APPENDIX E

Flood Study Report

Attachment E Hydrological Calibration Report

NARRABRI TO NORTH STAR SUBMISSIONS PREFERRED INFRASTRUCTURE REPORT

IR_1432

Technical and Approvals Consultancy Services: Narrabri to North Star

Hydrological Model Calibration Report

March 2019

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Glossary

1D	One-dimensional
2D	Two-dimensional
AEP	Annual Exceedance Probability
ARR1987	Australian Rainfall & Runoff 1987
ARR2016	Australian Rainfall & Runoff 2016
CL	Continuing loss (rainfall) – a hydrological model parameter
FFA	Flood Frequency Analysis
FLIKE	A software program for undertaking Flood Frequency Analysis using the latest methods prescribed in ARR2106
HHIP	Hydrological and Hydraulic Investigation Plan
IL	Initial loss (rainfall) – a hydrological model parameter
k_c	A routing parameter in the RORB hydrological model
km	Kilometre
km^2	Square kilometres
m	A routing parameter in the RORB hydrological model
m^3/s	Cubic metres per second
mAHD	Metres above Australian Height Datum
MIKE FLOOD	An industry standard hydraulic modelling software program
ML	Megalitres
ML/day	Megalitres per day
OEH	Office of Environment & Heritage, a NSW state government body
Pluviograph	A rain gauge that measures rainfall at a sub-daily frequency
RORB	An industry standard hydrological modelling software program
RFFE	Regional Flood Frequency Estimation
SRTM	Shuttle Radar Topography Mission
TA	Technical Advisor
XP-RAFTS	An industry standard hydrological modelling software program

Executive Summary

This report documents the hydrological model calibration process and results for the Narrabri to North Star project. The hydrological models provide inflows to the hydraulic models which are used to assess track and formation flood immunity, performance of cross drainage (culverts and bridges) under existing and proposed upgrade conditions and impacts of the project on flooding in adjacent land. These inflows are a key design input and therefore gaining an acceptable calibration of the models that produce the inflows is a critical project activity.

The project area is located within three major river basins – the Namoi, Gwydir and Border Rivers Basins. Majority of the project area is located within the Gwydir Basin, and crosses the major river floodplains of the Mehi and Gwydir Rivers. The project area also crosses Gil Gil and Croppa Creeks which are part of the Macintyre River regional system. The section of the project crossing the Gwydir-Mehi system is the only section that is subject to regional scale flooding, with all other sections subject to local catchment flooding processes.

The existing rail corridor crosses over twenty significant creek channels and numerous additional unnamed minor creeks and tributaries. There are approximately 230 cross drainage sub-catchments associated with the existing rail corridor.

The project area is covered by seven separate hydrological models, one of which was originally developed in 2016 on behalf of Moree Plains Shire Council and subsequently used for Phase 2 of the project (Feasibility Design Stage), with the other six developed for this study to inform the detailed design of the project. The new models were all developed using the updated national hydrology guideline Australian Rainfall & Runoff 2016 (ARR2016). The list of hydrological models covering the project area from south to north is as follows:

- NAMOI01: new model developed for this study; covers project chainage 575km to 590km; includes minor local creek catchments north of Narrabri;
- GWYDIR01: new model developed for this study; covers project chainage 590km to 619km; includes Ten Mile, Bulldog and Mehan Creeks;
- GWYDIR02: new model developed for this study; covers project chainage 619km to 666km; includes Gurley, Tycannah and Halls Creeks;
- Moree Regional Flood Model: existing model originally developed for Council and used in Phase 2; covers project chainage 666km to 682km; includes the regional Gwydir-Mehi system and its floodplain around the township of Moree; part of Tycannah Creek and Marshalls Pond Creek;
- GWYDIR03: new model developed for this study; covers project chainage 682km to 709km; includes minor local creek catchments east of the Camurra hairpin;
- MACINTYRE01: new model developed for this study; covers project chainage 709km to 727km; includes Gil Gil Creek; and
- MACINTYRE02: new model developed for this study; covers project chainage 727km to 760.46km; includes Croppa Creek.

The Moree Regional Flood Model was successfully calibrated for Council's 2016 Flood Study and has recently been adopted as the floodplain management planning model for Moree and the surrounding area. The model was updated for Phase 2 of the project and used to inform the Phase 2 design. The Phase 2 version of the model was subject to several checks and found to be suitable for detailed design. No further calibration of this model was attempted for this study but this document includes a review of the original calibration process to confirm the model's accuracy and suitability for use in design. The review concluded that the regional model is suitable for use in detailed design, with an update of the local catchment model to Australian Rainfall & Runoff 2016 required to inform the flood impact assessment for the project.

Due to a lack of hydrological data in the region around the project area, it has only been possible to calibrate two of the new hydrological models, as follows:

- The GWDIR02 model, which includes Tycannah Creek, a tributary of the Mehi River. This model was calibrated to the Tycannah Creek flow gauge record at Horseshoe Lagoon (gauge 418032), which is located approximately 20km upstream of the rail alignment. The model was calibrated to events that occurred in July 1998, January 2004, November 2011 and February 2012; and
- The MACINTYRE02 model, which includes Croppa Creek, a tributary of Whalan Creek and part of the Macintyre River system. This model was calibrated to the Croppa Creek flow gauge record at Tulloona Bore (gauge 416034), which is located approximately 15km downstream of the alignment. The model was calibrated to events that occurred in May 1983, July 1984 and April 1988.

The Tycannah Creek model calibration achieved good results for the July 1998 and January 2004 events. The calibration to the 2011 and 2012 events, which are the highest on record, required unusually high parameter values to fit recorded flows and was therefore less successful, likely due to issues with the flow record not estimating the total floodplain flow for high order events. The Croppa Creek model calibration achieved good results for all events analysed. Overall, the calibration was considered reasonable.

The hydrological model parameters determined from the calibration process were applied to the other four new hydrological models developed for the project. All six models were then run for several theoretical (or ‘design’) floods, and the peak design flows at the existing rail cross drainage sub-catchments were compared to flow estimates derived at Phase 2, and those derived from the Regional Flood Frequency Estimation (RFFE) method.

The new model peak flow estimates did not achieve close agreement with the Phase 2 estimates but this was to be expected given that the Phase 2 hydrology was based on the old hydrology guideline Australian Rainfall & Runoff 1987 and the method used for Phase 2 did not include detailed modelling. However, the comparison between the two datasets was within a reasonable range with no extreme outliers. The new model peak flow estimates agreed reasonably well with the RFFE method, taking into consideration the limitations of the RFFE method, and were also found to produce more conservative (i.e. higher) flow estimates than the Moree Regional Flood Model in areas where both models overlap.

Based on the calibration results and the checks against the RFFE and Moree Regional Flood Model flow estimates, the hydrological models developed for the project are considered to produce credible design flow estimates and to be a suitable basis for the flooding assessment and design of cross drainage structures.

1 Introduction

1.1 Purpose of Report

The purpose of this report is to document the calibration of the hydrological models developed for the Narrabri to North Star project. The hydrological models are rainfall runoff models that are used to define inflow boundary conditions s to the hydraulic models, which are then used for the following purposes:

- To assess the flood immunity of the rail corridor under existing and future conditions;
- To assess the performance of existing and upgraded cross drainage culverts and bridges; and
- To assess impacts of the project on flooding in adjacent land.

The inflows defined by the hydrological models are therefore critical design inputs and it is vital that the models are calibrated against flow measurements recorded in the catchments traversed by the project.

1.2 Objectives of the Calibration Process

The objectives of the hydrological model calibration process are as follows:

- To obtain a reasonable fit to reliable flow data recorded at gauges within the subject catchments for several historical flood events; and
- To define a reasonable and credible set of model parameters from the calibration event modelling that can be used as model input parameters for design events, i.e. the theoretical events, that are used to assess flood immunity, culvert performance and flood impacts.

1.3 Report Status

This report is currently at draft status and subject to review by ARTC and the Technical Advisor (TA).

1.4 Related Documents

This report should be read in conjunction with the following related documents:

- Technical and Approvals Consultancy Services: Narrabri to North Star, Hydrological and Hydraulic Investigation Plan (3-0001-260-IHY-00-PL-0001): Preliminary investigation plan that sets out the methodology for the project flood study; and
- Technical and Approvals Consultancy Services: Narrabri to North Star, Flood Study Report for Separable Portion 1 (3-0001-260-IHY-00-RP-0002): Main flood report for the project that documents the design event modelling process, cross drainage design and flood impact assessment.

2 Methodology

2.1 Overview

The hydrological modelling methodology was developed following a review of available flood modelling and hydrological data for the region as documented in the Hydrological and Hydraulic Investigation Plan (HHIP) (document reference 3-0001-260-IHY-00-PL-0001) and Section 3 of this report.

The project crosses the major regional floodplain of the Gwydir-Mehi system, over a distance of approximately 14km north and east of Moree. In this regional floodplain, the project interacts with an extensive flooding process involving conveyance of very large flow volumes and sensitive adjacent land uses including the town of Moree, individual dwellings and farm buildings, irrigation areas and associated infrastructure, the Newell Highway and local roads.

Outside of the regional floodplain the project crosses local floodplain systems where flood extents are less significant and the adjacent land is either less sensitive or has a lesser concentration of sensitive elements. Two separate methodologies were developed that considered the availability of previous flood studies and data:

- Gwydir-Mehi regional floodplain: This area was assessed using an existing flood model developed on behalf of Moree Plains Shire Council, which is the established flood risk management tool for the area and is used by Council in assessing and planning future development within the floodplain, including future flood risk management schemes for Moree and environs; and
- Other local floodplain systems: In these areas, some hydrological modelling had been undertaken by Office of Environment & Heritage (OEH), but no hydraulic models had been previously developed. The approach taken in these areas was to develop new hydrological and hydraulic models of the catchments that drain through the rail corridor, including accurate sub-catchment breakdowns to represent all cross drainage structures under the rail line, in order to accurately assess flood behaviour and hydraulic conditions around the existing and upgraded cross drainage structures.

The following sections provide an overview of the two separate methodologies that were adopted.

2.1.1 Gwydir-Mehi Regional Floodplain

The section of the project that crosses the Gwydir-Mehi regional floodplain (from 666 km to 682 km) is covered by an existing flood model that was completed in December 2016 for Moree Plains Shire Council's flood study for Moree and environs (WRM, January 2017). The flood model shows that the regional floodplain approaches 9km in total width through this area for the 1, 2 and 5% AEP events, reducing to approximately 5km in width for the 10% AEP event. For large events such as the 1% AEP, overtopping of the rail occurs over approximately 6km. The existing rail has lower than 10% AEP flood immunity at some locations within this section.

The flood study and model were developed by WRM on behalf of Council and the model was made up of the following components:

- Hydrological model compromising:
  - Main regional river inflows to the upstream boundary of the model based on Flood Frequency Analysis (FFA) of the gauge record at Gravesend. The FFA method used in the study was based on the latest Australian Rainfall & Runoff 2016 (ARR2016) hydrology guideline. This represents the primary inflow to the model domain; and
  - Local catchment inflows to the modelled area from smaller tributaries and creek systems that enter the Gwydir-Mehi floodplain downstream of Gravesend. These inflows were modelled using the XP-RAFTS hydrological modelling software program and based on the now superseded Australian Rainfall & Runoff 1987 (ARR1987) guideline. These inflows are significant but only constitute approximately 15% of the peak flow that passes through Moree; and

- Hydraulic model developed in the MIKE FLOOD hydraulic modelling software program. This includes a combined one-dimensional (1D) model representation of hydraulic control structures (such as culverts), and a two-dimensional (2D) model representation of overland flow over floodplains. The flexible mesh version of the software was used to develop the model, which allows the model resolution to be increased in areas of considerable interest, or hydraulic complexity.

For Phase 2 of the project, the model was reviewed in detail and updated to generate a baseline model, which included refinements of the model topographic representation along the rail corridor, and inclusion of additional rail and Newell Highway culverts identified from the Phase 2 survey that were not represented in the original model (Jacobs 2017). The results from the updated model were checked against those from the original model received from Council, and found to be in close agreement. The updated baseline model was then modified to test upgrade design options for Phase 2.

Given that a detailed flood model has been established for this section of the project, the hydrological modelling methodology for this section included a detailed review of the existing hydrological model to confirm that it is suitable for use in detailed design. The review included the following steps:

- Review of the reported model calibration to confirm that a robust calibration process was undertaken and reliable results were obtained;
- Check of the FFA of the Gravesend gauge record to confirm that the main inflow to the model is defined correctly; and
- Check of the changes to flows predicted by the local catchment XP-RAFTS model when updated to the latest ARR2016 guideline, to determine if this update should be adopted, given that the model is currently being used to define local council floodplain management and planning policies.

2.1.2 Local Catchment Floodplains

The remaining 164.5 km (89%) of the project lying outside the Gwydir-Mehi regional floodplain interacts with local catchment flooding processes, rather than regional processes, although some of these local catchments are relatively large and extend up to 1,000 km^2. Within the local catchments, there are approximately 57 main catchments that are made up of smaller sub-catchments that combine in big flood events, and approximately 166 sub-catchments draining to individual cross drainage structures. Some of these catchments have been modelled by the Office of Environment & Heritage (OEH) as part of their flood management planning program, including:

- Thalaba Creek in the Gwydir River Basin;
- Tycannah Creek in the Gwydir River Basin; and
- Gil Gil Creek in the Border Rivers Basin.

The HHIP proposed to develop these models to cover the project area; however, review of the models identified that they were too coarse and did not adequately cover all sub-catchments draining to the project area to be suitable for development for use in the detailed design. Instead, new hydrological models were developed using the RORB software program (as also used by OEH in the modelling studies listed above and by GHD in Phase 2 of the project) and calibrated where data allowed. Table 2.1 below lists the new hydrological models developed for the project. An overview of the railway corridor, main watercourses and the RORB model extents is provided in Figure 2.1 and Figure 2.2. Maps showing the layouts of the RORB models are provided in Appendix A. Maps showing the extent of the N2NS RORB models compared to the extent of the OEH RORB models are provided in Appendix B.

The RORB models are used to provide inflow hydrographs to the hydraulic models developed for the rail corridor and the land upstream and downstream of the corridor. The hydraulic models route the RORB model inflows through the topography around the corridor and through the rail cross drainage structures, taking account of the hydraulic connectivity of the cross drainage sub-catchments and how they combine and influence each other's hydraulic behaviour during high flow events.

Table 2.1 New hydrological models developed for the project

Model name	Project chainage	River basin	Significant creek systems included	Relevant OEH model
NAMOI01	575 to 590 km	Namoi	Minor local creeks only	-
GWYDIR01	590 to 619 km	Gwydir	Ten Mile Creek, Bulldog Creek, Mehan Creek	Thalaba Creek
GWYDIR02	619 to 666 km	Gwydir	Gurley Creek, Tycannah Creek	Tycannah Creek
GWYDIR03	682 to 709 km	Gwydir	Minor local creeks only	-
MACINTYRE01	709 to 727 km	Border Rivers	Gil Gil Creek	Gil Gil Creek
MACINTYRE02	727 to 760.46 km	Border Rivers	Croppa Creek	-

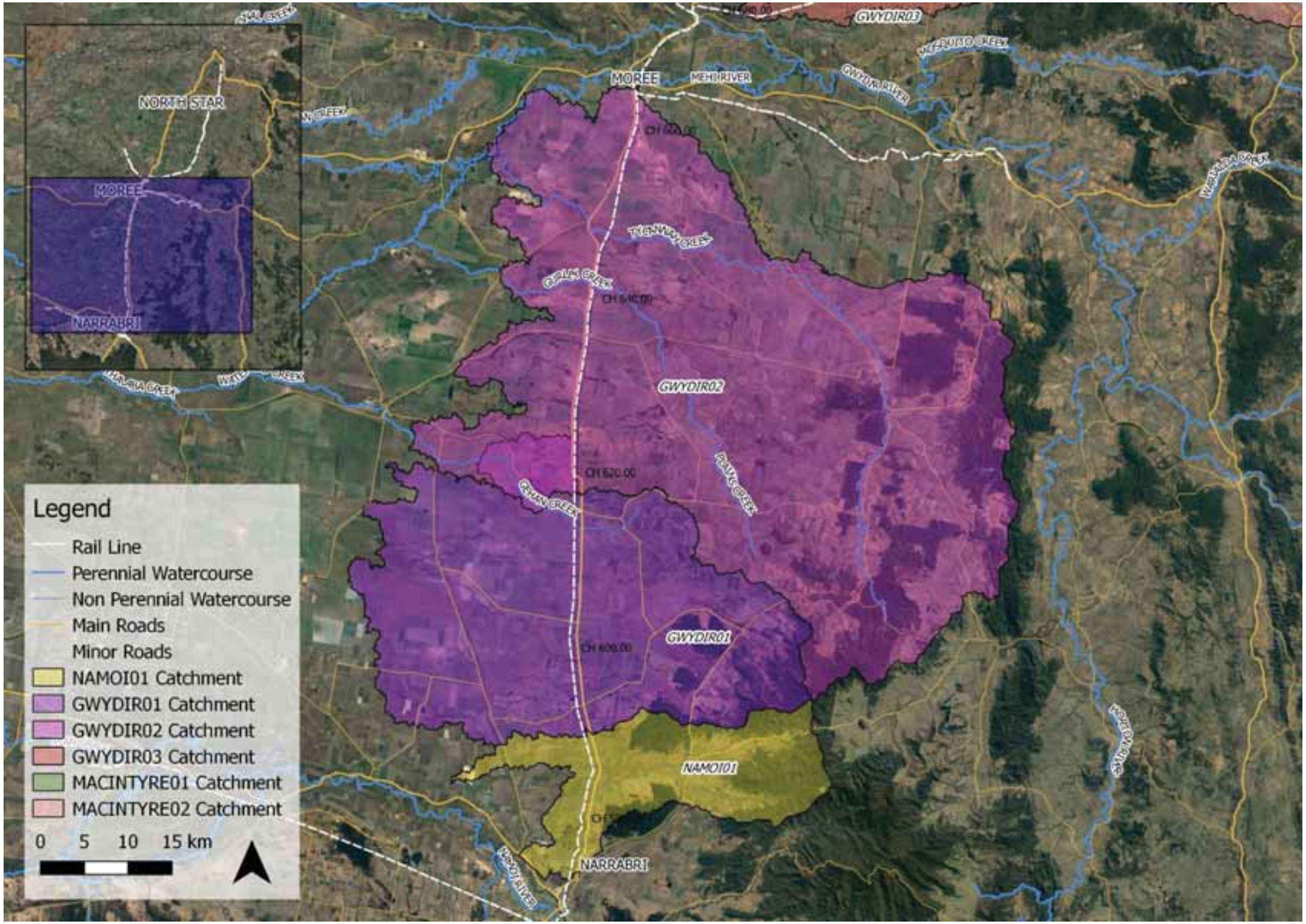

Figure 2.1 N2NS study area and extent of NAMOI01, GWYDIR01 and GWYDIR02 flood models

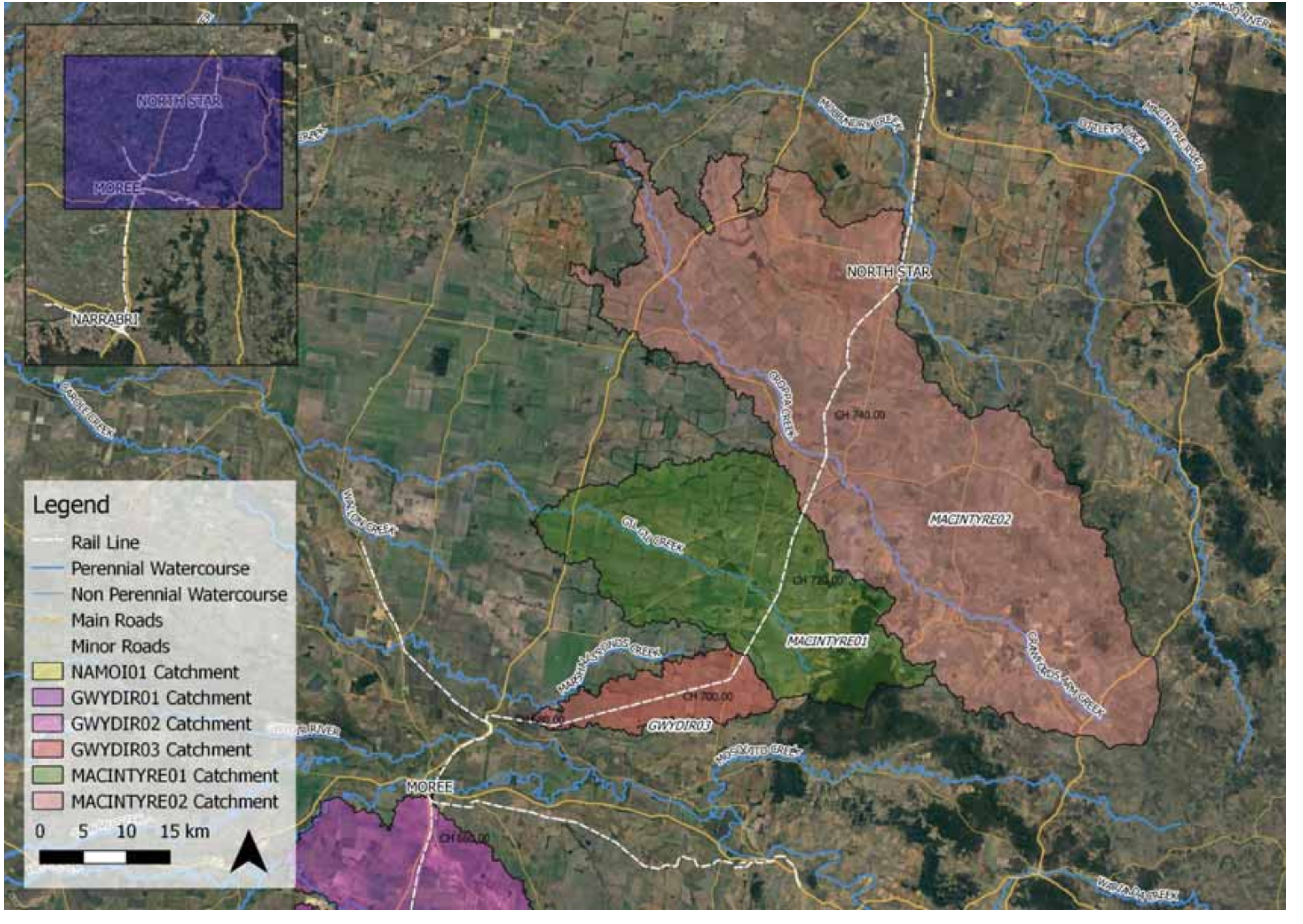

Figure 2.2 N2NS study area and extent of GWYDIR03, MACINTYRE01 and MACINTYRE02 flood models

The hydrological modelling process for the local catchment models is shown below in Figure 2.3.

Identify streamflow gauge records suitable for calibration and select previous storm and flood events for calibration

⇩

Build hydrological models of catchments draining to the gauge locations

⇩

Calibrate the hydrological models to at least 3 previous flood events and adopt the model parameters that achieve the best fit across the selected calibration events and use these parameters to establish the design event hydrological models

⇩

Apply the ARR2016 Regional Flood Frequency Estimation (RFFE) method at the rail cross drainage sub-catchments and check the design model flow estimates against RFFE predictions for the same range of design events.
Undertake a further hydrological model check by comparing design model flow estimates to design flows reported at Phase 2 of the project

Figure 2.3 Hydrological modelling process for local catchment floodplains

2.2 Detailed Methodologies

2.2.1 Flood Frequency Estimation

2.2.1.1 At Site Flood Frequency Analysis

As part of the review of the Moree Regional Flood Model, a check of the FFA was undertaken for the flow gauge record on the Gwydir River at Gravesend Road Bridge. The approach followed the method for at site FFA given in ARR2016 Book 3, Chapter 2, and involved the following steps:

- Obtain the annual maxima flow series from the data provided in the Flood Study Report (WRM, January 2017); and
- Use FLIKE (an extreme value analysis software package that calculates the probability of flood events based on historical records) to fit the data to a probability function and fit method. The Log Pearson III (LP III) probability model and Bayesian fit method were adopted in the analysis, as per the Flood Study Report.

The flow records at the gauge sites used for calibration of the RORB models (see Section 2.2.2 below) were found to be unsuitable for at site FFA due to issues with the data records. These issues are discussed further in Section 3.

2.2.1.2 Regional Flood Frequency Estimation Method

The RFFE model provided by the Australian Rainfall & Runoff Data Hub (http://rffe.arr-software.org/) was used to obtain peak flow estimates for all cross drainage sub-catchments draining to the existing rail corridor.

2.2.2 RORB Model Calibration

Based on the data available for model calibration (see Section 3), the following two RORB hydrological models were calibrated for the project:

1. GWYDIR02 model: Calibrated to the Tycannah Creek flow gauge record at Horseshoe Lagoon (gauge 418032), which is located approximately 20 km upstream of the existing rail corridor.
2. MACINTYRE02 model: Calibrated to the Croppa Creek low gauge record at Tulloona Bore (gauge 416034), which is located approximately 15 km downstream of the existing rail corridor.

The methodology involved the following steps:

- Review flow records to identify potential previous events for calibration;
- Review rainfall records to identify completeness and reliability of rainfall data relating to above events;
- Confirm calibration events based on review of available rainfall data and determine most reliable gauges to define calibration rainfall dataset and any adjustment factors required to gauges used to fill data gaps; and
- Run the models with the calibration rainfall datasets and vary the following RORB hydrological model parameters until a reasonable fit to the observed flow hydrographs is obtained:
  - Flood routing parameter k_c. k_c is the principal parameter within RORB and is a function of catchment area, catchment non-linearity and discharge;
  - Initial Loss (IL). IL is the initial rainfall lost at the start of an event to represent initial catchment wetting when no runoff is produced. IL varies by soil type and is specified in the range of 5 to 35 mm; and
  - Continuing Loss (CL). CL is the continuing loss rate that occurs during an event due to infiltration once the catchment is saturated. CL also varies by soil type and is specified in the range of 0.5 to 25 mm/hour.

2.2.2.1 GWYDIR02 Model Calibration at Tycannah Creek

Refer to Appendix A for an overview of the GWYDIR02 model extent, sub-catchment breakdown and calibration gauge location. The historic flood events selected for calibration are listed below in Table 2.2.

Table 2.2 Calibration events for GWYDIR02 model

Gauge	Calibration events
Tycannah Creek at Horseshow Lagoon (gauge 418032)	July 1998 January 2004 November 2011 February 2012

2.2.2.2 MACINTYRE02 Model Calibration at Croppa Creek

Refer to Appendix A for an overview of the MACINTYRE02 model extent, sub-catchment breakdown and calibration gauge location. The historic flood events selected for calibration are listed below in Table 2.3.

Table 2.3 Calibration events for MACINTYRE02 model

Gauge	Calibration events
Croppa Creek at Tulloona Bore (gauge 416034)	May 1983 July 1984 April 1988

3 Data Review

This section describes the flow gauge and rainfall data available for use in the calibration process.

3.1 Flow Gauge Data

3.1.1 Overview

A preliminary review of the flow gauge datasets identified the following historical events that had the potential for calibration:

- February 1976;
- May 1983;
- July 1984;
- April 1988;
- July 1998;
- January 2004;
- November 2011; and
- February 2012.

These events were selected because they were considered significant events for the Gwydir Valley. The February 1955 flood event was a significant event across most of NSW but there is insufficient gauge and rainfall data to use this event for calibration. The February 1976 flood event is a key event that OEH use for floodplain planning purposes, however, there were very few active gauges that recorded this event in the catchments surrounding the project area.

The above dates were selected for an in-depth review of the flow gauge and rainfall datasets within the subject catchments to identify events suitable for calibration.

3.1.2 Namoi River Basin 419

The project corridor crosses minor local catchments on the northern boundary of the Namoi River Basin north of Narrabri which are covered by the new NAMOI01 model developed for this project. There are no flow gauges with these local catchments. There are gauges on the main Namoi River through Narrabri but these flow records monitoring the main regional river flow are not suitable for use in calibrating the local catchment models.

3.1.3 Gwydir River Basin 418

3.1.3.1 Overview

Refer to Figure 3.1 for an overview of flow gauges within the Gwydir River Basin around the project area and Table 3.1 for a list of the available flow gauge records. Several of the gauges listed were used to calibrate the Moree Regional Flood Model, as noted in Table 3.1.

Tycannah Creek lies within the Gydir River Basin and joins the Mehi River downstream of Moree. This creek is included in the new GWYDIR02 model developed for this project. Flow records for this creek are discussed in the next section.

Thalaba Creek also lies within the Gwydir River Basin and joins the Barwon River downstream of Collarnebri. The upper reaches of this creek are included in the new GWYDIR01 model. However, there are no flow gauges available for this creek system.

Figure 3.1 Gwydir River Basin stream gauges near the project area

Table 3.1 List of available Gwydir River Basin flow gauges (excluding Tycannah Creek)

Station	Station Number	Opened (closed)	Comment
Gwydir River at Pallamallawa	418001	1891	Used in Moree Regional Flood Model calibration
Mehi River at Moree	418002	1937	Used in Moree Regional Flood Model calibration
Gwydir River at Yarraman Bridge	418004	1928	Used in Moree Regional Flood Model calibration
Carole Creek at D/S Regulator (Bells Crossing)	418011	1939	
Gwydir River at Gravesend Road Bridge	418013	1936	Used in Moree Regional Flood Model calibration
Gwydir River D/S Boolooroo Weir (Carole Creek)	418036	1972	
Gwydir River at D/S Tareelaroi Weir	418042	1976	
Gwydir River at Tareelaroi Weir Storage Gauge	418043	1976	
Mehi River D/S Tareelaroi Regulator	418044	1976	
Gwydir River at Boolooroo Weir Storage Gauge	418051	1979	
Moomin Creek at Glendello	418060		
Gwydir River (South Arm) at D/D Tyreel Offtake Regulator	418063	1985	
Gwydir River at Tyreel Storage Gauge	418065	1987	
Carole Creek at Midkin Crossing (D/S Marshalls Ponds)	418086	2005	
Mehi River at Chinook	418087	2005	
Marshall Ponds at Newell Highway	418089	2005	
Slaughterhouse Creek at Biniguy	418092	2011	
Biniguy	418094	1955	

3.1.3.2 Tycannah Creek

OEH has developed a RORB model of the Tycannah Creek catchment and attempted to calibrate this model to flood events that occurred in November 2011 and February 2012. Tycannah Creek is one of the significant creek crossings located within the GWYDIR02 model developed for this project. A summary of the available flow gauge records within the Tycannah Creek catchment is provided in the following sections.

TYCANNAH CREEK AT HORSESHOE LAGOON (GAUGE 418032)

- Record commenced in 1971;
- Catchment area to the gauge is 1,037km^2 (from OEH, 2016);
- Records daily maximum water levels and heights and maximum hourly heights and flows;
- 260 gaugings were taken between 13 July 1971 and 28 July 2017;
- Gauge zero height is at 250.196 mAHD; and
- Significant events in the record are as follows:
  - 11 February 1976 – total daily volume measured 30,483 ML, no sub-daily measurements of water level are available for this event;
  - 28 July 1998 – maximum hourly height measured as 6.612m which converts to a maximum flow of 147m^3/s (12,700 ML/day);
  - 17 January 2004 – maximum hourly height measured as 7.194m which converts to a maximum flow of 221m^3/s (19,100 ML/day);
  - 26 November 2011 – maximum instantaneous flow of 448m^3/s was recorded (38,776 ML/day); and
  - 2 February 2012 – maximum instantaneous flow of 559m^3/s was recorded (48,306 ML/day).

Observed flows for the July 1998, January 2004, November 2011 and February 2012 events are plotted in Figure 3.2 to Figure 3.5. Both flow rates and water level datasets are available from the WaterNSW streamflow database. Conversion of water levels to flow rates requires an additional calculation step as the rating curve is based on a logarithmic interpolation. To avoid this additional step, the flow rates directly downloaded from this database were used in current calibration and the rating curves were not used. Although in theory both methods are expected to yield identical results, the flow rates were believed to have higher accuracy due to the potential additional site knowledge or data that WaterNSW may have and which may have contributed to their method of estimating flows.

The following limitations associated with this flow record should be noted:

- The Moree and Environs Flood Study (WRM 2017) noted that the gauge is only rated to the in-channel flows and not the floodplain flows, and therefore the flow record would not be reliable for flow events with significant floodplain flow bypassing the main channel. Anecdotal evidence suggests that Tycannah Creek overflows to White Swamp during large flood events, and this occurred during the February 2012 flood (WRM 2017). Therefore the site is considered unsuitable for FFA as this would produce unreliable peak flow estimates for large events where significant bypassing of the channel occurs. This is discussed further in Section 4.2.1; and
- The quality of historical records in the July 1998 flood event were poor and the available water level data are estimates (commentary code 51), while the quality of January 2004 water level data is expected to be good (commentary code 32).

WEAH WAA CREEK AT TERRY HIE HIE (GAUGE 418028)

Data for this gauge could not be extracted from Pinneena. Details of the gauge indicate it was abandoned in 1971 due to flood damage. It has therefore not been considered for calibration.

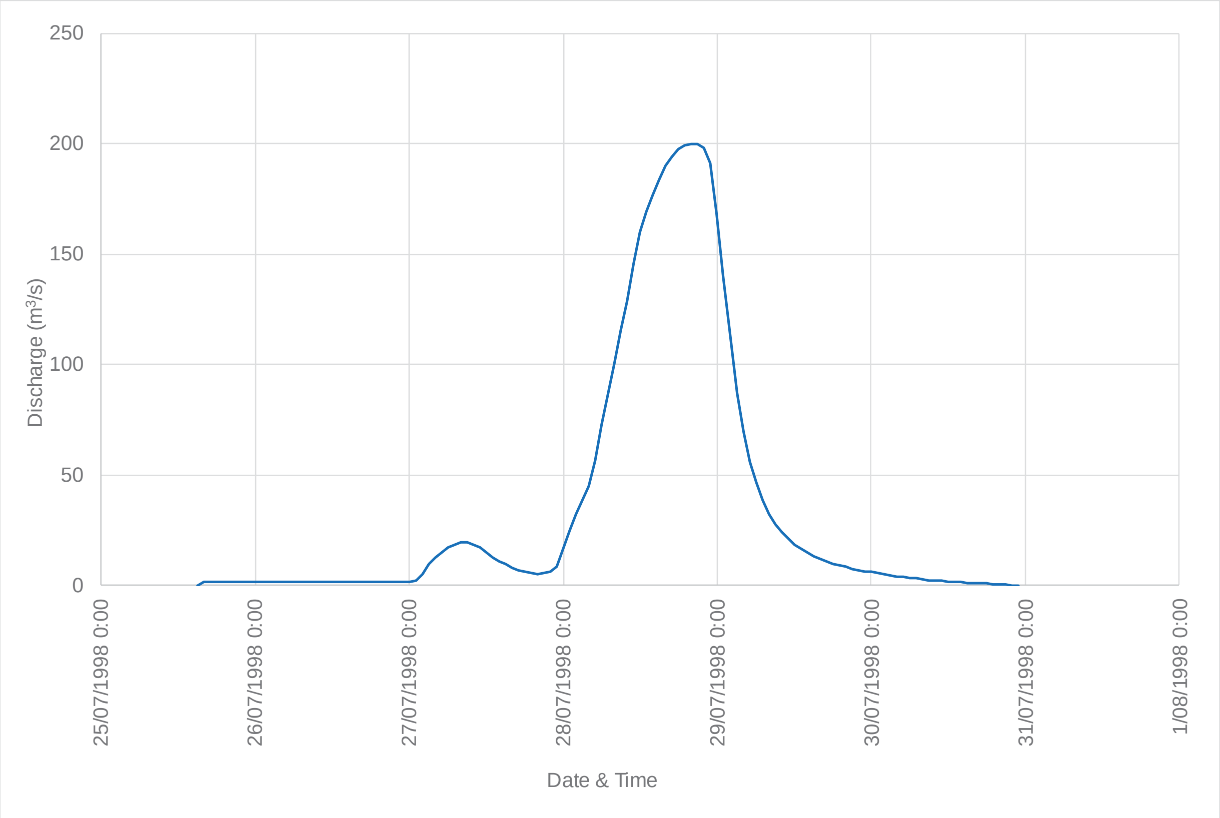

Figure 3.2 Observed flow for Tycannah Creek at Horseshoe Lagoon (418032) for July 1998 event

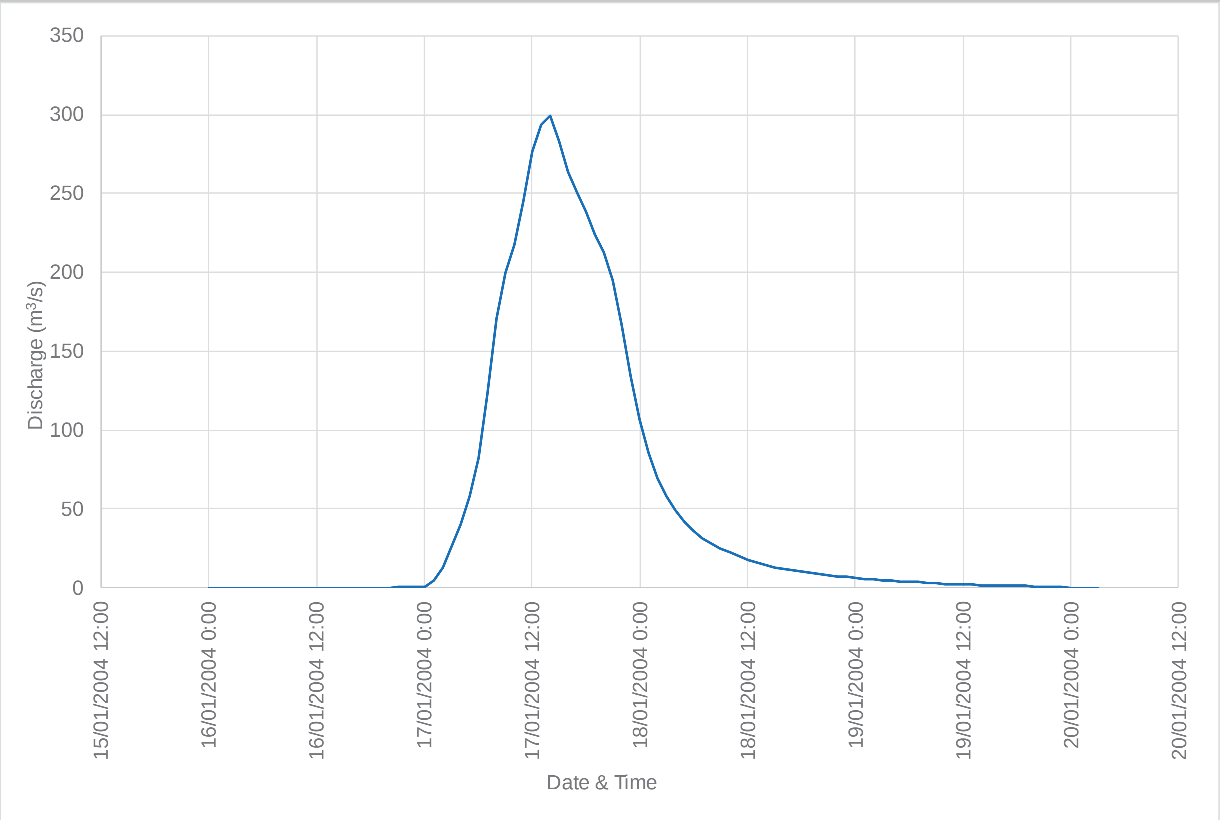

Figure 3.3 Observed flow for Tycannah Creek at Horseshoe Lagoon (418032) for January 2004 event

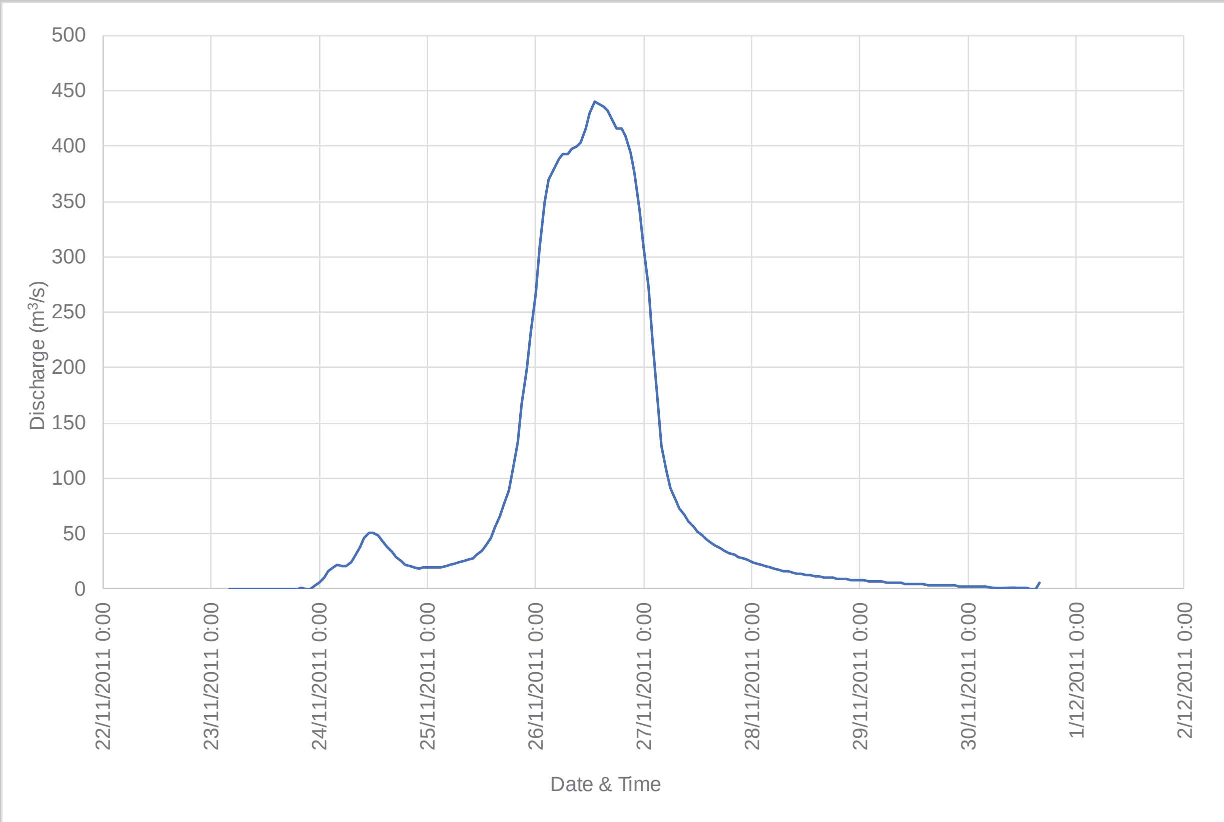

Figure 3.4 Observed flow for Tycannah Creek at Horseshoe Lagoon (418032) for November 2011 event

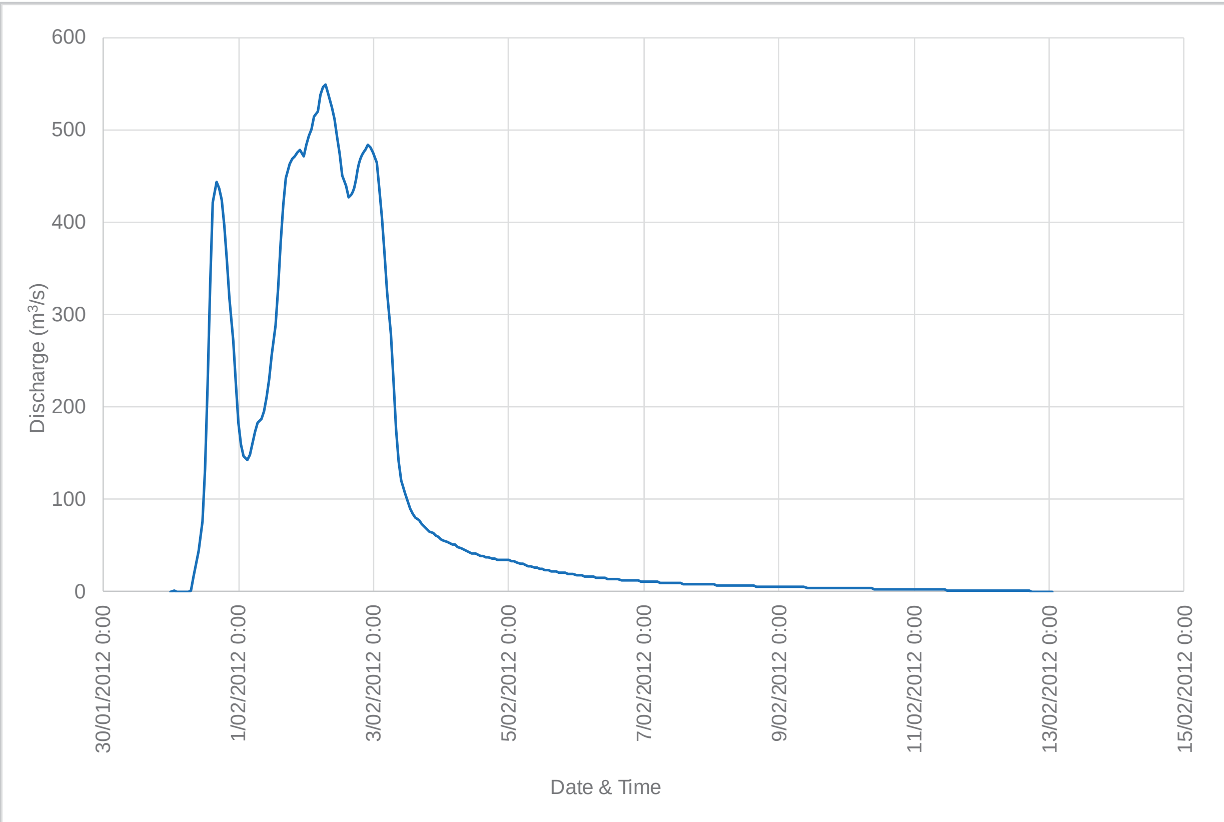

Figure 3.5 Observed flow for Tycannah Creek at Horseshoe Lagoon (418032) for February 2012 event

3.1.4 Border Rivers Basin 416

3.1.4.1 Overview

Refer to Figure 3.6 for an overview of flow gauges within the Border Rivers Basin around the project area.

Gil Gil Creek lies within the Border Rivers Basin and joins the Boomi River approximately 100km west of the existing rail corridor. The upper reaches of this creek are included in the new MACINTYRE01 model developed for this project. Flow records for this creek are discussed in the next section.

Croppa Creek also lies within the Border Rivers Basin and is a tributary of Whalan Creek which joins the Boomi River approximately 100km west of the existing rail corridor. The upper and middle reaches of Croppa Creek are included in the new MACINTYRE02 model. Flow records for this creek are discussed in Section 3.1.4.3.

Figure 3.6 Border Rivers River Basin stream gauges near the project area

3.1.4.2 Gil Gil Creek at Boolataroo (Gauge 416054)

OEH has developed a RORB model of the Gil Gil Creek catchment. Flows are recorded at the Gil Gil Creek at Boolataroo gauge 416054; however, for reasons not explained, OEH did not attempt to calibrate this model (OEH, 2016). The following summarises key information from the gauge record:

- Record commenced in 1996;
- Catchment area to the gauge is not provided but appears to be > 1,000km^2;
- Records daily maximum water levels and heights and instantaneous heights and flows;
- Records indicate that the site may be affected by upstream dams or backwater effects;
- 119 gaugings were taken between 8 March 1994 and 15 November 2017; and
- Significant events in the record are as follows:
  - 31 July 1998 – maximum flow recorded was 169.4m^3/s (14,639 ML/day);
  - 19 January 2004 – maximum flow recorded was 239.6m^3/s (20,700 ML/day);
  - 27 November 2011 – maximum flow recorded was 352.1m^3/s (30,418 ML/day); and
  - 4 February 2012 – maximum flow recorded was 290.7m^3/s (25115 ML/day).

Observed flows (based on the conversion of level to flow using the gauge rating) for the July 1998, January 2004, November 2011 and February 2012 events are plotted in Figure 3.7 to Figure 3.10.

While the upper reaches of Gil Gil Creek are crossed by the alignment and included in the new MACINTYRE01 model, the gauge is located over 70km downstream of the existing rail corridor and therefore flow characteristics at the gauge, which are noted to display the influences of upstream dams and backwater effects, are unlikely to represent those in the upper reaches that are crossed by the rail corridor. Rather than calibrate the MACINTYRE01 model to this gauge, the model parameters were adopted based on those from the calibrated OEH ROB model and those obtained from the calibration of the MACINTYRE02 model to the Croppa Creek gauge (see next section).

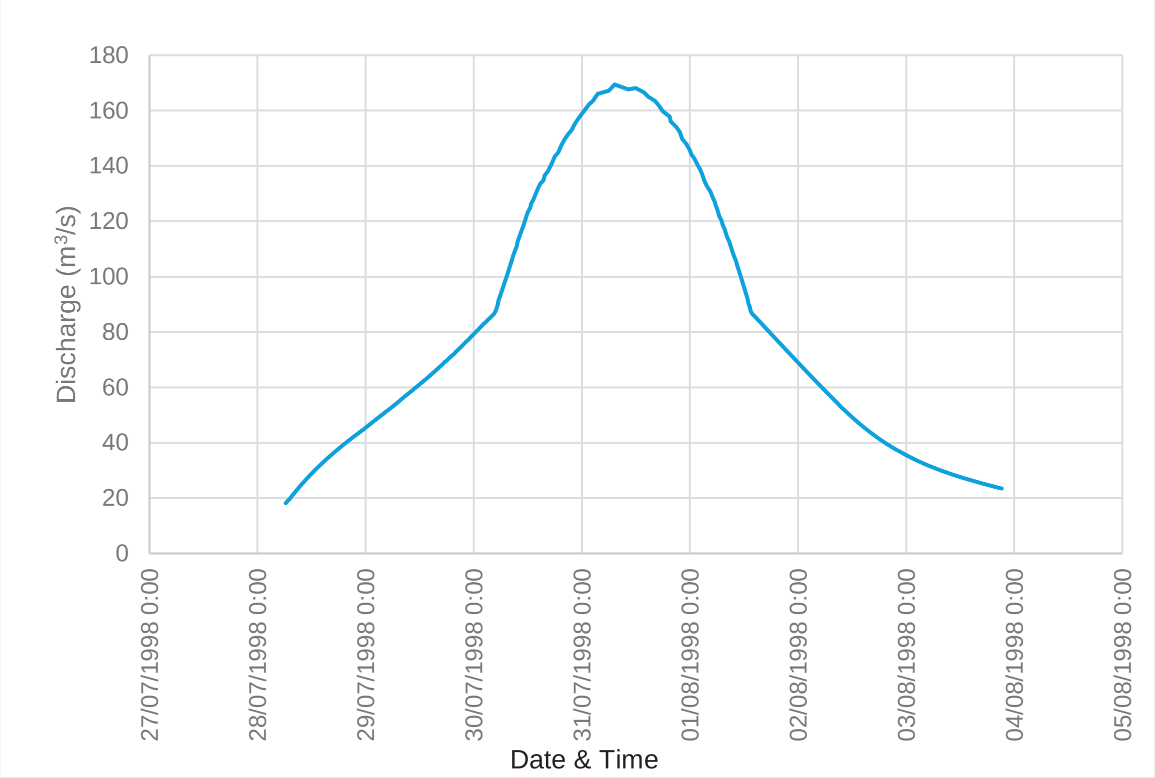

Figure 3.7 Observed flow for Gil Gil Creek at Boolataroo (416054) for July 1998 event

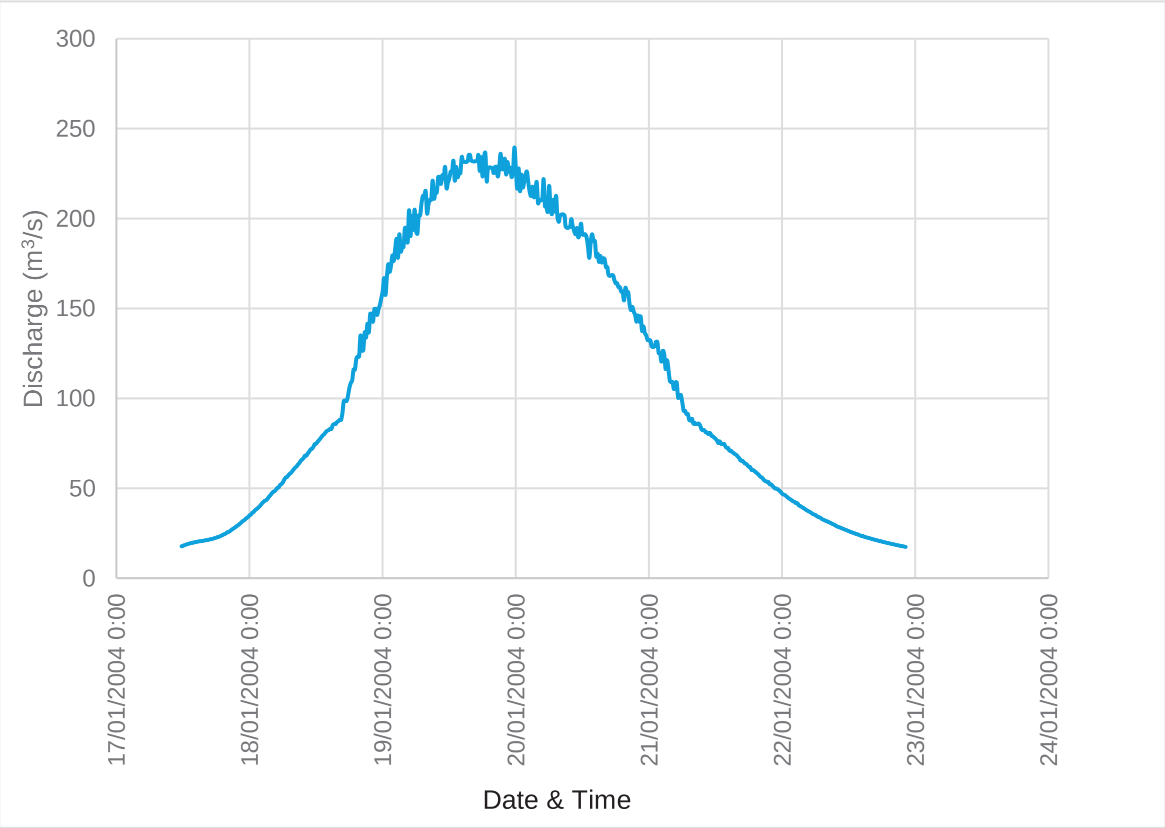

Figure 3.8 Observed flow for Gil Gil Creek at Boolataroo (416054) for January 2004 event

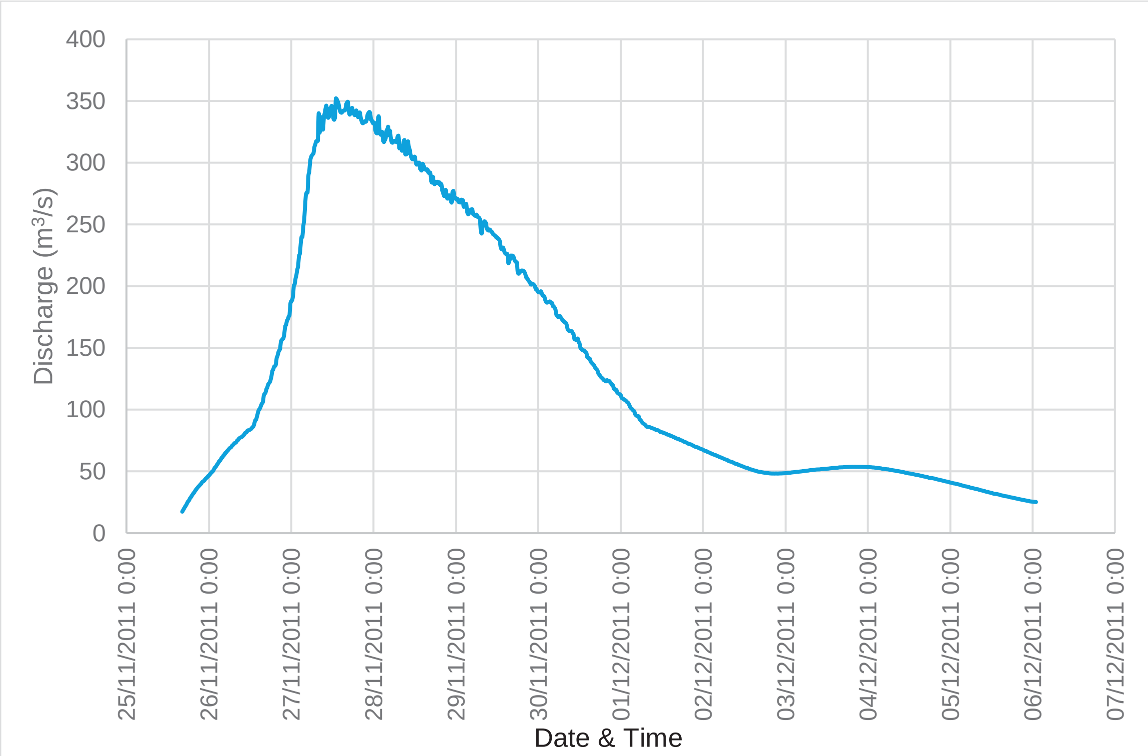

Figure 3.9 Observed flow for Gil Gil Creek at Boolataroo (416054) for November 2011 event

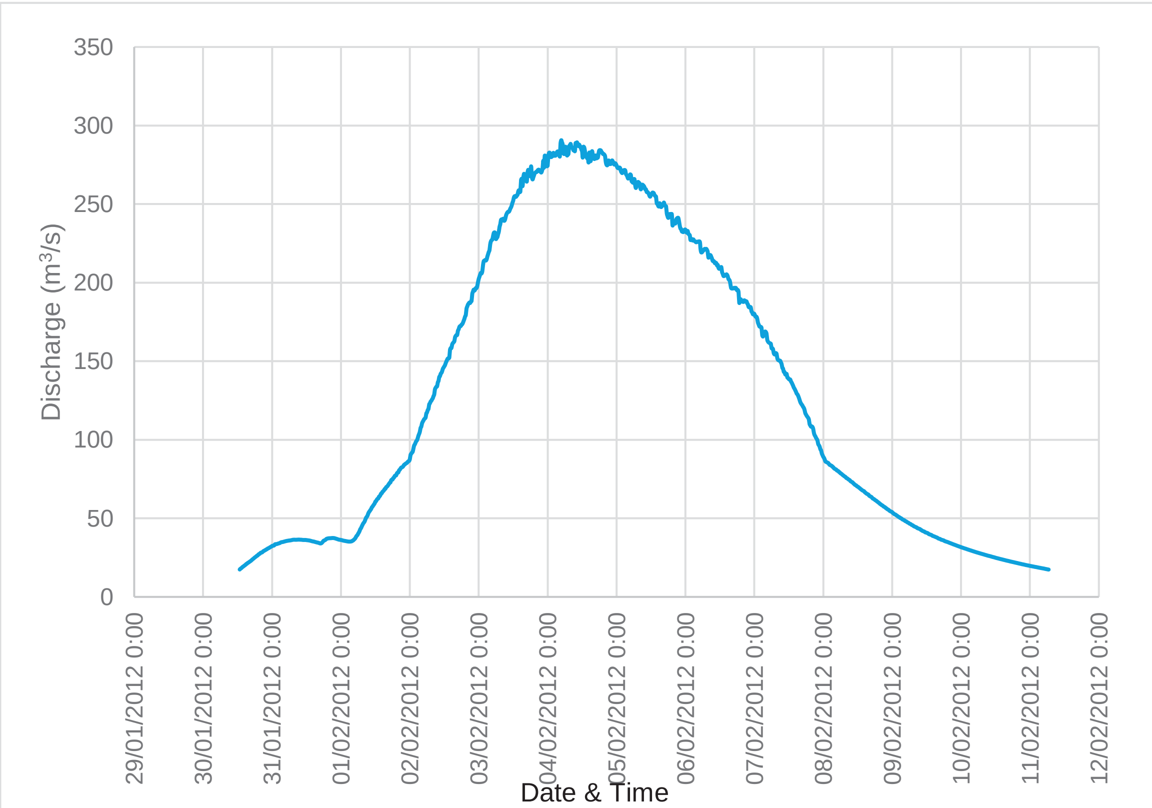

Figure 3.10 Observed flow for Gil Gil Creek at Boolataroo (416054) for February 2012 event

3.1.4.3 Croppa Creek at Tulloona Bore (Gauge 416034)

The existing rail corridor crosses the middle reaches of the Croppa Creek system, which are included in the MACINTYRE02 model. The Croppa Creek at Tulloona Bore gauge 416034 is located approximately 15km downstream of the rail corridor and was used to calibrate the MACINTYRE02 model. The following summarises key information from the gauge record:

- Record commenced in 1972 and closed in 1989;
- Catchment area to the gauge is 1,280km^2;
- Records hourly water level values;
- No data is available for the previous events discussed, i.e. February 1976, July 1998, January 2004, November 2011 and February 2012; and
- Significant events in the record are as follows:
  - 3 May 1983 – a peak water level of 5.7m was recorded which has been estimated to equal a discharge of 220m^3/s;
  - 29 July 1984 – a peak water level of 6.179m was recorded which has been estimated to equal a discharge of 266m^3/s; and
  - 4 April 1988 - a peak water level of 4.595m was recorded which has been estimated to equal a discharge of 110m^3/s.

The following limitations of the gauge record are noted:

- As the site only has a 17-year period of record, it is not considered suitable for FFA;
- Data is not available for the full length of operation of the gauge. Significant periods of no data occur in the record; and
- A rating table of the available data is provided with the gauge information. This rating is limited and does not allow precise flow estimation for some of the recorded water levels. A rating relationship has been estimated and used to generate the hourly flow series from the water level data.

Observed flows for the May 1983, July 1984 and April 1988 events are plotted in Figure 3.11 to Figure 3.13. The recorded peak water level is also included on the plots to show the shape of the hydrograph as the discharge estimates are uncertain due to the limitations in rating table information.

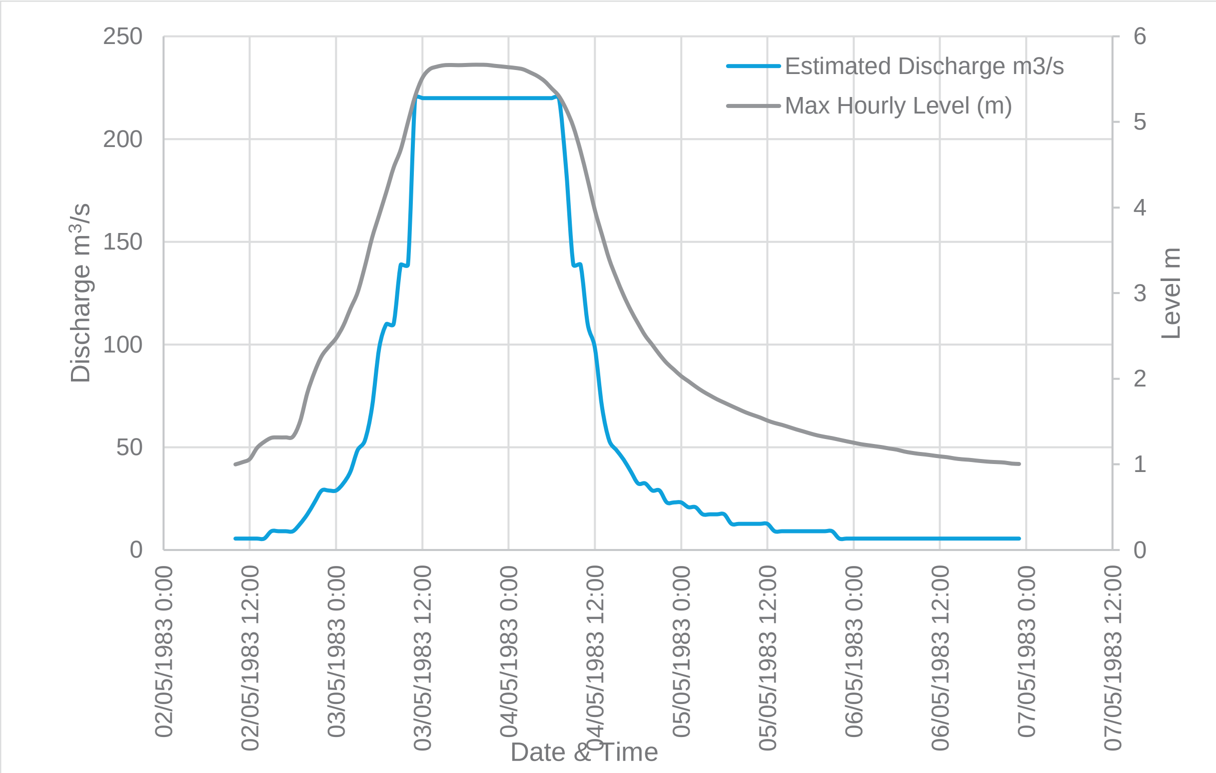

Figure 3.11 Observed flow for Croppa Creek at Tulloona Bore (416034) for May 1983 event

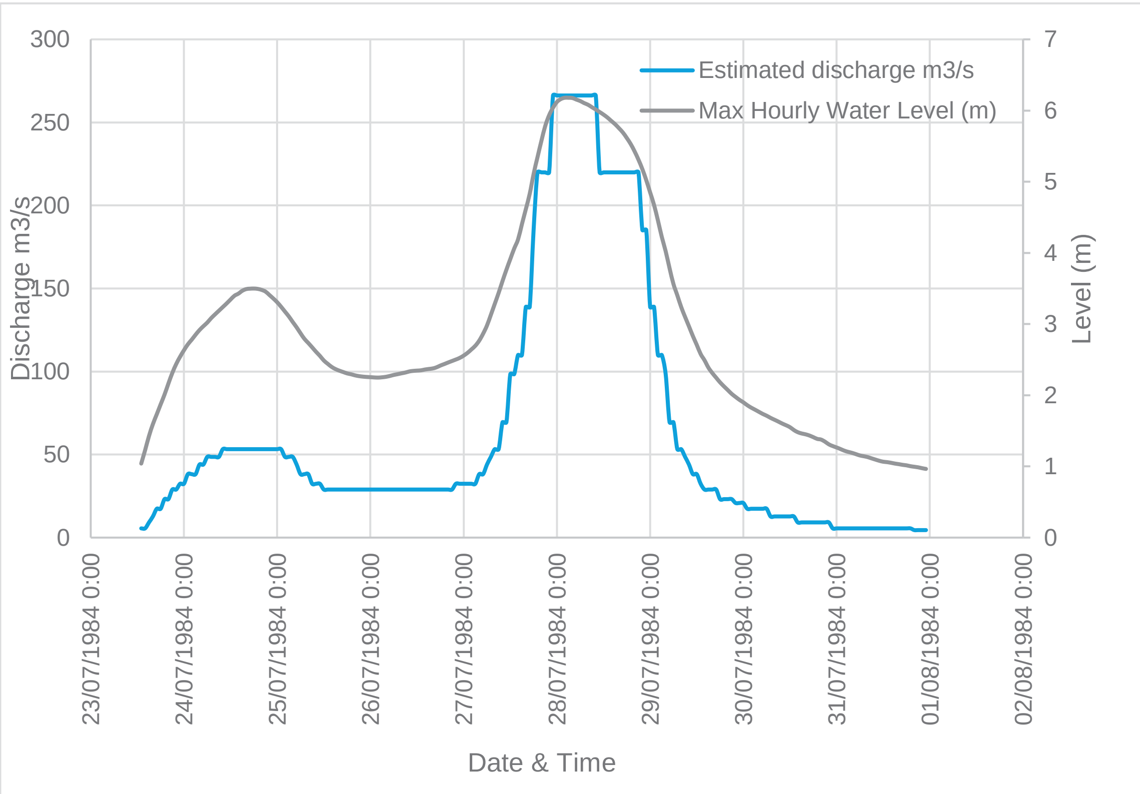

Figure 3.12 Observed flow for Croppa Creek at Tulloona Bore (416034) for July 1984 event

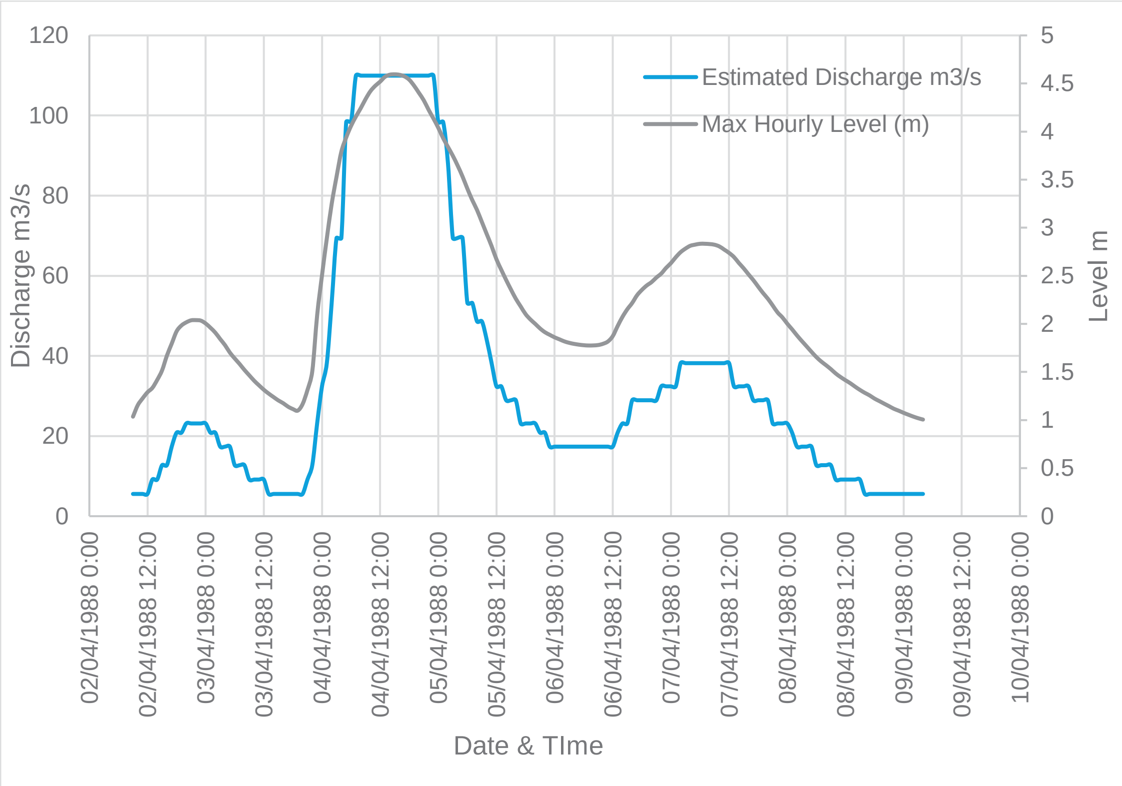

Figure 3.13 Observed flow for Croppa Creek at Tulloona Bore (416034) for April 1988 event

3.1.5 Conclusion

From a review of the gauge data and its quality, and the proximity of gauges to the project area, the gauges and events listed in Table 3.2 were selected for calibration of a sample of the new hydrological models developed for the project.

Table 3.2 Selected calibration events based on review of flow records

Gauge	Calibration events	Models for calibration
Gwydir Basin 418		
Tycannah Creek at Horseshoe Lagoon (gauge 418032)	July 1998 January 2004 November 2011 February 2012	GWYDIR02
Border Rivers 416		
Croppa Creek at Tulloona Bore (gauge 416034)	May 1983 July 1984 April 1988	MACINTYRE02

3.2 Rainfall Data

3.2.1 Overview

Figure 3.14 shows the locations of regional rain gauges around the project area. The rainfall data collected at these gauges was analysed for the calibration events identified in Table 3.2 above.

Rainfall depths for the events of interest are provided in Table 3.3. In addition to the calibration events, rainfalls for the large historic events of February 1955 and February 1976 are provided for comparison.

Daily recorded rainfall depths for the calibration events are presented in Figure 3.15 to Figure 3.21.

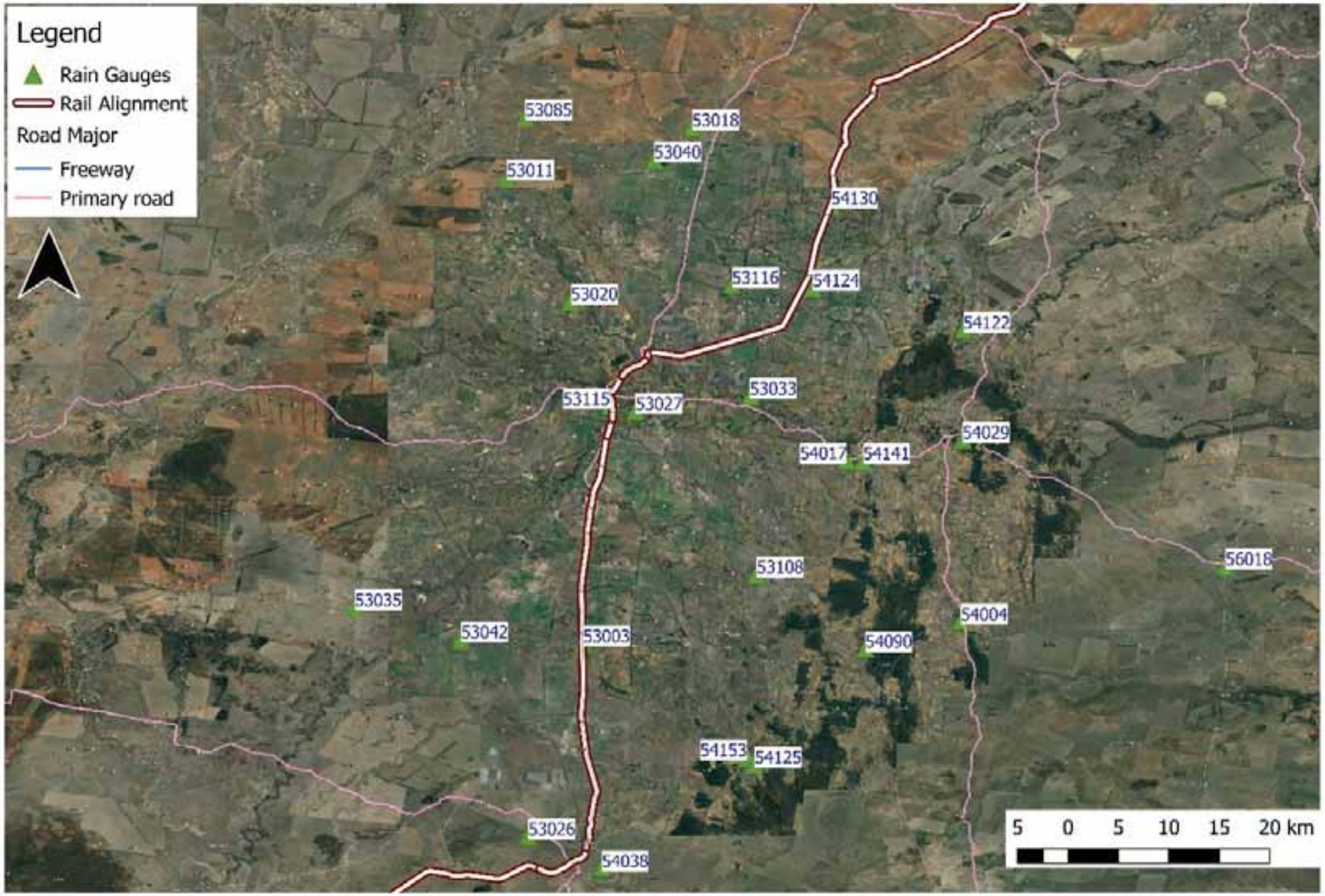

Figure 3.14 Rain gauges located around the project area

Table 3.3 Rainfall depths for events of interest

Station name	Station number	Type	Date of record	Rainfall Depths (mm)								
				Feb 55	Feb 76	May 83	Jul 84	Apr 88	Jul 98	Jan 04	Nov 11	Feb 12
Bellata Post Office	53003	Daily	1912	116.6	130.4	43	42.4	152.2	55.2	160.8	134.2	289.4
Garah Post Office	53011	Daily	1906	95.2	284	37	117	244	62	204.6	198.8	193.8
Croppa Creek (Krui Plains)	53018	Daily	1914	184.4	277.6	90	92.4	293	48.6	128.8	214	122.8
Ashley (Midkin)	53020	Daily	1906 (closed)	115.9	205							
Narrabri (Mollee)	53026	Daily	1926	87.4	51	38.6	85.8	136.4	44.6	136.8		270.4
Moree Post Office	53027	Daily	1879	107.4								
Pallamallawa	53033	Daily	1913	124.2	200.8	79.2	138.4	173.2	88	194	261.4	167.4
Bellata (Aberfeldie)	53035	Daily	1902	105.1	174.6	31.8	88.2	214.8	40.2	246.4	188.6	265
Moree Comparison	53048	Pluviograph	1960 (closed)				128.7	205.2				
Ashley (The Prairies)	53040	Daily	1928	151.4	237.6	94	104	282	43.5	155		
Garah (Ulinga)	53042	Daily	1936	97.8	286.2	0	0	218.8	47.8	169.8	210.2	172
Garah (Delvin)	53085	Daily	1967		292.2	42	105.4	209.2	47.4	155.6	262	153.4
Terry Hie Hie	53108	Daily	1981			58.6	75	110.3				
Moree Aero	53115	Pluviograph	1995						47.4	155.2	221.4	217.2
Moree (Oodnadatta)	53116	Daily	1994	246.4				0	65	206.2	288.4	138.4
Bingara Post Office	54004	Daily	1878	158.2	175.5	77.8	129.8	179.8	110	257.2	195.5	182.1
Gravesend PO	54017	Daily	1885	195.2	198.4	95.2	115.5	156.6	91.4	246.6	271.9	141.8
Warialda Post Office	54029	Daily	1878		253	119.6	125.4	198.4	69.8	208.5	210.6	111.4
Narrabri Airport	54038	Pluviograph	2001					0		191	173.2	29.4
Bingara (Pallal)	54090	Daily	1900					0			232	305
Pindari Dam	54104	Daily	1971				92.6	180.4				
Warialda	54122	Daily	1966		257.6	105.4	121.8	166.8	65.4	145.6	168.6	73
Crooble Station	54124	Daily	1967		217.8	98.4	107.8	217.4	51	147.2	214.4	103
Caroda (roseberry park)	54125	Daily	1900		144.2	60.4		157.4	96	197	219.9	276.4

Station name	Station number	Type	Date of record	Rainfall Depths (mm)								
				Feb 55	Feb 76	May 83	Jul 84	Apr 88	Jul 98	Jan 04	Nov 11	Feb 12
Croppa Creek (Belford Street)	54130	Daily	1966		250.5	127.4	103.2	213.6	61.8	121.7		97.8
Gwydir River (Gravesend Rd)	54141	Daily	1967	271.3				0		200.5	301.5	126
Caroda (paleroo)	54153	Daily	1967	195.4	164.3	55.5	168.5	110.4	96.8	207	195	312.6
Inverll research station	56018	Pluviograph	1969		171	109.6	121.9	165.8	89.4	148.6	170.8	63.8

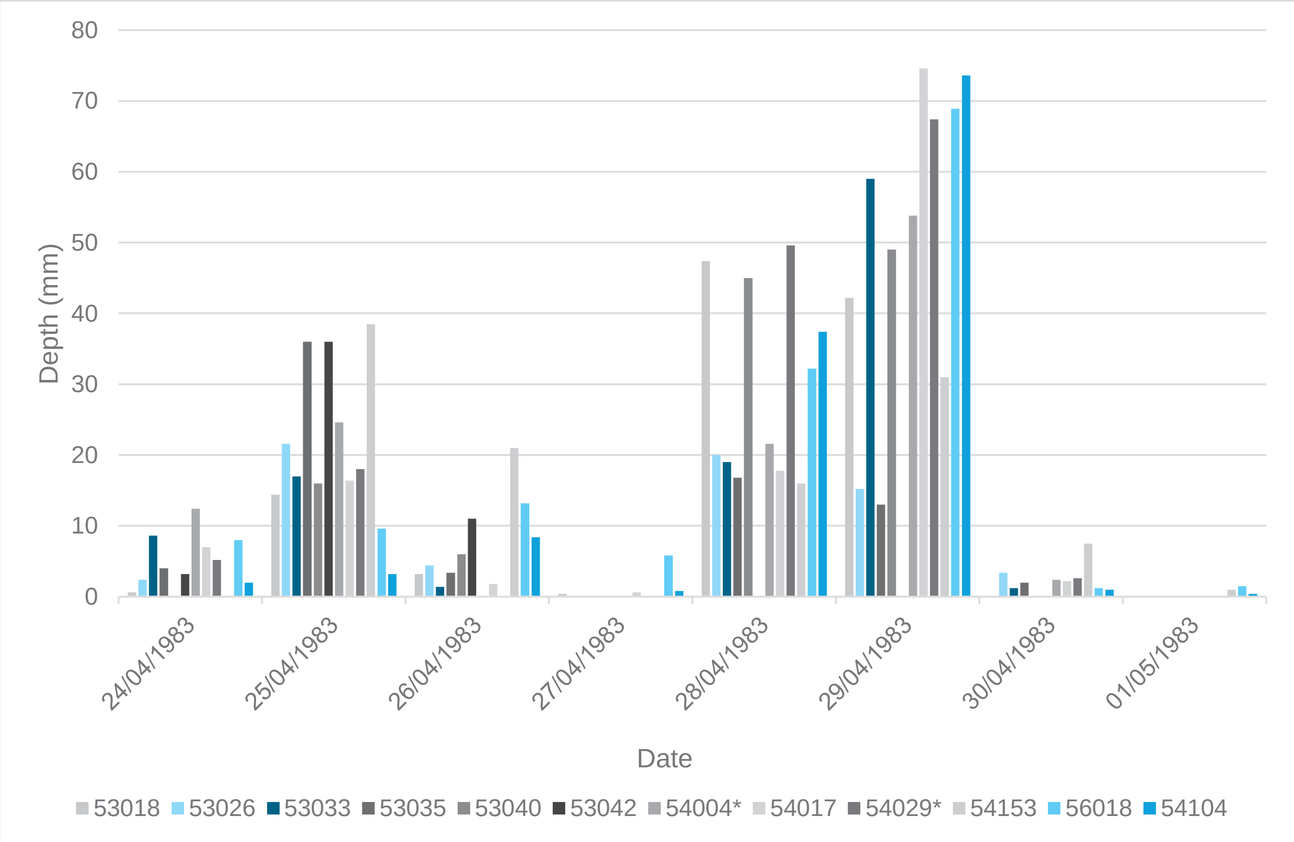

Figure 3.15 Daily rainfall depths for May 1983 event

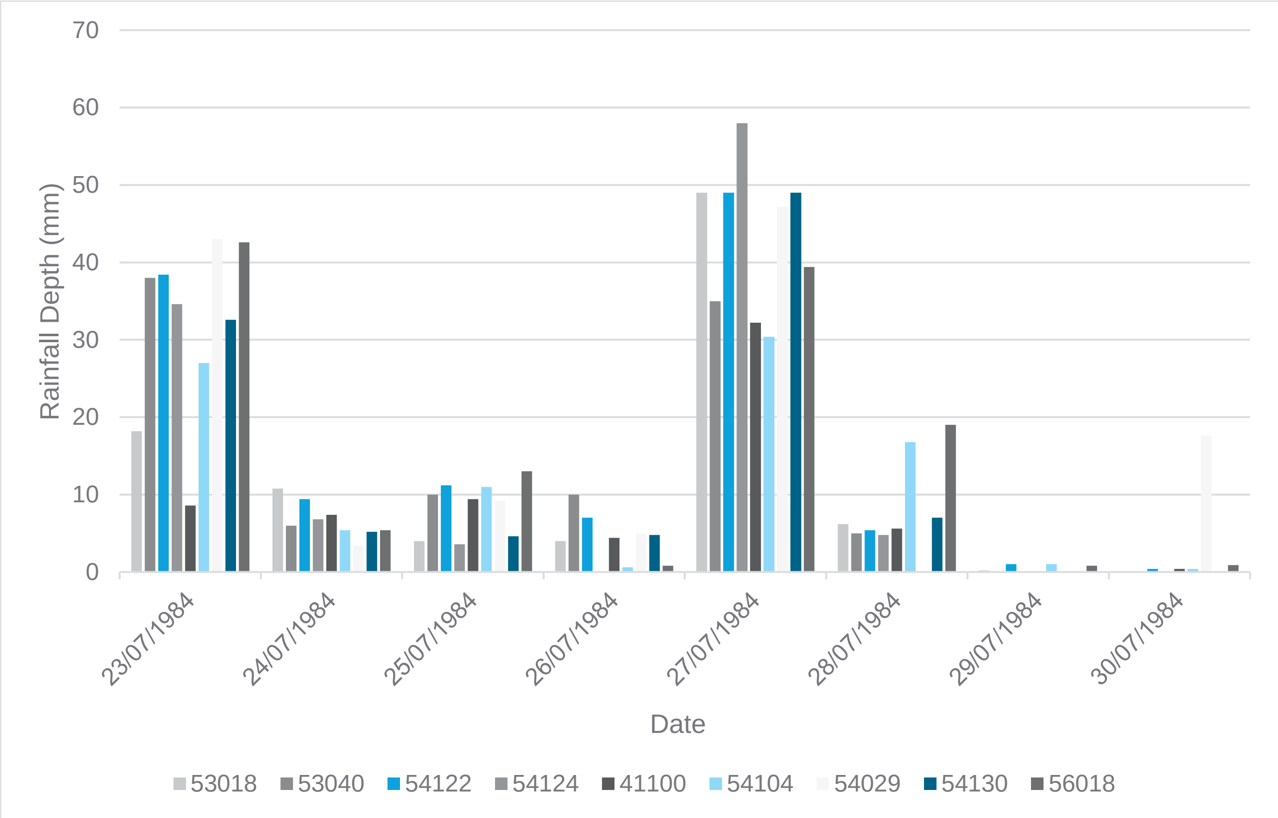

Figure 3.16 Daily rainfall depths for July 1984 event

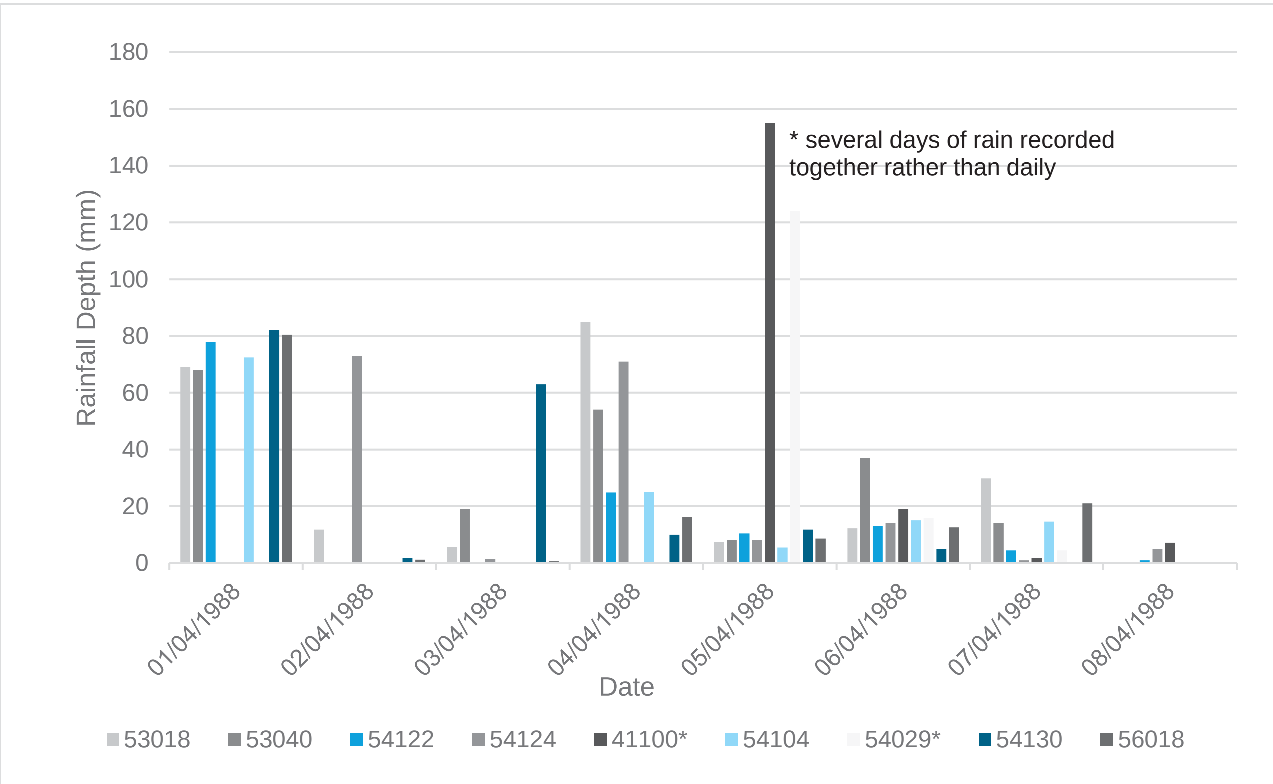

Figure 3.17 Daily rainfall depths for April 1988 event

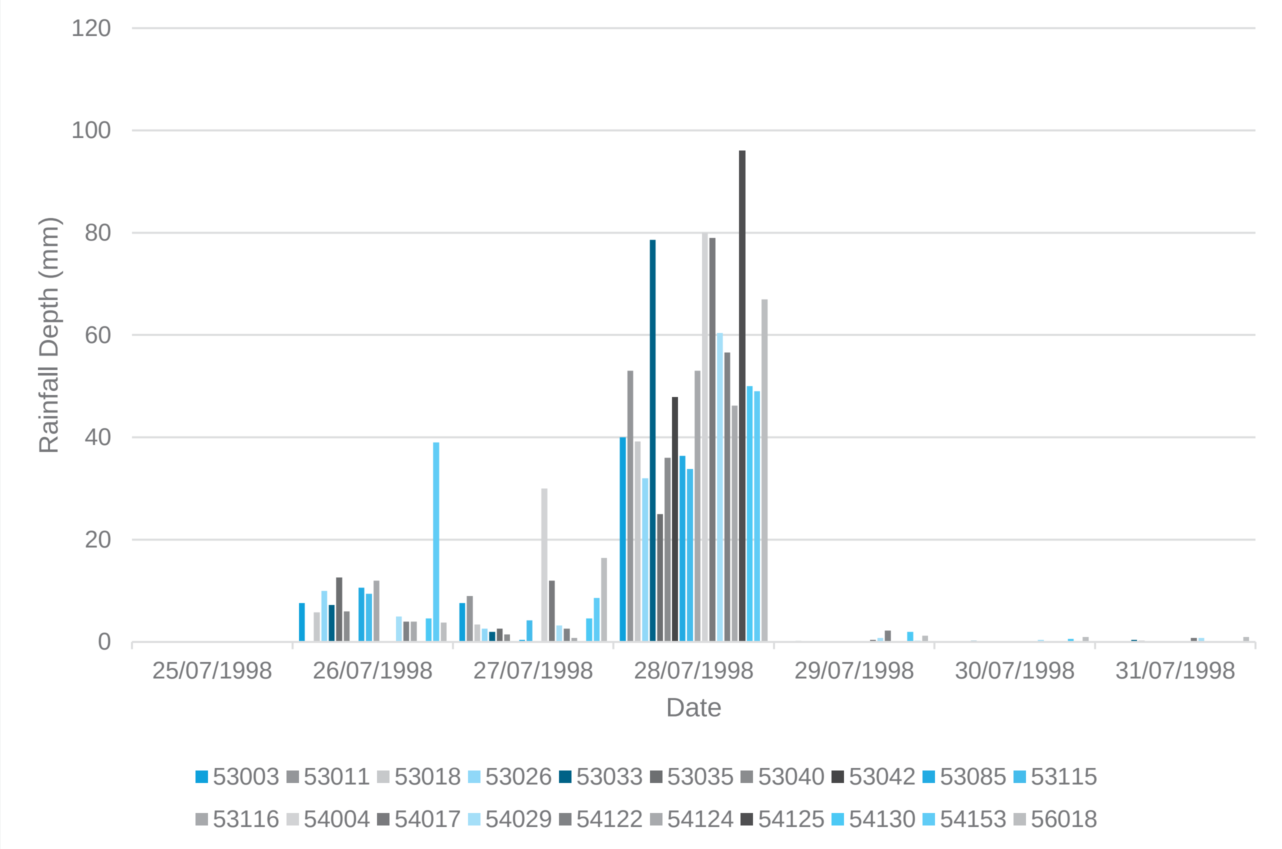

Figure 3.18 Daily rainfall depths for July 1998 event

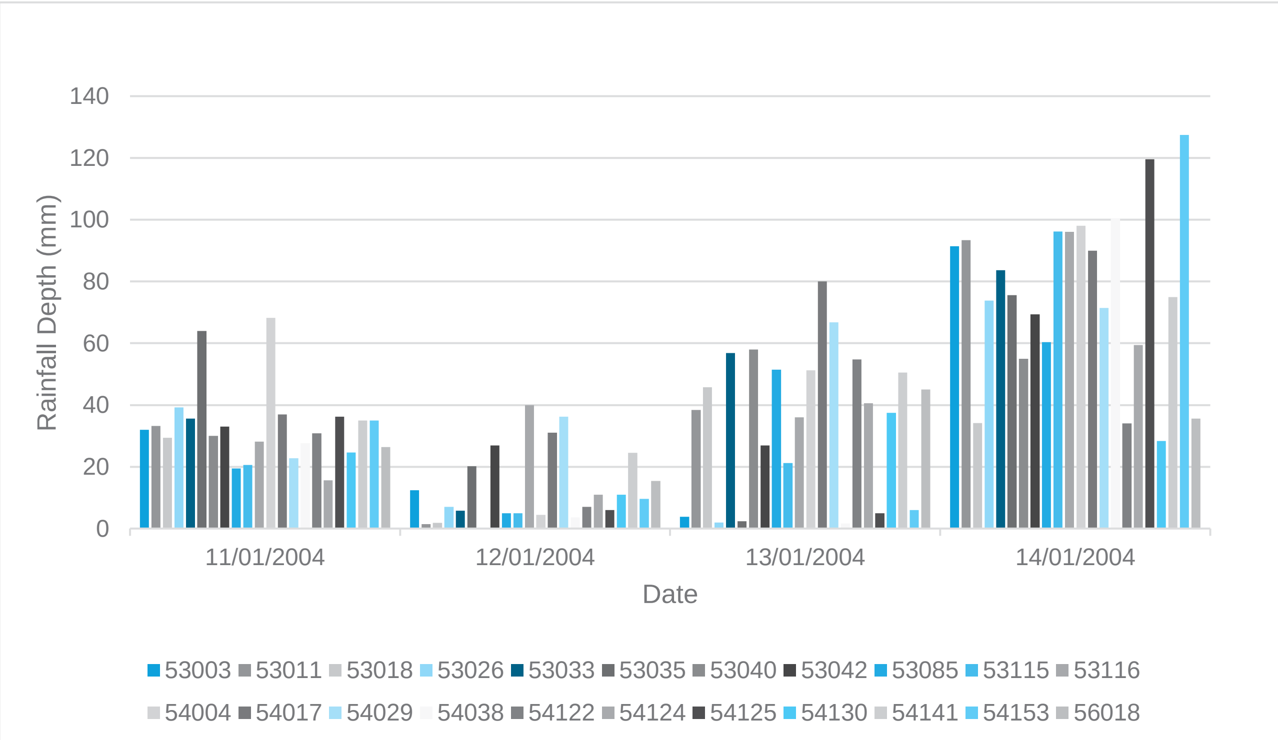

Figure 3.19 Daily rainfall depths for January 2004 event

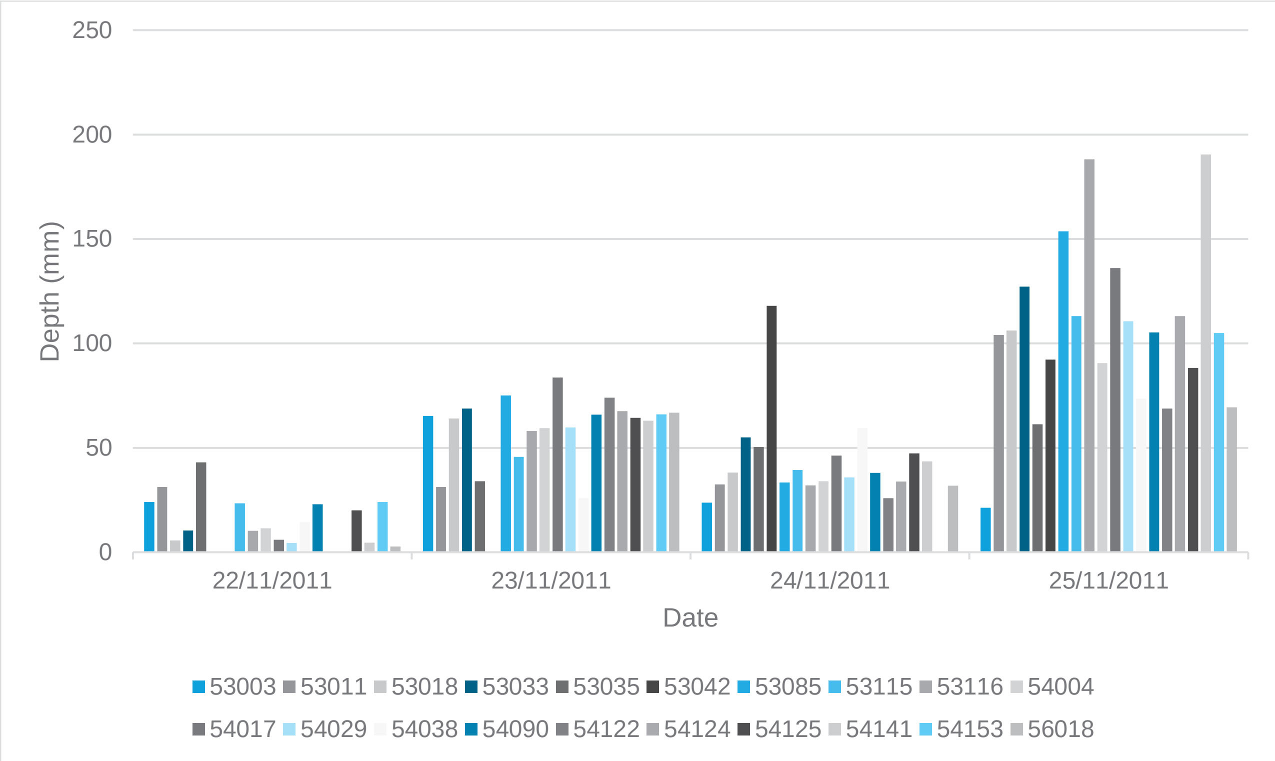

Figure 3.20 Daily rainfall depths for November 2011 event

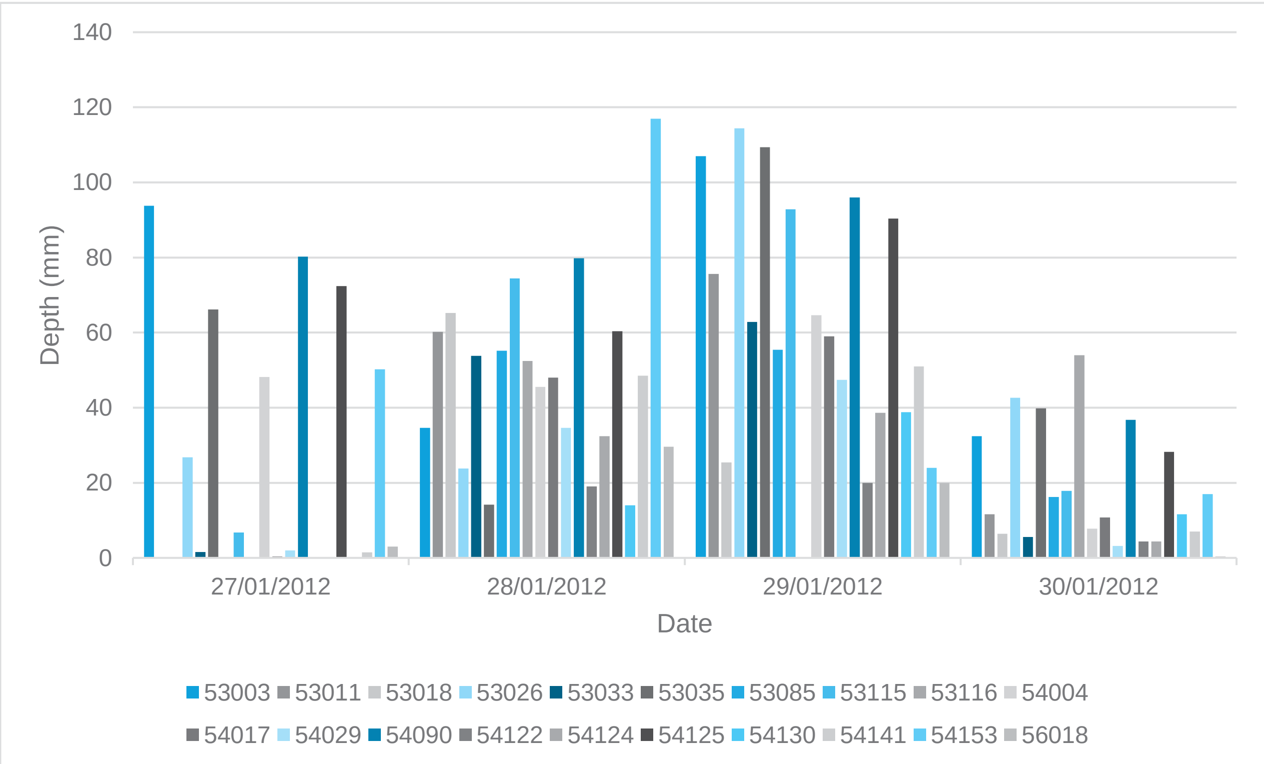

Figure 3.21 Daily rainfall depths for February 2012 event

3.2.2 Sub-Daily Rainfall Data

Sub-daily rainfall data is preferred when attempting to calibrate hydrological models to observed flow records. Sub-daily rainfall data is limited in the area surrounding the project. Table 3.3 identifies three active pluviograph stations; Moree Aero (53115), Narrabri Airport (54038) and Inverell Research Station (56018). Inverell is at the very eastern edge of the Gwydir catchment and is located on the catchment divide between the coastal plains and the western plains. However, the data has been reviewed given the limited data available closer to the project area.

Moree Aero (53115) only has minute rainfall depths for January 2004, November 2011 and February 2012. Narrabri Airport (54038) only has minute rainfall depths for November 2011 and it failed to record the February 2012 event. Inverell Research Station (56018) has data for November 2011 and February 2012. The Moree Comparison station (53048) has data for the May 1983, July 1984 and April 1988 events. Hourly data has been used to derive the calibration rainfall datasets. The hourly data for each of these events at the available stations is presented in Figure 3.22 to Figure 3.30 below.

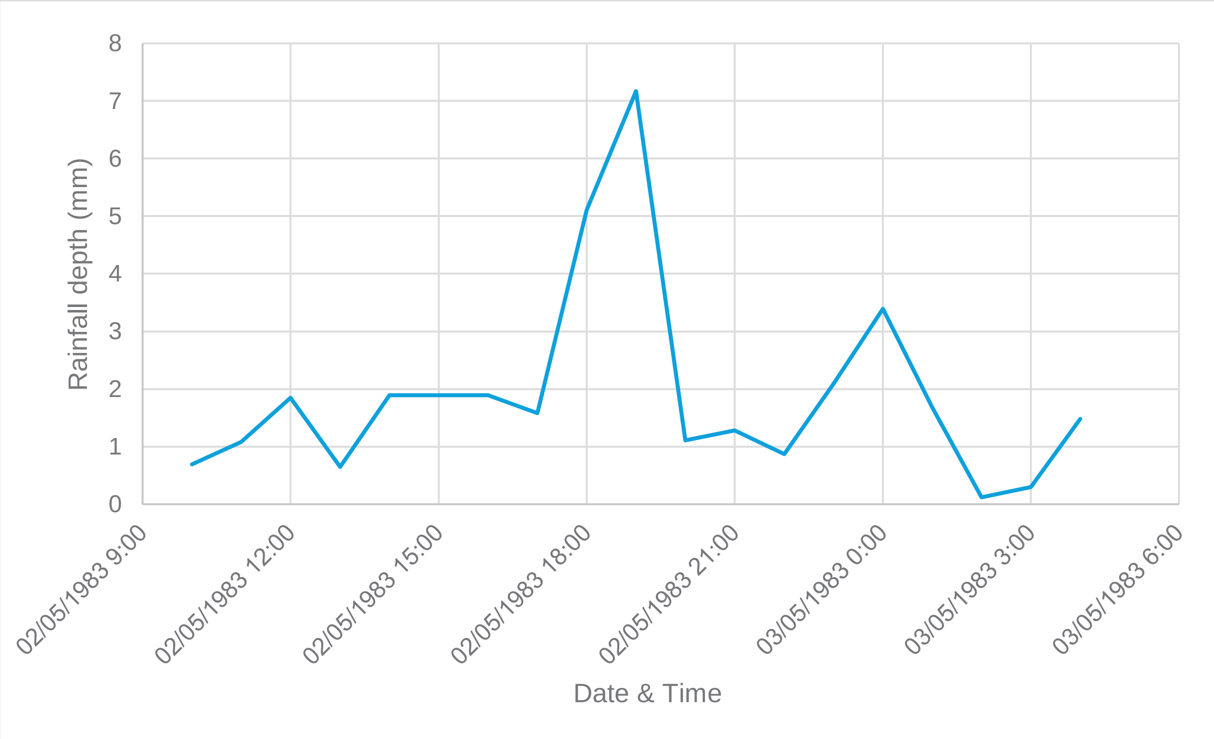

Figure 3.22 Hourly rainfall depths at Moree Comparison station (53048) for May 1983 event

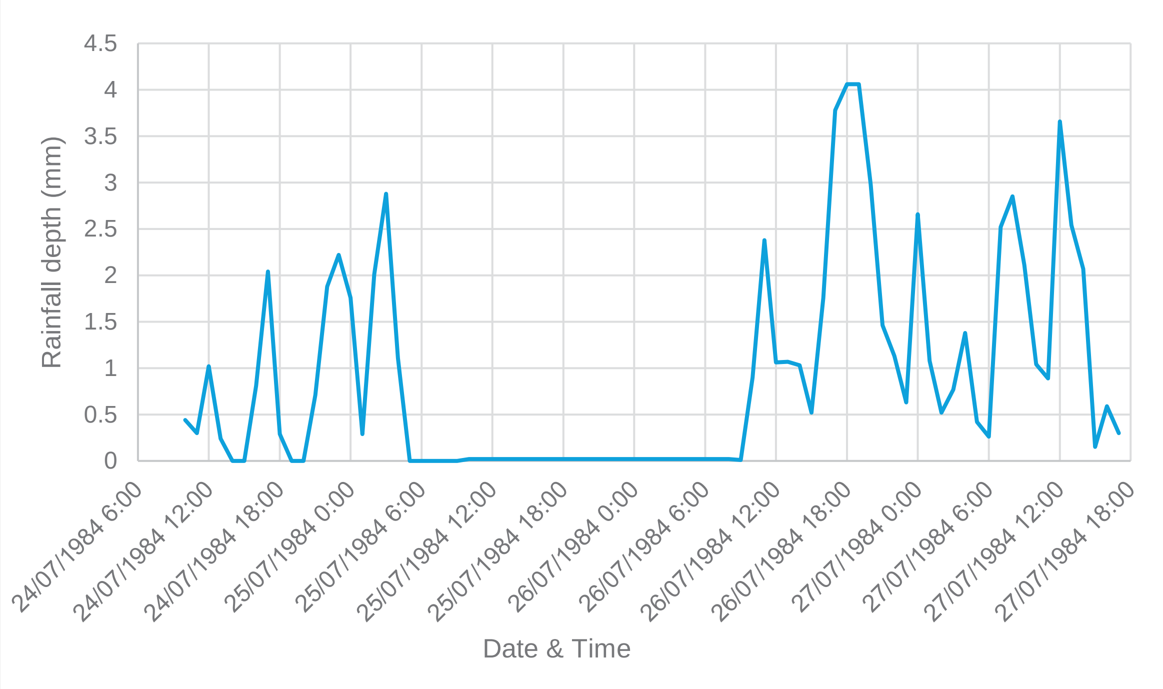

Figure 3.23 Hourly rainfall depths at Moree Comparison station (53048) for July 1984 event

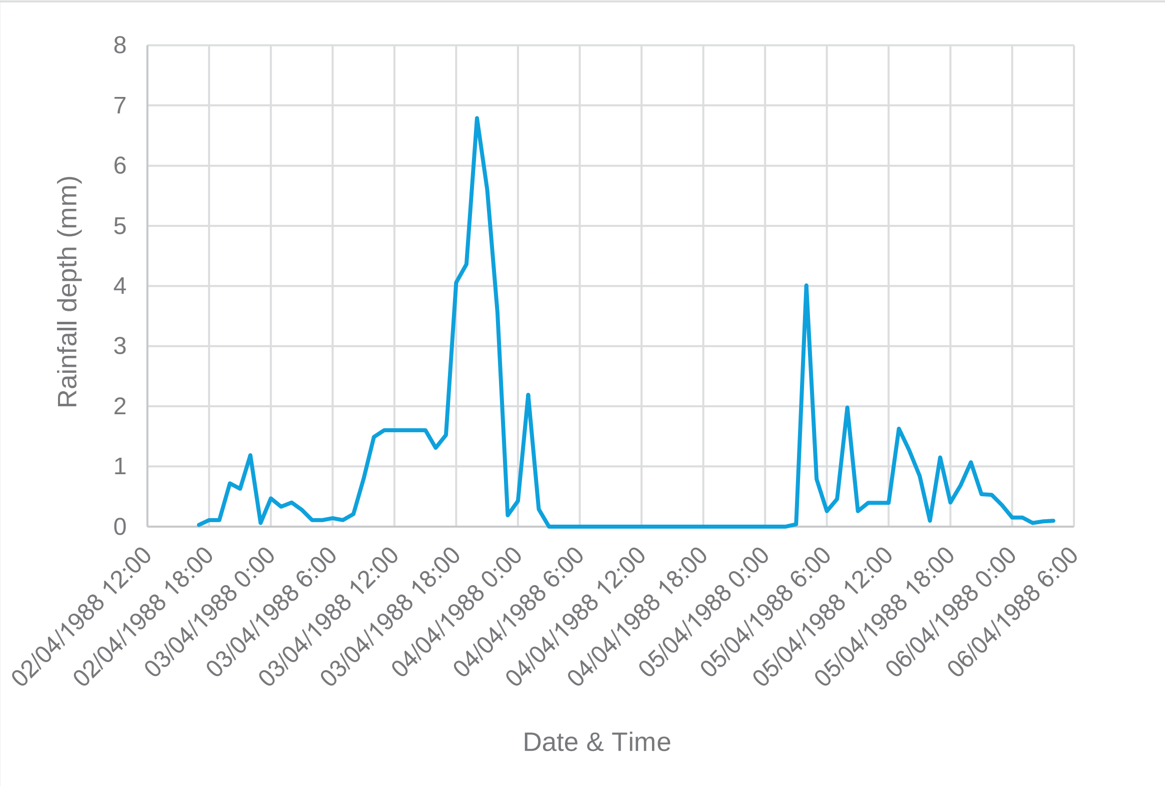

Figure 3.24 Hourly rainfall depths at Moree Comparison station (53048) for April 1988 event

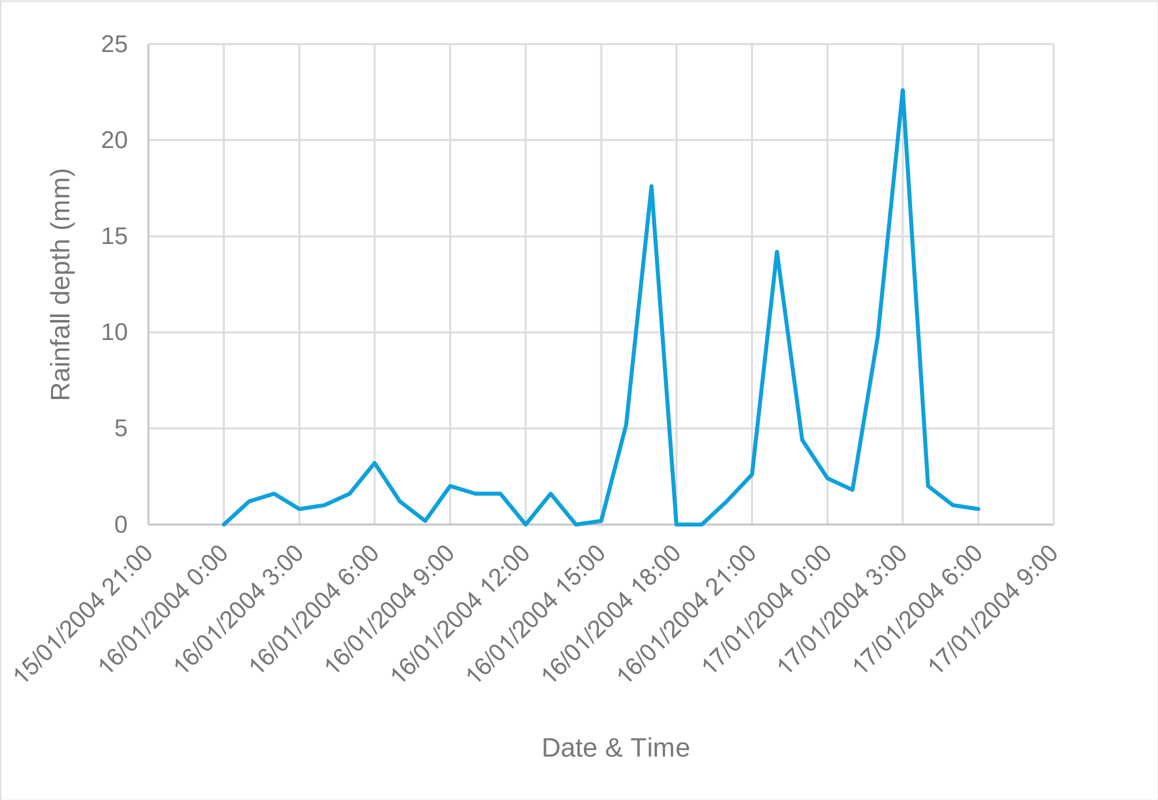

Figure 3.25 Hourly rainfall depths at Moree Aero station (53115) for January 2004 event

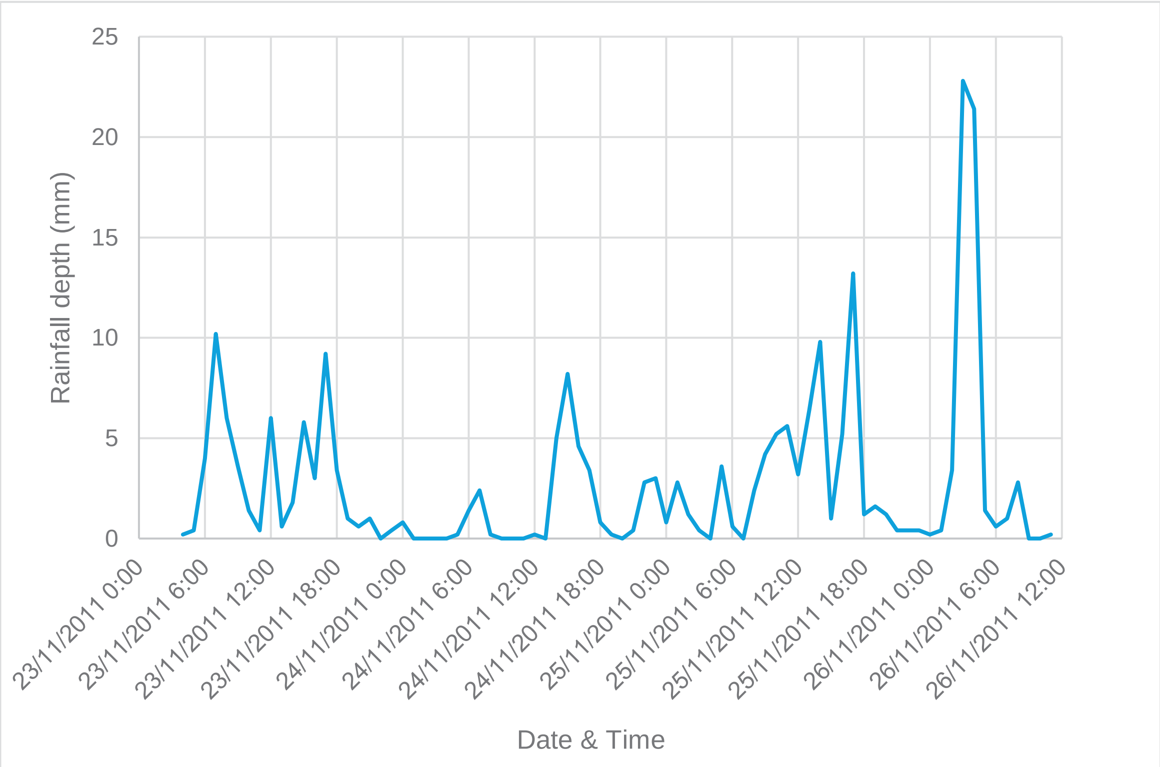

Figure 3.26 Hourly rainfall depths at Moree Aero station (53115) for November 2011 event

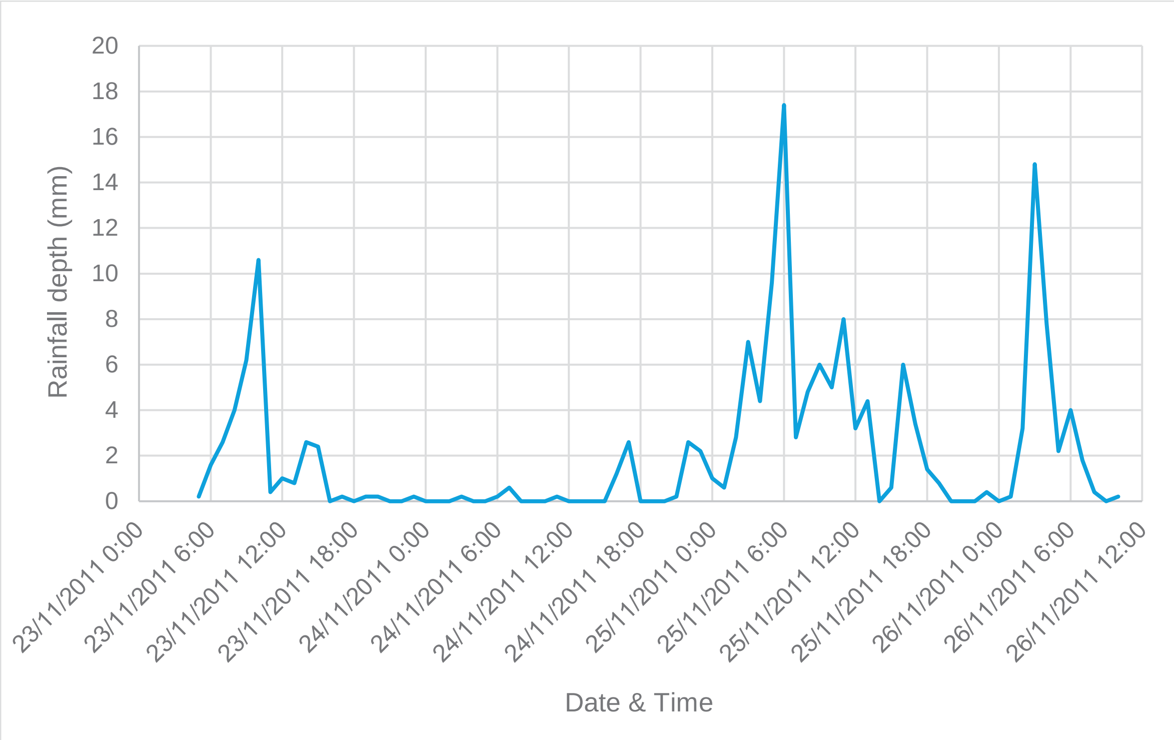

Figure 3.27 Hourly rainfall depths at Narrabri Airport station (54038) for November 2011 event

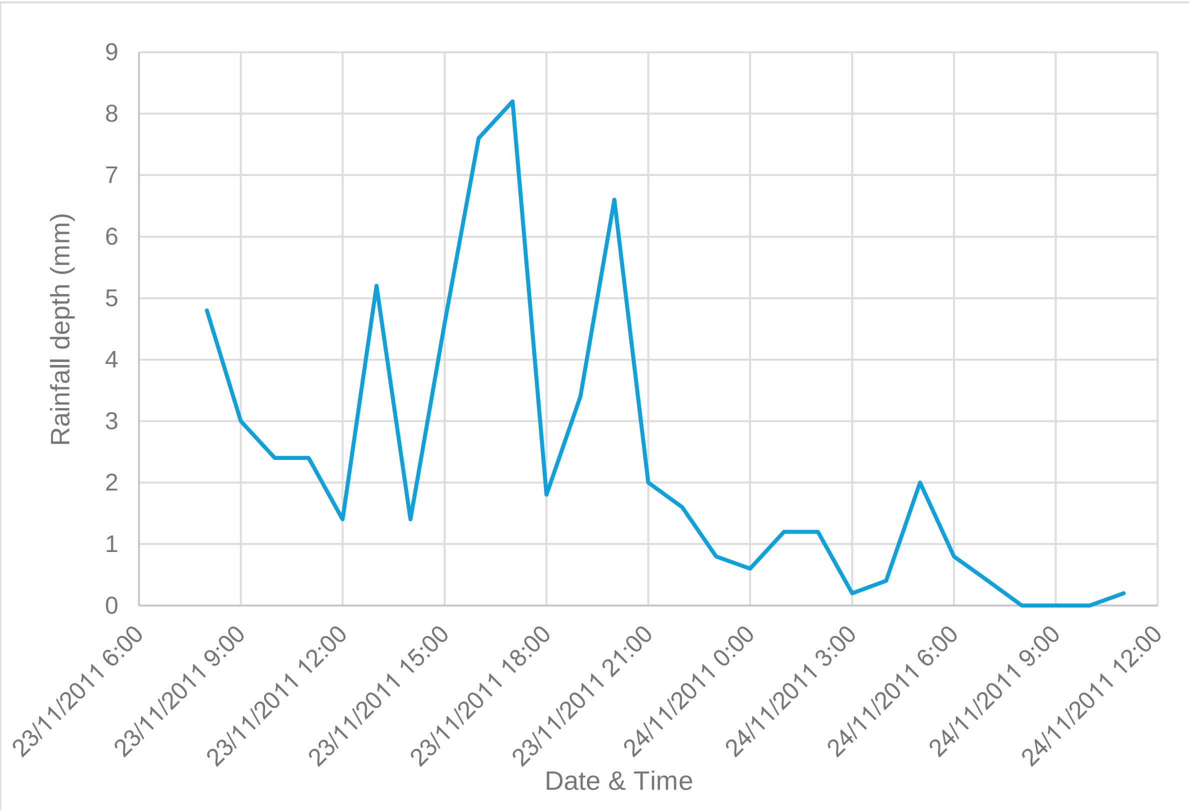

Figure 3.28 Hourly rainfall depths at Inverell Research Station (56018) for November 2011 event

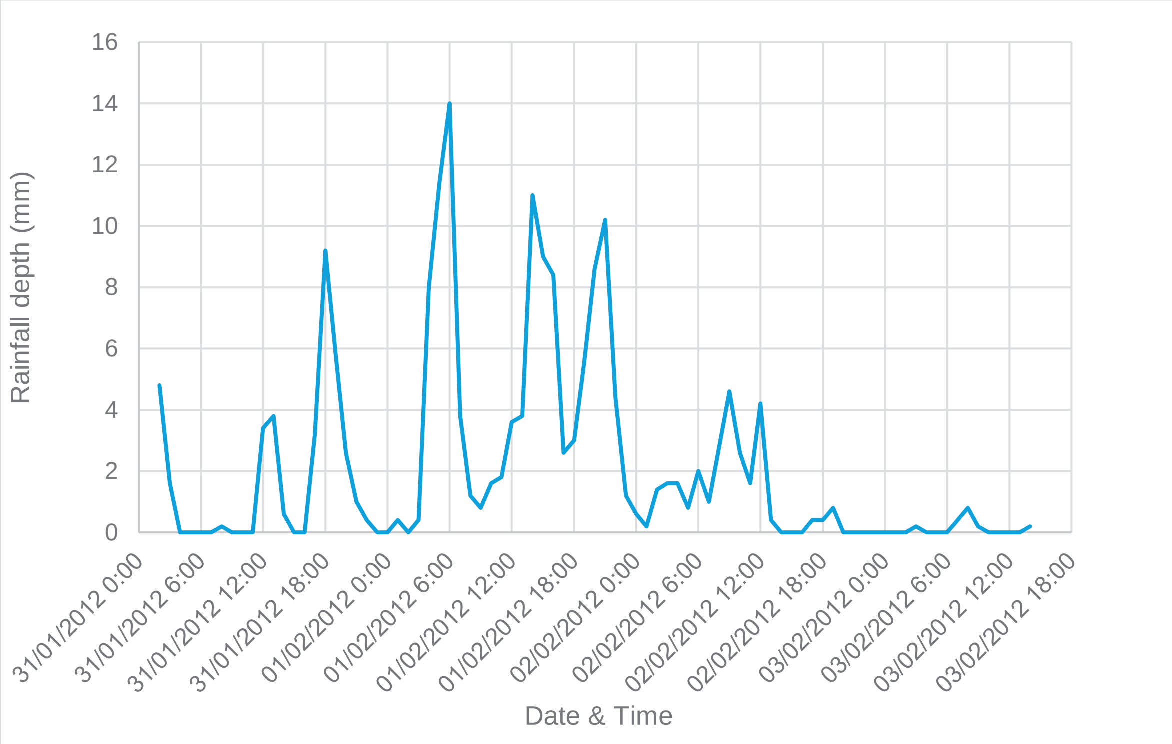

Figure 3.29 Hourly rainfall depths at Moree Aero station (53115) for February 2012 event

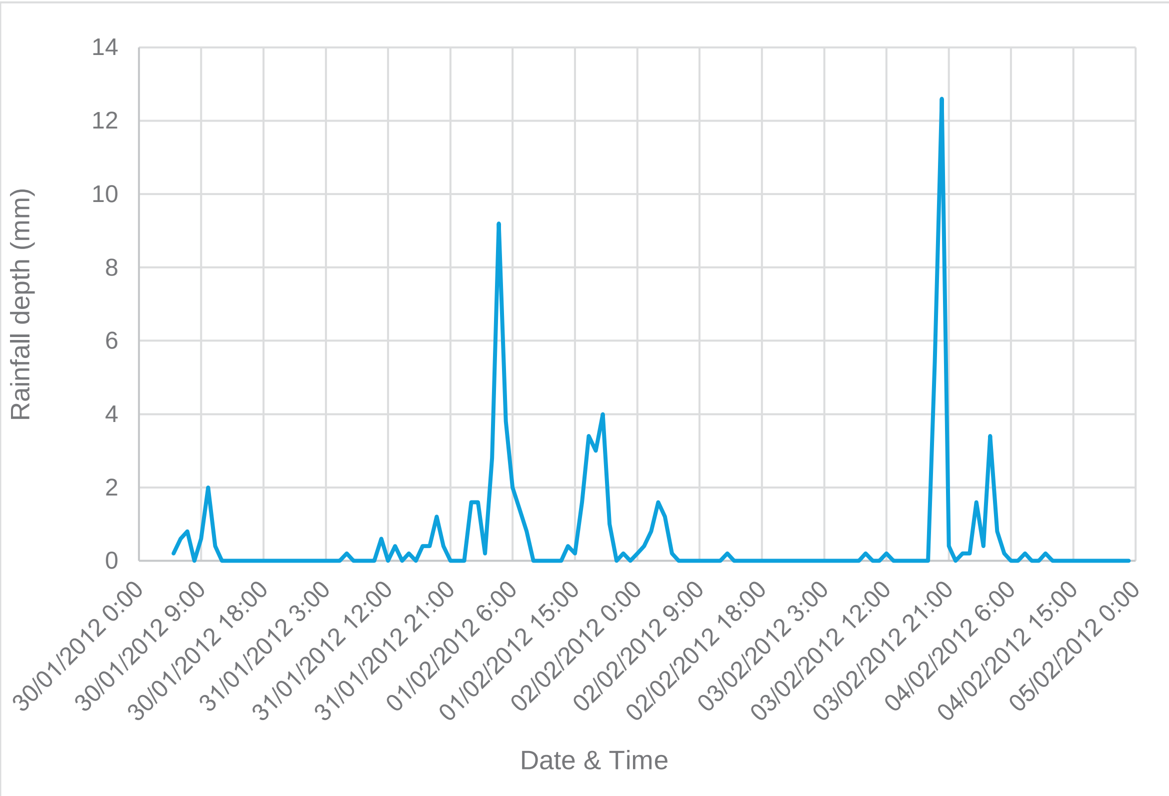

Figure 3.30 Hourly rainfall depths at Inverell Research Station (56018) for February 2012 event

3.2.3 Generation of Sub-Daily Rainfall Datasets for Calibration

Tycannah Creek and Croppa Creek catchments contain no daily or sub-daily rainfall stations. It was therefore necessary to use rainfall data from other adjacent catchments to generate the sub-daily rainfall datasets for calibration of the models for these catchments.

Book 2, Chapter 2 of ARR2016 provides advice on analysing sub-daily and daily data to generate the temporal pattern of rainfall for a particular event. The adopted approach is outlined below:

- Map all rainfall station locations within and around the Tycannah and Croppa Creek catchments;
- Identify sub-daily rainfall stations and stations that recorded data for the calibration events;
- Create Thiessen polygons using rainfall stations with data;
- Calculate the percentage (by area) of each rainfall station polygon within the catchment boundary;
- Multiply the percentage by the total rainfall depth recorded at each gauge and sum to provide a catchment rainfall depth;
- Factor the sub-daily temporal pattern by the ratio of catchment rainfall depth to sub-daily station rainfall depth (per event);
- If more than one sub-daily pattern is available, contour/grid the total rainfall depths and estimate the spatial influence of each pattern based on the total event rainfall depths; and
- Apply the generated temporal pattern to the RORB model for calibration.

The approach is demonstrated in the next section which explains how the sub-daily rainfall datasets were generated for the Tycannah Creek catchment for the November 2011 and February 2012 events.

3.2.3.1 Generation of Calibration Datasets for November 2011 and February 2012 Events for Tycannah Creek

The rainfall data available for the November 2011 and February 2012 events included 10 daily and 2 sub-daily rainfall stations. These stations are presented in Figure 3.31, which shows rainfall depths for the November 2011 event. The Inverell Research Station sub-daily dataset was disregarded from this analysis as it is located approximately 109 km to the east of the centroid of the catchment.

Thiessen polygons were used to spatially distribute the rainfall for the November 2011 and February 2012 events. The polygons are presented in Figure 3.32. The percentage of each polygon within the total Tycannah Creek catchment area was then calculated. These percentages where then used to estimate the total rainfall depth within the Tycannah Creek catchment for both the November 2011 and February 2012 rainfall events. The calculations are provided in Table 3.3.

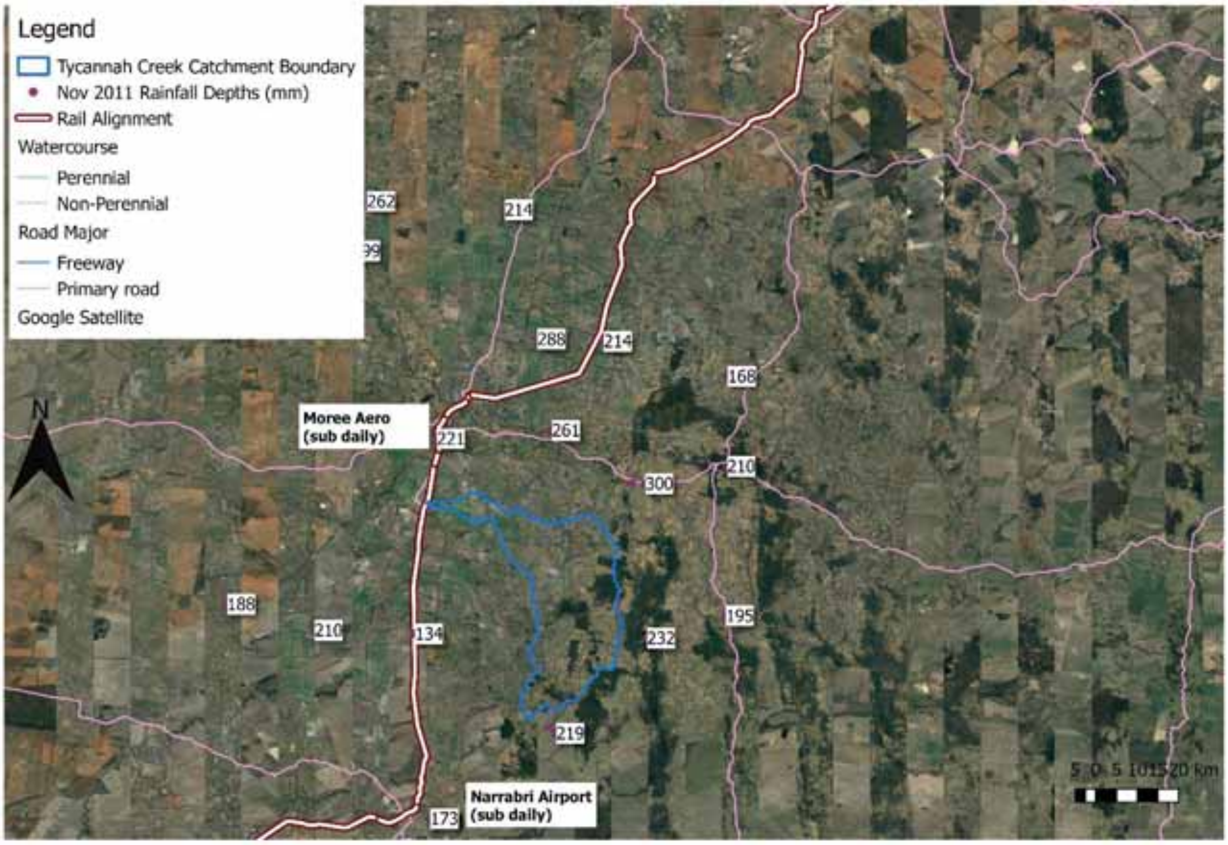

Figure 3.31 Daily and sub-daily rainfall data around the Tycannah Creek catchment for November 2011 event

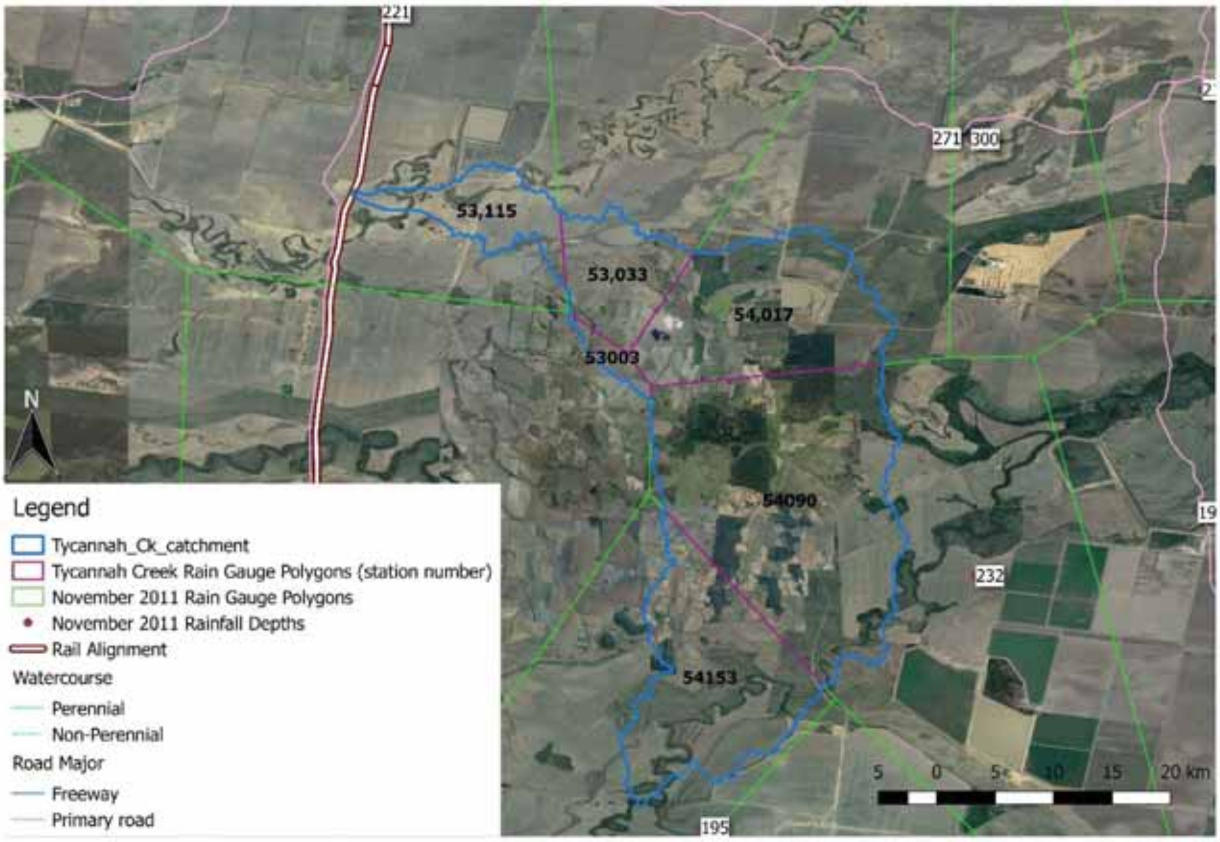

Figure 3.32 Thiessen polygons for Tycannah Creek catchment

Table 3.4 Estimation of total rainfall depths in Tycannah Creek for November 2011 and February 2012 events

Rain Gauge	Polygon area within Tycannah catchment	Percentage of polygon area within Tycannah catchment	Recorded depth for November 2011 event (mm)	Factored depth for November 2011 event (mm)	Recorded depth for February 2012 event (mm)	Factored depth for February 2012 event (mm)
53033	86.165	8%	261.4	22.2	167.4	14.2
53003	14.820	1%	134.2	2.0	289.4	4.2
54153	217.848	21%	195.0	41.9	312.6	67.1
54017	211.721	21%	271.9	56.7	141.8	29.6
54090	413.530	41%	232.0	94.5	305.0	124.3
53115	70.742	7%	221.4	15.4	217.2	15.1
TOTALS	**1014.826**	**100%**	**-**	**232.7**	**-**	**254.6**

Based on the calculations in Table 3.4, the weighted total event rainfall depths within the Tycannah Creek catchment for the November 2011 and February 2012 events are 232.7 mm and 254.6 mm respectively. For the November 2011 rainfall event the Moree Aero (53115) and Narrabri Airport (54038) hourly rainfall depths are available. The Moree Aero gauge is approximately 55 km from the centroid of the Tycannah Creek catchment and the Narrabri Airport is approximately 72 km from the centroid. The patterns are different as shown in Figure 3.33 for the November 2011 event.

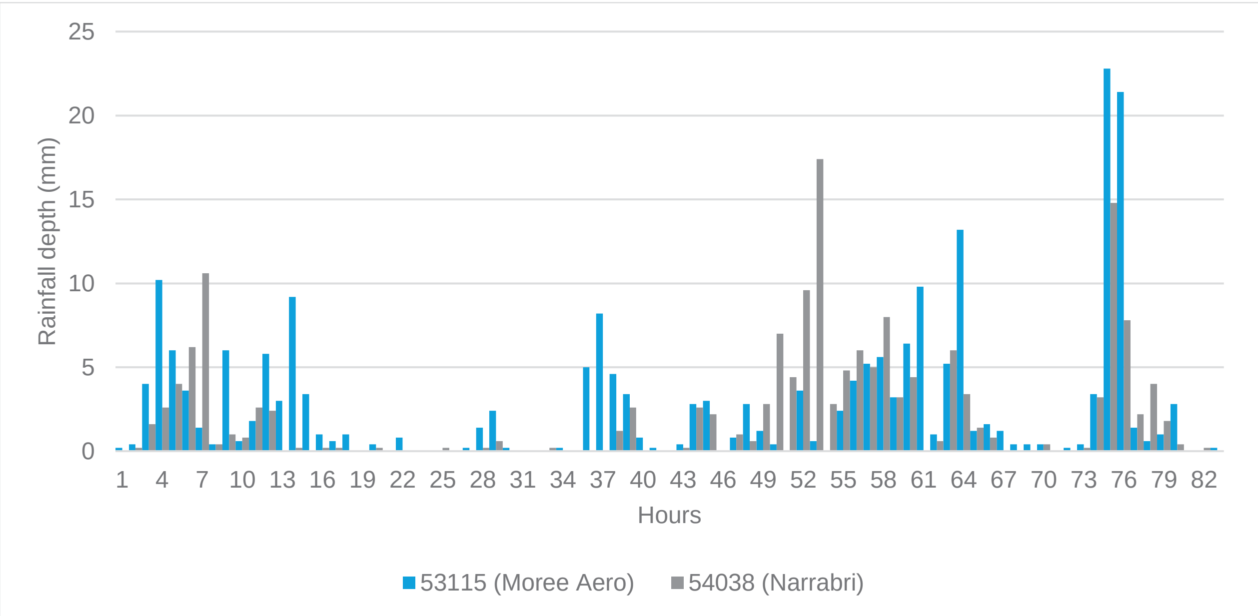

Figure 3.33 Hourly rainfall data for the November 2011 event at Moree Aero (53115) and Narrabri Airport (54038)

Weighted averages of the hourly rainfall patterns at these two stations were generated based on the distances from the stations to the Tycannah Creek catchment centroid and applied to the total rainfall depths estimated for the Tycannah Creek catchment to generate the hourly rainfall datasets for the November 2011 and February 2012 calibration events.

4 Results

4.1 Review of Moree Regional Flood Model Calibration and Hydrological Models

4.1.1 Review of Model Calibration

The Moree Regional Flood Model has a hydrological component which is made up of main regional river inflows at the upstream boundary at Gravesend and local catchment inflows between Gravesend and Moree. The upstream boundary inflows are defined from measured flow records at Gravesend while the local catchment inflows are modelled using the XP-RAFTS software package.

The hydrology model was calibrated by applying measured flows at the Gravesend flow gauge and corresponding measured rainfalls over the local catchments in the XP-RAFTS model. The model was calibrated to the following events:

- February 1955;
- July 1998;
- January 2004;
- November 2011; and
- February 2012.

The calibrated flows from the hydrological model were then applied to the MIKE FLOOD hydrodynamic model and the hydrodynamic model was calibrated to observed water levels and flows at numerous gauge locations for the same events. The calibration process is documented in detail in the Flood Study Report (WRM, 2017),

The hydrological model accurately predicted the timing of the flood events but tended to overpredict the peak flows at the gauge sites. An example is shown below in Figure 4.1. The report suggests this lack of fit of high flows is due to the inability of the rating curves developed for the flow gauges to account for the total floodplain flow. This is a reasonable conclusion and supported by the gauge records and common issues in other regional river catchments that experience very high flows.

When the calibrated hydrological model inflows were applied to the hydrodynamic model, a better match to recorded flows and a very good match to recorded water levels was obtained. An example is shown in Figure 4.2.

The Flood Study Report concludes that the calibration was acceptable given the very good fit achieved by the model to in-channel flows for all calibration events and the model calibrated well to observed levels on the floodplain for the highest events of February 1955 and February 2012, and in particular, a good agreement to observed flood levels was obtained around the existing rail corridor and Newell Highway for the February 2012 event.

This review found that the model calibration process was rigorous and made best use of all available data. Given the complexity of the modelled Gwydir-Mehi system and the good calibration results achieved for four out of the five events simulated, it is considered that the model provides a sound basis for the detailed design phase of this project.

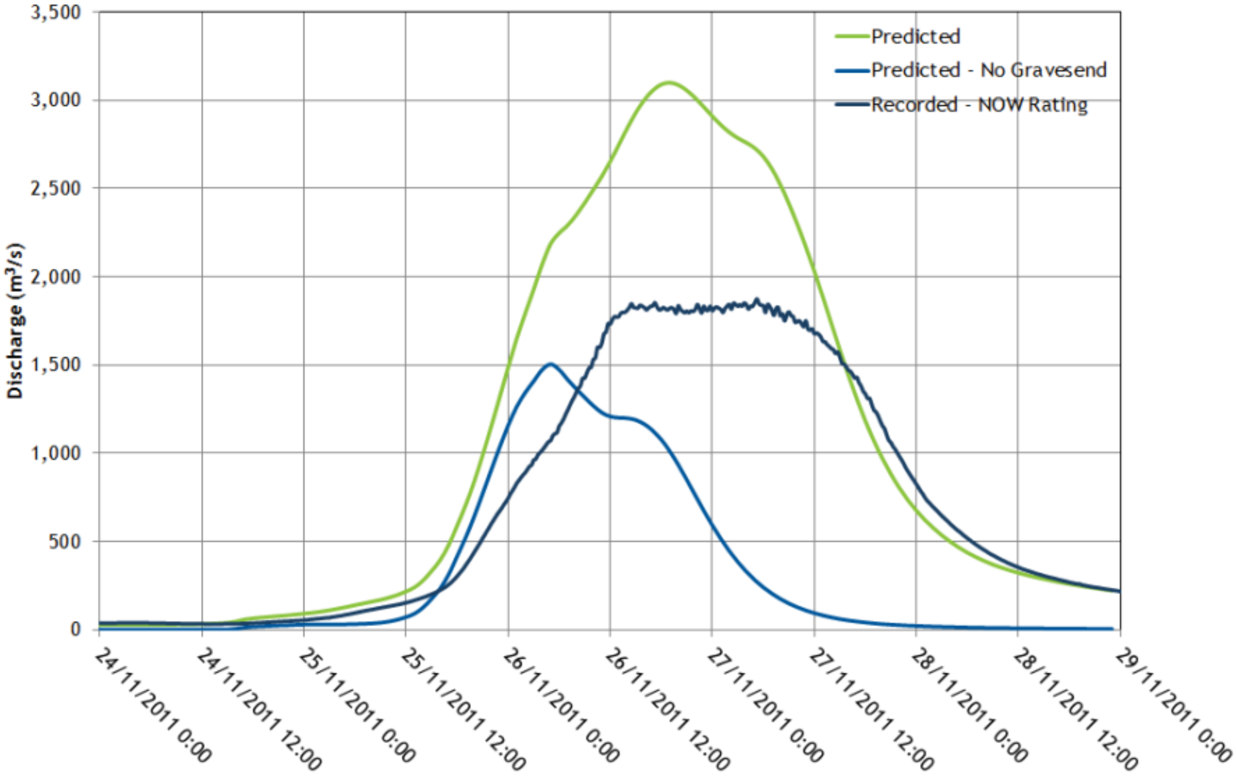

Figure 4.1 Moree Regional Flood Model hydrological model calibration result at Gwydir River at Pallamallawa (418001) for November 2011 event (WRM, 2017)

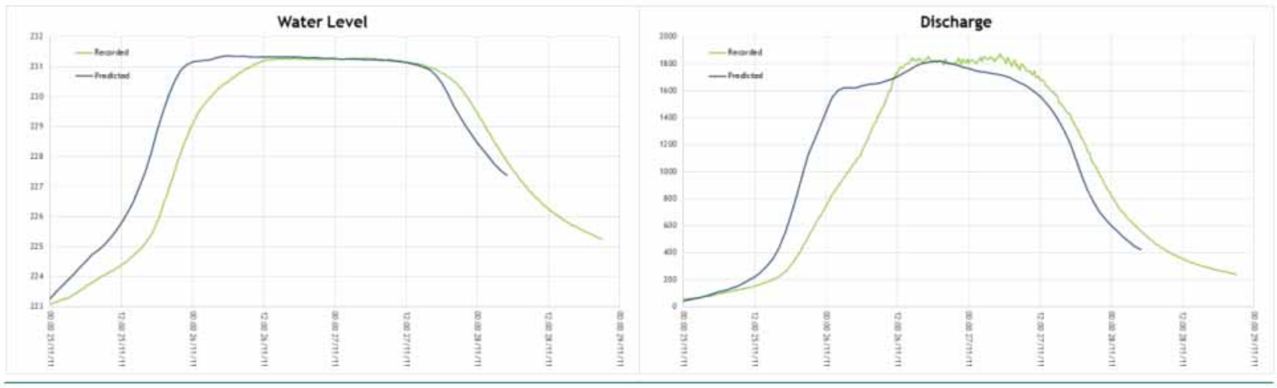

Figure 4.2 Moree Regional Flood Model hydrodynamic model calibration result at Gwydir River at Pallamallawa (418001) for November 2011 event (WRM, 2017)

4.1.2 Gravesend Flood Frequency Analysis

The main river inflow at Gravesend is a key input to the model as it represents the regional river inflows from the 11,020km^2 catchment upstream. The inflows were generated using FFA of the flow records from the Gwydir River at Gravesend Road Bridge gauge (418013) to generate peak flows which were then fitted to an inflow hydrograph shape determined from the records of historical floods.

The gauge record dates from 1936 to the present, giving a record length of 80 years. The Flood Study Report (WRM, 2017) describes a procedure that was used to supplement this data with a further 45 years of peak flow estimates by developing a correlation between the Gravesend flow record and the record for the Gwydir River at Pallamallawa (418001) which dates from 1891. Further adjustments were made to the FFA to account for the impact of Copeton Dam and additional information suggesting that the 1955 flood event was the highest on record since 1860.

A Log-Pearson Type III (LP III) distribution was fitted to the annual series of recorded (and inferred from the correlation with the Pallamallawa gauge) peak flood flows at Gravesend using the Bayesian inference methodology recommended in ARR2106 using the FLIKE software. This methodology allows the user to more accurately consider historic data outside the gauged record, as well as allowing the user to censor low flows to improve the fit for the larger events.

A check of the FFA produced for the Flood Study was undertaken by IRDJV using the same FFA method but only the period of record dataset at the Gravesend gauge (from 1937 to 2015) as all the additional information used by WRM in adjusting account for Copeton Dam and the flood history to 1860 was not available from the Flood Study Report. The IRDJV check obtained a very similar result from FLIKE, confirming that the flow estimates are reliable. Table 4.1 provides the WRM peak flows obtained from the FFA of the gauge record and the adopted flows based on the FFA of the extended dataset using all historical data and the relevant adjustments.

Table 4.1 Peak main river inflows at Gravesend based on FFA of gauge record only and gauge record supplemented with historical data (WRM, 2017)

Event	Peak flow based on FFA of gauge record (m^3/s)	Peak flow based on FFA of gauge record supplemented with historical data – ADOPTED (m^3/s)
20% AEP	1,300	1,280
10% AEP	2,300	2,180
5% AEP	3,570	3,240
2% AEP	5,620	4,820
1% AEP	7,570	6,110
0.5% AEP	10,310	7,520

4.1.3 Local Catchment XP-RAFTS Model

The local catchment between Gravesend and Moree extends to approximately 2,300 km^2 and includes the following creeks and tributaries:

- Halls Creek;
- White Swamp;
- Marshalls Pond Creek;

- Mia Mia Creek;
- Eatons Ponds Creek;
- Slaughterhouse Creek;
- Mosquito Creek; and
- Tycannah Creek.

For the 2017 Flood Study an XP-RAFTS model was developed to represent the local catchment inflows. The XP-RAFTS model consists of total of 55 sub-catchments which were delineated from the Shuttle Radar Topography Mission (SRTM) satellite data. The modelled sub-catchments range in size from 16km^2 to 115km^2.

The flow gauge records and the XP-RAFTS modelling demonstrate that the local catchment peaks around 12 to 15 hours earlier than the peak river inflow at Gravesend for historical events. The local catchment does not govern the flood behaviour through Moree but does contribute to the peak flow, particularly in the higher events.

Figure 4.3 shows the water level hydrograph result from the MIKE FLOOD model at the rail line crossing of the main Gwydir channel north of Moree, at approximate chainage 675km. The hydrograph demonstrates an initial peak water level occurring approximately 36 hours before the main peak level. The main peak level is approximately 600mm higher than the initial peak level. This pattern is evident in the model inflow boundary condition at Gravesend and is likely to be due to significant local sub-catchments upstream of Gravesend that peak before the main river peak flow occurs from the total upstream catchment.

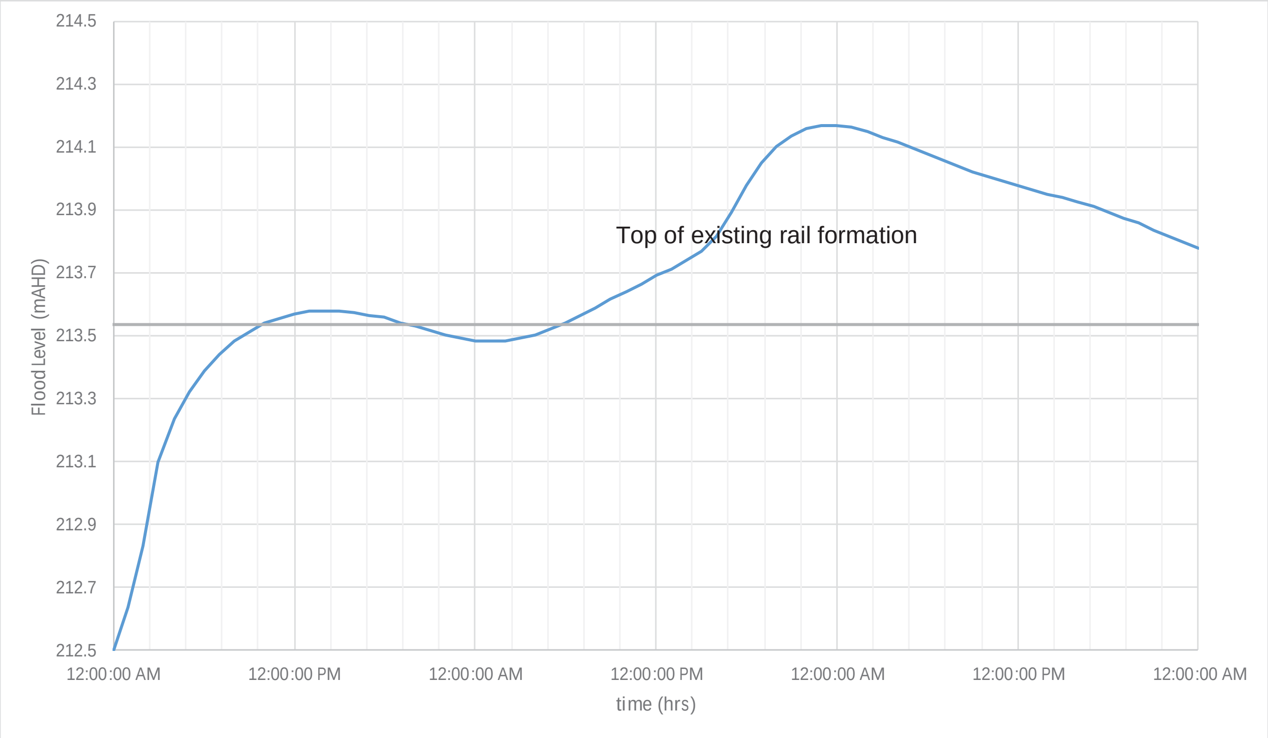

Figure 4.3 Moree Regional Flood Model 1% AEP water level hydrograph at rail crossing of Gwydir River

Table 4.2 gives the peak 1% AEP design flows for the main river inflow at Gravesend and the local catchment creek systems to show the relative contribution of each catchment. The flows given in the table are for the 48-hour design storm which was found to be the critical duration (i.e. the storm duration that produces worst case flooding) for flooding through Moree and environs.

Table 4.2 1% AEP 48 hour storm peak flows for main river and local catchments in the Moree Regional Flood Model

Catchment	Peak flow for 1% AEP event (m^3/s)
Main river inflow at Gravesend	6,110
Halls Creek	71
White Swamp	62
Marshalls Pond Creek	221
Mia Mia Creek	194
Eatons Ponds Creek	124
Slaughterhouse Creek	205
Mosquito Creek	558
Tycannah Creek	890

The XP-RAFTS local catchment model is based on the ARR1987 guideline, which does not include the latest rainfall intensity, frequency and duration parameters and rainfall temporal patterns provided in ARR2016. Updating the model to ARR2016 was considered, however, it should be noted that the model has been recently formally adopted by Moree Plains Shire Council for floodplain management planning purposes and as of December 2018 the Flood Study was subject to community consultation. Therefore, any changes to the established model and its associated predictions of flood behaviour need to be carefully considered given that Council is currently at a sensitive stage in its floodplain management planning process.

The impact of updating the XP-RAFTS model to ARR2016 was tested by updating a sample number of upstream sub-catchments to ARR2016 and comparing the resulting flows to the original ARR1987 model. The results of this test are provided in Table 4.3 below. The key findings from the test were that ARR2016 produces higher flows for shorter duration storms (e.g. the 12-hour storm) which are critical in the local catchments, whereas ARR1987 produces higher flows for the 48-hour storm, which is critical for the regional catchment and which governs the worst-case flood behaviour in the study area. The ARR1987 model can therefore be considered to provide conservatively high flow values for the local catchments for the critical flood in the area around Moree.

The local catchment model does not govern flood behaviour around the rail corridor and the primary flood risk to the corridor is governed by the main river inflow at Gravesend. Therefore, for design purposes it is proposed to adopt the existing ARR1987 local catchment hydrology model to be consistent with the established flood model for the area. However, the flood impact assessment will update the ARR1987 model to ARR2016 for the purposes of assessing impacts of the project on flooding in the local catchments for non-critical shorter duration storms (e.g. as the 12-hour storm) to assess the potential for higher impacts occurring under shorter duration, higher frequency and locally critical flood events than under the critical regional flood event.

Table 4.3 Results of tests of updating XP-RAFTS model to ARR2016

Model sub-catchment	Sub-catchment area (km^2)	Critical storm in local sub-catchment				Critical storm in regional catchment (48 hour storm)			
		Storm duration (hours)	ARR1987* peak flow (m^3/s)	ARR2016** peak flow (m^3/s)	Difference in peak flow	ARR1987* peak flow (m^3/s)	ARR2016 median storm	ARR2016** peak flow (m^3/s)	Difference in peak flow
1 (upper sub-catchment of Mosquito Creek)	56.7	12	202	217	+7%	152	Storm 5	99	-35%
12 (sub-catchment of Gwydir River downstream of Gravesend)	109.2	12	335	458	+37%	253	Storm 5	197	-22%
42 (upper sub-catchment of Halls Creek)	43.4	48	51	73	+42%	51	Storm 9	56	+10%

Notes:

*ARR1987 method uses losses from the Flood Study Report (WRM, 2017) of IL = 30 mm, CL = 4.5 mm/hour

**ARR2016 method uses Ensemble Storms and losses of IL = 26 to 69 mm, CL = 0 to 0.7 mm/hour

4.1.4 Conclusions

The review of the Moree Regional Flood Model has found the following:

- The hydrological model calibration did not achieve a close agreement to the flow record, however this is very likely due to the limitations in the rating curves at the gauges and their inability to capture the entire floodplain flow. The hydrodynamic model calibration achieved close agreement to the water level and flow records, giving confidence that the model simulated the historical flood behaviour accurately;
- The main river inflow to the model, which governs the model predictions of regional flooding around Moree, is based on a detailed FFA that considers all available historical data and the impact of major storages in the upstream catchment. The FFA was based on the latest methods recommended in ARR2016 and was independently checked and confirmed by IRDJV; and
- Local catchment inflows do not govern the regional scale flood behaviour predicted by the model but do contribute to flood behaviour for high order events. The local catchment hydrology model is based on the superseded ARR1987 guideline and was found to produce conservatively high flow estimates for the critical regional flood event. If updated to ARR2016 the model will likely produce higher flow estimates for short duration locally critical flood events, but not for the critical regional flood event which will dictate the detailed design of the rail upgrade.

Based on these conclusions, it is proposed to adopt the existing Moree Regional Flood Model for use in design, with an update to the local catchment hydrology model to ARR2016 to be undertaken as part of the flood impact assessment for short duration locally critical events. The approach to adopt the previous model is further justified on the basis that the model is currently the best available tool for defining flood risk within this area and has been through a comprehensive process of review and adoption by Council as the basis for their flood risk management planning in the area.

4.2 Calibration of GWYDIR02 and MACINTYRE02 RORB Models

4.2.1 GWYDIR02 Model Calibration at Tycannah Creek

4.2.1.1 Calibration Results

The calibration of the GWYDIR02 model to the Tycannah Creek gauge at Horseshoe Lagoon (418032) did not yield consistent results. Although the physiographic characteristics of the catchment were accurately represented in the model, and the best available data were used to define and include the spatial and temporal patterns of rainfall in the model, this did not result in a conclusive calibration of the GWYDIR02 model. The model parameters are given below in Table 4.4 and the calibration plots are present in Figure 4.4 to Figure 4.7.

Table 4.4 Parameters used in GWYDIR02 / Tycannah Creek model calibration

Event / Study	m	k_c	Initial loss (mm)	Continuing loss (mm per hour)
Jul 1998 event	0.80	96	20	0.61
Jan 2004 event	0.80	70	20	11.07
Nov 2011 event	0.80	135	30	5.71
Feb 2012 event	0.80	270	20	1.75
Gwydir Floodplain Management Plan* (OEH, 2016)	0.80	70.9	25	2.32
Notes: **Model parameters from the OEH study are averaged for all calibration events assessed*				

As shown in Table 4.4, the July 1998 and January 2004 flood events yielded k_c values of 96 and 70, respectively, through a relatively good calibration, although the timing of the peak flow occurrence did not match the records. However, the November 2011 and February 2012 events, which were the highest flood events on record, resulted in k_c parameters of 135 and 270, respectively.

The Moree and Environs Flood Study (WRM 2017) noted that the gauge is only rated to the in-channel flows and not the floodplain flows, and therefore the flow record would not be reliable for flow events with significant floodplain flow bypassing the main channel. In large flood events with significant bypassing flow the gauge record would therefore be expected to underestimate peak flows. The results in Table 4.4 show that the RORB model achieved a good agreement to the peak observed flows for the lower events of July 1998 and January 2004 with reasonable k_c values input to the model. For the higher events of November 2011 and February 2012, very high k_c values were required to match the observed peak flows, which had the effect of introducing more storage into the catchment to reduce the runoff. Given that the recorded peak flows for the 2011 and 2012 events were on average 50% higher than those recorded for the 1998 and 2004 events, it is likely that significant bypassing floodplain flow occurred during the larger events which would explain the difficulty in calibrating the model to these events. It is noted that a similar issue occurred for the Moree and Environs Flood Study (WRM 2017) in which the RAFTS model used for that study did not achieve a good calibration to the recorded peak flows. In that study the RAFTS model was found to overestimate peak flows for the same four calibration events, with the highest overprediction of 87% obtained for the 2012 event.

Because the Tycannah Creek calibration attempts did not yield consistent k_c values, the adopted k_c was based on an alternate method, as described in Section 5.3 below.

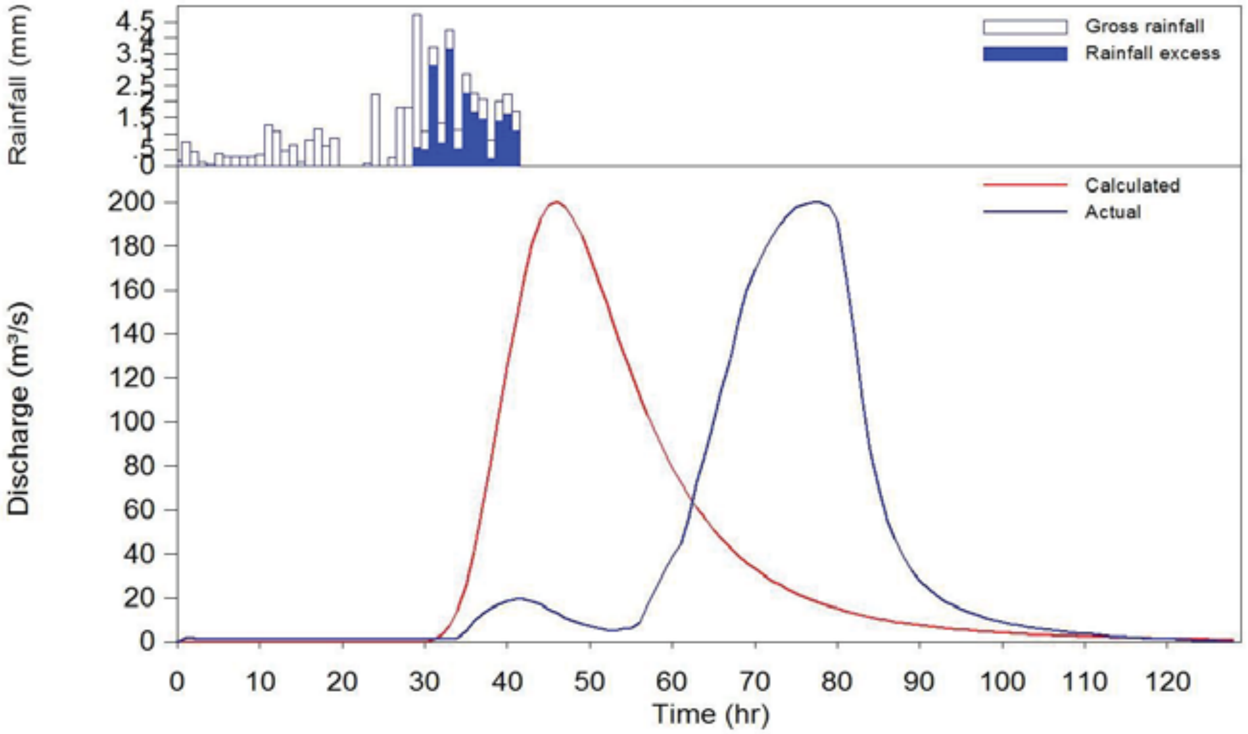

Figure 4.4 Calibration result for GWYDIR02 model (Tycannah Creek at Horseshoe Lagoon) for July 1998 event

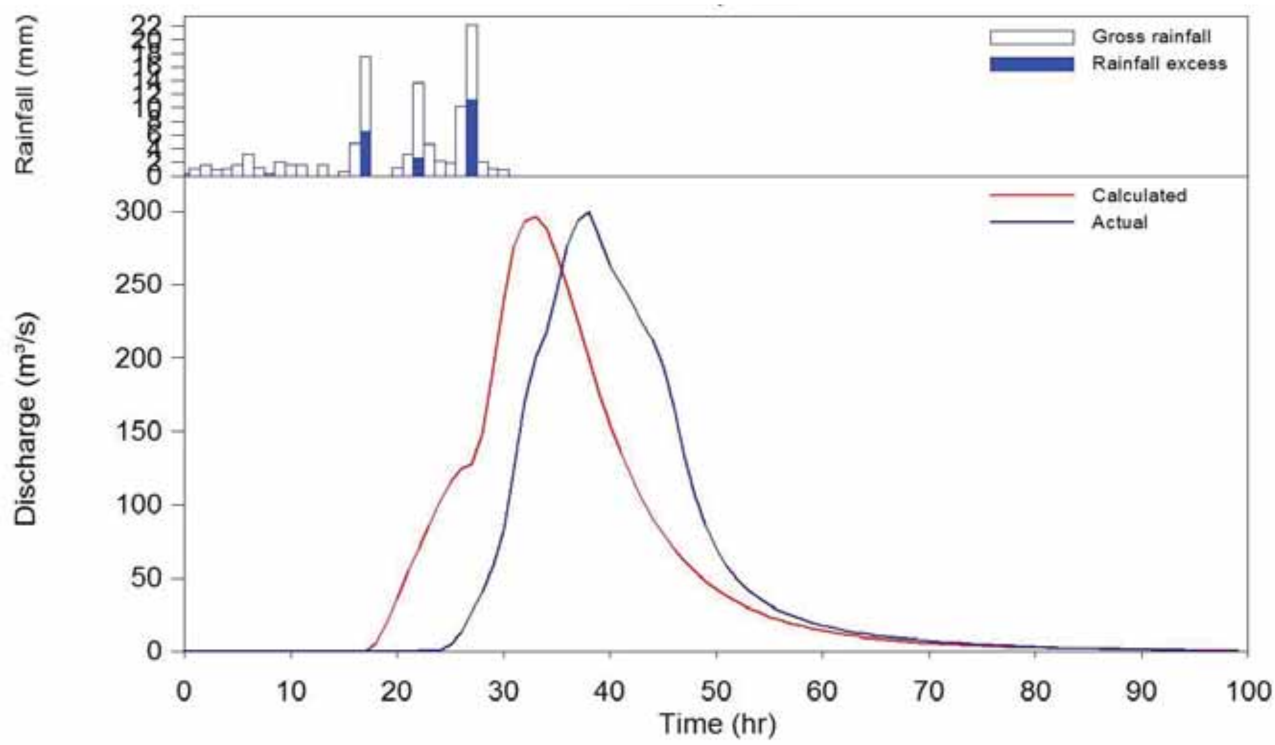

Figure 4.5 Calibration result for GWYDIR02 model (Tycannah Creek at Horseshoe Lagoon) for January 2004 event

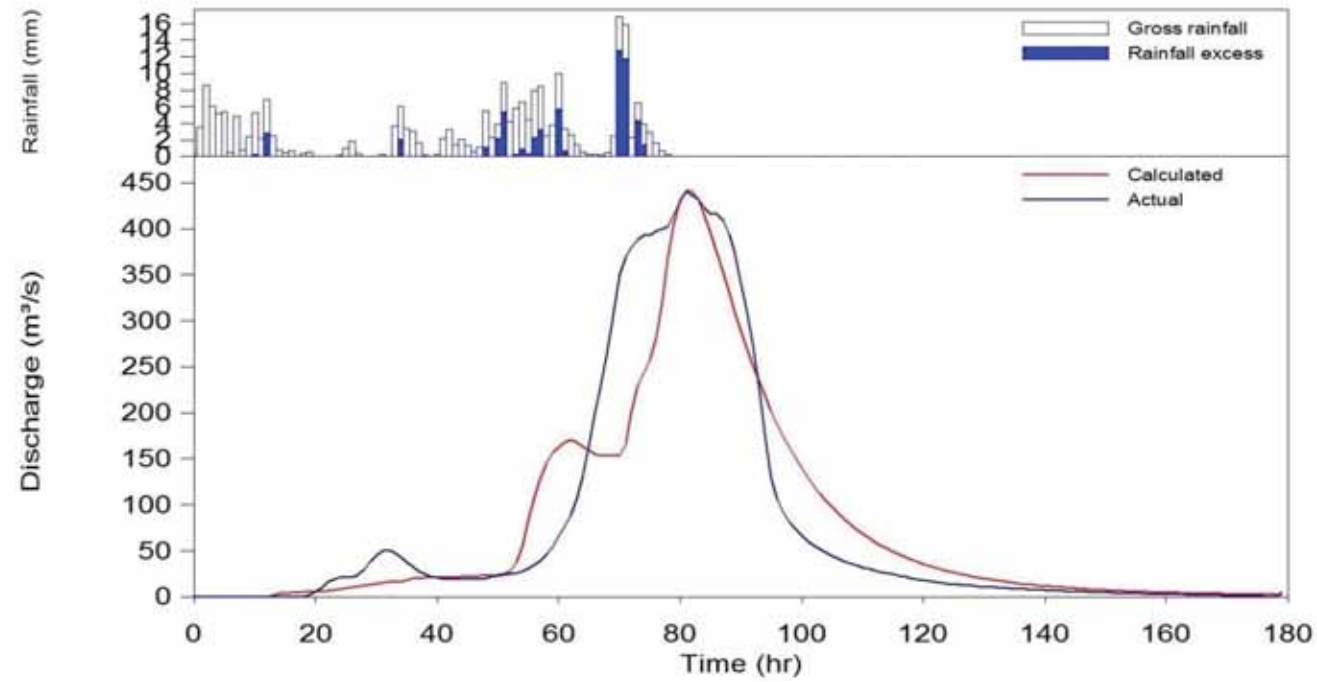

Figure 4.6 Calibration result for GWYDIR02 model (Tycannah Creek at Horseshoe Lagoon) for November 2011 event

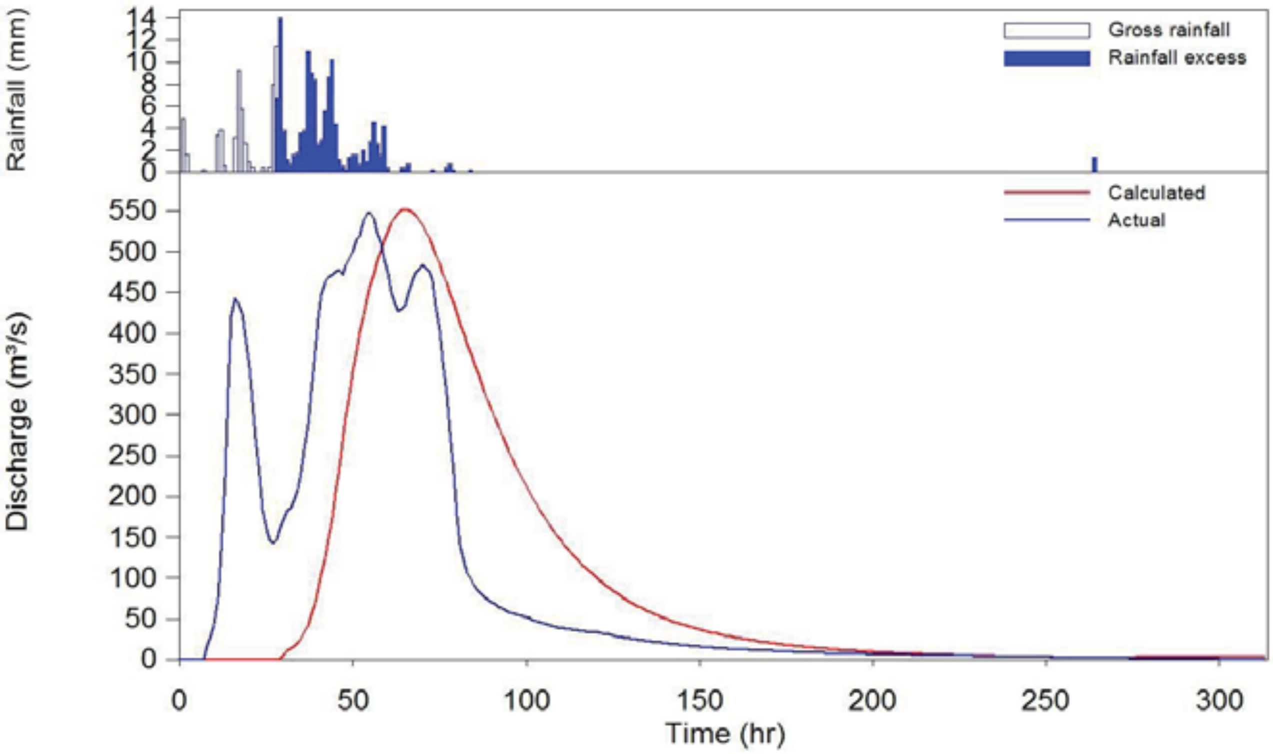

Figure 4.7 Calibration result for GWYDIR02 model (Tycannah Creek at Horseshoe Lagoon) for February 2012 event

4.2.1.2 Return Period Estimates for Calibration Events

An FFA was undertaken on the stream gauge records to estimate the return period of the calibration events. Due to the issues with the gauge flow records noted in Section 3.1.3.2, the analysis has been done on the water level records rather than the flow records. The results are provided in Table 4.5.

Table 4.5 Return period estimates for of calibration events for Tycannah Creek at Horseshoe Lagoon

Event	Return Period* (1 in Y years)
July 1998	3.6
January 2004	7.4
November 2011	21.4
February 2012	57.0
*Based on 46-year period of record	

4.2.2 MACINTYRE02 Model Calibration at Croppa Creek

4.2.2.1 Calibration Results

The calibration of the MACINTYRE02 model to the Croppa Creek gauge at Tulloona Bore (416034) yielded reasonable results. The model parameters are given below in Table 4.6 and the calibration plots are presented in Figure 4.8 to Figure 4.10.

Table 4.6 Parameters used in MACINTYRE02 / Croppa Creek model calibration

Event / Study	m	k_c	Initial loss (mm)	Continuing loss (mm per hour)
May 1983 event	0.80	65	15	0.05
Jul 1984 event	0.80	73	27	0.04
April 1988 event	0.80	64	20	2.1

Based on the above results, a k_c value of 67 (average of the three calibrated values) will be adopted for the MACINTYRE02 design event model, with IL and CL values in accordance with the ARR2016 recommended values.

The above results were also used to establish design event model parameters for the other uncalibrated models, as described in Section 5.3.

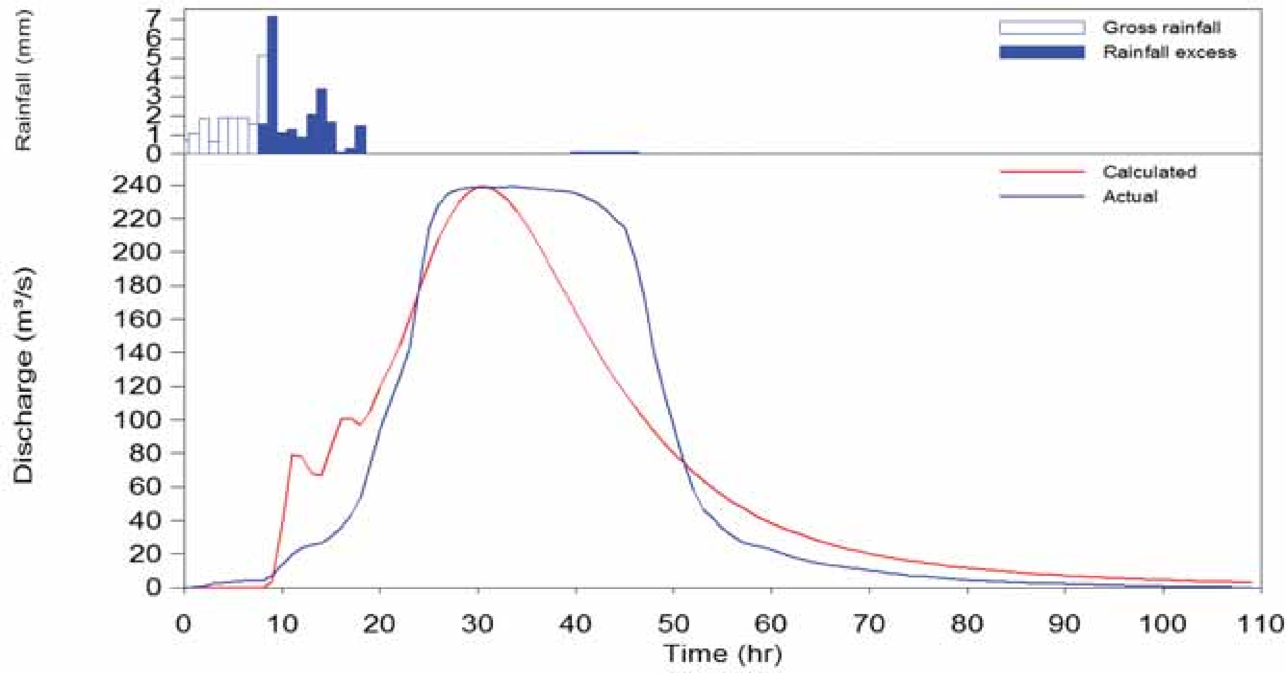

Figure 4.8 Calibration result for MACINTYRE02 model (Croppa Creek at Tulloona Bore) for May 1983 event

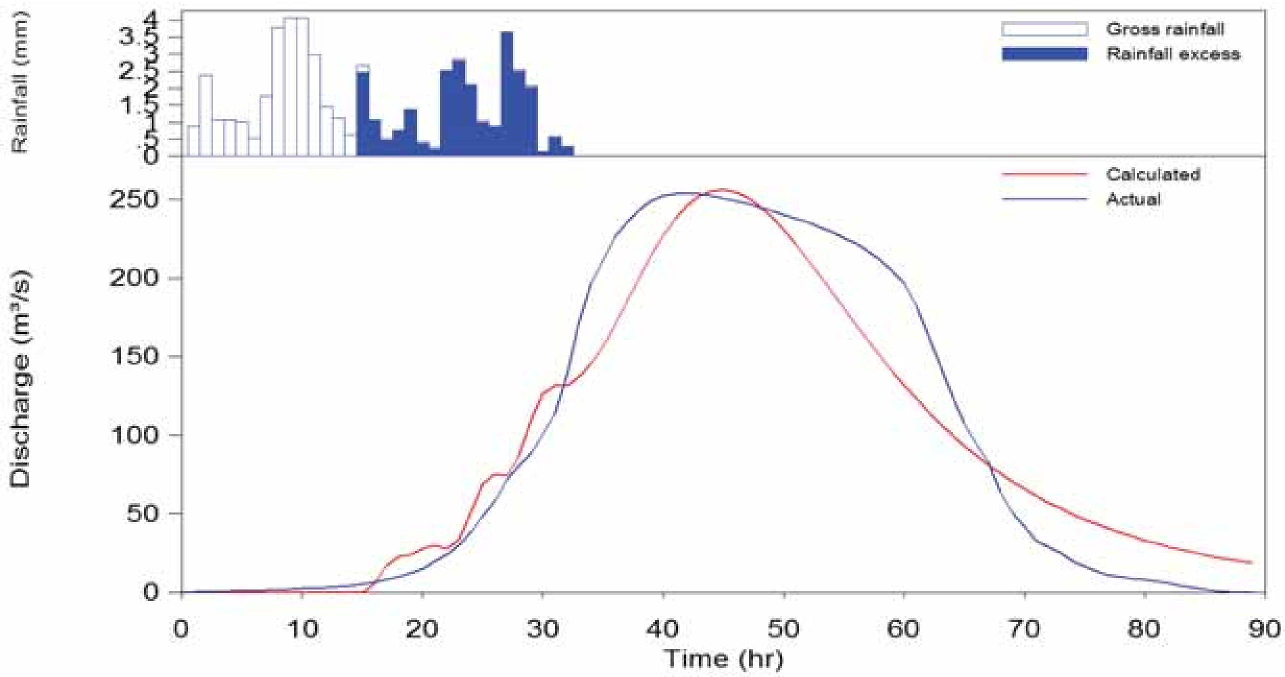

Figure 4.9 Calibration result for MACINTYRE02 model (Croppa Creek at Tulloona Bore) for July 1984 event

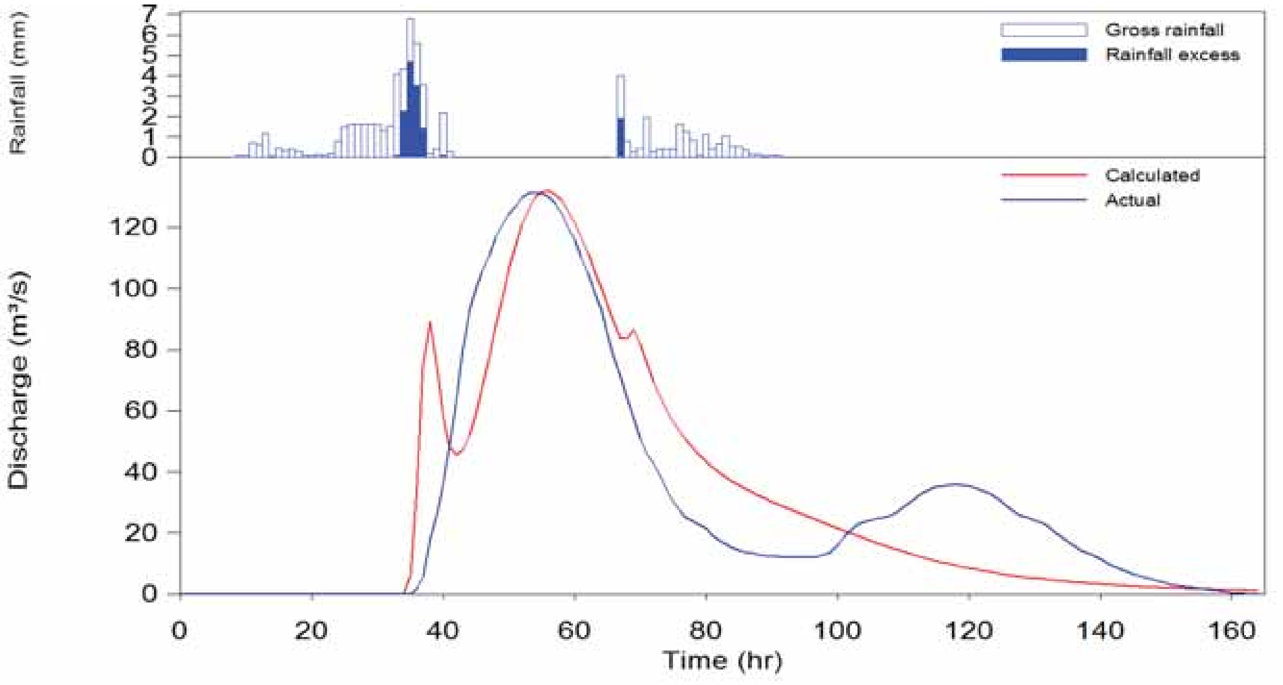

Figure 4.10 Calibration result for MACINTYRE02 model (Croppa Creek at Tulloona Bore) for April 1988 event

4.2.2.2 Return Period Estimates for Calibration Events

An FFA was undertaken on the stream gauge records to estimate the return period of the calibration events. As for the GWYDIR02 analysis, the FFA has been done on the water level records rather than the flow records. The results are provided in Table 4.7.

Table 4.7 Return period estimates for of calibration events for Croppa Creek at Tulloona Bore

Event	Return Period* (1 in Y years)
May 1983	6.4
July 1984	17.0
April 1988	3.9
*Based on 17-year period of record	

5 Design Event Modelling

5.1 Introduction

As demonstrated in the previous section, calibration could only be attempted for two out of the six RORB models developed for the project due to the lack of available stream gauge data. For the two models that were calibrated, the data only provided two reliable events of relatively low return period for the GWYDIR02 model (July 1998 and January 2004) and three events of relatively low return period for the MACINTYRE02 model.

It was therefore necessary to validate the RORB models by comparing their flow predictions to other methods of flow estimation. The RFFE method was selected for validation of the RORB models and the validation was undertaken by comparing design peak flow estimates generated by RORB to those generated by RFFE for a range of events.

This section outlines the RORB design event modelling approach and the results of sensitivity analyses carried out on key design modelling parameters. Full details of the design modelling approach, including detailed catchment maps of the cross drainage structures, is presented in the Flood Study Report (3-0001-260-IHY-00-RP-0002). The validation of the RORB models against RFFE is described in Section 6.

5.2 Adopted RORB Design Modelling Parameters

5.2.1 k_c Parameter

The flood routing parameter k_c is the principal parameter within RORB and is a function of catchment area, catchment non-linearity and discharge.

The RORB Manual (Monash University, 1990) suggests using Equation (1) below to set the starting k_c value for calibration:

$$k_c = 2.2A^{0.5}\left(\frac{Q_p}{2}\right)^{0.8-m} \quad (1)$$

which for *m*=0.8 reduces to:

$$k_c = 2.2A^{0.5} \quad (2)$$

In ARR2016, a number of empirical equations are provided for estimation of the k_c parameter based on studies on natural catchments in NSW (ARR2016, Book 7, Section 6.2.1). They include catchments with different sizes and at various locations across the state. The ARR2016 recommended method to estimate the k_c parameter for NSW catchments is given below in Equation (3).

$$k_c = 1.18A^{0.46} \quad (3)$$

From a review of the empirical equations developed for NSW catchments, it becomes evident that in all established relationships k_c is directly proportional to A^x, where the power x is close to 0.5 (A is the catchment area in km^2):

$$k_c \propto A^{0.5} \tag{4}$$

Equation (4) was used to estimate the k_c values for all the subject catchments, using the calibrated k_c value for the Croppa Creek RORB model.

This was done through implementation of Equation (5) to calculate the k_c value for any arbitrary catchment with area (A) using the Croppa Creek parameters.

$$\frac{k_c}{(k_c)_{Cr}} = \left(\frac{A}{A_{Cr}}\right) \tag{5}$$

where:

$A_{Cr} = 1834.3\ km^2$

$(k_c)_{Cr} = 67$

are the catchment area and the calibrated k_c value for the Croppa Creek catchment, respectively. Implementation of Equation (5) over Equations (2) and (3) has the advantage of reflecting the local hydrologic characteristics of the area through implementation of the calibrated Croppa Creek parameters, which provide a better representative of site-specific hydrologic conditions.

The k_c values adopted in the RORB models are summarised in Table 5.1.

Table 5.1 Adopted k_c values in design event RORB models

RORB Model	Total catchment area (km²)	Adopted k_c value	Notes
NAMOI01	415.4	31.9	Calculated from Eq.(5)
GWYDIR01	1264.9	55.6	Calculated from Eq.(5)
GWYDIR02	2537.0	78.8	Calculated from Eq.(5)
GWYDIR03	153.9	19.4	Calculated from Eq.(5)
MACINTYRE01	703.1	41.4	Calculated from Eq.(5)
MACINTYRE02	1834.3	67.0	From calibration

It should be noted that the estimated k_c value for the GWYDIR02 catchment using this method is close to the values used in the July 1998 and January 2004 event calibrations of the GWYDIR02 / Tycannah Creek model. It is also in close agreement with the calibrated value for this creek adopted in the Gwydir Floodplain Management Plan (OEH, 2016). Validation of the adopted k_c values is discussed further in Section 6.4.

5.2.2 Initial and Continuing Losses

Rainfall losses for each model were generated from the ARR2016 datahub website. These are given in Table 5.2 and compared to the loss values adopted for calibration of the GWYDIR02 and MACINTYRE02 models.

Table 5.2 Adopted initial and continuing loss values in design event RORB models

RORB Model	Design model initial loss (mm)	Calibration model initial loss (mm)	Design model continuing loss (mm)	Calibration model continuing loss (mm)
NAMOI01	42	Model not calibrated	0.8	Model not calibrated
GWYDIR01	57	Model not calibrated	0.2	Model not calibrated
GWYDIR02	56	20 to 30	0.4	0.61 to 11.07
GWYDIR03	54	Model not calibrated	0.1	Model not calibrated
MACINTYRE01	52	Model not calibrated	0.3	Model not calibrated
MACINTYRE02	58	15 to 27	0.1	0.04 to 2.1

The table demonstrates that the initial loss values used in the design models are considerably higher than those used in the calibration models. This is discussed further in Section 5.4.2.

5.3 RORB Design Modelling Methodology

The RORB design modelling method utilised the ensemble event approach as described in ARR2016, Book 4, Chapter 3 and shown in Figure 5.1. Each flood event was run for a range of standard durations and for an ensemble of 10 temporal patterns within each duration. Results were extracted for the critical flow at each culvert crossing separately and the median of these flows was selected as the design flow for each AEP event.

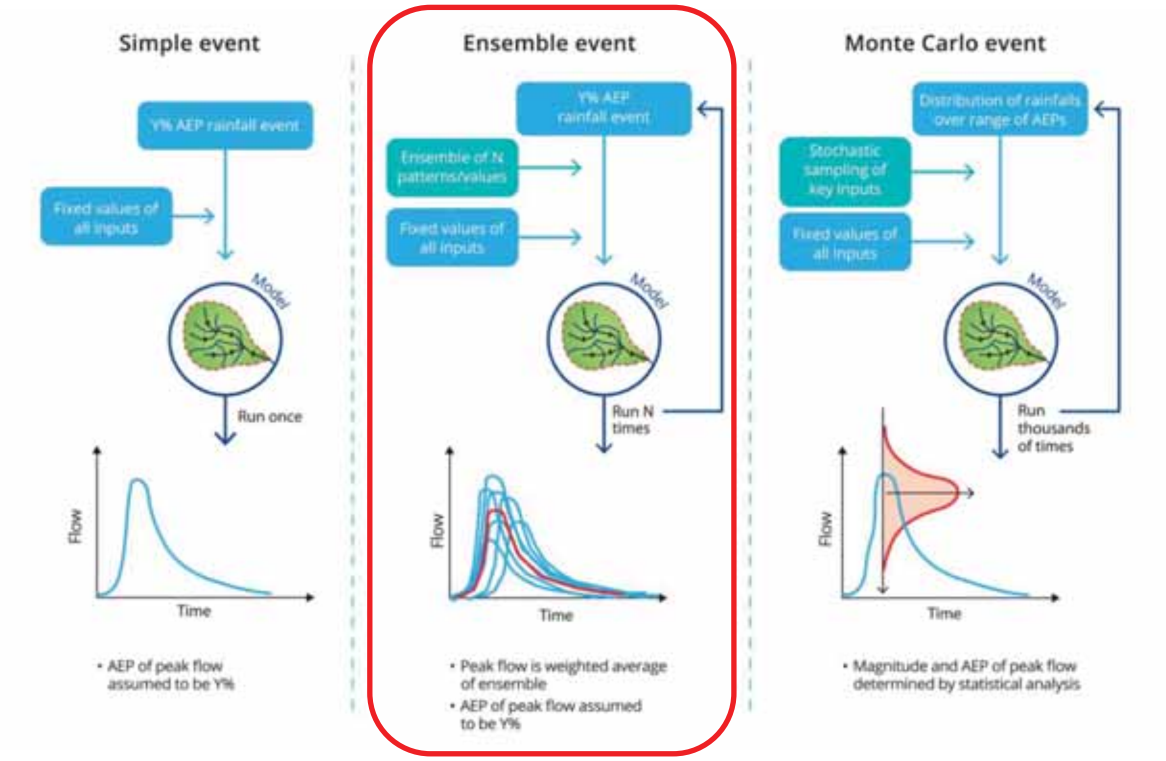

Figure 5.1 ARR2016 approaches to estimation of peak flow

Source: ARR design guidelines Book 4 Chapter 3 (ARR 2016) http://book.arr.org.au.s3-website-ap-southeast-2.amazonaws.com/

The design modelling scenarios for RORB were set up using the software program Storm Injector (Catchment Simulation Solutions, 2018). Storm Injector sets up appropriate combinations of storm durations, Areal Reduction Factors (ARFs) and point and areal temporal patterns and for input to RORB. Table 5.3 provides the key inputs to the RORB model that were set up within Storm Injector based on the variable upstream catchment size to each rail cross drainage culvert. In addition to those given in Table 5.3, the following key inputs were also provided to RORB / Storm Injector:

- 2016 Intensity-Frequency-Duration design rainfalls: obtained from Bureau of Meteorology website;
- Initial and continuing losses and pre-burst depths: obtained from the ARR2016 data hub; and
- k_c parameter: as per Section 5.2.1.

Table 5.3 Key hydrological inputs to RORB / Storm Injector

Upstream catchment size	Storm duration	Areal Reduction Factor (ARF)	Temporal Pattern
<1 km^2	All durations	ARF = 1 (as per ARR2016 Book 2, Chapter 4, Table 2.4.1)	Point temporal patterns for all catchments < 75km^2 (as per ARR2016 Book 2, Chapter 5, Section 5.9.1)
1 to 10 km^2	All durations	ARF = 1 (based on calculations as per ARR2016 Book 2, Chapter 4, Table 2.4.1 which produced values very close to 1 in all cases)	Point temporal patterns for all catchments < 75km^2 (as per ARR2016 Book 2, Chapter 5, Section 5.9.1)
10 to 75 km^2	All durations	ARF varies (calculated by Storm Injector as per ARR2016 Book 2, Chapter 4, Table 2.4.1)	Point temporal patterns for all catchments < 75km^2 (as per ARR2016 Book 2, Chapter 5, Section 5.9.1)
>75 km^2	< 12 hours	ARF varies (calculated by Storm Injector as per ARR2016 Book 2, Chapter 4, Table 2.4.1)	Point temporal patterns were adopted for < 12 hour duration storms as ARR2016 has not produced areal temporal patterns for these durations. There is no guidance for this case in ARR2016.
	=/> 12 hours	ARF varies (calculated by Storm Injector as per ARR2016 Book 2, Chapter 4, Table 2.4.1)	As per ARR2016 Book 2, Chapter 5, Section 5.6.3 different areal temporal patterns were used between: - 75km^2 – 150km^2 - 150km^2 – 350km^2 - 350km^2 – 750km^2 - 750km^2 – 1750km^2 There were no catchments in the project >1750km^2.

The RORB models were set up and run separately for each culvert using the inputs in Table 5.3 for the ensemble suite of temporal patterns. At each culvert, the critical duration and temporal pattern for that culvert was determined as follows:

- The critical temporal pattern was selected as the ‘first above median’ from the set of temporal patterns for every duration separately; and
- The maximum in any duration was selected (from the set of ‘first above medians’ determined above) to find the critical duration (and corresponding critical temporal pattern).

The output from this process was the critical duration and temporal pattern for every individual culvert with the associated critical flow for a range of return periods (AEPs).

5.4 RORB Design Model Sensitivity Tests

Sensitivity tests were undertaken for two key RORB parameters that have most influence on the flow estimates: the k_c parameter and the initial loss value. The tests and results are described in the following sections.

5.4.1 k_c Parameter

The design models have adopted k_c values determined from the RORB model calibration. This parameter represents the amount of storage in the catchment. Low values of k_c reduce the amount of storage and increase the peak flow in the runoff hydrograph. ARR2016 Book 7, Chapter 6, Section 6.2.1 provides a method for calculating k_c using the following formula:

$k_c = 1.18 \times A^{0.46}$ (where A = catchment area in km^2)

Table 5.4 provides the values of k_c used in the design models compared to those calculated using ARR2016.

Table 5.4 k_c parameter used in design and calculated from ARR2016

Model	k_c adopted in design models	k_c calculated using ARR2016
NAMOI01	31.9	18.9
GWYDIR01	55.6	31.5
GWYDIR02	78.8	43.4
GWYDIR03	19.4	12.0
MACINTYRE01	41.4	24.1
MACINTYRE02	67.0	37.4

A sensitivity test was undertaken to assess the impact of the default ARR2016 values of k_c on peak flow. The results are provided in Table 5.5 for the 1% AEP event for a selection of the largest rail cross drainage sub-catchments within each model area.

Table 5.5 Results of sensitivity analysis of k_c parameter

Model	Sub-catchment	1% AEP peak flow using design model k_c values	1% AEP peak flow using k_c values calculated from ARR2016	Difference
NAMOI01	582.605	300	391	31%
	586.200	473	651	38%
GWYDIR01	600.500	391	556	42%
	614.650	196	266	36%
GWYDIR02	627.340	316	403	28%
	641.540	1,019	1,478	45%
	647.605	1,530	2,048	34%
	660.610	222	307	38%
GWYDIR03	699.880	98	124	27%
MACINTYRE01	716.850	611	848	39%
	721.030	47	71	52%
MACINTYRE02	735.115	1,324	1,818	37%
	740.665	605	937	55%

The results show that using the default ARR2016 values for k_c increases peak flows by 27 to 55% for the 1% AEP event. This confirms that the impact of using the default values is to significantly increase flows. As the k_c parameter is established through calibration, the current adopted values for design will continue to be used rather than the ARR2016 default values.

5.4.2 Initial Loss

Table 5.6 provides the initial losses used in the RORB design hydrology models, which are default values obtained from ARR2016, and those used in the model calibration. The table shows that the values used in design are significantly higher than those used in calibration.

Table 5.6 Initial losses used in design and calibration

Model	Initial loss – design (mm)	Initial loss – calibration (mm)
NAMOI01	42	Model not calibrated
GWYDIR01	57	Model not calibrated
GWYDIR02	56	20 to 30
GWYDIR03	54	Model not calibrated
MACINTYRE01	52	Model not calibrated
MACINTYRE02	58	15 to 27

If the calibration initial loss values were adopted for design, then design flows may be considerably increased. A sensitivity test has been undertaken on the initial loss values in the RORB design models by setting the values to 25mm across all models. The resulting peak design flows are as shown in Table 5.7 for a selection of the largest rail cross drainage sub-catchments within each model area.

Table 5.7 Results of sensitivity analysis of initial loss

Model	Sub-catchment	1% AEP peak flow using design model initial losses (m^3/s)	1% AEP peak flow using initial loss of 25mm (m^3/s)	Difference
NAMOI01	582.605	300	330	10%
	586.200	473	519	10%
GWYDIR01	600.500	391	481	23%
	614.650	196	241	23%
GWYDIR02	627.340	316	367	17%
	641.540	1,019	1,261	24%
	647.605	1,530	1,628	6%
	660.610	222	250	13%
GWYDIR03	699.880	98	117	20%
MACINTYRE01	716.850	611	727	19%
	721.030	47	65	41%
MACINTYRE02	735.115	1,324	1,533	16%
	740.665	605	764	26%

The table shows that reducing the initial loss to 25mm has the effect of increasing the 1% AEP peak flows by 6 to 41% for the largest sub-catchments within the project area. For the three largest sub-catchments where the 1% AEP peak flows exceed 1,000m^3/s, the increases are 6 to 24%. Such increases are likely to increase design flood levels at the rail corridor significantly.

Selection of the appropriate initial loss value is covered in ARR2016 Book 5, Chapter 3. The relevant section is reproduced below:

3.3.3.1. At-site Event Data

If there is long-term pluviograph and streamflow data available at the site of interest it may be possible to directly estimate loss values for a number of events. In order to undertake such an analysis the streamflow should be free of significant regulation or diversion and land use within the catchment should be stationary over the period of data being analysed.

The events to be analysed should be selected carefully to ensure that the sample of events is not biased. The selection of high runoff events for loss derivation is likely to be biased towards wet antecedent conditions (ie. losses tend to be too low). Ideally, events should be selected on the basis of rainfall to remove this bias. However the selection and analysis of events by rainfall is problematic because it requires consideration of a representative duration of the rainfall and there may be little or no runoff generated from some intense bursts of rainfall if the antecedent conditions are dry.

The main limitation of deriving losses directly from the analysis of recorded data is that they may not be compatible with the other design inputs and hence suitable for design flood estimation. That is, although the loss values may reflect the loss response observed for a number of events on the catchment, this does not guarantee that their application with other design inputs results in unbiased estimates of floods. For this reason, it is also desirable to reconcile design values with independent flood frequency estimates where possible (refer **Section 3.3.3.3**).

Figure 5.2 Excerpt from ARR2016 Book 5, Chapter 3 relating to initial loss

This guidance recommends selection of initial loss values that are not biased towards low values obtained from calibration to events that had wet antecedent conditions in the catchment.

Analysis of the rainfall data used in the RORB model calibration found significant rainfall occurred in the days or weeks preceding the peak of the events, as demonstrated in Table 5.8 below.

Table 5.8 Antecedent rainfall for calibration events

Event	Average* total antecedent rainfall prior to start of RORB model simulation (mm)	Number of days of preceding rainfall prior to start of RORB model simulation
July 1998	76.3	16
January 2004	72.4	6
May 1983	57.2	6
July 1984	44.9	3
April 1988	57.2	1
*Total rainfall averaged across all relevant rain gauges		

This explains the low initial loss values used for calibration of the RORB models. Rather than adopt the low initial loss values from calibration, the design models have used the default ARR2016 initial loss values, which is consistent with the guidance provided in ARR2016 Book 5, Chapter 3, Section 3.3.3.1 reproduced above.

6 RORB Model Validation

6.1 Introduction

This section describes the validation of the RORB design models against the RFFE method and against other flood studies undertaken in the region. The validation against RFFE was done by comparing the RORB model peak flow estimates at the existing rail cross drainage structures to those produced by the RFFE method. A further check against the Phase 2 hydrology study (GHD, 2017) was also undertaken, however, given the limitations of the Phase 2 study, this comparison is of lesser importance.

6.2 Design Flow Estimate Comparisons with RFFE

6.2.1 Comparison of RORB Design Flows to RFFE at Gauge Locations

To undertake a general consistency check on the RORB design models, the peak flow predictions from the models were checked against RFFE flow estimates at the gauge locations. The results are provided in Table 6.1 below.

Table 6.1 Comparison of RORB design model peak flows against RFFE at gauge locations used for calibration

Gauge	Average of peak flow estimates (m^3/s)							
	1% AEP		2% AEP		5% AEP		10% AEP	
	RFFE Expected Value	RORB design model	RFFE Expected Value	RORB design model	RFFE Expected Value	RORB design model	RFFE Expected Value	RORB design model
Tycannah Creek at Horseshoe Lagoon	2,080	2,069	1,510	1,673	940	1,160	620	912
Croppa Creek at Tulloona Bore	2,370	1,824	1,720	1,587	1,070	1,250	710	1,009

For the 1 and 2% AEP events the peak flows agree reasonably well, with RORB producing peak flows between 11% higher to 23% lower than RFFE. The methods diverge for the lower events, with RORB producing peak flows between 17 and 23% higher than RFFE for the 5% AEP event and between 42 and 47% higher than RFFE for the 10% AEP event. The check is considered to indicate reasonable agreement between the methods for the higher events. Further discussion on the comparison between RORB and RFFE is provided in the next section.

6.2.2 Comparison of RORB Design Flows to RFFE at Existing Rail Cross Drainage Structures

Comparisons of the design models to RFFE peak flow estimates at the existing rail cross drainage structures were made. The results of these comparisons are provided in Appendix C. Table 6.2 provides a summary of the comparisons by catchment area range for a range of events. Table 6.3 summarises the variance between the two flow datasets.

Table 6.2 Summary of average peak flows determined from RFFE and RORB models for different catchment sizes

Catchment Area Range (km^2)	Average of peak flow estimates (m^3/s)							
	1% AEP		2% AEP		5% AEP		10% AEP	
	RFFE Expected Value	RORB design model	RFFE Expected Value	RORB design model	RFFE Expected Value	RORB design model	RFFE Expected Value	RORB design model
>100 (9 catchments)	832.0	729.7	606.0	592.2	377.3	425.3	249.5	321.4
50 to 100 (3 catchments)	204.0	187.6	148.7	153.2	92.6	119.5	61.4	93.4
10 to 50 (15 catchments)	83.7	86.0	61.4	70.0	38.4	55.5	25.4	43.6
1 to 10 (59 catchments)	18.3	24.6	13.1	19.0	8.2	14.0	5.5	11.0
<1 (71 catchments)	3.3	4.0	2.4	3.2	1.5	2.1	1.0	1.6

Table 6.3 Variance between RFFE and RORB model peak flow estimates

AEP	Variance (RORB/RFFE)
1%	96%
2%	106%
5%	125%
10%	144%

The results show a reasonable agreement between the RORB models and the RFFE. The highest variance occurs for the lower order events (5 and 10% AEP) for which the RORB models predict significantly higher peak flows.

A review of the raw data that was used to generate the RFFE flows was undertaken. The RFFE uses a flood frequency estimation model to extrapolate flows from the closest 15 gauged catchments. The accuracy of the model depends on similarity between the gauged catchments and the local catchment, and is considered to have limited accuracy where the subject catchment parameters are not well reflected by the RFFE dataset.

Figure 6.1 provides a of an example catchment within the N2NS project area (marked C) compared to the gauged catchments used by RFFE (marked 1 to 15). It is noted that the distance between the gauged catchments from the project area range from 76km to 161km with an average of 127km and are located close to the Great Dividing Range. This would indicate that there may be poor representation of the N2NS project area within the RFFE model.

Figure 6.1 N2NS catchment locations in relation to RFFE gauged catchments

It is noted that the areas of the RFFE gauged catchment dataset range between 156km^2 and 970km^2 with no representation of smaller catchments. Figure 6.2 provides a graph of catchment areas for the RFFE gauged catchment dataset.

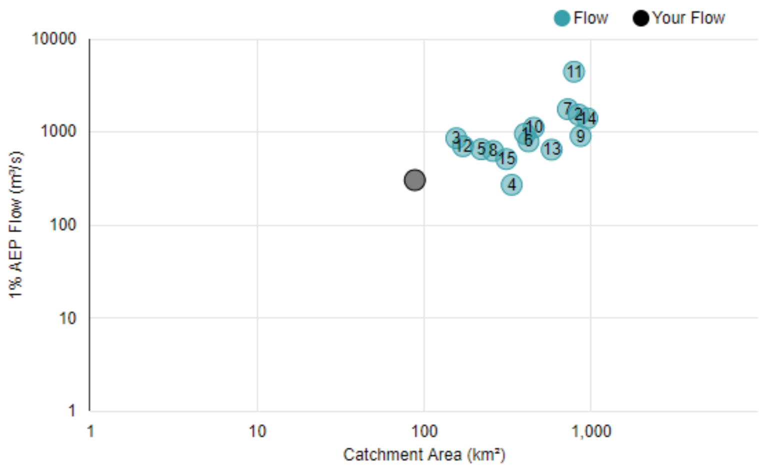

Figure 6.2 1% AEP flow vs catchment area for RFFE gauged catchments

As larger catchments have increased storage, it is expected that a greater attenuation effect (and thus lower peak flows) would be apparent in lower AEP events. As no smaller catchments are represented in the RFFE dataset, it would be expected to see underestimation in the RFFE results of peak flows in low AEP events but not in the higher AEP events.

It is also noted that the RORB models were calibrated against a range of lower order storms, which would indicate higher accuracy in low order flood event flow estimates produced by RORB than the RFFE.

In summary, the RORB models are considered to compare well with the RFFE flow estimates for the higher order events (1% and 2% AEP) and to be more accurate than the RFFE in predicting flows for lower order events (5% and 10% AEP).

6.2.3 Assessment of Reliability of RFFE in the N2NS Project Area

Additional data was extracted from the RFFE web-based tool to check key parameters at the RFFE data points and how these compare to those for the rail sub-catchments in the N2NS project area. Key data has been extracted from the RFFE website for 7 of the RFFE data points that are located closest to the N2NS project area. The data is provided in Table 6.4 below.

Table 6.4 RFFE data at selected data points closest to N2NS project area

Site ID	Distance to Croppa Creek gauge (km)	Distance to Tycannah Creek gauge (km)	Catchment Area (km^2)	Mean Annual Rainfall (mm)	Shape Factor	Bias Correction Factor	1% AEP Flow Estimate (m^3/s)
416020	71.4	85.2	402	775	0.81	-0.541	955
416305	88.9	142.1	335	686	0.70	-0.923	271
416312	102.1	144.0	422	750	0.70	-0.916	792
418017	106.8	54.1	842	742	0.76	0.420	1,537
418025	120.7	59.3	156	772	0.77	-0.505	856
418027	145.5	70.8	220	805	0.98	1.030	652
419051	174.48	92.4	454	656	0.96	-1.335	1,112

Table 6.5 provides the same data from the RFFE calculations at a selection of N2NS rail sub-catchments of comparable size to those at the RFFE data points.

Table 6.5 RFFE data at selected N2NS rail sub-catchments

Sub-catchment	RORB model	Catchment Area (km^2)	Mean Annual Rainfall (mm)	Shape Factor	Bias Correction Factor	1% AEP Flow Estimate (m^3/s)
586.200	NAMOI01	143	560	1.27	-0.406	369
600.500	GWYDIR01	229	560	0.96	-0.307	528
735.115	MACINTYRE02	721	620	0.95	-0.311	1,740
740.665	MACINTYRE02	281	620	1.15	-0.325	736

The tables show that the RFFE flow estimates compare reasonably well when taking into account the significantly lower rainfall that occurs in the N2NS project area and the shape factors, with the highest variance between flow estimates occurring for shape factors that differ significantly, e.g.

- The 1% AEP flow estimate for RFFE site ID 418017 (catchment area 842 km^2, mean annual rainfall 742 mm) is significantly lower than that for sub-catchment 735.115 (catchment area 721 km^2, mean annual rainfall 620 mm), which is a counter-intuitive result. However, the shape factors for both data points differ significantly (0.76 and 0.95); and
- The 1% AEP flow estimate for RFFE site ID 418027 (catchment area 220 km^2, mean annual rainfall 805 mm) is comparable to that for sub-catchment 600.500 (catchment area 229 km^2, mean annual rainfall 560 mm). The shape factors for both data points are very similar in this example (0.98 and 0.96).

The above assessment suggests that RFFE produces consistent results between the area containing the nearest RFFE data points and the N2NS project area, considering the variability of rainfall between the two areas, in cases where catchment shape factors are similar. The overall conclusion is that RFFE produces reasonable results for the N2NS project area even though the RFFE data points are located on average 127km east of the N2NS project area.

6.2.4 Sensitivity Analysis at Selected Rail Crossings

The largest discrepancy between RORB and RFFE occurs at:

- Structure 600.5 in the GWYDIR01 model, where the RORB 1% AEP flow is 74% of the RFFE value; and
- Structure 735.12 in the MACINTYRE02 model, where the RORB 1% AEP flow is 76% of the RFFE value.

Hydraulic model sensitivity tests were undertaken for these structures which involved factoring up the inflows by 1.333 to match the peak flows predicted by RFFE and then checking the sensitivity of the model predictions of flood level to flow at the structure locations. The results are provided in the table below.

Table 6.6 Results of flow sensitivity analysis at selected cross drainage structures

Structure chainage	Existing conditions 1% AEP event peak flood level at structure		1% AEP peak flood level difference (mm)	Current design bridge soffit level (mAHD)	Current design top of rail level (mAHD)
	Un-factored flows (mAHD)	Flows factored by 1.333 (mAHD)			
600.5	237.76	238.05	290	236.95	238.53
735.12	275.22	275.53	310	274.59	277.19

The table shows that increasing the flows by 33% increases the peak 1% AEP flood level at the structures by approximately 300mm. However, in both flow cases (un-factored and factored) and at both locations the bridge soffits are surcharged and the top of rail levels are not exceeded. Therefore, the increased flows do not change the hydraulic conditions at the bridge (surcharged deck occurs in both flow cases) and the top of rail remains well above the 1% AEP flood level. The impact of the increased flow would however result in a reduced flood immunity to the top of formation away from the bridges.

The results show that in areas where the highest discrepancies between RORB and RFFE occur, the flood risk to the crossing structures and the rail is not significantly increased if the higher RFFE flows were adopted.

6.3 Design Flow Estimate Comparisons with Phase 2 Hydrology Study

Comparisons of the RORB model flow estimates were also made to those from the Phase 2 hydrology study (GHD, 2017), which was based limited RORB modelling and the Probabilistic Rational Method, which is no longer recommended by ARR2016. The results of these comparisons are provided in Appendix C. Table 6.7 provides a summary of the comparisons by catchment area range for a range of events.

Table 6.7 Summary of average peak flows determined from Phase 2 hydrology study and RORB models for different catchment sizes

Catchment Area Range (km^2)	Average of peak flow estimates (m^3/s)							
	1% AEP		2% AEP		5% AEP		10% AEP	
	Phase 2	RORB design model	Phase 2	RORB design model	Phase 2	RORB design model	Phase 2	RORB design model
>100 (9 catchments)	573.6	729.7	405.9	592.2	213.3	425.3	128.2	321.4
50 to 100 (3 catchments)	207.3	187.6	146.5	153.2	76.6	119.5	45.9	93.4
10 to 50 (15 catchments)	87.5	86.0	61.7	70.0	32.2	55.5	19.2	43.6
1 to 10 (59 catchments)	20.9	24.6	14.7	19.0	7.6	14.0	4.5	11.0
<1 (71 catchments)	1.4	4.0	1.0	3.2	0.5	2.1	0.3	1.6

The table demonstrates that the RORB models produce significantly higher peak flow estimates in most cases, with the difference increasing with event probability. Close agreement between the two studies was not expected given the limitations of the Phase 2 study, however, the methods produce reasonably consistent results for the 1% and 2% AEP events, with the RORB models producing more conservative flow predictions.

6.4 Validation of k_c Values and Design Flow Estimate Comparisons with Other Studies

The k_c values adopted in the RORB design models were generated by applying the calibrated k_c value at Croppa Creek within the MACINTYRE02 RORB model to the other RORB models using a catchment area weighting method (refer to Section 5.2.1). This approach produced significantly higher k_c values, and lower flows, than would be obtained if the standard ARR2016 equation was used, as described in Section 5.4.1. Also, as described in the previous section, the RORB models produce lower peak flow estimates that the RFFE method for the 1% AEP event and for some of the larger catchments crossed by the rail corridor. The analyses show that the method used to set the k_c values across the project produces lower flows for the higher events within the larger catchments when compared against the standard ARR2016 equation for estimating k_c and the RFFE method. Therefore, the following further checks were undertaken to provide confidence in the RORB model k_c values and design flow estimates.

While the k_c values were established from the limited calibration of the Croppa Creek model, the following additional independent checks validate the approach and the adopted values:

- When the value of k_c obtained from the Croppa Creek model calibration was applied to the Tycannah Creek model (located within the Gwydir system near Moree) using the catchment area weighting approach, the k_c value obtained for Tycannah Creek (79) was close to the k_c values used to calibrate the Tycannah Creek RORB model to the two most reliable recorded flow events records (96 for the July 1998 event and 70 for the January 2004 event; and
- The adopted k_c value of 79 for the Tycannah Creek system (and surrounding GWYDIR02 model) is also close to the value of 71 used by OEH in their separate RORB model developed for the Gwydir Floodplain Management Plan (OEH, 2016).

It should be noted that the k_c values mentioned above are significantly higher than the value of 43 that would be obtained when applying the standard equation from ARR2016. Sensitivity tests show that if the k_c value of

43 was used then the peak 1% AEP flows would be increased by 28 to 45% for the 4 largest sub-catchments in the GWYDIR02 model (see Section 5.4.1).

A further check was undertaken against the Moree Regional Flood Model (WRM, 2017), which is the only other major flood study undertaken in the region. As described in Section 2.1.1, the hydrological model developed for the flood study was a combination of FFA for the main river inflow and a RAFTS model covering the local catchments around Moree. The RAFTS model of the local catchments included the Tycannah Creek and Halls Creek systems south of Moree, both of which are also represented in the GWYDIR02 RORB model. The WRM RAFTS model was based on ARR1987 hydrological modelling methods and inputs. Comparisons of the RAFTS and RORB predictions of the 1% AEP peak flow were made at the 3 locations shown below in Figure 6.3 – at the confluence of Tycannah Creek and Berrygill Creek, at the location of flow gauge 418032 on Tycannah Creek and at the rail crossing of Halls Creek. The results of the comparison are provided in Table 6.8 below.

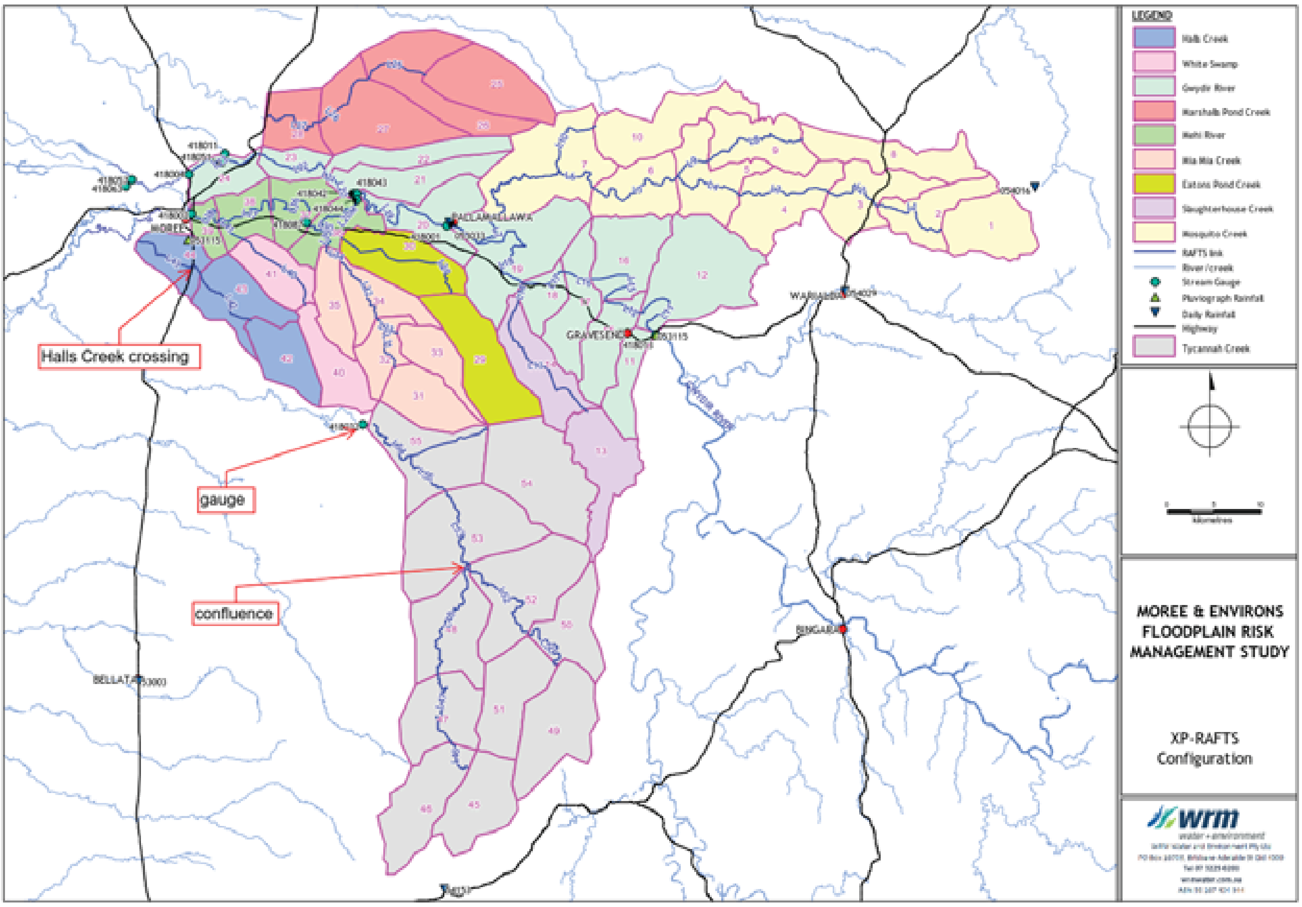

Figure 6.3 Overview of WRM RAFTS model and locations of comparison with GWDIR02 RORB model (figure sourced from WRM, 2017)

Table 6.8 Comparison of 1% AEP event peak flows and critical durations between RAFTS and RORB models

Location	WRM RAFTS Model (ARR1987)		N2NS RORB Model (ARR2016)		1% AEP Peak Flow Comparison (RORB/RAFTS)
	1% AEP Peak Flow (m^3/s)	1% AEP Critical Duration (minutes)	1% AEP Peak Flow (m^3/s)	1% AEP Critical Duration (minutes)	
Halls Creek crossing	125	1,080	222	720	178%
Tycannah Creek at Horseshoe Lagoon (gauge 418032)	1,138	1,080	1,509	1,080	133%
Confluence of Tycannah Creek and Berrygill Creek (near gauge 418028)	910	1,080	1,170	1,080	129%

The comparison shows that the RORB model produces significantly higher flows at all three locations. While some difference would be expected due to the different ARR versions used, this check provides confidence that the RORB models are not underestimating the 1% AEP event flows (as was suggested by the RFFE comparison in Section 6.2) when compared to the other major flood study undertaken in the region.

7 Conclusions

The Moree Regional Flood Model originally developed by Moree Plains Shire Council and further updated and adopted for Phase 2 of the project has been reviewed for adequacy for detailed design purposes. The review focussed on the hydrological modelling component of the regional model, including the calibration process, the FFA that defines the primary flow inputs and the local catchment model that contributes significant flow to the hydrodynamic model. The review concluded that the regional model is suitable for use in detailed design, with an update of the local catchment model to ARR2016 required to inform the flood impact assessment for the project.

Six new hydrological models were developed for the project to cover the project area outside the extent of the Moree Regional Flood Model. Due to the limited flow and rainfall datasets available in the region, calibration could only be attempted for two of these models. Calibration to observed flow records was attempted for Tycannah Creek in the Gwydir River Basin and Croppa Creek in the Border Rivers Basin. Good calibration was achieved for the Croppa Creek model. Good calibration was achieved for two out of four calibration events for Tycannah Creek; however, inconsistent parameters were obtained from the calibration of the two other higher flood events at this location, likely due to issues with the flow record not estimating the total floodplain flow for high order events. Overall, the calibration was considered reasonable. The calibration results were used to define design RORB model parameters for the other uncalibrated models using a catchment area based approach.

The peak design flow estimates generated by the RORB models were checked against estimates produced by the RFFE. The RORB models were found to compare well with the RFFE flow estimates for the higher order events (1% and 2% AEP) and to predict higher flows for lower order events (5% and 10% AEP). Limitations with the RFFE method suggest that the RORB models are more reliable for flow estimation for the lower order events.

The peak design flow estimates generated by the RORB models were also checked against estimates produced by the Phase 2 hydrology study. The RORB models were found to produce significantly higher peak flow estimates in most cases, with the difference increasing with event probability. Close agreement between the two studies was not expected given the limitations of the Phase 2 study, however, the methods produce reasonably consistent results for the 1% and 2% AEP events, with the RORB models producing more conservative flow predictions.

The peak design flow estimates for the 1% AEP event for the GWYDIR02 RORB model were also checked at three locations that overlap with the Moree Regional Flood Model RAFTS model. This comparison showed that the RORB model produces more conservative (i.e. higher) design flow estimates than the regional model.

Based on the calibration results and the checks against the RFFE and the Moree Regional Flood Model, the new RORB models developed for the project are considered to produce credible design flow estimates and to be a suitable basis for the flooding assessment and design of cross drainage structures.

8 References

- Australian Rainfall and Runoff: A Guide to Flood Estimation; Commonwealth of Australia; Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M, Testoni I, (Editors); 2016.
- Gwydir Floodplain Management Plan, Flood Modelling Report; Office of Environment & Heritage; May 2016.
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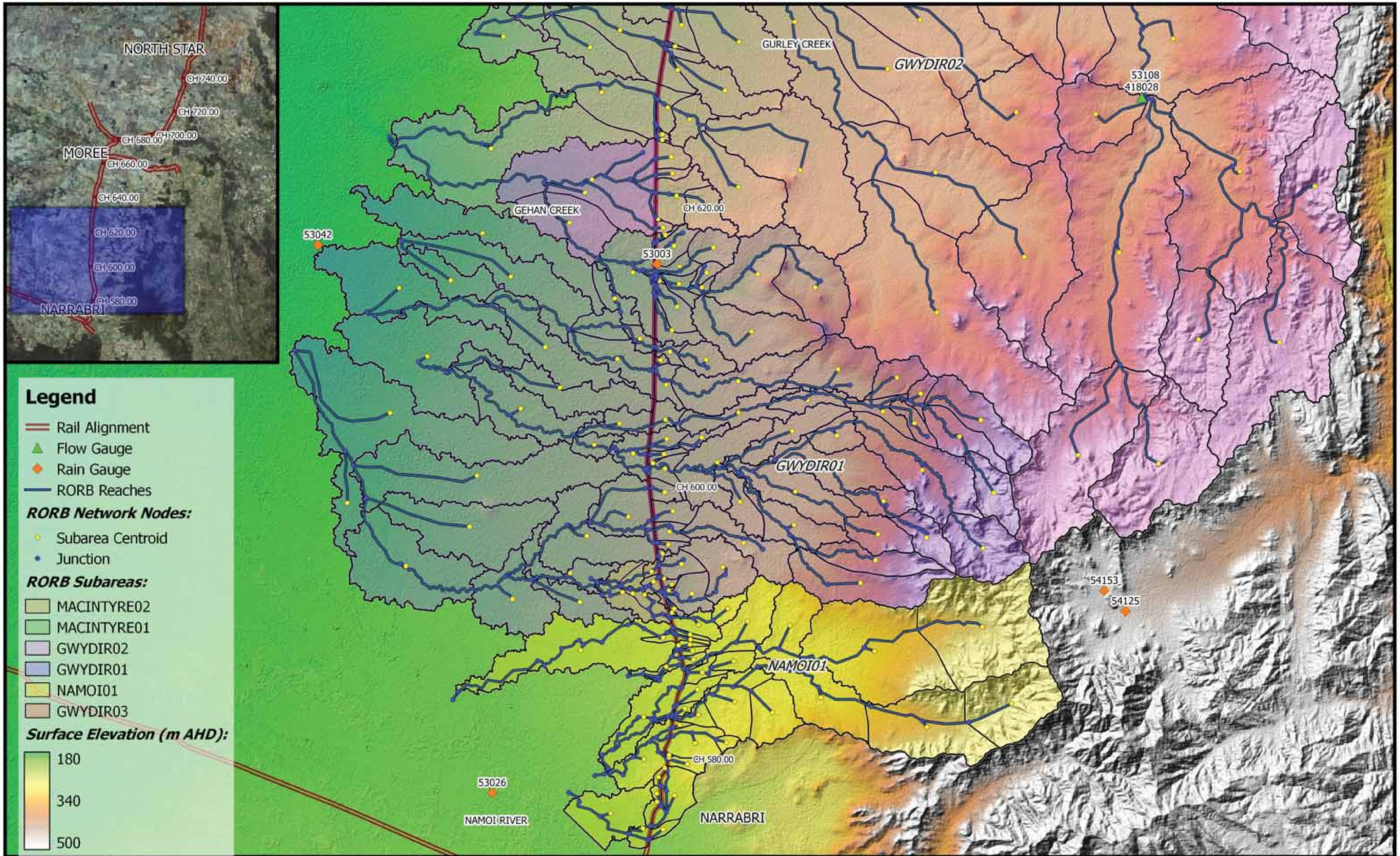
- RORB – Version 4, Runoff Routing Program: User Manual; Laurenson, EM and Mein, RG; Monash University Department of Civil Engineering; 1990.

Appendix A

N2NS RORB Model Layout Maps



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Coordinate system: GDA94 MGA Zone 55
Scale ratio correct when printed at A3

MOTT MACDONALD

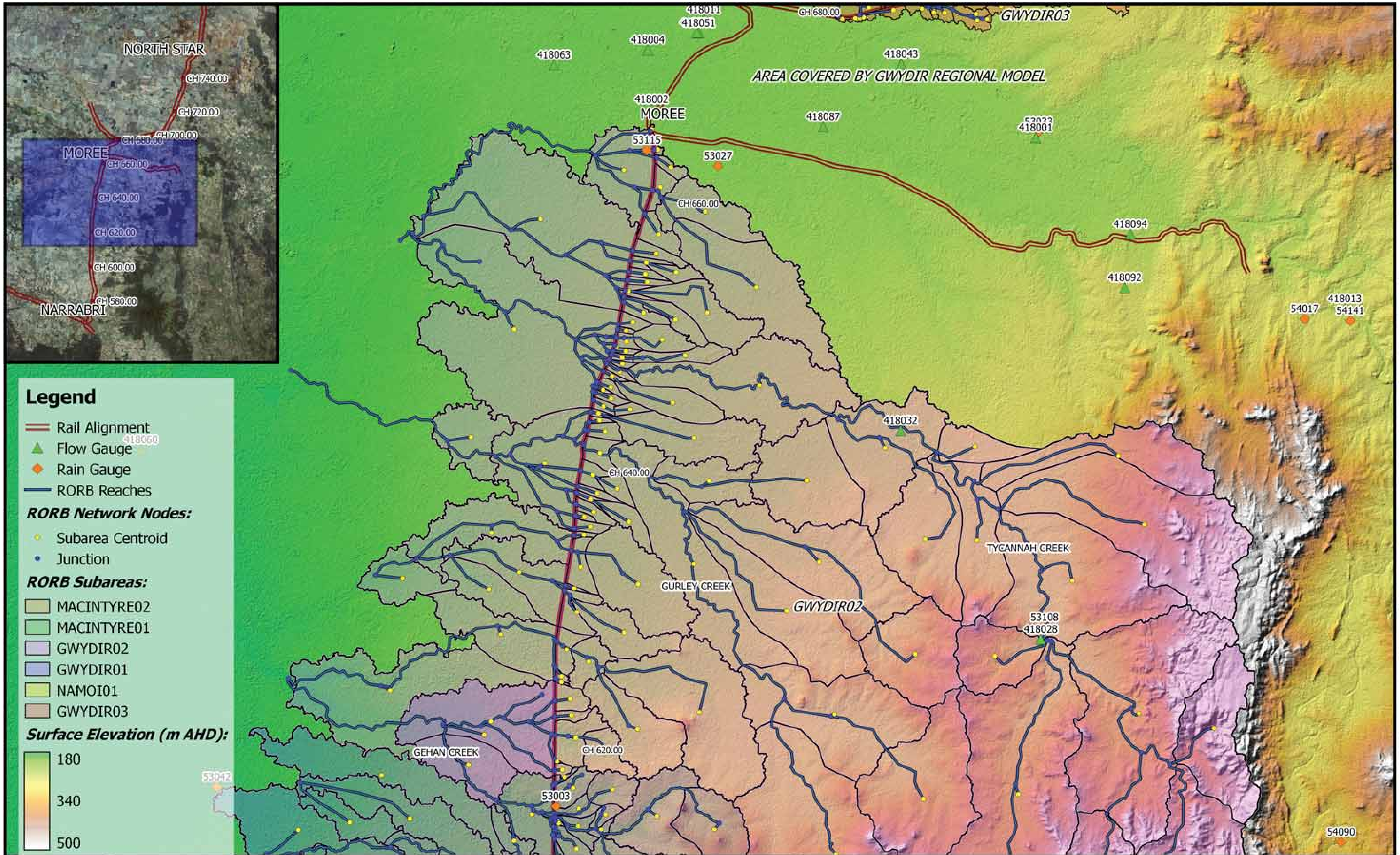
Date: 13/12/2018

Rev: B

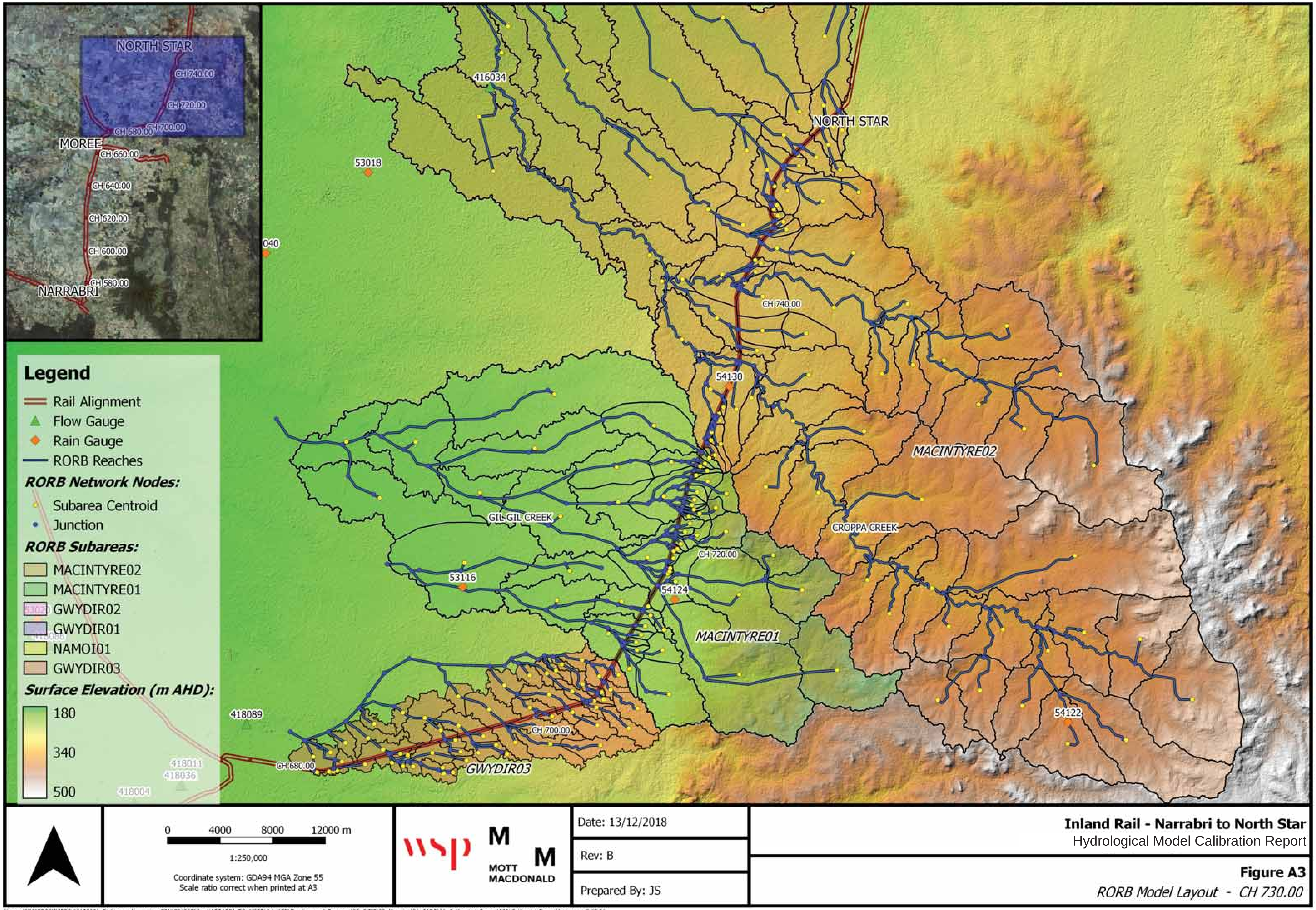
Prepared By: JS

Inland Rail - Narrabri to North Star
Hydrological Model Calibration Report

Figure A1
RORB Model Layout - CH 600.00



0 4000 8000 12000 m 1:250,000 Coordinate system: GDA94 MGA Zone 55 Scale ratio correct when printed at A3	wsp / MOTT MACDONALD	Date: 13/12/2018 Rev: B Prepared By: JS	**Inland Rail - Narrabri to North Star** Hydrological Model Calibration Report **Figure A2** *RORB Model Layout - CH 630.00*



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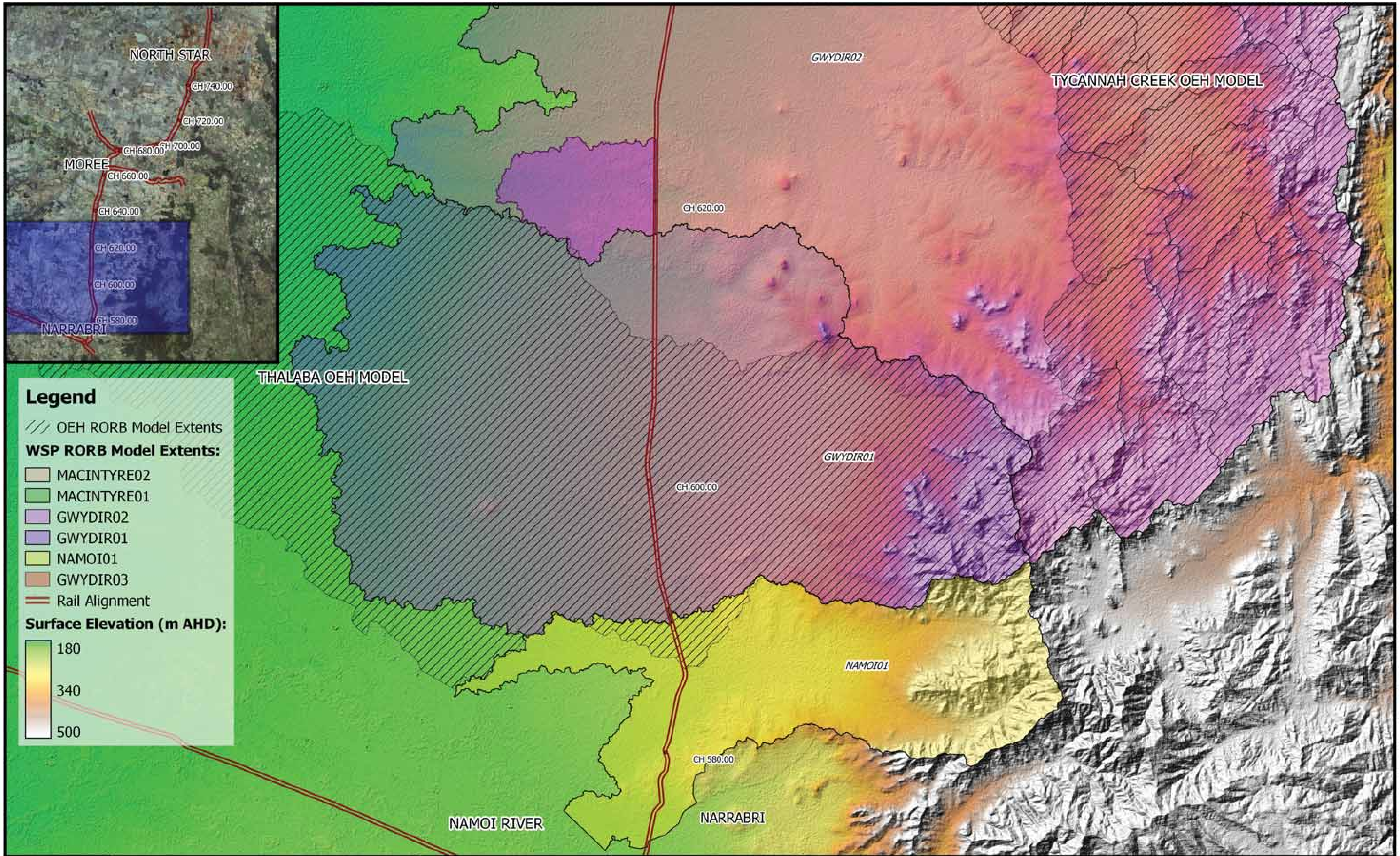
Hydrological Model Calibration Report

Figure A3

RORB Model Layout - CH 730.00

Appendix B

Comparison of OEH and N2NS RORB Model Extents



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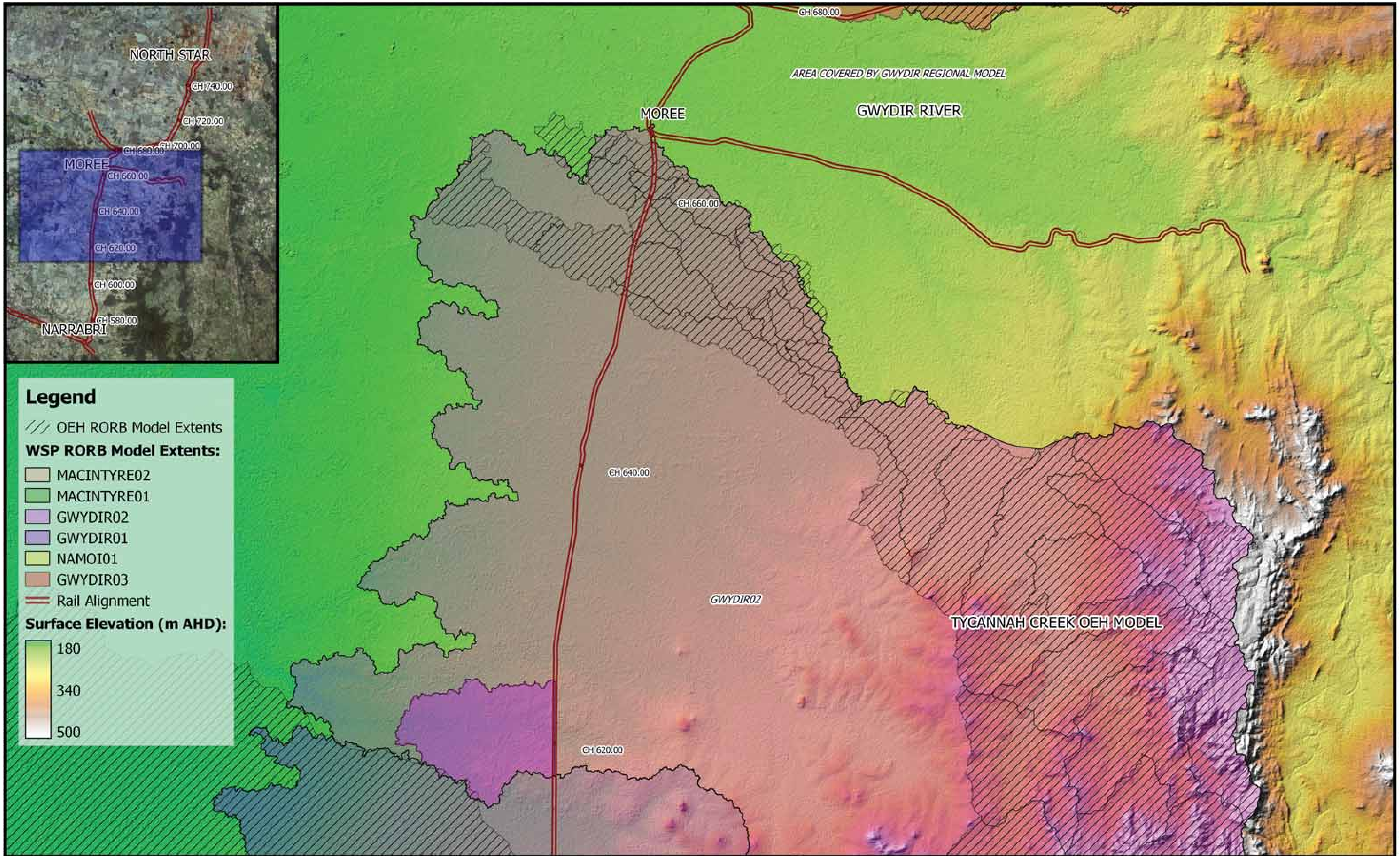
Hydrological Model Calibration Report

Figure B1
OEH RORB Model Layout Comparison - CH 600.00



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Coordinate system: GDA94 MGA Zone 55
Scale ratio correct when printed at A3

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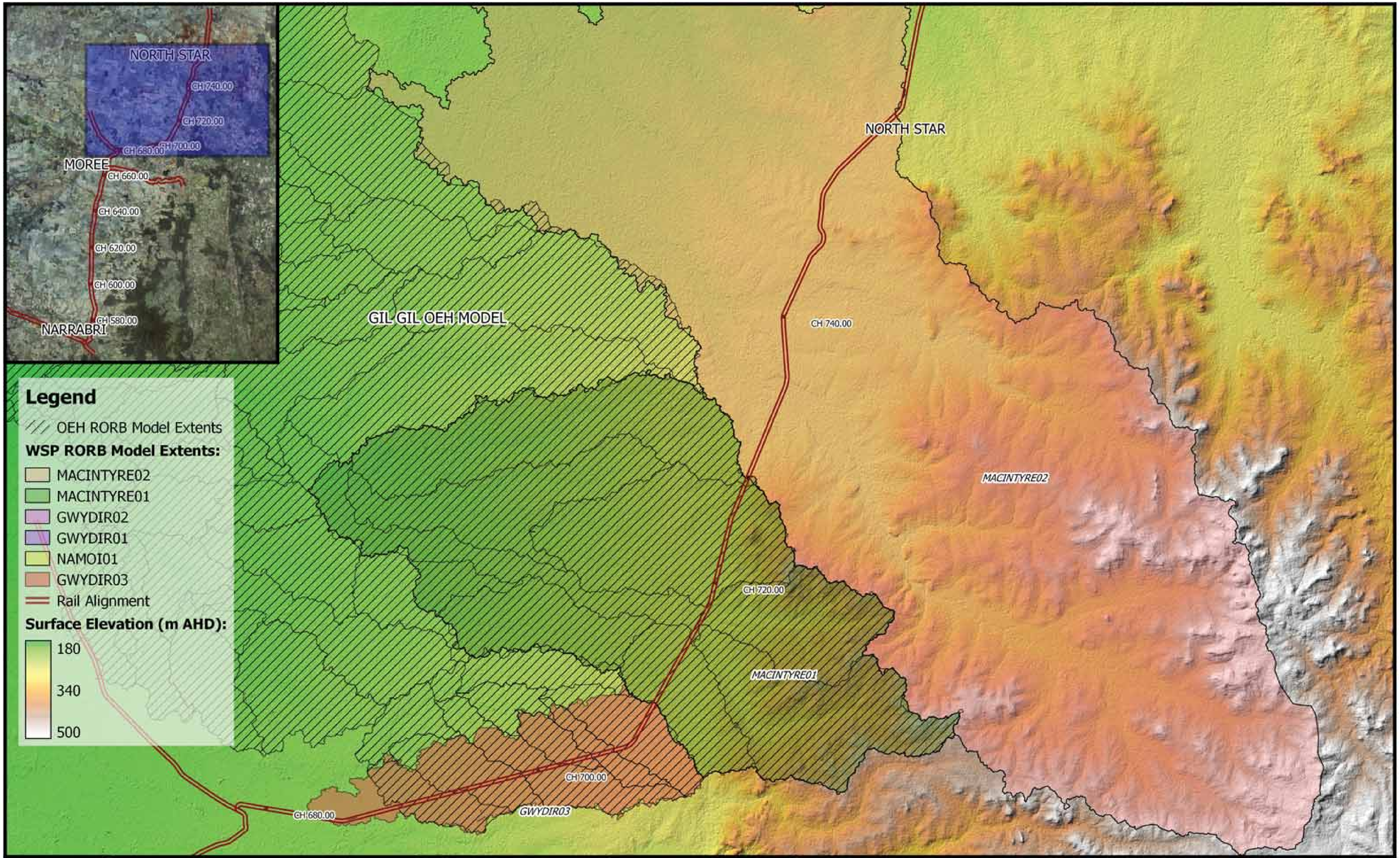
Inland Rail - Narrabri to North Star
Hydrological Model Calibration Report

Figure B2
OEH RORB Model Layout Comparison - CH 630.00

\\apsyd6f03\PROJ\I\IRDJV\2135661_Parkes_to_Narromine_P2N\PS106716__NARRABRI_TO_NORTH\4_WIP\Flooding_and_Drainage\03_QGIS\02_Mapping\01_30DD\01_Calibration_Report\30%ComparisonWithOEH.qgs - 2.18.21



0 4000 8000 12000 m

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Coordinate system: GDA94 MGA Zone 55
Scale ratio correct when printed at A3

wsp

MOTT MACDONALD

Date: 13/12/2018

Rev: B

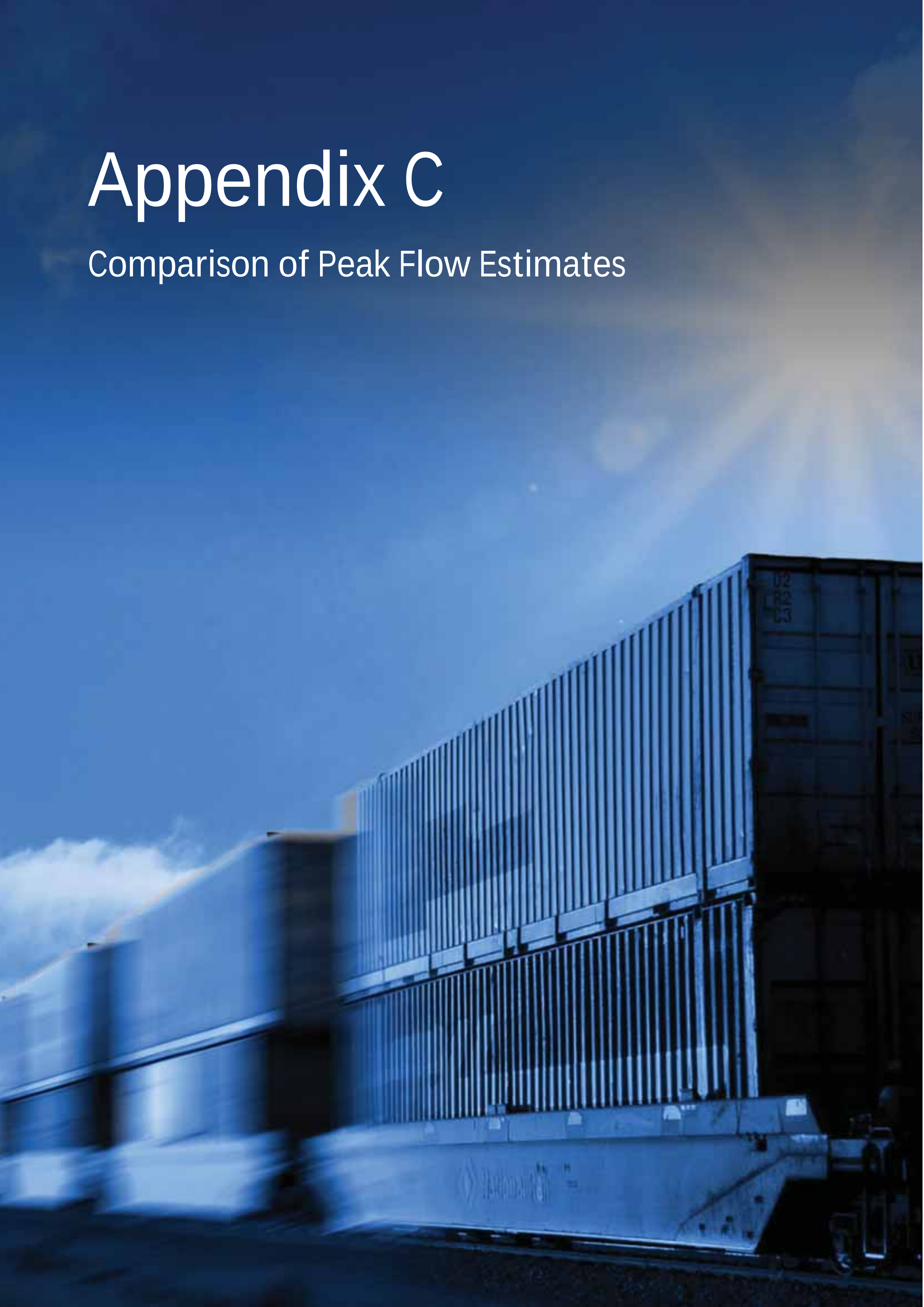
Prepared By: JS

Inland Rail - Narrabri to North Star
Hydrological Model Calibration Report

Figure B3
OEH RORB Model Layout Comparison - CH 730.00

Appendix C

Comparison of Peak Flow Estimates



NAMOI01 MODEL

Cross Drainage Catchment Chainage	IRDJV Catchment Area (km^2)	Phase 2 Chainage	Phase 2 Catchment Area (km^2)	Peak Flow Estimates (m^3/s)											
				1% AEP			2% AEP			5% AEP			10% AEP		
				RORB	RFFE	Phase 2	RORB	RFFE	Phase 2	RORB	RFFE	Phase 2	RORB	RFFE	Phase 2
573.360	1.513	573.360	0.051	13.6	8.2	0.8	10.6	6.1	0.6	7.8	3.8	0.3	6.0	2.6	0.2
574.375	8.888	574.375	0.009	70.1	37.3	0.2	59.4	27.4	0.1	43.3	17.3	0.1	30.9	11.5	0.0
574.405		574.405	0.012			0.3			0.2			0.1			0.1
576.030	0.084	576.030	0.023	1.0	0.8	0.4	0.8	0.6	0.3	0.6	0.4	0.2	0.5	0.2	0.1
576.185	0.732			7.7	4.9		6.2	3.6		4.1	2.3		3.4	1.5	
577.445	0.638			7.1	4.6		5.6	3.4		3.8	2.1		3.1	1.4	
578.725	0.046			0.7	0.6		0.5	0.4		0.4	0.3		0.3	0.2	
579.585	0.016	579.585	0.000	0.3	0.2	0.0	0.2	0.2	0.0	0.2	0.1	0.0	0.1	0.1	0.0
581.180	9.792	581.180	2.620	64.5	47.5	18.3	54.9	34.8	12.9	40.4	21.9	6.7	28.5	14.6	4.0
581.800	0.608	581.800	0.026	6.8	4.4	0.5	5.4	3.2	0.3	3.6	2.0	0.2	3.0	1.4	0.1
582.605	115.311	582.605	10.500	299.5	289.0	50.3	262.1	212.0	35.4	179.4	133.0	18.4	133.6	88.8	11.0
582.837	1.068	582.837	0.000	11.1	7.1	0.0	8.7	5.2	0.0	6.0	3.3	0.0	4.9	2.2	0.0
583.430	3.493	583.430	230.000	26.6	16.9	447.0	20.5	12.4	315.0	15.5	7.8	166.0	12.4	5.2	99.9
586.200	142.739	586.200	0.022	473.4	369.0	0.4	385.0	270.0	0.3	271.3	170.0	0.2	191.8	113.0	0.1
587.090	1.264	587.090	30.100	13.8	8.4	107.0	11.0	6.1	75.1	7.9	3.9	39.3	6.6	2.6	23.5
587.700	0.993	587.700	1.460	10.2	6.3	11.8	8.1	4.6	8.3	5.9	2.9	4.3	4.8	1.9	2.6
587.835	0.759	587.835	0.041	7.9	5.1	0.7	6.3	3.7	0.5	4.6	2.4	0.3	3.8	1.6	0.1
588.815	0.836	588.815	2.830	7.4	4.8	19.4	5.7	3.5	13.6	4.2	2.2	7.1	3.2	1.5	4.2
589.300	0.831	589.300	0.006	6.8	4.6	0.1	5.2	3.4	0.1	3.9	2.1	0.1	3.0	1.4	0.0
590.020	8.280	590.020	4.990	51.3	36.0	29.3	43.4	26.3	20.6	30.4	16.5	10.7	23.6	11.0	6.4
590.225	0.164	590.225	1.070	2.9	2.0	9.3	2.3	1.4	6.5	1.6	0.9	3.4	1.1	0.6	2.0

DI 01 MODEL

Cross Drainage Catchment Chainage	IRDJV Catchment Area (km^2)	Phase 2 Chainage	Phase 2 Catchment Area (km^2)	Peak Flow Estimates (m^3/s)											
				1% AEP			2% AEP			5% AEP			10% AEP		
				RORB	RFFE	Phase 2	RORB	RFFE	Phase 2	RORB	RFFE	Phase 2	RORB	RFFE	Phase 2
591.685	0.769	591.685	0.027	4.2	3.7	0.5	3.1	2.7	0.3	2.5	1.7	0.2	2.0	1.1	0.1
591.766	21.512	591.766	0.010	71.5	74.3	0.2	64.4	54.3	0.2	49.3	34.1	0.1	40.0	22.7	0.0
591.925	0.138	591.925	0.003	1.2	1.0	0.1	1.0	0.7	0.0	0.6	0.5	0.0	0.5	0.3	0.0
592.075	0.754	592.075	26.700	5.6	4.8	97.7	4.0	3.5	68.9	2.8	2.2	36.0	2.4	1.5	21.5
593.060	0.172	593.060	0.217	1.6	1.6	2.7	1.2	1.2	1.9	0.7	0.7	1.0	0.6	0.5	0.6
593.820	5.158	593.820	0.002	29.9	25.6	0.1	20.1	18.7	0.0	16.9	11.8	0.0	13.4	7.8	0.0
595.520	1.483	595.520	0.684	10.4	7.8	6.6	7.0	5.7	4.7	5.4	3.6	2.4	4.6	2.4	1.4
596.430	20.418	596.430	8.380	59.7	58.9	42.8	56.1	43.0	30.1	40.5	27.0	15.7	33.2	17.9	9.3
597.230	4.622	597.230	1.600	30.5	22.2	12.6	20.9	16.2	8.9	16.1	10.2	4.6	13.2	6.8	2.7
599.445	2.936	599.445	64.500	20.4	16.5	183.0	13.6	12.1	129.0	10.6	7.6	67.2	8.9	5.0	40.3
600.500	228.948	600.500	5.760	391.3	528.0	32.6	290.3	385.0	23.0	235.2	241.0	11.9	176.3	160.0	7.1
600.800	0.165	600.800	3.340	1.2	1.1	21.9	0.8	0.8	15.4	0.6	0.5	8.0	0.5	0.3	4.8
601.865	1.465	601.865	125.000	9.7	7.5	291.0	6.7	5.5	206.0	5.1	3.5	108.0	4.2	2.3	64.8
602.450	6.239	602.450	0.000	26.4	24.9	0.0	19.8	18.2	0.0	16.7	11.4	0.0	13.0	7.6	0.0
603.850	86.697	603.850	0.000	144.0	171.0	0.0	119.6	125.0	0.0	86.7	78.0	0.0	63.6	51.8	0.0
607.830	47.150	607.830	136.000	122.4	126.0	307.0	96.1	91.6	218.0	72.4	57.3	114.0	55.8	38.0	68.5
608.070	0.695	608.070	17.700	5.2	4.4	73.0	3.9	3.2	51.4	2.6	2.0	26.8	2.2	1.3	16.0
609.550	9.471	609.550	45.000	33.3	35.2	141.0	25.6	25.7	100.0	20.8	16.1	52.2	18.1	10.7	31.3
613.190	4.611	613.190	1.130	16.9	17.4	9.7	12.5	12.7	6.8	10.5	8.0	3.5	8.9	5.3	2.1
613.990	5.854	613.990	12.200	25.1	24.3	55.8	19.5	17.7	39.4	16.3	11.1	20.5	12.5	7.4	12.2
614.445	0.200	614.445	0.000	1.9	1.7	0.0	1.5	1.2	0.0	0.8	0.8	0.0	0.7	0.5	0.0
614.650	83.413	614.650	7.870	196.1	230.0	40.8	155.2	167.0	28.8	122.2	104.0	15.0	92.2	69.1	8.9
614.930	10.453	614.930	0.000	35.2	35.7	0.0	31.8	26.0	0.0	24.1	16.2	0.0	19.4	10.8	0.0
614.960		614.960	94.700			239.0			169.0			88.6			52.9
616.170	5.263	616.170	0.899	34.4	26.0	8.2	23.3	18.9	5.7	18.4	11.8	3.0	15.0	7.9	1.8
617.075	0.658	617.075	0.000	4.5	3.7	0.0	3.1	2.7	0.0	2.3	1.7	0.0	1.9	1.1	0.0

DI 0 MODEL

Cross Drainage Catchment Chainage	IRDJV Catchment Area (km^2)	Phase 2 Chainage	Phase 2 Catchment Area (km^2)	Peak Flow Estimates (m^3/s)											
				1% AEP			2% AEP			5% AEP			10% AEP		
				RORB	RFFE	Phase 2	RORB	RFFE	Phase 2	RORB	RFFE	Phase 2	RORB	RFFE	Phase 2
618.025	0.999	618.025	4.500	9.7	7.2	27.2	7.3	5.3	19.2	5.1	3.3	9.9	3.5	2.2	5.9
619.030	0.134	619.030	0.025	0.9	1.1	0.5	0.7	0.8	0.3	0.5	0.5	0.2	0.3	0.3	0.1
620.610	17.023	620.610	1.320	88.2	72.7	10.9	70.5	52.9	7.7	54.9	33.0	4.0	42.9	21.9	2.4
621.855	1.730	621.855	0.061	12.2	9.2	1.0	9.5	6.7	0.7	6.4	4.2	0.3	5.3	2.8	0.2
623.030	4.054	623.030	17.600	29.0	23.5	72.8	24.3	17.1	51.3	14.6	10.7	26.8	12.3	7.1	16.0
624.755	0.209	624.755	1.380	2.5	1.4	11.3	1.9	1.0	8.0	1.1	0.7	4.1	1.0	0.4	2.5
625.520	0.583	625.520	3.760	3.4	3.4	23.9	2.8	2.5	16.8	1.7	1.6	8.7	1.4	1.0	5.2
627.230		627.230	0.001			0.0			0.0			0.0			0.0
627.340	107.459	627.340	0.000	315.2	294.0	0.0	246.8	214.0	0.0	184.0	133.0	0.0	147.2	88.1	0.0
627.490		627.490	108.000			262.0			185.0			97.1			58.1
630.870		630.870	0.001			0.0			0.0			0.0			0.0
631.085	21.293	631.085	2.940	85.0	74.2	19.9	69.2	54.0	14.0	56.1	33.7	7.3	41.9	22.3	4.3
631.525	0.453	631.525	0.000	2.2	2.8	0.0	1.9	2.1	0.0	1.1	1.3	0.0	1.0	0.9	0.0
633.720	42.366	633.720	16.500	184.7	148.0	69.4	151.5	108.0	48.9	125.0	67.3	25.5	93.3	44.5	15.3
635.090	0.632	635.090	0.000	3.0	4.0	0.0	2.5	2.9	0.0	1.5	1.8	0.0	1.3	1.2	0.0
635.355	1.854	635.355	44.600	14.1	10.0	140.0	11.3	7.3	99.1	7.3	4.6	51.7	6.1	3.0	31.0
636.650	0.704	636.650	0.000	5.9	4.4	0.0	4.8	3.2	0.0	2.7	2.0	0.0	2.3	1.4	0.0
637.120		637.120	0.001			0.0			0.0			0.0			0.0
637.230	2.495	637.230	0.000	24.4	12.6	0.0	18.7	9.2	0.0	12.8	5.7	0.0	10.6	3.8	0.0
638.080	6.724	638.080	0.000	17.1	23.2	0.0	13.7	16.9	0.0	11.1	10.6	0.0	8.2	7.0	0.0
638.460	1.603	638.460	0.000	13.9	7.7	0.0	9.5	5.6	0.0	7.3	3.5	0.0	6.0	2.3	0.0
639.690	6.272	639.690	0.000	49.7	28.7	0.0	35.9	20.9	0.0	26.7	13.1	0.0	21.3	8.7	0.0
641.540	556.724	641.540	310.000	1019.0	1140.0	550.0	824.5	827.0	392.0	622.6	515.0	205.0	488.4	340.0	123.0
642.315	2.763	642.315	9.070	5.7	14.3	45.2	3.9	10.4	31.9	3.0	6.5	16.6	2.4	4.3	9.9
643.160	0.643	643.160	1160.000	10.7	4.0	1420.0	9.1	2.9	1000.0	5.2	1.8	530.0	4.5	1.2	318.0
643.910	28.466	643.910	2.970	89.3	80.1	20.1	71.0	58.3	14.1	57.5	36.4	7.3	49.3	24.1	4.4
643.965		643.965	0.826			7.7			5.4			2.8			1.7
644.910	2.980	644.910	0.001	25.7	13.4	0.0	20.3	9.7	0.0	15.8	6.1	0.0	12.2	4.0	0.0
645.415	1.400	645.415	28.600	6.7	7.2	103.0	4.7	5.3	72.4	3.5	3.3	37.9	2.9	2.2	22.7
645.850	0.343	645.850	0.000	2.0	2.4	0.0	1.7	1.8	0.0	1.0	1.1	0.0	0.8	0.7	0.0
645.995		645.995	5.810			32.8			23.1			12.0			7.1
646.090	1.267	646.090	0.000	9.4	7.2	0.0	7.8	5.3	0.0	4.8	3.3	0.0	4.0	2.2	0.0
647.095	13.942	647.095	0.000	53.8	44.2	0.0	42.5	32.2	0.0	34.4	20.1	0.0	28.4	13.3	0.0
647.254		647.254	0.001			0.0			0.0			0.0			0.0
647.605	1014.826	647.605	26.500	1529.0	1760.0	97.3	1213.0	1280.0	68.6	842.5	795.0	35.9	602.8	525.0	21.5
647.850	0.746	647.836	0.000	2.3	4.1	0.0	1.6	3.0	0.0	1.2	1.9	0.0	1.0	1.2	0.0
648.170		648.170	0.356			4.0			2.8			1.5			0.9
648.320	7.012	648.320	1.970	37.2	22.2	14.8	30.5	16.2	10.4	23.2	10.1	5.4	16.4	6.7	3.2
648.565	0.695	648.565	0.000	2.7	3.9	0.0	2.0	2.9	0.0	1.4	1.8	0.0	1.2	1.2	0.0
649.115	0.049	649.115	60.400	0.6	0.5	174.0	0.4	0.4	123.0	0.3	0.2	64.4	0.2	0.2	38.6
649.520	8.054	649.520	0.000	42.1	32.1	0.0	29.5	23.4	0.0	23.4	14.6	0.0	17.9	9.7	0.0
650.260	0.473	650.260	0.349	3.1	3.2	3.9	2.7	2.3	2.8	1.5	1.5	1.4	1.3	1.0	0.8
650.610	1.914	650.610	8.860	11.6	11.0	44.5	9.6	8.0	31.3	5.9	5.0	16.3	5.0	3.3	9.7
652.440	2.966	652.440	0.001	21.0	13.7	0.0	14.4	10.0	0.0	11.1	6.3	0.0	9.0	4.1	0.0
652.636	6.193	652.636	1.250	21.2	20.9	10.5	16.9	15.2	7.4	14.0	9.5	3.8	10.3	6.3	2.3
653.070	19.860	653.070	1.520	9.0	61.4	12.1	8.0	44.7	8.5	4.5	27.9	4.4	3.8	18.5	2.6
653.540		653.540	0.001			0.0			0.0			0.0			0.0
653.620	1.546	653.620	0.000	30.9	7.4	0.0	24.1	5.4	0.0	16.1	3.4	0.0	13.4	2.3	0.0
654.445	1.525	654.445	11.800	7.7	8.0	54.6	5.5	5.8	38.5	4.0	3.6	20.0	3.3	2.4	11.9
655.170	4.977	655.170	0.000	85.4	19.9	0.0	67.1	14.5	0.0	53.7	9.0	0.0	44.3	6.0	0.0
655.895	3.401	655.895	0.000	19.4	16.1	0.0	14.0	11.7	0.0	10.3	7.3	0.0	8.3	4.8	0.0
658.850	5.273	658.850	27.500	39.3	24.3	99.8	27.5	17.7	70.6	20.8	11.1	36.7	16.9	7.3	22.0
660.610	73.671	660.610	1.560	222.6	211.0	12.4	184.8	154.0	8.7	149.7	95.9	4.5	124.4	63.4	2.7
663.050	2.105	663.050	4.710	14.8	10.2	28.2	11.5	7.5	19.8	7.7	4.7	10.3	6.4	3.1	6.1
663.350	0.460	663.351	91.500	5.2	4.2	233.0	3.9	3.1	165.0	2.6	1.9	86.0	1.7	1.3	51.9
664.905	0.150	664.905	0.018	1.9	1.7	0.3	1.4	1.3	0.2	1.0	0.8	0.1	0.6	0.5	0.1

Chainage does not match Phase 2 chainage

DI 0 MODEL

Cross Drainage Catchment Chainage	IRDJV Catchment Area (km^2)	Phase 2 Chainage	Phase 2 Catchment Area (km^2)	Peak Flow Estimates (m^3/s)											
				1% AEP			2% AEP			5% AEP			10% AEP		
				RORB	RFFE	Phase 2	RORB	RFFE	Phase 2	RORB	RFFE	Phase 2	RORB	RFFE	Phase 2
684.897	0.270	684.897	0.000	3.2	2.8		2.7	2.0	0.0	1.8	1.3	0.0	1.1	0.8	0.0
686.404	0.280	686.404	0.353	3.0	2.5	4.0	2.6	1.8	2.8	1.7	1.1	1.4	1.1	0.7	0.9
686.440	0.040	686.440	0.019	0.4	0.3	0.4	0.3	0.3	0.3	0.2	0.2	0.1	0.1	0.1	0.1
686.495	1.870	686.495	0.064	12.7	10.4	1.0	10.7	7.6	0.7	6.9	4.7	0.4	5.4	3.1	0.2
690.820	12.530	690.830	13.400	57.9	59.7	59.9	45.9	43.5	42.2	36.1	27.1	21.9	28.4	17.9	13.1
691.025	0.530	691.025	0.000	4.9	4.0	0.0	3.9	2.9	0.0	2.7	1.8	0.0	1.9	1.2	0.0
695.211	2.300	695.210	2.190	17.9	16.0	16.0	14.6	11.7	11.2	9.2	7.3	5.8	7.6	4.8	3.5
696.990	6.040	696.990	6.290	43.1	32.0	34.7	29.8	23.3	24.5	22.6	14.6	12.7	18.2	9.6	7.6
699.880	23.300	699.880	19.200	97.6	99.8	77.4	78.2	72.6	54.6	63.7	45.2	28.5	46.7	29.9	17.0
702.380	2.270	702.380	0.000	0.3	0.4	0.0	0.3	0.3	0.0	0.2	0.2	0.0	0.1	0.1	0.0
703.065	1.210	703.065	1.030	12.3	10.1	9.1	10.2	7.4	6.4	6.9	4.6	3.3	4.4	3.0	2.0
704.790	6.492	704.790	5.810	47.2	35.7	32.8	38.8	26.0	23.1	24.8	16.2	12.0	20.1	10.7	7.1
706.250	6.815	706.250	7.370	33.9	31.2	38.9	27.6	22.7	27.5	22.4	14.2	14.3	17.0	9.4	8.5
706.675	0.125	706.675	0.000	1.5	1.3	0.0	1.2	0.9	0.0	0.8	0.6	0.0	0.5	0.4	0.0
707.400	0.707	707.400	0.704	6.7	5.4	6.8	5.3	3.9	4.8	3.7	2.5	2.5	2.6	1.6	1.5
707.565	0.252	707.565	0.147	3.1	2.7	2.0	2.6	1.9	1.4	1.7	1.2	0.7	1.0	0.8	0.4
708.435	7.404	708.435	7.540	36.5	35.3	39.7	29.6	25.7	27.9	24.1	16.1	14.5	18.2	10.6	8.7
709.740	1.970	709.740	1.360	15.3	14.4	11.2	13.5	10.5	7.8	8.1	6.6	4.1	6.8	4.3	2.4

MA IN E01 MODEL

Cross Drainage Catchment Chainage	IRDJV Catchment Area (km^2)	Phase 2 Chainage	Phase 2 Catchment Area (km^2)	Peak Flow Estimates (m^3/s)											
				1% AEP			2% AEP			5% AEP			10% AEP		
				RORB	RFFE	Phase 2	RORB	RFFE	Phase 2	RORB	RFFE	Phase 2	RORB	RFFE	Phase 2
711.500	23.938	711.500	25.000	104.0	104.0	93.6	84.5	75.7	66.0	65.8	47.2	34.4	52.4	31.2	20.6
711.620	5.833	711.620	5.010	32.2	29.9	29.5	24.2	21.8	20.7	19.4	13.6	10.8	15.2	9.0	6.4
711.775	0.933	711.775	0.443	7.5	6.8	4.7	6.4	4.9	3.3	4.3	3.1	1.7	3.2	2.0	1.0
712.540	0.995	712.540	3.470	8.1	7.3	22.5	6.9	5.3	15.8	4.6	3.3	8.2	3.4	2.2	4.9
713.340		713.340	0.001			0.0			0.0			0.0			0.0
713.350	1.283	713.350	0.188	12.2	9.8	2.4	9.8	7.1	1.7	6.6	4.5	0.9	4.5	3.0	0.5
714.610	2.509	714.610	2.290	19.0	15.6	16.5	15.9	11.3	11.6	9.6	7.1	6.0	7.9	4.7	3.6
714.820	0.203	714.820	0.000	2.4	2.0	0.0	2.0	1.5	0.0	1.5	0.9	0.0	1.0	0.6	0.0
716.850	177.706	716.850	175.000	611.2	632.0	369.0	476.0	460.0	262.0	308.2	286.0	137.0	247.4	189.0	81.8
718.044	0.124	718.044	0.011	1.5	1.4	0.2	1.2	1.0	0.2	0.9	0.6	0.1	0.6	0.4	0.0
718.200	0.105	718.200	0.002	1.1	1.1	0.1	0.9	0.8	0.0	0.6	0.5	0.0	0.4	0.3	0.0
718.390	0.161	718.390	0.024	1.8	1.8	0.4	1.4	1.3	0.3	1.0	0.8	0.2	0.7	0.5	0.1
718.900	0.584	718.900	0.237	5.5	5.4	2.9	4.4	3.9	2.0	3.0	2.4	1.1	2.1	1.6	0.6
719.905	0.063	719.905	0.000	0.8	0.8	0.0	0.7	0.6	0.0	0.5	0.3	0.0	0.3	0.2	0.0
720.175		720.175	0.334	3.5	3.0	3.8	2.8	2.2	2.7	1.9	1.4	1.4	1.2	0.9	0.8
720.740		720.740	0.308	6.2	5.2	3.6	5.0	3.8	2.5	3.4	2.4	1.3	2.4	1.6	0.8
721.030		721.030	11.100	46.5	43.8	52.2	38.2	31.9	36.8	30.1	19.9	19.2	24.0	13.2	11.4
721.170	2	721.170	0.086	6.4	5.5	1.3	5.2	4.0	0.9	3.5	2.5	0.5	2.5	1.6	0.3
721.645		721.645	1.190	10.7	9.0	10.1	9.2	6.6	7.1	6.2	4.1	3.7	4.5	2.7	2.2
722.820	22	722.820	0.150	4.4	4.0	2.0	3.7	2.9	1.4	2.5	1.8	0.7	1.8	1.2	0.4
723.005	2	723.005	2.050	15.3	14.4	15.2	11.6	10.5	10.7	9.3	6.5	5.6	7.3	4.3	3.3
723.225		723.225	0.024	2.9	2.7	0.5	2.7	2.0	0.3	1.7	1.3	0.2	1.3	0.8	0.1
723.600		723.600	1.020	4.7	3.7	9.0	3.8	2.7	6.3	2.6	1.7	3.3	1.9	1.1	2.0
723.875		723.875	0.669	6.8	5.9	6.5	5.6	4.3	4.6	3.7	2.7	2.4	2.7	1.8	1.4
724.620		724.620	1.420	11.9	10.2	11.6	9.9	7.5	8.1	6.5	4.7	4.2	4.8	3.1	2.5
725.275		725.275	7.150	40.6	33.8	38.1	30.3	24.6	26.8	22.5	15.3	14.0	17.9	10.1	8.3
725.545		725.545	0.002			0.0			0.0			0.0			0.0
725.590	2	725.590	0.189	3.1	2.7	2.4	2.5	1.9	1.7	1.8	1.2	0.9	1.2	0.8	0.5
726.115	2	726.115	0.753	5.3	5.0	7.1	4.8	3.6	5.0	3.1	2.3	2.6	2.4	1.5	1.5
726.540	2	726.540	0.114	2.9	2.4	1.6	2.3	1.8	1.1	1.7	1.1	0.6	1.1	0.7	0.3
726.960		726.960	1.160	4.6	4.0	9.9	3.7	2.9	7.0	2.5	1.8	3.6	1.7	1.2	2.2
727.695	1.115	727.695	0.089	10.5	8.6	1.3	8.4	6.3	0.9	5.6	3.9	0.5	3.9	2.6	0.3

MA IN E0 MODEL

Cross Drainage Catchment Chainage	IRDJV Catchment Area (km^2)	Phase 2 Chainage	Phase 2 Catchment Area (km^2)	Peak Flow Estimates (m^3/s)											
				1% AEP			2% AEP			5% AEP			10% AEP		
				RORB	RFFE	Phase 2	RORB	RFFE	Phase 2	RORB	RFFE	Phase 2	RORB	RFFE	Phase 2
728.430	1.523	728.430	2.110	12.3	9.7	15.6	10.3	7.1	10.9	6.1	4.4	5.7	5.2	2.9	3.4
728.910	0.260	728.910	0.154	3.1	2.5	2.0	2.4	1.8	1.4	1.5	1.1	0.7	1.0	0.8	0.4
729.700	0.151	729.700	0.001	1.9	1.4	0.0	1.5	1.0	0.0	1.0	0.6	0.0	0.6	0.4	0.0
729.960	2.194	729.960	1.800	15.3	12.5	13.8	10.5	9.1	9.7	8.1	5.7	5.0	6.4	3.8	3.0
730.390	0.416	730.390	0.001	5.3	4.1	0.0	4.2	3.0	0.0	2.7	1.9	0.0	1.7	1.2	0.0
730.570	0.152	730.570	0.427	1.9	1.5	4.6	1.5	1.1	3.2	1.0	0.7	1.7	0.6	0.5	1.0
732.010	0.478	732.010	0.110	5.0	3.7	1.6	3.7	2.7	1.1	2.6	1.7	0.6	1.8	1.1	0.3
734.945	10.234	734.945	10.700	49.4	45.7	51.0	38.6	33.3	35.9	31.7	20.8	18.7	23.3	13.7	11.2
735.115	720.899	735.115	719.000	1324.0	1740.0	1000.0	1127.0	1270.0	711.0	798.9	789.0	372.0	636.1	521.0	225.0
736.210	2.578	736.210	0.719	20.6	15.5	6.9	17.2	11.3	4.8	10.3	7.1	2.5	8.7	4.7	1.5
736.300		736.300	0.168			2.2			1.5			0.8			0.5
737.555	16.218	737.555	15.500	76.6	71.8	66.4	60.9	52.3	46.9	50.0	32.6	24.4	35.1	21.6	14.5
740.665	281.363	740.665	282.000	604.7	736.0	516.0	505.4	536.0	364.0	385.3	334.0	191.0	269.1	221.0	115.0
740.945	0.121	740.945	0.000	1.4	1.1	0.0	1.0	0.8	0.0	0.7	0.5	0.0	0.5	0.3	0.0
741.345	0.963	741.345	0.076	7.7	6.2	1.2	6.4	4.5	0.8	3.9	2.8	0.4	3.2	1.9	0.2
742.110		742.110	2.150			15.8			11.1			5.7			3.4
742.240	0.823	742.240	0.000	6.5	5.3	0.0	5.5	3.9	0.0	3.3	2.4	0.0	2.8	1.6	0.0
742.690	0.111	742.690	0.000	1.1	0.9	0.0	0.8	0.7	0.0	0.6	0.4	0.0	0.4	0.3	0.0
744.555	41.547	744.555	45.100	163.3	154.0	142.0	129.8	112.0	99.8	102.7	70.2	52.3	90.3	46.4	31.2
745.410	1.820	745.410	0.000	14.0	10.8	0.0	11.4	7.9	0.0	7.2	4.9	0.0	6.0	3.3	0.0
746.025	0.938	746.025	0.561	7.6	6.5	5.7	6.7	4.7	4.0	3.9	2.9	2.1	3.3	2.0	1.2
746.600	0.607	746.600	0.000	6.3	5.1	0.0	4.7	3.7	0.0	3.3	2.3	0.0	2.3	1.5	0.0
747.905	0.197	747.905	0.121	2.5	1.8	1.7	2.0	1.3	1.2	1.3	0.8	0.6	0.8	0.6	0.4
748.425	0.478	748.425	0.433	5.0	3.7	4.6	3.7	2.7	3.3	2.6	1.7	1.7	1.7	1.1	1.0
749.450	0.138	749.450	0.112	1.7	1.7	1.6	1.3	1.3	1.1	0.8	0.8	0.6	0.5	0.5	0.3
750.965	25.352	750.965	25.600	115.3	113.0	94.9	90.4	82.7	66.8	75.2	51.6	34.9	57.7	34.1	20.9
751.130	2.071	751.130	1.420	17.8	13.8	11.5	14.5	10.1	8.1	8.4	6.3	4.2	7.2	4.2	2.5
752.490	0.240	752.490	0.059	3.4	2.6	0.9	2.7	1.9	0.7	1.7	1.2	0.3	1.0	0.8	0.2
753.100	4.242	753.100	4.580	35.8	26.4	27.6	29.4	19.3	19.4	17.2	12.0	10.1	14.6	8.0	6.0
755.225	4.961	755.225	0.629	33.1	26.0	6.2	22.6	19.0	4.4	17.7	11.8	2.3	13.9	7.8	1.3
755.490	0.090	755.495	5.040	1.4	1.1	29.6	1.2	0.8	20.8	0.7	0.5	10.8	0.4	0.3	6.4
755.490				1.4			1.2			0.7			0.4		
755.975	1.185	755.975	0.715	12.0	9.5	6.9	9.1	7.0	4.8	6.1	4.4	2.5	4.5	2.9	1.5
757.003	2.044	757.003	2.690	15.3	12.4	18.6	11.7	9.1	13.1	7.9	5.7	6.8	6.5	3.8	4.1
758.215		758.215	0.105			1.5			1.1			0.5			0.3
758.255		758.255	0.154			2.1			1.4			0.7			0.4

Chainage does not match Phase 2 chainage