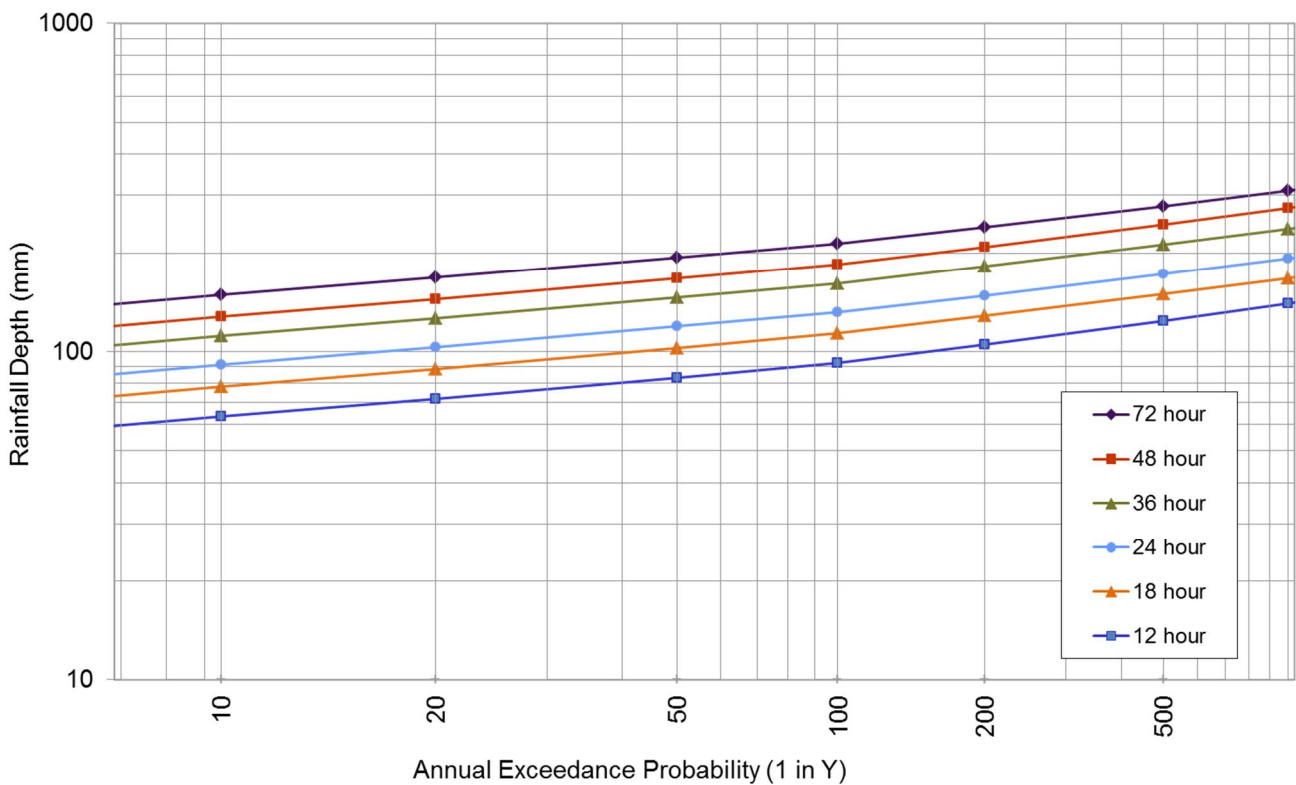


Figure 7-1 Design rainfall intensity-frequency-duration inputs for catchment to Delatite River at Tonga Bridge gauge

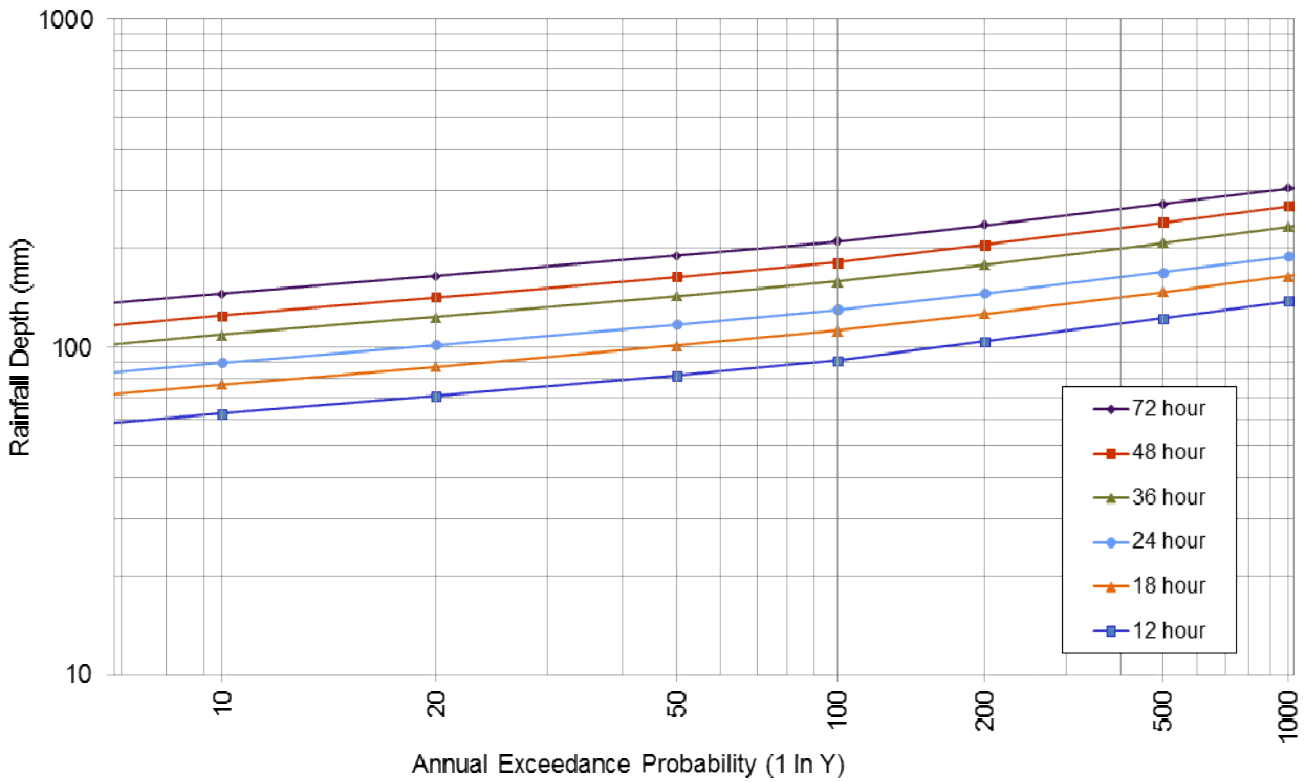


Figure 7-2 Design rainfall intensity-frequency-duration inputs for catchment to Delatite River at Eildon Dam tailwater

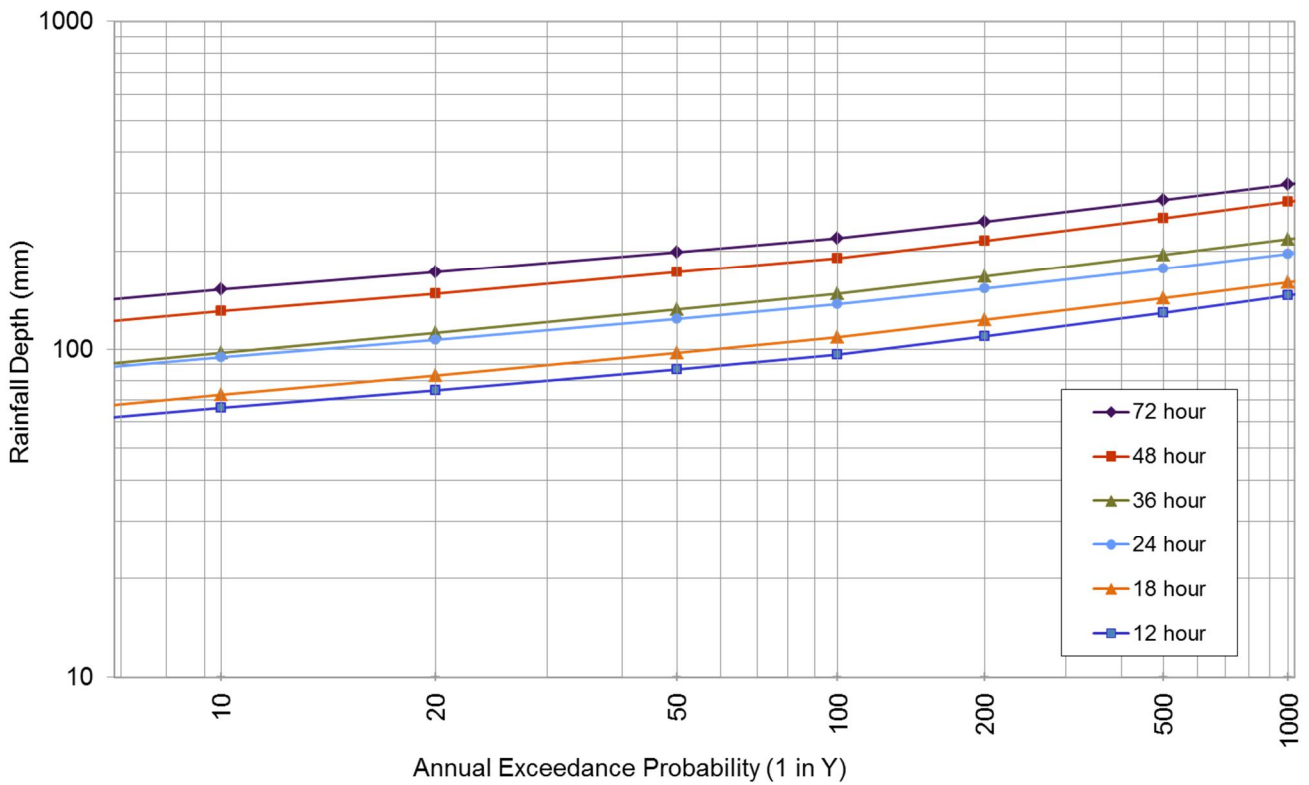


Figure 7-3 Design rainfall intensity-frequency-duration inputs for catchment to Fords Creek at Mansfield gauge

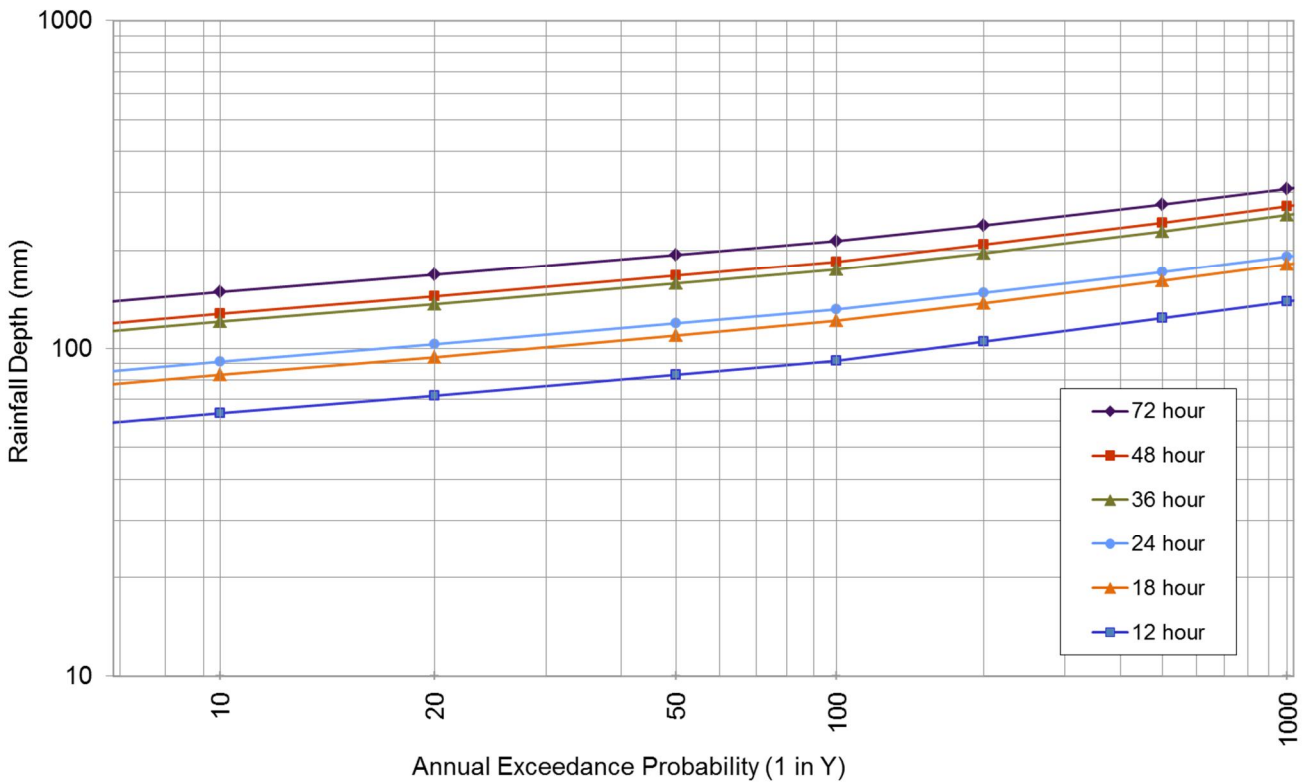


Figure 7-4 Design rainfall intensity-frequency-duration inputs for catchment to Howqua River at Glen Esk gauge

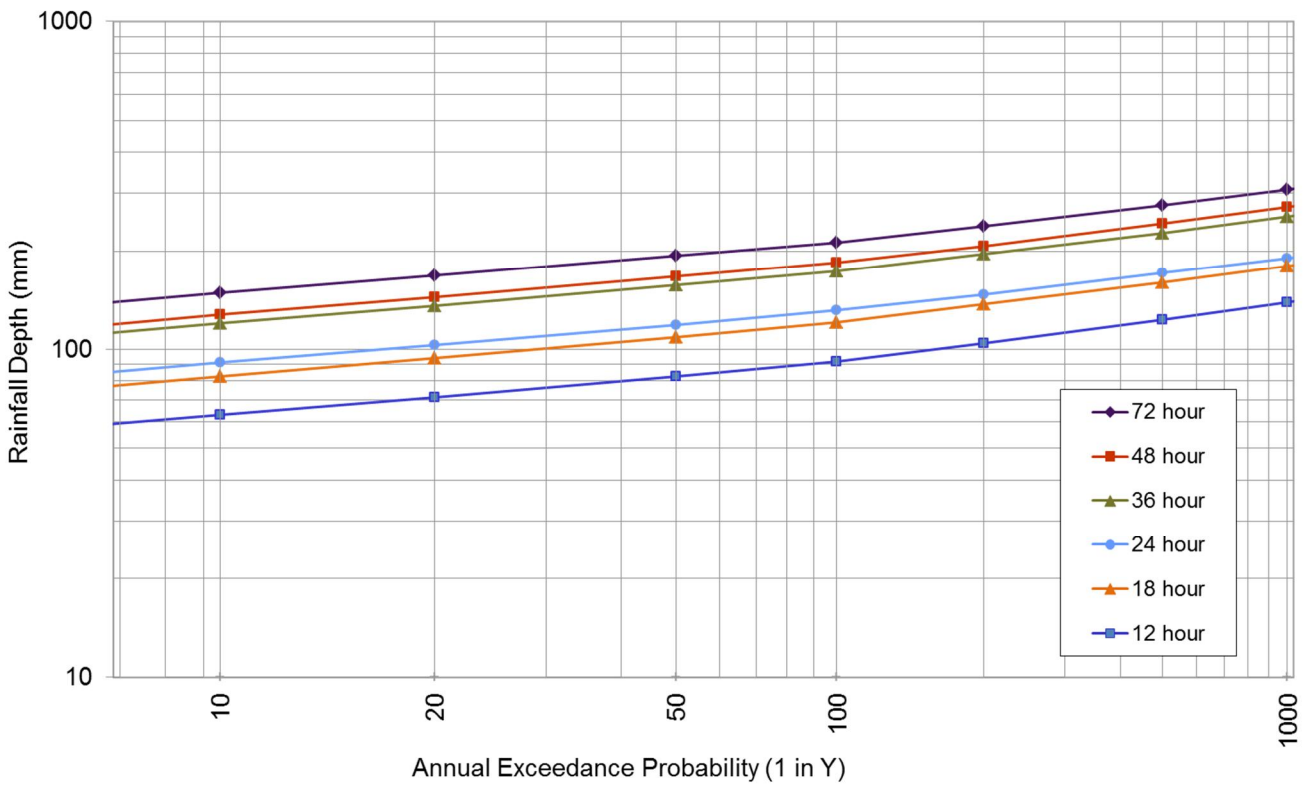


Figure 7-5 Design rainfall intensity-frequency-duration inputs for catchment to Howqua River to Eildon Dam tailwater

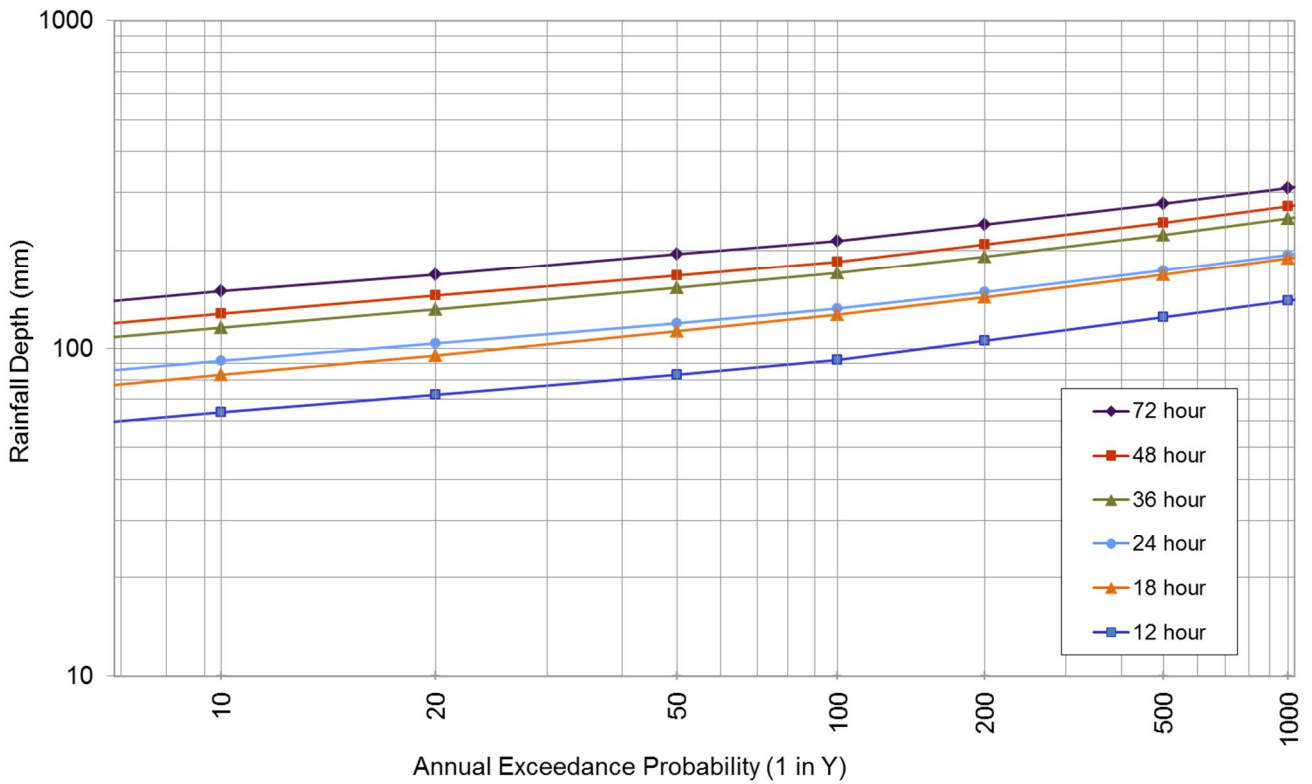


Figure 7-6 Design rainfall intensity-frequency-duration inputs for catchment to Upper Goulburn River at upstream of Snake Creek gauge

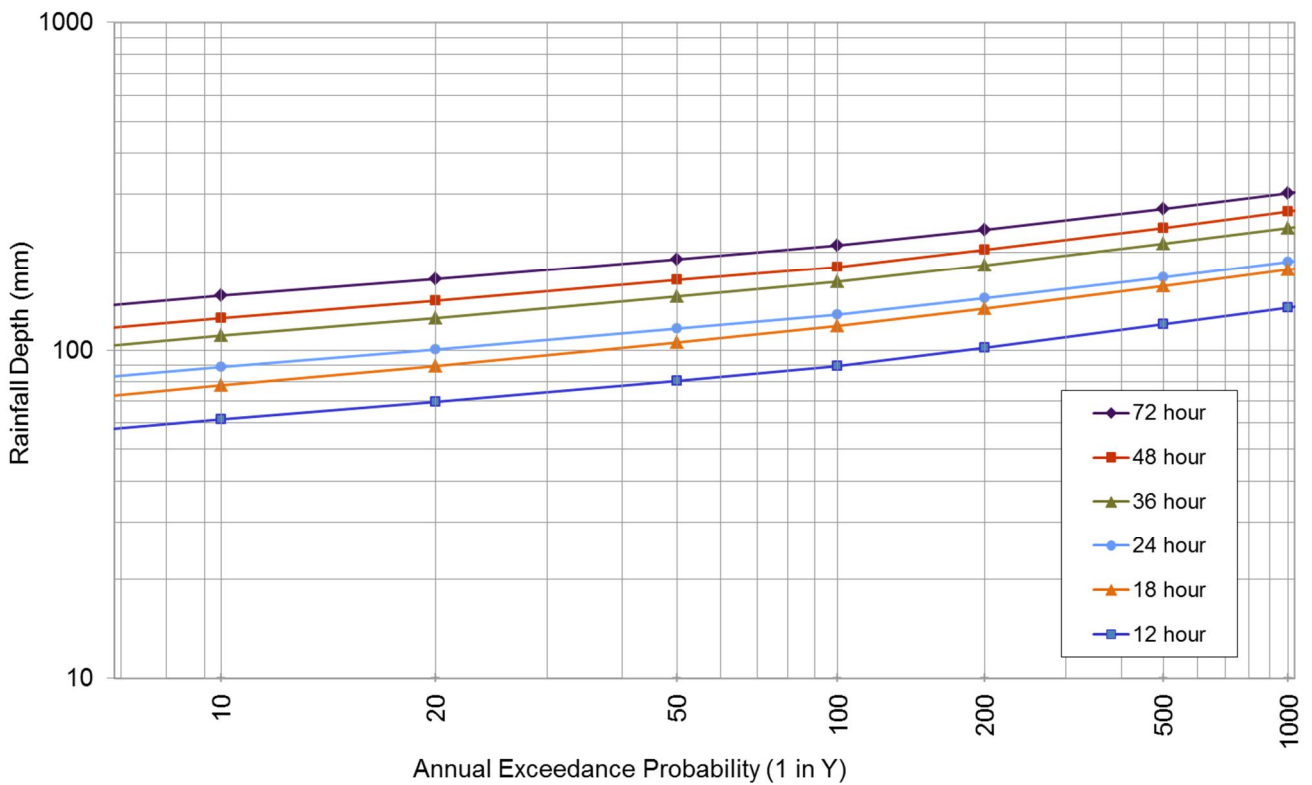


Figure 7-7 Design rainfall intensity-frequency-duration inputs for catchment to Upper Goulburn River at Dohertys gauge

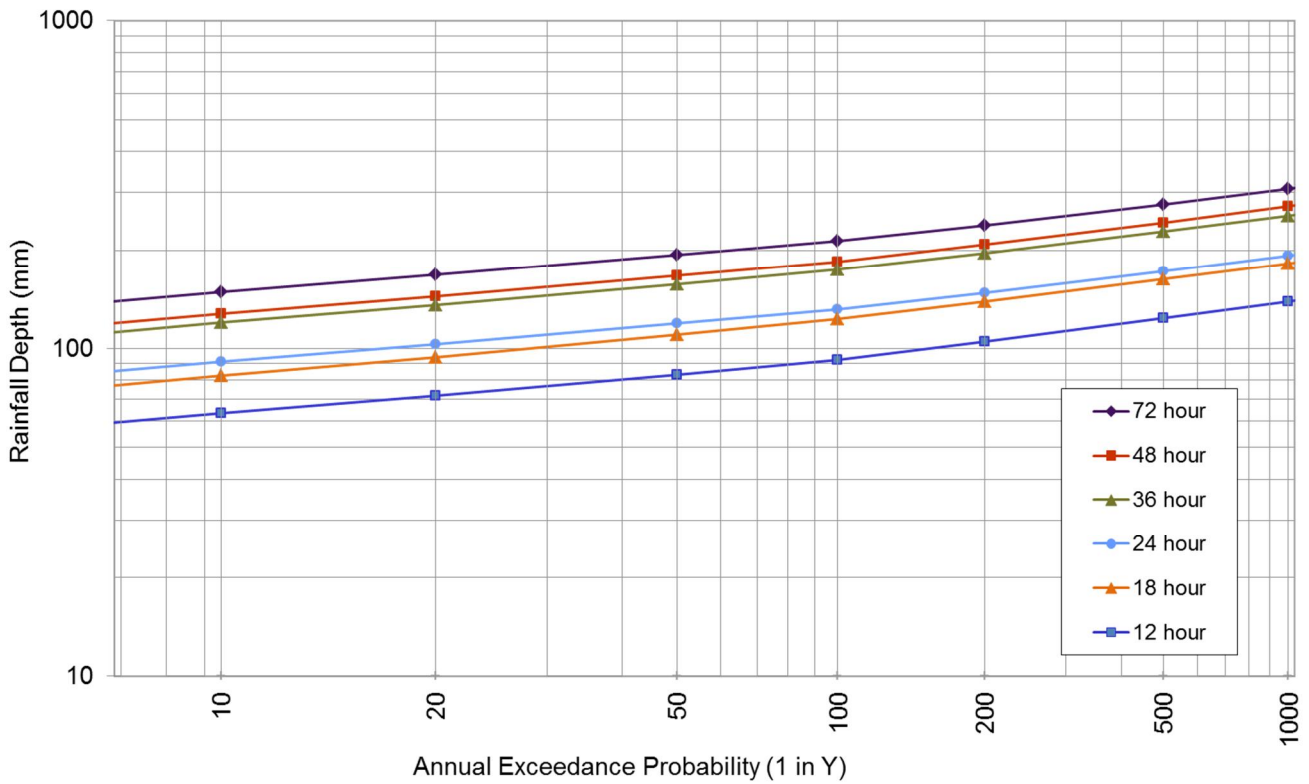


Figure 7-8 Design rainfall intensity-frequency-duration inputs for catchment to Jamieson River at Gerrang Bridge gauge

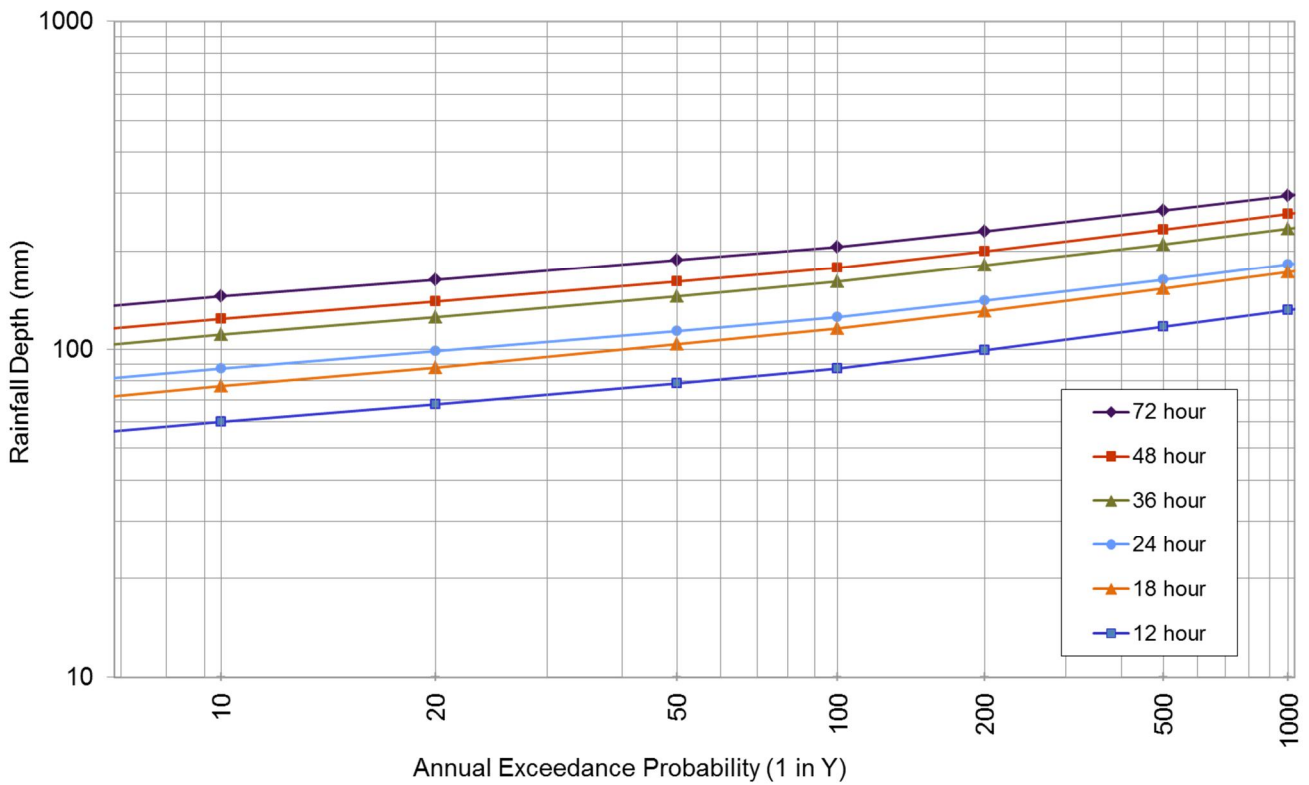


Figure 7-9 Design rainfall intensity-frequency-duration inputs for catchment to Upper Goulburn River at Eildon Dam tailwater

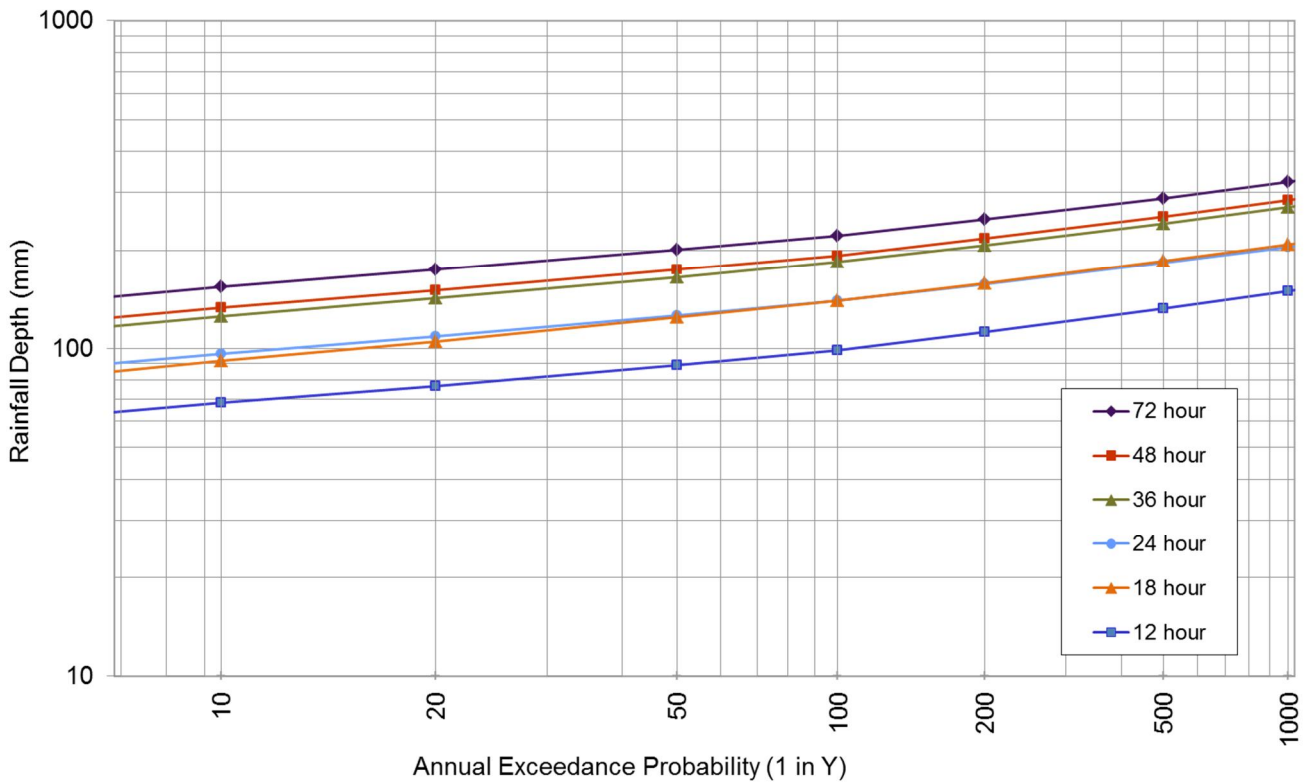


Figure 7-10 Design rainfall intensity-frequency-duration inputs for catchment to Upper Goulburn River at Woods Point

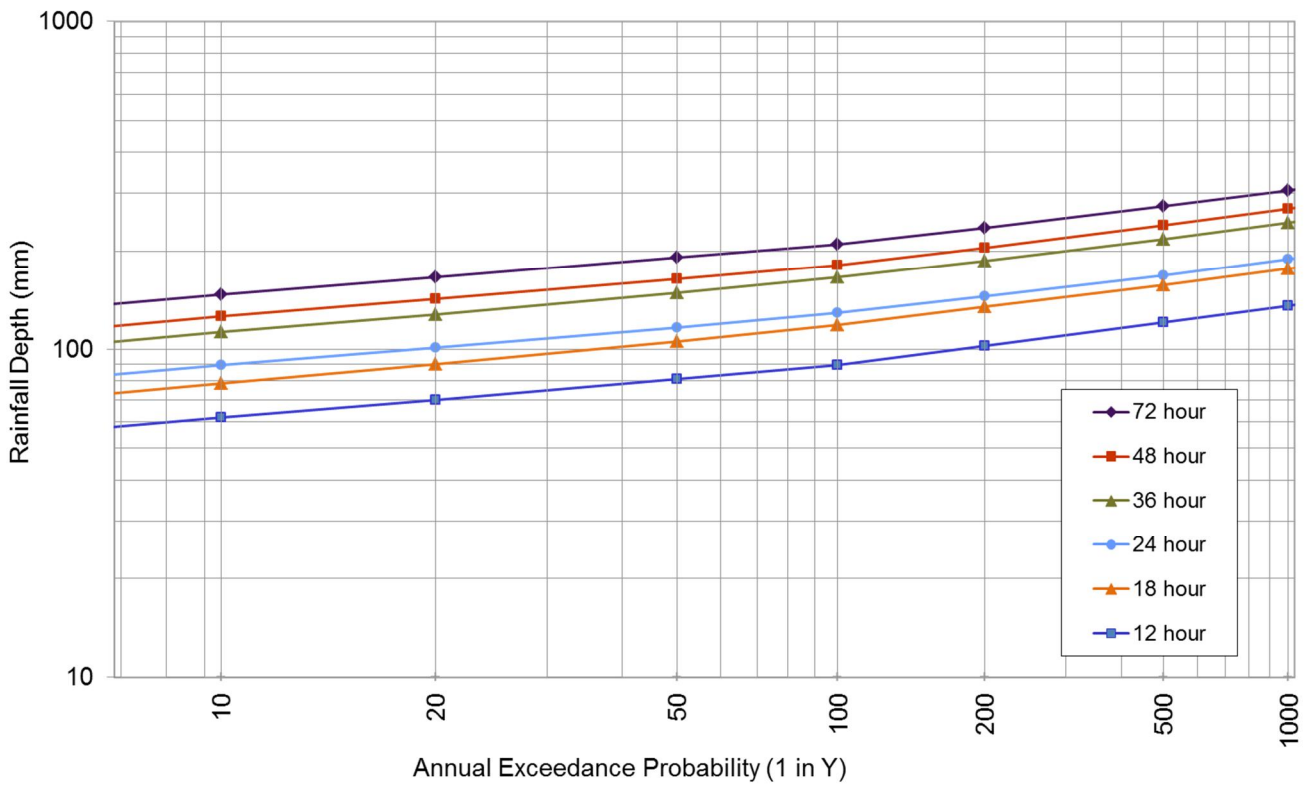


Figure 7-11 Design rainfall intensity-frequency-duration inputs for catchment to Big River at Jamieson

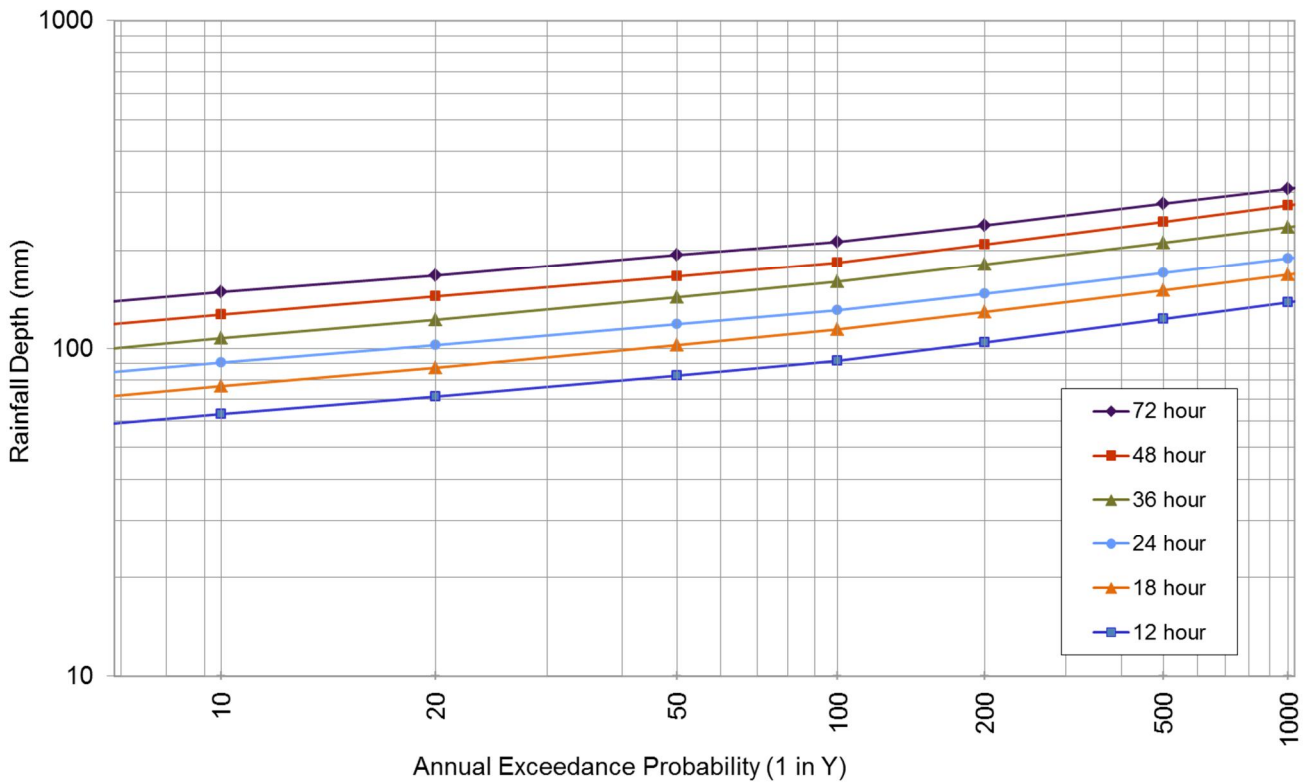


Figure 7-12 Design rainfall intensity-frequency-duration inputs for catchment of Broken River to Lake Nillahcootie

### 7.3 Reservoir drawdown for Lake Nillahcootie

The storage volume of Lake Nillahcootie at full supply level is 40,400 ML. There is a significant probability that Lake Nillahcootie will be drawn down somewhat below full supply level before the occurrence of the flood and this reservoir drawdown may contribute significantly to reduce flood outflows from Lake Nillahcootie. The probability distribution of initial reservoir drawdown for Lake Nillahcootie was derived from analysis of historical water levels in Lake Nillahcootie over the period between 1 January 1994 and 31 December 2014. This period was coincident with the period that was used for analysis of annual maxima outflow floods from Lake Nillahcootie, which were used to verify the RORB model parameters for the Broken River RORB model. The reservoir drawdown curve for Lake Nillahcootie that was adopted in RORB is shown in Figure 7-13.

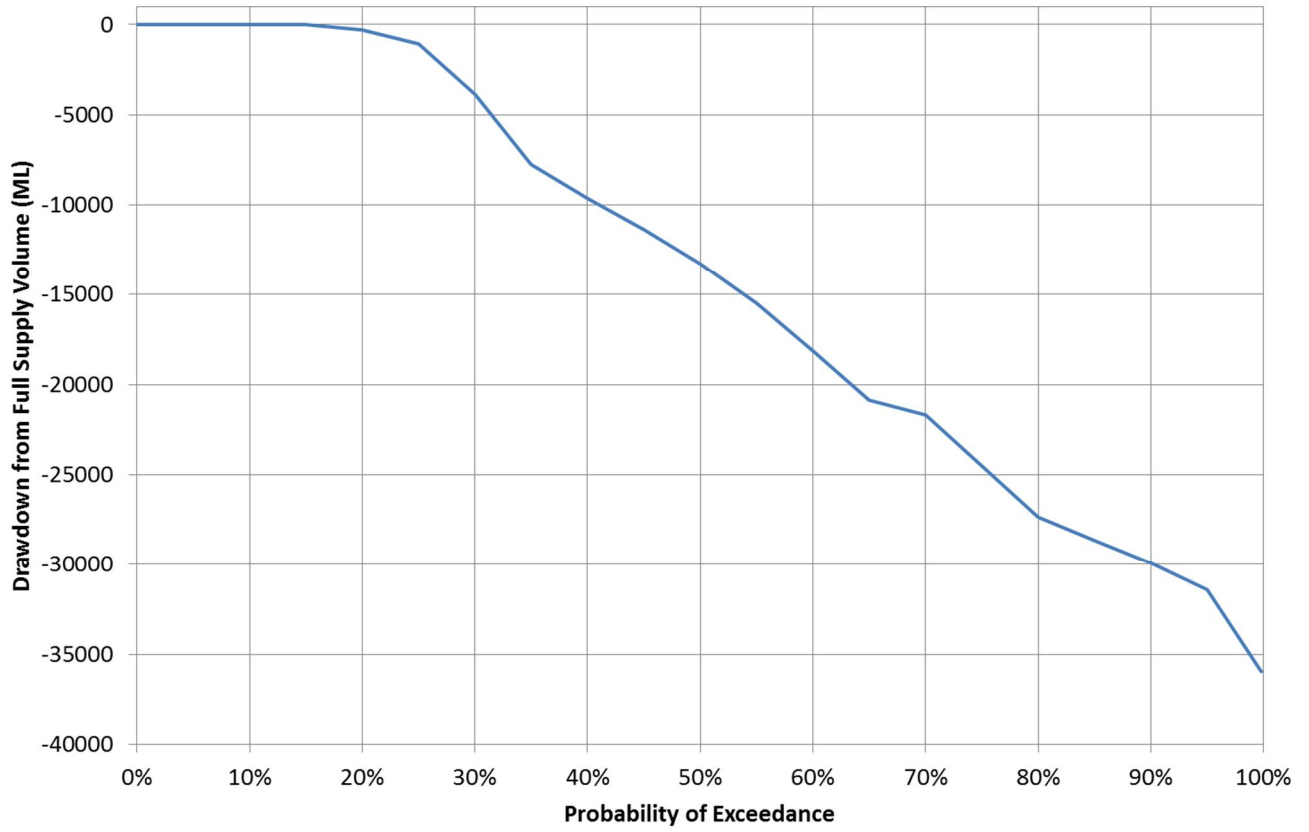


Figure 7-13 Probability distribution of reservoir drawdown in Lake Nillahcootie that was adopted for RORB verification runs

## 8. Verification of RORB model parameters to flood frequency analysis and development of adopted flood frequency curve from gauged and RORB modelled peak flows

### 8.1 Approach

There have been several years since the RORB models were last calibrated or verified to gauged flows. The RORB models were adjusted to include more subareas and then re-calibrated to new flood events. The models were also run in a Monte-Carlo simulation framework. As a result of all of these changes to the RORB models, verification demonstrates that the adjusted overall model framework reproduces flood quantiles from flood frequency analysis to gauged flows.

At each flow gauge location there are therefore two possible estimates of the design flood peak at each AEP: one from the flood frequency analysis that was fitted to the gauged peaks and one produced from the RORB model. Whilst the intent of the verification process for the RORB model is to produce estimates from each of these two methods that are similar to one another, the two estimates will not produce identical peak flows at each AEP.

A merged flood frequency curve was therefore produced at each gauge location as a compromise between the results of the gauged flood frequency analysis and the verified RORB model results. For the more common events, it was assumed that the flood frequency analysis to recorded peaks would produce the more accurate estimate of the true flood quantile. Conversely, at the rarer end of the frequency distribution it was assumed that the RORB model results would produce more accurate estimate of the true flood quantile, as in this range there is considerably more confidence in estimating the frequency distribution of design rainfalls (and deriving flows from the RORB model) than in extrapolating the flood frequency curve fitted to flow peaks. The merged flood frequency quantiles were therefore produced using the following equations:

$$Q_{10\%Merged} = 1.0Q_{10\%FFAGauged} + 0Q_{10\%RORBModel}$$

$$Q_{5\%Merged} = 0.8Q_{5\%FFAGauged} + 0.2Q_{5\%RORBModel}$$

$$Q_{2\%Merged} = 0.6Q_{2\%FFAGauged} + 0.4Q_{2\%RORBModel}$$

$$Q_{1\%Merged} = 0.4Q_{1\%FFAGauged} + 0.6Q_{1\%RORBModel}$$

$$Q_{0.5\%Merged} = 0.2Q_{0.5\%FFAGauged} + 0.8Q_{0.5\%RORBModel}$$

$$Q_{0.2\%Merged} = 0Q_{0.2\%FFAGauged} + 1.0Q_{0.2\%RORBModel}$$

For reporting purposes, the merged flood frequency quantiles were then rounded at each gauge to the nearest 10 m<sup>3</sup>/s.

The RORB models were run for rainfall event durations of 12, 18, 24, 36, 48 and 72 hours. For several of the catchments, the critical duration from the initial Monte-Carlo simulation runs was identified to be 72 hours. Since the catchment area to each of the gauges was less than 700 km<sup>2</sup>, the 72 hour duration was considered to be infeasibly long and it was identified that this critical duration was likely to be an artefact of some of the temporal patterns that were included in the sample of possible temporal patterns for the 72 hour duration from the GSAM catalogue. When the 72 hour duration was excluded (hence only considering durations between 12 and 48 hours inclusive) more reasonable critical durations were obtained, typically in the range between 12 and 36 hours, depending upon the catchment.

## 8.2 Verification of RORB models and adopted flood frequency curves at gauge locations upstream of Lake Eildon

Parameters of the Eildon Northern Area RORB model were verified to flood frequency analysis fitted to gauged flows for the Delatite River at Tonga Bridge and Fords Creek at Mansfield. Adopted parameter values for these two interstation areas are shown in Table 8-1. As shown in Figure 8-1 and Figure 8-2, excellent fits were obtained between the RORB model and the flood frequency analysis at both gauges. An initial loss continuing loss (IL/CL) model was adopted for representing losses in the Eildon Northern Area RORB model. The  $k_c$  and  $m$  parameters of the RORB models were consistent with the values obtained from calibration to historical flood events for each of the interstation areas.

Table 8-1 Adopted parameters of Eildon Northern Area RORB model after verification of peaks from Monte-Carlo simulations to flood frequency analysis at flow gauges

Gauged Catchment	Total Area Upstream (km <sup>2</sup> )	$d_{av}$ (km)	$k_c$	$m$	Initial Loss (mm)	Continuing Loss Rate (mm/h)
Delatite River at Tonga Bridge	368.0	24.47	27	0.8	58	2.5
Fords Creek at Mansfield	116.8	11.75	5.5	0.8	75	1.0

Parameters of the Eildon Southern Area RORB model were verified to flood frequency analysis fitted to gauged flows for the Howqua River at Glen Esk, Goulburn River upstream of Snake Creek, Goulburn River at Doherteys, Jamieson River at Gerrang Bridge and Big River at Jamieson streamflow gauges. Adopted parameter values for these five interstation areas are shown in Table 8-2. As shown in Figure 8-3 to Figure 8-7, excellent fits were obtained between the RORB model and the flood frequency analysis at all five gauges when an initial loss runoff coefficient model was adopted. The flood frequency curves show that reasonable fits could be obtained in the Southern Area catchments using an IL/CL model but the fits were poorer than the runoff coefficient model. The IL/CL model was found to have difficulty in matching the slope of the at-site flood frequency curve across the range between 10% and 0.02% at several of the southern area gauges. The  $k_c$  and  $m$  parameters of the RORB models were consistent with the values obtained from calibration to historical flood events for each of the interstation areas.

Table 8-2 Adopted parameters of Eildon Southern Area RORB model after verification of peaks from Monte-Carlo simulations to flood frequency analysis at flow gauges

Gauged Catchment	Total Area Upstream (km <sup>2</sup> )	$d_{av}$ (km)	$k_c$	$m$	Initial Loss (mm)	Runoff Coefficient
Howqua River at Glen Esk	368.0	36.77	45	0.8	80	39%
Goulburn River upstream of Snake Creek	327.1	28.78	47	0.8	55	47%
Goulburn River at Doherteys	701.6	32.41	39	0.8	55	47%
Jamieson River at Gerrang Bridge	361.8	37.91	39	0.8	55	49%
Big River at Jamieson	627.4	40.22	60	0.8	40	48%

Adopted flood frequency quantiles at each of the gauge locations are listed in Table 8-3.

Table 8-3 Flood quantiles at streamflow gauge locations from flood frequency analysis, verified RORB model simulations and adopted quantiles merged from the two methods (and then rounded to nearest 10 m<sup>3</sup>/s)

Interstation Area	Method	Flood Quantiles (m <sup>3</sup> /s) at AEP					
		10%	5%	2%	1%	0.5%	0.2%
Delatite River at Tonga Bridge	FFA	195	260	365	462	578	766
	RORB	197	257	373	486	614	822
	Merged	190	260	370	480	610	820
Fords Creek at Mansfield	FFA	104	150	233	317	427	627
	RORB	129	170	215	258	304	373
	Merged	100	150	230	280	330	370
Howqua River at Glen Esk	FFA	90	107	129	146	163	186
	RORB	84	111	133	152	178	219
	Merged	90	110	130	150	170	220
Goulburn upstream of Snake Creek	FFA	92	120	165	206	254	332
	RORB	100	124	149	174	201	248
	Merged	90	120	160	190	210	250
Goulburn River at Dohertys	FFA	181	210	244	268	291	319
	RORB	164	205	245	285	326	400
	Merged	180	210	240	280	320	400
Jamieson River at Gerrang Bridge	FFA	133	161	199	228	260	303
	RORB	135	164	193	228	262	319
	Merged	130	160	200	230	260	320
Big River at Jamieson	FFA	166	209	273	328	389	482
	RORB	187	218	265	301	352	424
	Merged	170	210	270	310	360	420

Note that the legend used in Figure 8-1 to Figure 8-7 uses a consistent style (red line) to represent the *adopted approach*, which may be either an initial loss/continuing loss, or an initial loss/runoff coefficient approach.

Figure 8-4 presents the RORB results for Goulburn River upstream of Snake Creek gauge, where an initial loss/runoff coefficient approach has been adopted. While the fit of the adopted method does not look ideal for this particular gauge, it has been adopted because all other subcatchments in this area use the same method and it doesn't make sense to change it just for this subcatchment. This method still produces results within the confidence limits and is considered suitable for use in this context.

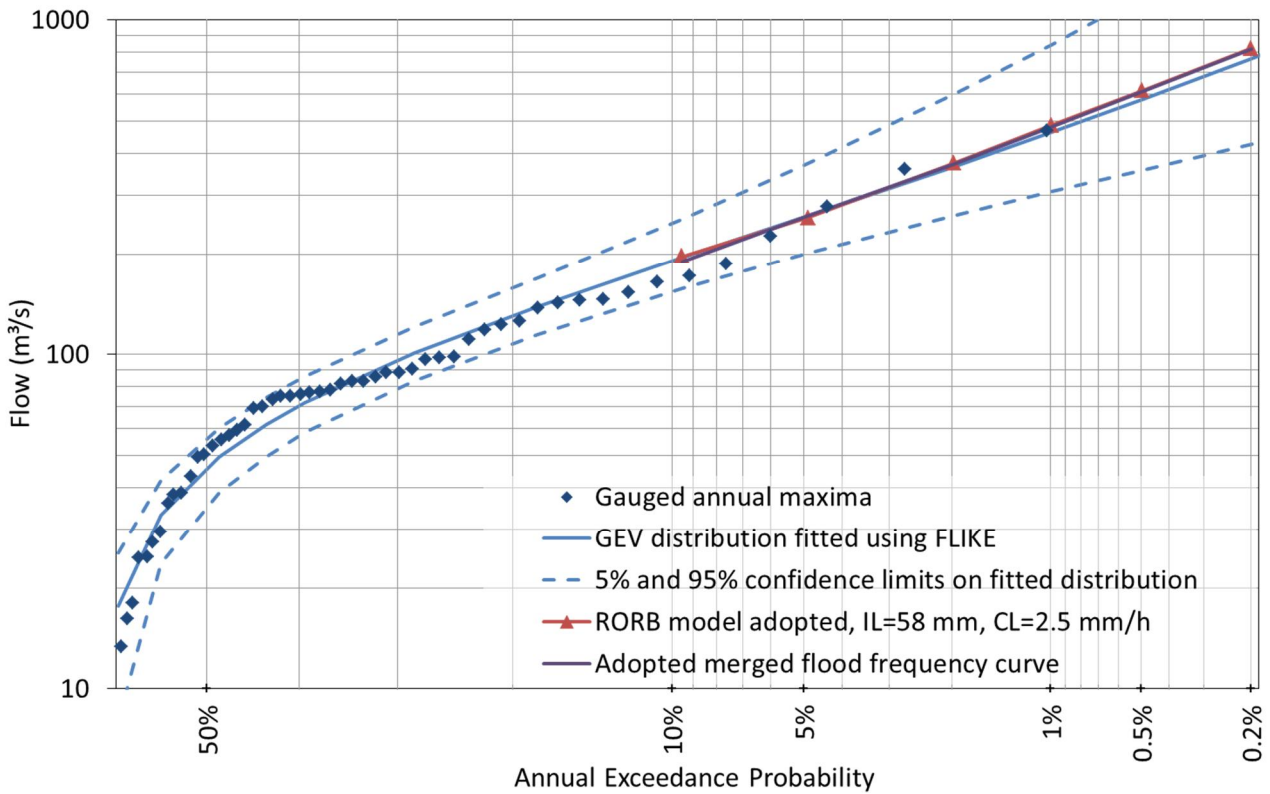


Figure 8-1 Adopted flood frequency curve and RORB model verified to flood frequency analysis of gauged peak flows for the Delatite River at Tonga Bridge gauge (405214)

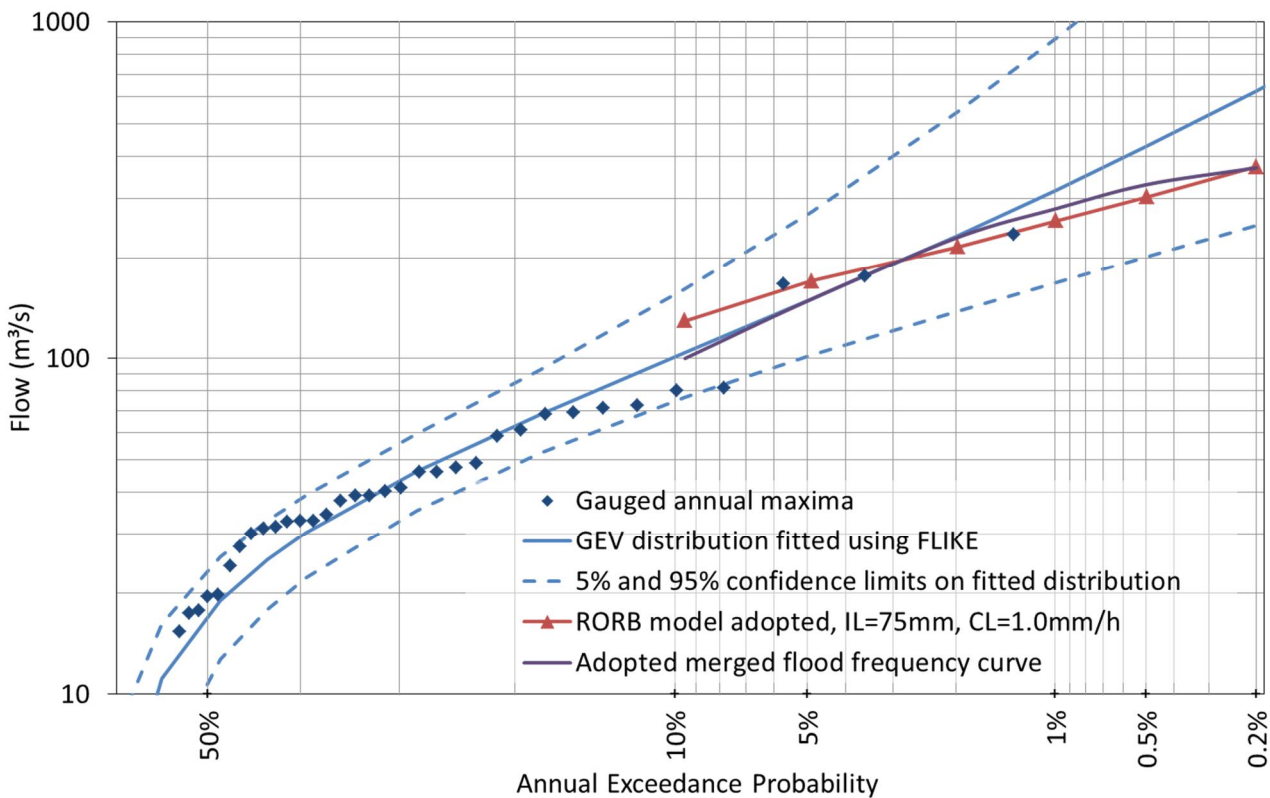


Figure 8-2 Adopted flood frequency curve and RORB model verified to flood frequency analysis of gauged peak flows for the Fords Creek at Mansfield gauge (405245)

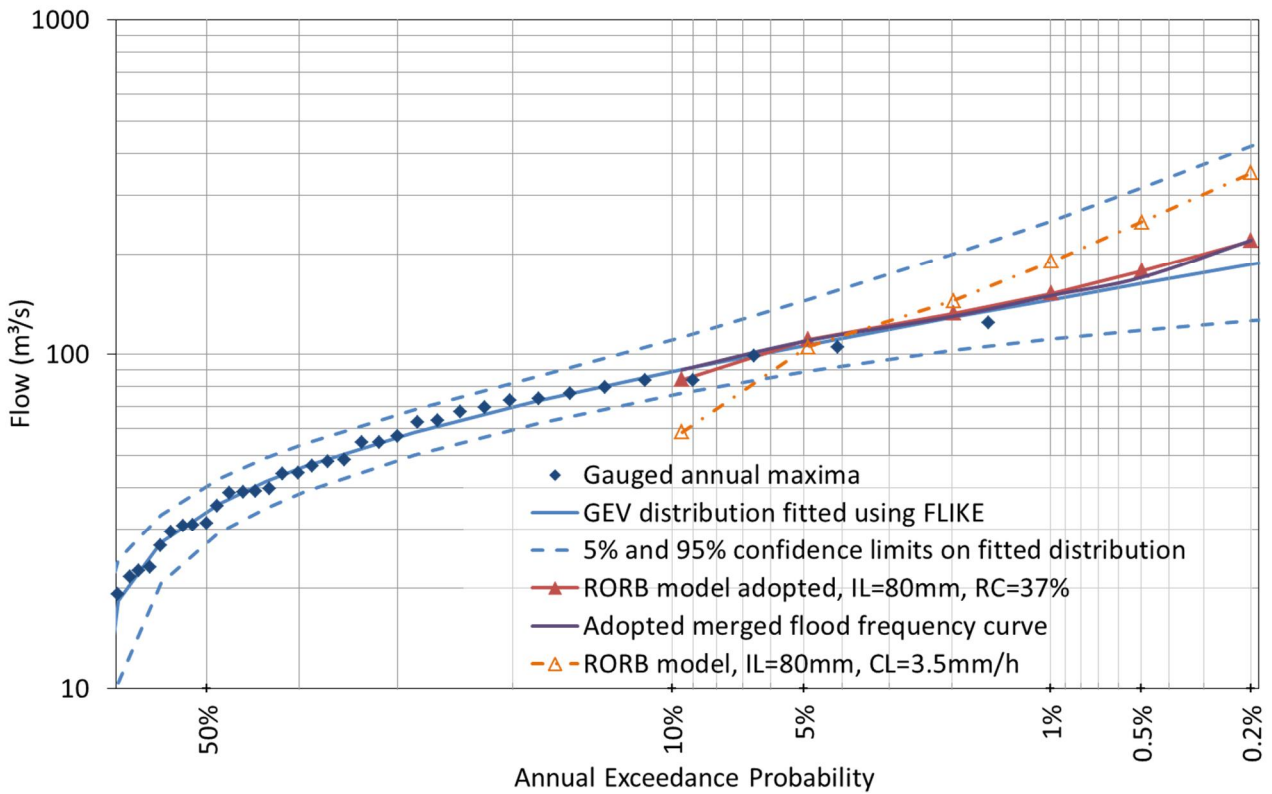


Figure 8-3 Adopted flood frequency curve and RORB model verified to flood frequency analysis of gauged peak flows for the Howqua River at Glen Esk gauge (405215)

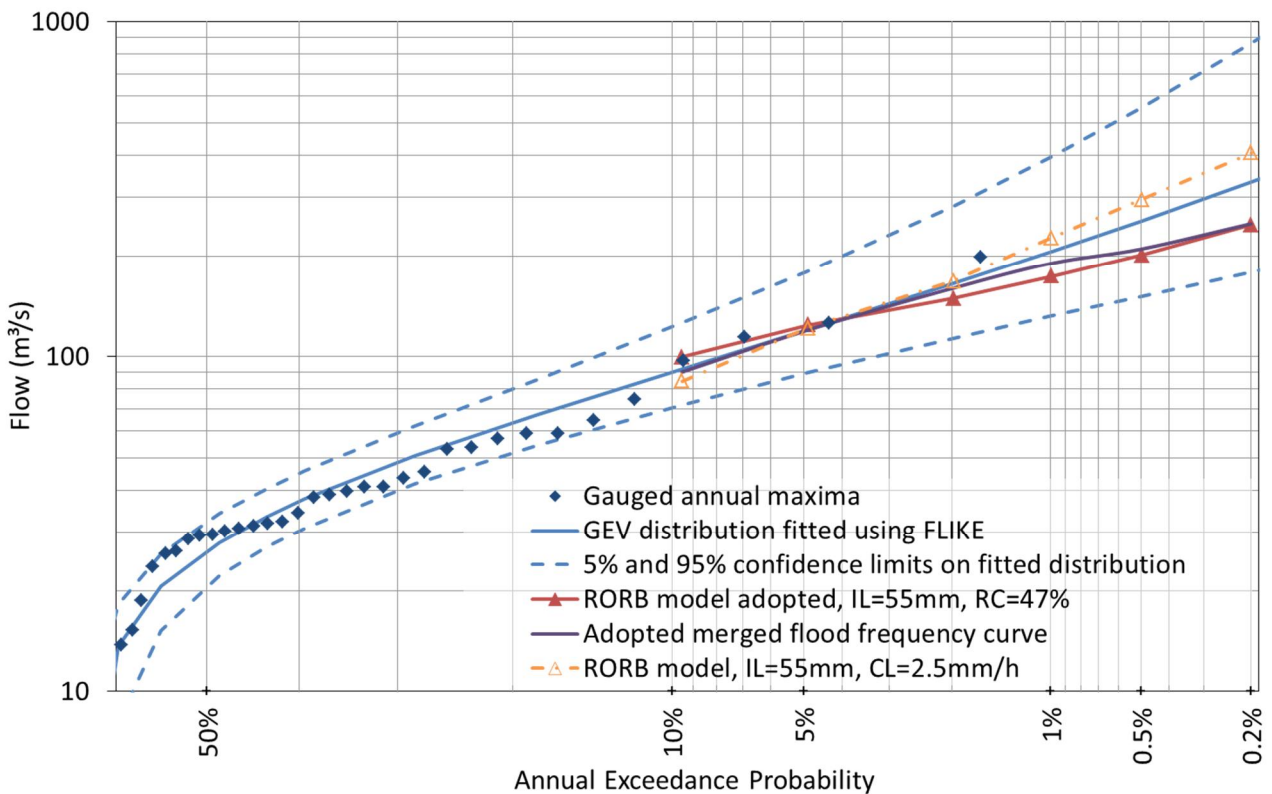


Figure 8-4 Adopted flood frequency curve and RORB model verified to flood frequency analysis of gauged peak flows for the Goulburn River upstream of Snake Creek gauge (405263)

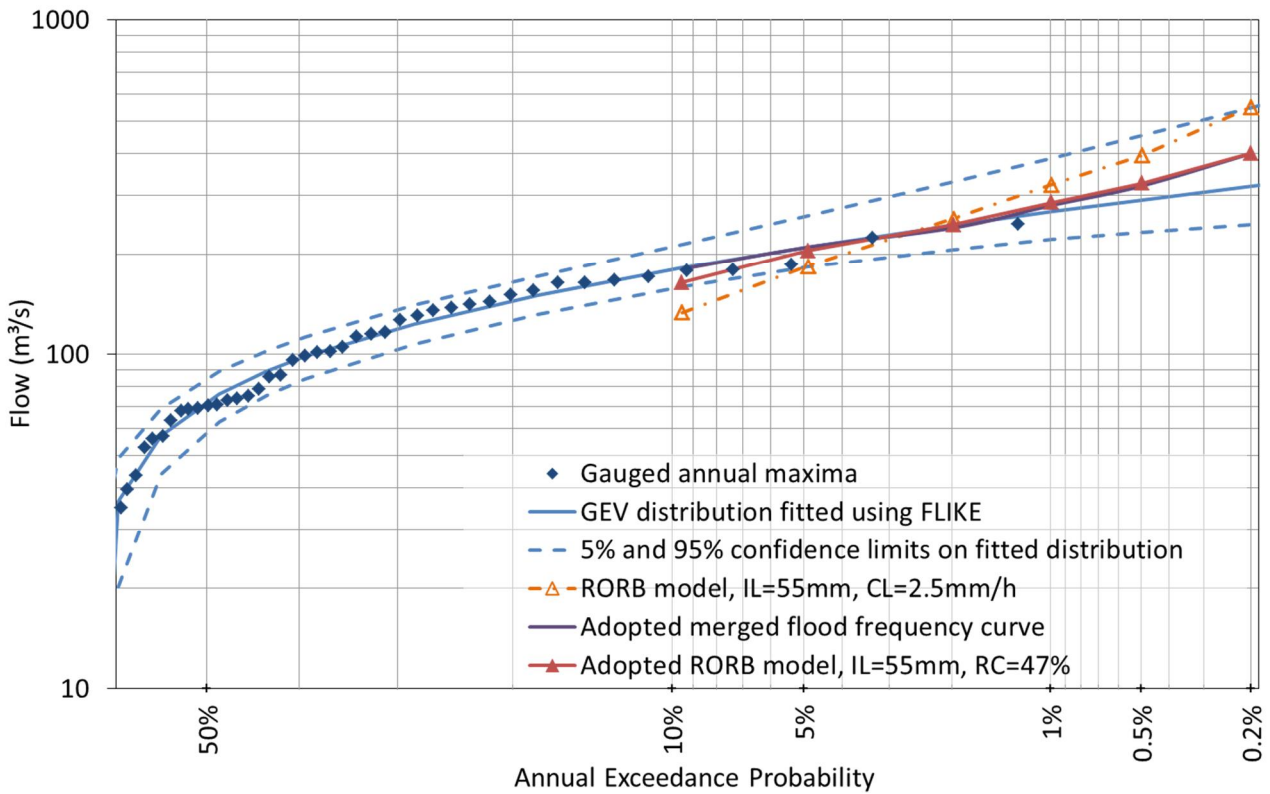


Figure 8-5 Adopted flood frequency curve and RORB model verified to flood frequency analysis of gauged peak flows for the Goulburn River at Dohertys gauge (405219)

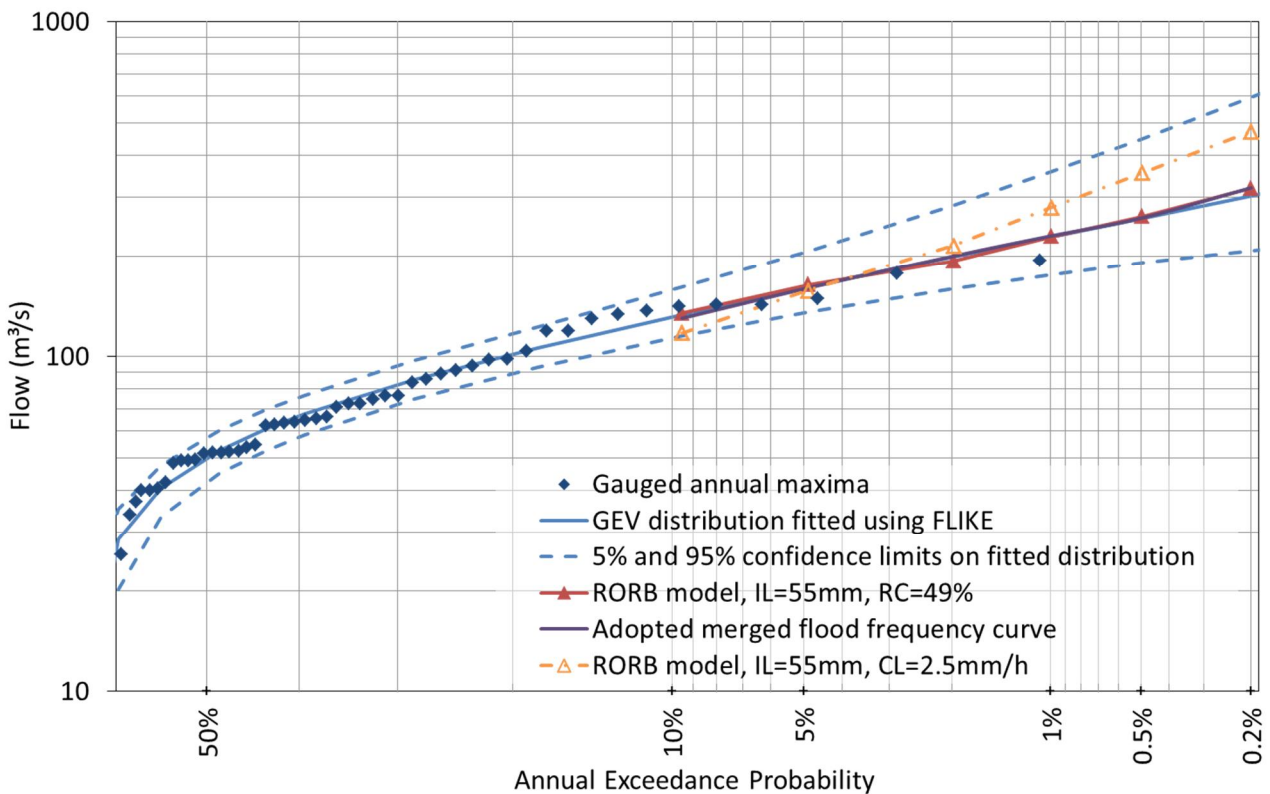


Figure 8-6 Adopted flood frequency curve and RORB model verified to flood frequency analysis of gauged peak flows for the Jamieson River at Gerrang Bridge gauge (405218)

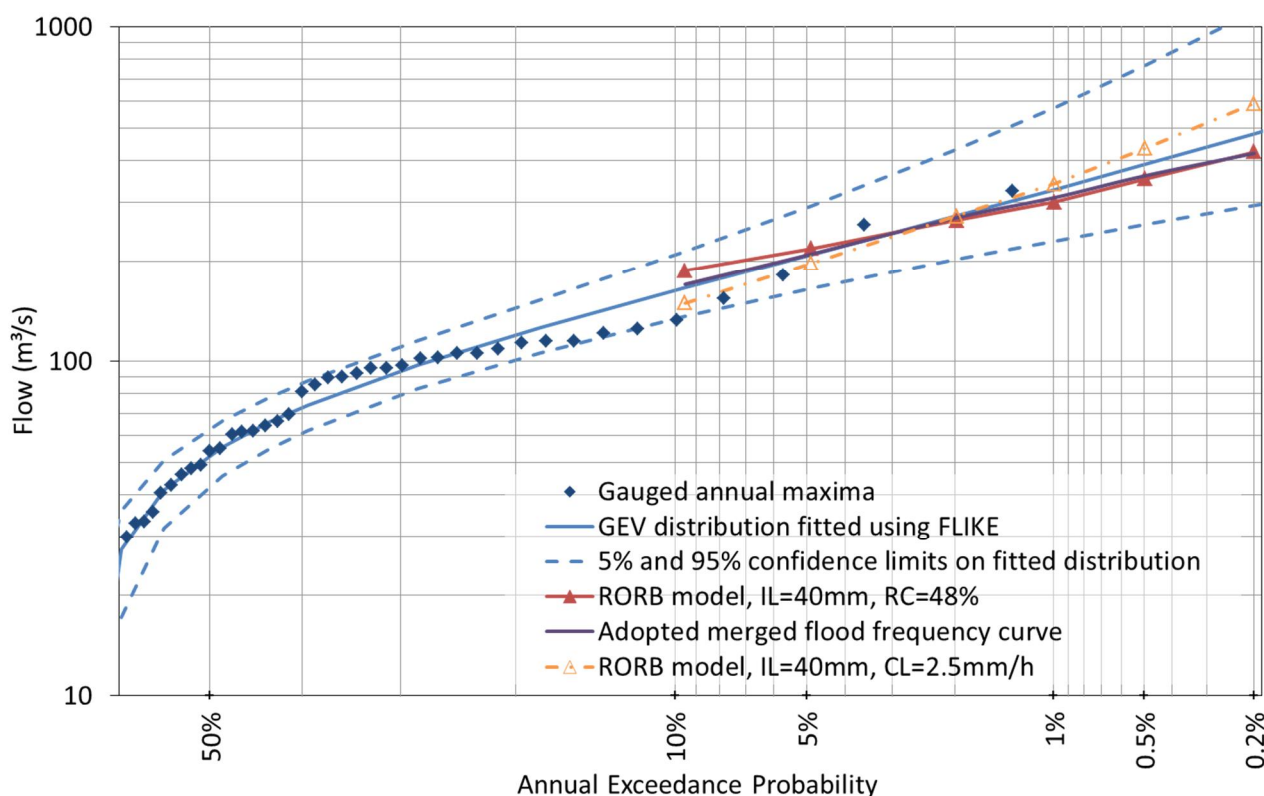


Figure 8-7 Adopted flood frequency curve and RORB model verified to flood frequency analysis of gauged peak flows for the Big River at Jamieson gauge (405227)

### 8.3 Verification of RORB models and adopted flood frequency curves for inflows to Eildon Dam

A modified approach was required in order to undertake the verification to flood frequency on inflows to Eildon Dam. The inflows to the upstream gauge locations were produced using separate Northern and Southern area RORB models for the two portions of the catchment upstream of Eildon. The Southern Area RORB model adopted an initial loss runoff coefficient model, whilst the Northern Area RORB model adopted an initial loss continuing loss model.

In order to model inflows to Eildon Dam, a single model was required and the RORB software required that a single loss model type was adopted for the whole catchment. For simulations of inflows to Eildon, an initial loss continuing loss model was adopted for the whole catchment RORB model (all subcatchments) because it was anticipated that future simulations of Lake Eildon, as a large dam, are likely to be concerned with floods in the very large and extreme range and extrapolation of RORB model simulations based upon an initial loss continuing loss model would be more defensible for the production of large and extreme floods than an initial loss runoff coefficient model.

Inflow flood volumes for the peak 24 hour period of events simulated from the RORB model were verified to the flood frequency analysis fitted to the series of historical annual maxima mean daily inflows to Lake Eildon. Parameters adopted for each of the interstation areas of the RORB model are shown in Table 8-4. The values of the routing parameters,  $k_c$  and  $m$  for each of the interstation areas for the whole of Eildon model were the same as those adopted for each of the subcatchment models. However, median initial loss and continuing loss rate parameters were adopted for the whole of Eildon model in order to verify the flood frequency analysis of historical inflows to Lake Eildon. This was possibly because the temporal patterns from the GSAM sample become less variable for the large catchment area of Eildon (3,874 km<sup>2</sup>) than for the individual subcatchments

(typically in the 300 – 1,100 km<sup>2</sup> range) and hence lower loss rates are required in the simulations on the larger catchment area in order to generate sufficient runoff during the simulation of design flood events.

Table 8-4 Adopted parameters for portions of RORB model used for verification of inflow floods to Lake Eildon

Interstation Area	Total Area (km <sup>2</sup> )	$d_{av}$ (km)*	$k_c$	$k_c / d_{av}$	Median IL (mm)	CL (mm/h)
Delatite at Tonga Bridge	349.5	24.47	27.00	1.10	29	1.25
Delatite to Eildon Tailwater	394.1	4.79	5.29	1.10	29	1.25
Fords Creek at Mansfield	116.9	11.75	5.50	0.47	37	0.50
Fords Creek to Eildon Tailwater	142.7	5.23	2.45	0.47	37	0.50
Remainder of Northern Area RORB model	1411.5	38.86	39.73	1.02	27	1.25
Howqua at Glen Esk	368.0	36.77	45.00	1.22	40	1.75
Howqua to Eildon Tailwater	394.8	6.96	7.66	1.22	40	1.75
Goulburn upstream of Snake Creek	327.1	28.78	47.00	1.63	27	1.25
Goulburn at Doherteyes	701.6	32.41	39.00	1.20	27	1.25
Jamieson at Gerrang Bridge	361.8	37.91	39.00	1.03	27	1.25
Goulburn to Eildon Tailwater	1109.5	5.94	7.15	1.20	27	1.25
Big River downstream of Frenchmans Creek	331.4	21.54	32.13	1.49	27	1.25
Big River to Eildon Tailwater	650.5	4.26	6.36	1.49	27	1.25
Remainder of Southern Area RORB model	2462.9	18.11	37.09	2.05	27	1.25

\* Note: whilst  $d_{av}$  is a conventional distance measured in km for most interstation areas, the routing distance is adjusted for the reaches in the model that are drowned within the reservoir of Lake Eildon to represent the reduction in routing travel times associated with increased average depth of flow.

Figure 8-8 shows that with the adopted parameters, a good fit was achieved to the flood frequency analysis that was fitted to historical annual maxima. Since the flood frequency analysis was performed using mean daily flow estimates, the RORB model results that were compared for verification were the distribution of mean inflow rate over the peak 24 hour period of each event. The RORB model with the same parameters produced peak (instantaneous) inflow rates to Lake Eildon that were between 6% and 8% larger than the mean inflow rate over the peak 24 hour period of each event (comparing green with red lines on Figure 8-8).

The whole Eildon RORB model was run in Monte-Carlo simulation mode with seasonal reservoir drawdown curves, from Sinclair Knight Merz (2004b). Table 8-3 lists the fitted quantiles after verification for mean daily inflow and peak instantaneous inflow to Lake Eildon.

Outflow is not presented here as it is extremely sensitive to model assumptions. In particular, assumptions around initial reservoir drawdown and gate operations will substantially alter the peak outflow rates.

The inflow and outflow flood frequency curves would be sensitive to baseflow inflows to Lake Eildon during flood events, the distribution of initial drawdown of Lake Eildon before floods and the flood operations procedures for Lake Eildon (i.e. the relationship between the water level in Lake Eildon, gate openings and hence outflow rates). The assumptions about all of these inputs were the same as those adopted by Sinclair Knight Merz (2004b).

Table 8-5 RORB model quantiles of inflows to Lake Eildon

Location	Hydrograph	Flood Quantiles of Mean Inflow for 24 hour period (m <sup>3</sup> /s) at AEP					
		10%	5%	2%	1%	0.5%	0.2%
Lake Eildon	Flood Frequency analysis of historical mean daily inflow maxima	850	1050	1300	1550	1800	2100
	Mean Daily Inflow from RORB model	850	1050	1400	1750	2100	2700
	Peak Inflow from RORB model	920	1100	1510	1870	2280	2890

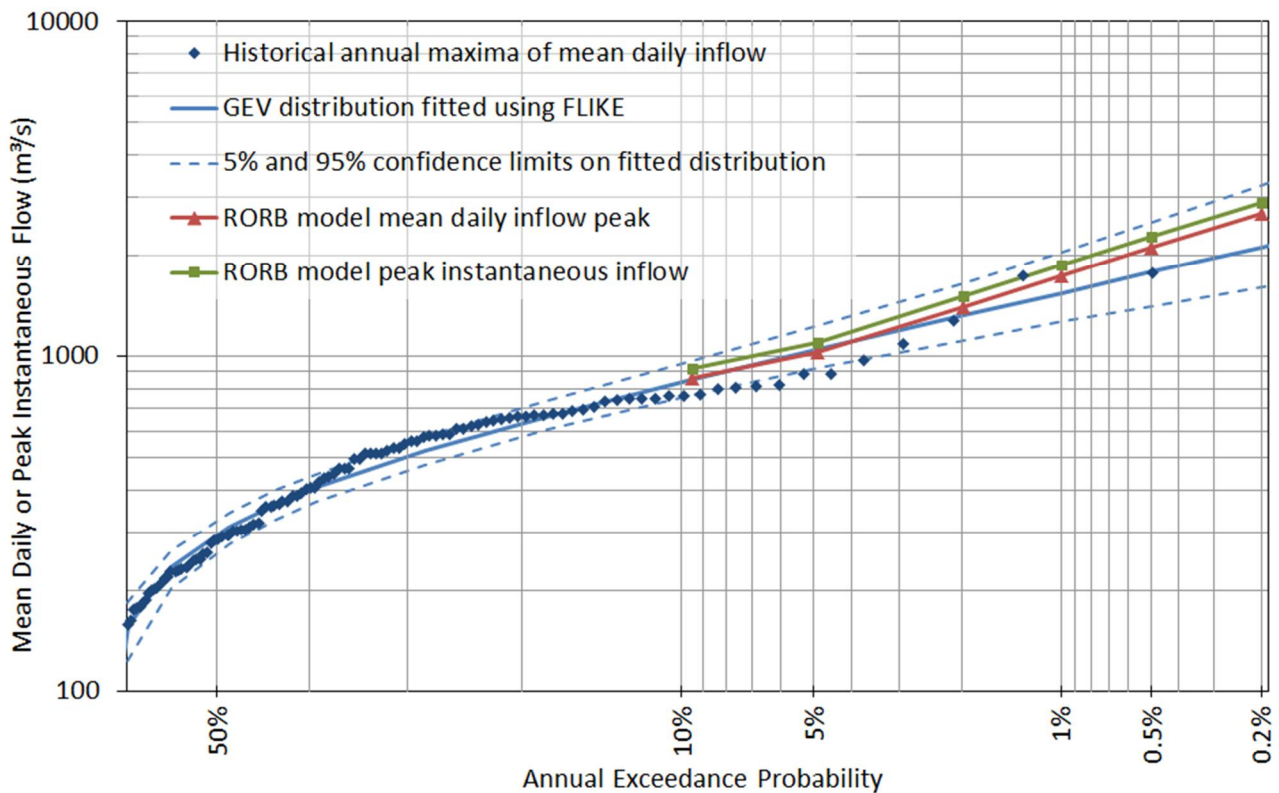


Figure 8-8 Adopted flood frequency curve and RORB model verified to flood frequency analysis of historical inflow maxima to Lake Eildon

## 8.4 Verification of RORB models and adopted flood frequency curves for outflows from Lake Nillahcootie

Parameters of the Broken River to Nillahcootie RORB model were verified to flood frequency analysis fitted to recorded historical annual maxima outflows from Lake Nillahcootie for the period between 1993 and 2014. Adopted parameter values are shown in Table 8-6. Figure 8-9 shows that with the adopted parameters, a good fit was achieved to the flood frequency analysis that was fitted to historical annual maxima of outflows from Lake Nillahcootie. The initial loss, continuing loss rate,  $k_c$  and  $m$  parameters of the RORB models were consistent with the values obtained from calibration to historical flood events.

The RORB model was run in Monte-Carlo simulation mode allowing for the joint probability of reservoir drawdown. The resulting peak outflow rates for the design events were between 28% and 50% less than the peak inflow rates, due to routing of the inflow floods through Lake Nillahcootie and the effect of drawdown. Table 8-7 lists the fitted quantiles after verification for peak inflow and outflow from Lake Nillahcootie.

Table 8-6 Adopted parameters of Broken River to Lake Nillahcootie RORB model after verification of peaks from Monte-Carlo simulations to flood frequency analysis at flow gauges

Gauged Catchment	Total Area Upstream (km <sup>2</sup> )	$d_{av}$ (km)	$k_c$	$m$	Initial Loss (mm)	Continuing Loss Rate (mm/h)
Broken River catchment to Lake Nillahcootie	415.9	30.61	28	0.8	40	1.0

Table 8-7 RORB model quantiles of inflows to and outflows from Lake Nillahcootie

Location	Hydrograph	Flood Quantiles (m <sup>3</sup> /s) at AEP					
		10%	5%	2%	1%	0.5%	0.2%
Lake Nillahcootie	Flood Frequency analysis of historical peak outflow maxima (1993-2014)	140	220	360	510	720	1100
	Peak Inflow from RORB model	320	400	500	600	700	860
	Peak Outflow from RORB model	160	240	310	400	490	620

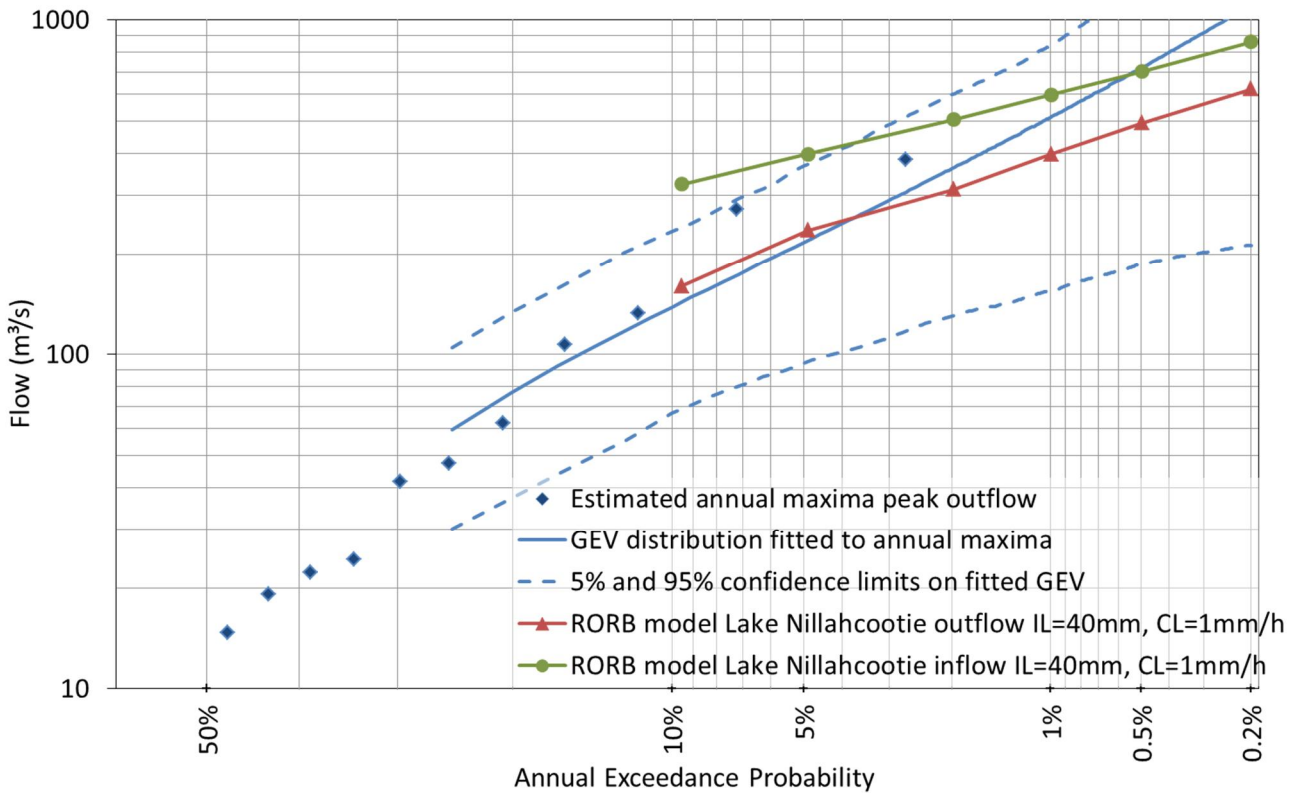


Figure 8-9 Adopted flood frequency curve and RORB model verified to flood frequency analysis of annual maxima of peak outflows from Lake Nillahcootie

## 9. Monte-Carlo simulation of design flood frequency curves at specified locations

Flooding is of particular interest at several locations in the catchment that are not particularly near to a streamflow gauge. At these other locations of interest, the RORB model provides the most viable means of estimating the flood quantiles. It was possible to adjust the quantiles from the RORB simulations using the percentage differences between the adopted (merged) flood quantiles and the RORB quantiles at a nearby streamflow gauge. The other locations of interest, where flood peak quantiles were required were:

- Goulburn River at Woods Point (peak flows from RORB adjusted by calculating percentage adjustments at Goulburn River at upstream of Snake Creek gauge);
- Goulburn River at Eildon Dam tailwater (percentage adjustments calculated from Goulburn River at Doherteys and Jamieson River at Gerrang Bridge gauges);
- Delatite River at Eildon Dam tailwater (percentage adjustments calculated from Tonga Bridge); and
- Howqua River at Eildon Dam tailwater (percentage adjustments calculated from Glen Esk).

Table 9-1 lists the adopted parameters of the Eildon Northern Area RORB model that were adopted for deriving inflow floods for the gauged areas and for the residual areas of the Delatite River and Fords Creek catchments and for the residual area of the Eildon Northern Area RORB model to Eildon Dam. The value of the  $k_c$  parameter for the Delatite and Fords Creek catchments were calculated by scaling the  $k_c/d_{av}$  ratio from the respective gauged portion of the catchment. Losses were also adopted from the relevant gauged portion of the catchment. For the residual area to Eildon Dam, the value of the  $k_c$  parameter was derived by adopting the same  $k_c/d_{av}$  ratio as was adopted in the SKM (1999), since the subcatchments and reaches had not been modified appreciably in this portion of the model since the SKM (1999) version of the catchment file.

Table 9-1 Adopted parameters for portions of the Eildon Northern Area RORB model after verification to flood frequency analysis

Interstation Area	Total Area (km <sup>2</sup> )	$d_{av}$ (km)*	$k_c$	$k_c / d_{av}$	IL (mm)	CL (mm/h)	Method for $k_c$ and losses
Delatite at Tonga Bridge	349.5	24.47	27.00	1.10	58	2.5	Verified to FFA at gauge
Delatite to Eildon Tailwater	394.1	4.79	5.29	1.10	58	2.5	Adopted from Tonga Bridge
Fords Creek at Mansfield	116.9	11.75	5.50	0.47	75	1.0	Verified to FFA at gauge
Fords Creek to Eildon Tailwater	142.7	5.23	2.45	0.47	75	1.0	Adopted from Mansfield gauge
Remainder of Northern Area RORB model	1411.5	38.86	39.73	1.02	62	2.1	$k_c / d_{av}$ from SKM (1999) model, losses mean of gauged portions of Northern Eildon catchment

\* Note: whilst  $d_{av}$  is a conventional distance measured in km for most interstation areas, the routing distance is adjusted for the reaches in the model that are drowned within the reservoir of Lake Eildon to represent the reduction in routing travel times associated with increased average depth of flow.

Table 9-2 lists the adopted parameters of the Eildon Southern Area RORB model that were adopted for deriving inflow floods for the gauged areas and for the residual areas of the Howqua, Goulburn and Big River catchments and for the residual area of the Eildon Southern Area RORB model to Eildon Dam. The value of the  $k_c$  parameter for the Howqua River, Big River and Goulburn/Jamieson River catchments were calculated by scaling the  $k_c/d_{av}$  ratio from the respective gauged portion of the catchment. Losses were also adopted from the relevant gauged portion of the catchment. For the residual area to Eildon Dam, the value of the  $k_c$  parameter was derived by adopting the same  $k_c/d_{av}$  ratio as was adopted in the SKM (1999), since the subcatchments and reaches had not been modified appreciably in this portion of the model since the SKM (1999) version of the catchment file.

Table 9-2 Adopted parameters for portions of the Eildon Southern Area RORB model after verification to flood frequency analysis

Interstation Area	Total Area (km <sup>2</sup> )	$d_{av}$ (km)*	$k_c$	$k_c / d_{av}$	IL (mm)	RC	Method for $k_c$ and losses
Howqua at Glen Esk	368.0	36.77	45.00	1.22	80	39%	Verified to FFA at gauge
Howqua at Ben Bullen	386.3	4.15	5.08	1.22	80	39%	Adopted from Glen Esk gauge
Howqua to Eildon Tailwater	394.8	2.11	2.58	1.22	80	39%	Adopted from Glen Esk gauge
Goulburn upstream of Snake Creek	327.1	28.78	47.00	1.63	55	47%	Verified to FFA at gauge
Goulburn at Doherteys	701.6	32.41	39.00	1.20	55	47%	Verified to FFA at gauge
Jamieson at Gerrang Bridge	361.8	37.91	39.00	1.03	55	49%	Verified to FFA at gauge
Goulburn to Eildon Tailwater	1109.5	5.94	7.15	1.20	55	47%	Adopted from Doherteys
Big River downstream of Frenchmans Creek	331.4	21.54	32.13	1.49	40	48%	Adopted from Big River at Jamieson gauge
Big River at Jamieson	627.4	18.68	27.87	1.49	40	48%	Verified to FFA at gauge
Big River to Eildon Tailwater	650.5	4.26	6.36	1.49	40	48%	Adopted from Big River at Jamieson gauge
Remainder of Southern Area RORB model	2462.9	18.11	37.09	2.05	55	46%	$k_c / d_{av}$ from SKM (1999) model, losses mean of gauged portions of Southern Eildon catchment

\* Note: whilst  $d_{av}$  is a conventional distance measured in km for most interstation areas, the routing distance is adjusted for the reaches in the model that are drowned within the reservoir of Lake Eildon to represent the reduction in routing travel times associated with increased average depth of flow.

Table 9-3 lists the flood quantiles derived at the relevant additional locations of interest: Delatite River at Eildon Tailwater, Howqua River at Eildon Tailwater, Goulburn River at Woods Point and Goulburn River at Eildon Tailwater (including both the Eildon and Jamieson River catchments).

Table 9-3 Flood quantiles at other locations of interest derived from RORB Monte-Carlo simulations, with adjustment of RORB quantiles based upon percentage adjustments at nearest flow gauge (and then rounded to nearest 10 m<sup>3</sup>/s for all sites except Goulburn River at Woods Point)

Interstation Area	Method	Flood Quantiles (m <sup>3</sup> /s) at AEP					
		10%	5%	2%	1%	0.5%	0.2%
Delatite River at Eildon Dam Tailwater	RORB	199	260	383	498	623	823
	% Adjustment to RORB	-4%	+1%	-1%	-1%	-1%	0%
	Adopted	190	260	380	490	620	820
Goulburn River at Woods Point	RORB	52	63	79	94	112	138
	% Adjustment to RORB	-10%	-3%	+7%	-9%	-4%	+1%
	Adopted	47	61	85	100	115	140
Goulburn River at Eildon Dam Tailwater	RORB	272	334	401	480	552	678
	% Adjustment to RORB	+5%	+1%	0%	-1%	-1%	0%
	Adopted	290	340	400	480	540	680
Howqua River at Eildon Dam Tailwater	RORB	85	111	133	154	179	218
	% Adjustment to RORB	+7%	-1%	-2%	-1%	-4%	0%
	Adopted	90	110	130	150	170	220

## 10. Generation of design flood hydrographs for representative single events

### 10.1 General Approach

The Monte-Carlo simulation approach constructs a flood frequency curve at a location from peak flows generated from thousands of runs of the RORB rainfall-runoff routing model. As it is infeasible to undertake a comparably large number of runs of a two-dimensional hydraulic flood model such as TUFLOW with current computer technology, the purpose of this component was to generate a single inflow hydrograph at each input location to the TUFLOW model that was representative of a flood with the nominated AEP. Hydrographs were generated using a single temporal pattern and with initial loss and continuing loss or runoff coefficient values for the single run to produce a single hydrograph from the RORB model with a flood peak (for the critical duration event) that matches the flood quantile for the same AEP and critical duration at the most relevant location for hydraulic modelling of flood events.

### 10.2 Delatite River Subcatchment

Design flood hydrographs were extracted from the RORB model for design flood events up to the 1 in 500 AEP for the catchment to the Delatite River at Eildon Dam tailwater. Hydrographs were generated with the temporal patterns, durations and loss parameters listed in Table 10-1. The loss parameters adopted to match to the flood frequency at the catchment outlet are reasonably consistent across durations and they are also relatively consistent with the median initial loss (65 mm) and median continuing loss rate (2.5 mm/h) that were adopted for the Monte-Carlo simulations, particularly as the probability distribution of initial loss is skewed (i.e. the median value is less than the mean and less than half the maximum value).

Hydrographs were extracted at locations where tributary inflows or local catchments enter the Delatite River system. The locations of the extracted hydrographs are shown in Figure 4-4.

Table 10-1 Parameters adopted for generation of representative hydrographs in Delatite River subcatchment

Event AEP (1 in Y)	Adopted Event Duration (hours)	Representative Temporal Pattern	RORB $k_c$ parameter to Tonga Bridge gauge	RORB $k_c$ parameter Tonga Bridge to Eildon Dam tailwater	Initial Loss (mm)	Continuing Loss (mm/h)
10	24	GSAM Unsmoothed	27	5.29	71	2.5
20	24	GSAM Unsmoothed	27	5.29	71.5	2.5
50	24	GSAM Unsmoothed	27	5.29	85	2.5
100	24	GSAM Unsmoothed	27	5.29	93	2.5
200	24	GSAM Unsmoothed	27	5.29	105	2.5
500	24	GSAM Unsmoothed	27	5.29	122	2.5

### 10.3 Fords Creek subcatchment

Flood hydrographs were generated for the entire Fords Creek subcatchment to Eildon Dam tailwater. However, the primary location of interest for flood estimation is the town of Mansfield and surrounding areas. Design flood hydrographs were therefore extracted from the RORB model for design flood events up to the 1 in 500 AEP with peak flows that matched the flood frequency quantiles on Fords Creek at the Mansfield streamflow gauge. Hydrographs were generated with the temporal patterns, durations and loss parameters listed in Table 10-2 for all of the inflow locations required, including those downstream of the flow gauge to Eildon Dam tailwater. The loss parameters adopted to match to the flood frequency at the Mansfield flow gauge are reasonably consistent

across durations and they are also relatively consistent with the median initial loss (75 mm) and median continuing loss rate (1.0 mm/h) that were adopted for the Monte-Carlo simulations.

Hydrographs were extracted at locations where tributary inflows or local catchments enter the Fords Creek system. The locations of the extracted hydrographs are shown in Figure 4-3.

Table 10-2 Parameters adopted for generation of representative hydrographs in Fords Creek subcatchment

Event AEP (1 in Y)	Adopted Event Duration (hours)	Representative Temporal Pattern	RORB $k_c$ parameter to Mansfield	Initial Loss (mm)	Continuing Loss (mm/h)
10	36	GSAM Unsmoothed	5.5	68	1.0
20	36	GSAM Unsmoothed	5.5	74	1.0
50	36	GSAM Unsmoothed	5.5	74	1.0
100	36	GSAM Unsmoothed	5.5	77	1.0
200	36	GSAM Unsmoothed	5.5	85	1.0
500	36	GSAM Unsmoothed	5.5	104	1.0

## 10.4 Big River subcatchment

Flood hydrographs were generated for the entire Big River subcatchment to Eildon Dam tailwater. However, the streamflow gauge on the Big River is relatively close to Eildon Dam tailwater and the main area of interest for design flood estimation is in the vicinity of the streamflow gauge. Design flood hydrographs were therefore extracted from the RORB model for design flood events up to the 1 in 500 AEP with peak flows that matched the flood frequency quantiles on Big River at Jamieson streamflow gauge. Hydrographs were generated with the temporal patterns, durations and loss parameters listed in Table 10-3. The loss parameters adopted to match to the flood frequency at the catchment outlet are reasonably consistent across durations and they are also relatively consistent with the median initial loss (40 mm) and runoff coefficient (48%) that was adopted for the Monte-Carlo simulations.

Hydrographs were extracted at locations where tributary inflows or local catchments enter the Big River system. The locations of the extracted hydrographs are shown in Figure 4-11.

Table 10-3 Parameters adopted for generation of representative hydrographs in Big River subcatchment

Event AEP (1 in Y)	Adopted Event Duration (hours)	Representative Temporal Pattern	RORB $k_c$ parameter to Jamieson gauge	Initial Loss (mm)	Runoff Coefficient
10	48	GSAM Unsmoothed	60	44	48%
20	48	GSAM Unsmoothed	60	43	48%
50	48	GSAM Unsmoothed	60	37	48%
100	48	GSAM Unsmoothed	60	34	48%
200	36	GSAM Unsmoothed	60	43	48%
500	36	GSAM Unsmoothed	60	56	48%

## 10.5 Upper Goulburn River and Jamieson River subcatchment

Flood hydrographs were generated for the entire Upper Goulburn and Jamieson River subcatchment to Eildon Dam tailwater. There were locations of interest through this subcatchment with very different upstream catchment areas (and hence times of concentration), with the main areas being (1) around Woods Point and (2) the Jamieson area, around the confluence of the Goulburn and Jamieson Rivers and extending upstream from the confluence.

For ease of running hydraulic model simulations, it was desirable to have a single set of design inflow hydrographs for each AEP that applied across the whole of the Upper Goulburn and Jamieson River catchment that would still allow for simulation of design events that with peak flows that were consistent with Monte-Carlo simulations at both of the main areas of interest. It was possible to achieve this objective.

Design flood hydrographs were first extracted from the RORB model for design flood events up to the 1 in 500 AEP for the Goulburn River at Woods Point. Hydrographs were generated with the temporal patterns, durations and loss parameters listed in Table 10-4. The loss parameters adopted to match to the flood frequency at the catchment outlet are reasonably consistent across durations and they are also relatively consistent with the median initial loss (55 mm) and runoff coefficient (47%) values that were adopted for the Monte-Carlo simulations.

Table 10-4 Parameters adopted for generation of representative hydrographs in Goulburn River upstream of Woods Point

Event AEP (1 in Y)	Adopted Event Duration (hours)	Representative Temporal Pattern	RORB $k_c$ parameter to Upstream of Snake Creek Gauge	Initial Loss (mm)	Runoff Coefficient
10	18	GSAM Unsmoothed	47	45	47%
20	18	GSAM Unsmoothed	47	44	47%
50	18	GSAM Unsmoothed	47	34	47%
100	18	GSAM Unsmoothed	47	33	47%
200	18	GSAM Unsmoothed	47	36	47%
500	18	GSAM Unsmoothed	47	35	47%

In the second stage of simulating design flood hydrographs for this subcatchment, hydrographs were then extracted from the RORB model for design flood events up to the 1 in 500 AEP for the catchment of the Goulburn River at Eildon Dam Tailwater. In these simulations, the hydrograph at Woods Point for each event was over-ridden with the hydrograph generated from the single inflow hydrograph simulations at Woods Point. A check was performed and it was found that this approach did not disturb the timing of the inflow floods downstream of the forced inflow at Woods Point.

Design flood hydrographs were generated with the temporal patterns, durations and loss parameters listed in Table 10-5. The loss parameters adopted to match to the flood frequency at the catchment outlet are reasonably consistent across durations and they are also relatively consistent with the median initial loss (55 mm) and runoff coefficient (47% for Goulburn River, 49% for Jamieson River) that were adopted for the Monte-Carlo simulations. Hydrographs were also generated for an event representative of a Major flood event at the Dohertys gauge (6 metres gauge height or approximately 295 m<sup>3</sup>/s peak flow), with the temporal patterns, durations and loss parameters listed in Table 10-6.

Hydrographs were extracted at locations where tributary inflows or local catchments enter the Goulburn River. The locations of the extracted hydrographs are shown in Figure 4-8 and Figure 4-9

Table 10-5 Parameters adopted for generation of representative hydrographs in Goulburn River to Eildon Dam Tailwater

Event AEP (1 in Y)	Adopted Event Duration (hours)	Representative Temporal Pattern	Interstation Area	RORB $k_c$ parameter	Initial Loss (mm)	Runoff Coefficient
10	48	GSAM Unsmoothed	Goulburn to Upstream of Snake Creek gauge	47	35	47%
			Goulburn from US Snake Creek to Doherteys	39	35	47%
			Jamieson River at Gerrang Bridge	39	35	49%
			To Eildon Tailwater	7.15	35	47%
20	48	GSAM Unsmoothed	Goulburn to Upstream of Snake Creek gauge	47	36	47%
			Goulburn from US Snake Creek to Doherteys	39	36	47%
			Jamieson River at Gerrang Bridge	39	36	49%
			To Eildon Tailwater	7.15	36	47%
50	48	GSAM Unsmoothed	Goulburn to Upstream of Snake Creek gauge	47	44	47%
			Goulburn from US Snake Creek to Doherteys	39	44	47%
			Jamieson River at Gerrang Bridge	39	44	49%
			To Eildon Tailwater	7.15	44	47%
100	48	GSAM Unsmoothed	Goulburn to Upstream of Snake Creek gauge	47	35	47%
			Goulburn from US Snake Creek to Doherteys	39	35	47%
			Jamieson River at Gerrang Bridge	39	35	49%
			To Eildon Tailwater	7.15	35	47%
200	48	GSAM Unsmoothed	Goulburn to Upstream of Snake Creek gauge	47	43	47%
			Goulburn from US Snake Creek to Doherteys	39	43	47%
			Jamieson River at Gerrang Bridge	39	43	49%
			To Eildon Tailwater	7.15	43	47%
500	48	GSAM Unsmoothed	Goulburn to Upstream of Snake Creek gauge	47	36	47%
			Goulburn from US Snake Creek to Doherteys	39	36	47%
			Jamieson River at Gerrang Bridge	39	36	49%
			To Eildon Tailwater	7.15	36	47%

Table 10-6 Parameters adopted for generation of representative hydrographs for Major Flood event in Goulburn River at Dohertys Gauge (295 m<sup>3</sup>/s estimated peak flow)

Event AEP (1 in Y)	Adopted Event Duration (hours)	Representative Temporal Pattern	Interstation Area		RORB $k_c$ parameter	Initial Loss (mm)	Runoff Coefficient
100	48	GSAM Unsmoothed	Goulburn to Upstream of Snake Creek gauge		47	39	47%
			Goulburn from US Snake Creek to Dohertys		39	39	47%
			Jamieson River at Gerrang Bridge		39	39	49%
			To Eildon Tailwater		7.15	39	47%

## 10.6 Howqua River subcatchment

Design flood hydrographs were extracted from the RORB model for design flood events up to the 1 in 500 AEP for the catchment of the Howqua River to Eildon Dam Tailwater. Hydrographs were generated with the temporal patterns, durations and loss parameters listed in Table 10-7. The loss parameters adopted to match to the flood frequency at the catchment outlet are reasonably consistent across durations and they are also relatively consistent with the median initial loss (80 mm) and runoff coefficient (39%) values that were adopted for the Monte-Carlo simulations.

Hydrographs were extracted at locations where tributary inflows or local catchments enter the Howqua River system. The locations of the extracted hydrographs are shown in Figure 4-6.

Table 10-7 Parameters adopted for generation of representative hydrographs in Howqua River subcatchment

Event AEP (1 in Y)	Adopted Event Duration (hours)	Representative Temporal Pattern	RORB $k_c$ to Glen Esk gauge	RORB $k_c$ Glen Esk to Eildon Tailwater	Initial Loss (mm)	Runoff Coefficient
10	48	GSAM Unsmoothed	45	7.66	50	39%
20	36	GSAM Unsmoothed	45	7.66	52	39%
50	36	GSAM Unsmoothed	45	7.66	61	39%
100	36	GSAM Unsmoothed	45	7.66	65	39%
200	36	GSAM Unsmoothed	45	7.66	75	39%
500	36	GSAM Unsmoothed	45	7.66	77	39%

## 10.7 Broken River catchment

Design flood hydrographs were extracted from the RORB model for design flood events up to the 1 in 500 AEP with peak flows that matched the flood frequency quantiles for inflows to Lake Nillahcootie from the Monte-Carlo simulations. Hydrographs were generated with the temporal patterns, durations and loss parameters listed in Table 10-8 for all of the inflow locations required. The loss parameters adopted to match to the flood frequency at Lake Nillahcootie are reasonably consistent across durations and they are also relatively consistent with the median initial loss (40 mm) and median continuing loss rate (1.0 mm/h) that were adopted for the Monte-Carlo simulations.

Hydrographs were extracted at locations where tributary inflows or local catchments enter the Broken River system. The locations of the extracted hydrographs are shown in Figure 4-13.

Table 10-8 Parameters adopted for generation of representative hydrographs in Broken River catchment to Lake Nillahcootie

Event AEP (1 in Y)	Adopted Event Duration (hours)	Representative Temporal Pattern	RORB $k_c$ parameter to Lake Nillahcootie	Initial Loss (mm)	Continuing Loss (mm/h)
10	24	GSAM Unsmoothed	28	35	1.0
20	24	GSAM Unsmoothed	28	38	1.0
50	24	GSAM Unsmoothed	28	42	1.0
100	24	GSAM Unsmoothed	28	43	1.0
200	24	GSAM Unsmoothed	28	47	1.0
500	24	GSAM Unsmoothed	28	48	1.0

## 11. Conclusions and recommendations

Inflow hydrographs were generated for the purposes of design flood estimation in the rural catchments of the tributaries of the Goulburn River upstream of Lake Eildon and for the Broken River upstream of Lake Nillahcootie.

Inflow flood hydrographs were generated for representative design flood events with AEPs of 1 in 10, 20, 50, 100, 200 and 500. The inflow hydrographs were supplied at multiple locations in each catchment for the nominated design flood events, suitable as inputs to a hydraulic flood simulation model, such as TUFLOW. The single design events were generated such that the peak from the single event generated in the RORB model matched the corresponding flood quantile from joint probability Monte-Carlo simulations with RORB. The flood quantiles from the Monte-Carlo simulations in turn were verified to flood quantiles produced from flood frequency analysis to historical maxima gauged flows. At most gauges, excellent matches were achieved between the RORB model simulated flood quantiles and the flood frequency analysis. The calibration of the RORB models was also checked by running a number of calibration flood events in each catchment and good fits were achieved for most events at most of the gauges.

There is more uncertainty associated with estimating the representative outflow flood hydrographs for Lake Eildon and Lake Nillahcootie. For both dams, the outflow flood frequency curve will be sensitive to the model assumptions made about the probability distribution of initial reservoir drawdown, thus increasing the uncertainty associated with fitting the RORB model parameters for the catchment upstream of the dam to fit derived inflow and outflow flood frequency curves for each site.

In the case of Lake Eildon, the flood operations procedures for the dam (notably the relationship between upstream water level and flood gate opening) will have a significant influence on the shape and the peak flow for the outflow flood hydrograph from Lake Eildon. This effect may be particularly noticeable for floods in the range of AEP modelled in this study (1 in 10 to 1 in 500). For these reasons, outflow flood hydrographs were not provided for Lake Eildon.

The outflow results presented for Lake Nillahcootie should be considered as indicative, considering the influence of dam operations. Further review should be undertaken of the assumptions about reservoir drawdown and gate operations before these results are relied upon for setting flood planning levels downstream of the reservoir.

## Important note about your report

The sole purpose of this report and the associated services performed by Jacobs is to generate inflow hydrographs for design flood events for the purpose of design flood modelling (to subsequently be undertaken by others) in agreed areas of the Goulburn and Broken River catchments, in accordance with the scope of services set out in the contract between Jacobs and Goulburn Broken Catchment Management Authority (GBCMA) the Client. That scope of services, as described in this report, was developed with GBCMA.

In preparing this report, Jacobs has relied upon, and presumed accurate, any information (or confirmation of the absence thereof) provided by GBCMA and/or from other sources. Except as otherwise stated in the report, Jacobs has not attempted to verify the accuracy or completeness of any such information. If the information is subsequently determined to be false, inaccurate or incomplete then it is possible that our observations and conclusions as expressed in this report may change.

Jacobs derived the data in this report from information sourced from GBCMA and/or available in the public domain at the time or times outlined in this report. The passage of time, manifestation of latent conditions or impacts of future events may require further examination of the project and subsequent data analysis, and re-evaluation of the data, findings, observations and conclusions expressed in this report. Jacobs has prepared this report in accordance with the usual care and thoroughness of the consulting profession, for the sole purpose described above and by reference to applicable standards, guidelines, procedures and practices at the date of issue of this report. For the reasons outlined above, however, no other warranty or guarantee, whether expressed or implied, is made as to the data, observations and findings expressed in this report, to the extent permitted by law.

This report should be read in full and no excerpts are to be taken as representative of the findings. No responsibility is accepted by Jacobs for use of any part of this report in any other context.

As agreed in our scope of work, the inflow flood hydrographs were calibrated to the September 1998 and September 2010 flood events and verified to flood frequency analysis on gauged flows at the nominated locations in the catchment. In time, future flood events may occur at these locations that are of similar size or larger in magnitude (flood peak and/or volume) than these events and should that occur, recalibration of the models used to generate the hydrographs should be considered. Jacobs have relied upon and presumed accurate the water level, gauged flows and rainfall data that we obtained from the client and other data collection agencies including the Bureau of Meteorology and Victorian Department of Environment and Primary Industries.

This report has been prepared on behalf of, and for the exclusive use of GBCMA and is subject to, and issued in accordance with, the provisions of the contract between Jacobs and GBCMA. Jacobs accepts no liability or responsibility whatsoever for, or in respect of, any use of, or reliance upon, this report by any third party

## 12. References

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## **Appendix A. Additional Information**