# APPENDIX

# H

# Hydrology and Flooding Technical Report

PART 1 OF 3

## Main Report

**NORTH STAR TO NSW/QUEENSLAND BORDER** ENVIRONMENTAL IMPACT STATEMENT

The Australian Government is delivering Inland Rail through the Australian Rail Track Corporation (ARTC), in partnership with the private sector.

# Inland Rail: North Star to NSW/QLD Border

## Appendix H - Hydrology and Flooding Technical Report

**Australian Rail Track Corporation**

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Revision: 3

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# Glossary

The following terms and acronyms are used within this document:

| Term or Acronym | Description |
|---|---|
| AAToS | Annual Average Time of Submergence (hrs/yr) |
| AEP | Annual Exceedance Probability |
| AHD | Australian Height Datum |
| ARR 2016 | Australian Rainfall and Runoff Guidelines – 2016 edition |
| ARTC | Australian Rail Track Corporation |
| BoM | Bureau of Meteorology |
| CC | Climate change |
| CL | Continuing loss rate (mm/hr) |
| DCDB | Digital Cadastral Data Base |
| DEM | Digital Elevation Model |
| Developed Case | Hydraulic modelling case with proposal in place |
| Disturbance footprint | The proposal disturbance footprint includes the rail corridor and other permanent works associated with the proposal (e.g. where changes to the road network are required) as well as the construction footprint where only temporary disturbance is proposed (e.g. laydown areas and compound sites). |
| DPIE | Department of Planning, Industry and Environment |
| EIS | Environmental Impact Statement |
| Existing Case | Hydraulic modelling case pre-proposal |
| FFA | Flood Frequency Analysis |
| FFJV | Future Freight Joint Venture |
| GIS | Geographic Information System |
| km | kilometres |
| LGA | local government area |
| LiDAR | Light Detection and Ranging |
| m | metres |
| mm | millimetres |
| m AHD | metres above Australian Height Datum |
| N/A | Not Applicable |
| NS2B | North Star to Border |
| NSW | New South Wales |
| DPIE | Department of Planning, Industry and Environment |
| PMF | Probable Maximum Flood |
| QLD | Queensland |
| RCBC | Reinforced concrete box culvert |
| RCP | Reinforced concrete pipe |
| RFFE | Regional Flood Frequency Estimation |
| SEARs | Critical State Significant Infrastructure Standard Secretary's Environmental Assessment Requirements |
| Flood study area | The limits of the proposal area as defined in the SEARs |
| the proposal | The North Star to Border proposal |

| Term or Acronym | Description |
|---|---|
| TOF | Top of formation level |
| TOR | Top of rail level |
| ToS | Time of Submergence (hrs) |
| FFA | flood frequency analysis |

# Executive summary

Inland Rail is a once-in-a-generation Program connecting regional Australia to domestic and international markets, transforming the way we move freight around the country. It will complete the 'spine' of the national freight network between Melbourne and Brisbane via regional Victoria, New South Wales and Queensland. This new 1,700 kilometres (km) line is the largest freight rail infrastructure project in Australia and is expected to commence operations in 2025.

The Inland Rail North Star to Border (NS2B) Proposal (the 'proposal') provides a connection between North Star in New South Wales (NSW) and the NSW and Queensland (QLD) Border. The proposal crosses the Macintyre River and its floodplain which are a part of the Border Rivers catchment. The proposal runs through Moree Plains, Gwydir and Goondiwindi local government areas (LGA).

The Macintyre River floodplain has experienced many floods including the 1956 and 1976 events and more recently the 1996 and 2011 flood events. The floodplain is generally used for farming practices and many landholders are reliant on characteristics of flooding across the floodplain for collection and storage of water for irrigation.

The purpose of this investigation was to better understand and quantify the existing flooding characteristics of the each of the high-risk waterways in the vicinity of the proposal alignment and to assess and mitigate any potential impacts on properties and infrastructure. The key objectives of the Report are to provide information on the data investigation, development and calibration of the hydrology and hydraulic models, document impacts and mitigation measures and to provide comment on the performance on the proposal design.

Available background information including existing hydrologic and hydraulic models, survey, streamflow data, available calibration information and anecdotal flood data was collected and reviewed. This data was sourced from a wide range of stakeholders was used to develop calibrated hydrologic and hydraulic models for the Macintyre River floodplain and associated waterways. These models were calibrated against multiple historical events and validated through stakeholder and community feedback.

Design flood estimation techniques in accordance with Australian Rainfall and Runoff 2016 (ARR 2016) were applied to the hydrologic and hydraulic models to determine Existing Case flood conditions on the Macintyre River floodplain. This modelling was undertaken for a range of design event from the 20% Annual Exceedance Probability (AEP) event up to the 1 in 10,000 AEP event and the Probable Maximum Flood (PMF).

A Developed Case was prepared using the Existing Case models and incorporating the proposal design. The Developed Case model was run for the same range of design events with results compared to determine impacts on peak water levels, flows, flood flow distribution, velocities and duration of inundation on the floodplain and, in particular, upon identified flood sensitive receptors.

The refinement of the proposal design was guided using hydraulic design criteria and flood impact objectives (refer Table 1) that were developed for the proposal. The flood impact objectives were initially developed based on a review of objectives used for other large infrastructure projects in rural and urban areas as well as consideration of industry practice and use of engineering judgement.

**Table 1 Flood impact objectives**

| Parameter | Objectives | | | | |
|---|---|---|---|---|---|
| Afflux[1] | Existing habitable and/or commercial and industrial buildings/premises (e.g. dwellings, schools, hospitals, shops) | Residential or commercial/industrial properties/lots where flooding does not impact dwellings/ buildings (e.g. yards, gardens) | Existing non-habitable structures (e.g. agricultural sheds, pump-houses) | Roadways | Agricultural and grazing land/forest areas and other non-agricultural land |
| | ≤ 10 mm | ≤ 50 mm | ≤ 100 mm | ≤ 100 mm | ≤ 200 mm with localised areas up to 400 mm |
| | Changes in peak water levels are to be assessed against the above proposed limits. It is noted that changes in peak water levels can have varying impacts upon different infrastructure/land and flood impact objectives were developed to consider the flood sensitive receptors in the vicinity of the proposal. It should be noted that in many locations the presence of existing buildings or infrastructure limits the afflux. | | | | |
| Change in duration of inundation[1] | Identify changes to time of inundation through determination of time of submergence (ToS).<br>For roads, determine the average annual time of submergence (AAToS) (if applicable) and consider impacts on accessibility during flood events.<br>Justify acceptability of changes through assessment of risk with a focus on land-use and flood sensitive receptors. | | | | |
| Flood flow distribution[1] | Aim to minimise changes in natural flow patterns and minimise changes to flood flow distribution across floodplain areas.<br>Identify any changes and justify acceptability of changes through assessment of risk with a focus on land-use and flood sensitive receptors. | | | | |
| Velocities[1] | Maintain existing velocities where practical.<br>Identify changes to velocities and impacts on external properties and waterway geomorphology.<br>Determine appropriate mitigation measures taking into account existing soil and geomorphological conditions.<br>Justify acceptability of changes through assessment of risk with a focus on land-use and flood sensitive receptors. | | | | |
| Hazard[1] | Identify changes to hazard categories and any impacts on external properties.<br>Justify acceptability of changes through assessment of risk with a focus on land-use and flood sensitive receptors. | | | | |
| Extreme event risk management | Consider risks posed to neighbouring properties for events larger than the 1% AEP event to ensure no unexpected or unacceptable impacts. | | | | |
| Climate change and blockage | Consider risks posed climate change and blockage in accordance with ARR 2016.<br>Undertake assessment of impacts associated with proposal alignment for both scenarios.<br>Consider additional sensitivity options as identified throughout the proposal development and as a result of stakeholder engagement. | | | | |
| Emergency management | Consider the impacts the proposal may have upon existing community emergency management arrangements for flooding as well changes to flood safety risks on private and public land including roads and pathways. | | | | |
| Compliance with Floodplain Management Plans | Check to ensure consistency with:<br>▪ Moree Plains Development Control Plan<br>▪ Border Rivers Floodplain Management Plan | | | | |

**Table notes:**

1 These flood impact objectives apply for events up to and including the 1% AEP event

Detailed hydrologic and hydraulic modelling was undertaken to meet the hydraulic design criteria and flood impact objectives, with a series of iterations undertaken to incorporate design refinement and stakeholder and community feedback.

The hydrologic and flooding assessment undertaken has demonstrated that the proposal is predicted to result in impacts on the existing flooding regime that generally comply with the flood impact objectives and that the proposal meets the hydraulic design criteria.

A comprehensive consultation exercise has been undertaken to provide the community with detailed information and certainty around the flood modelling and the proposal design. The consultation with stakeholders, including landholders, was undertaken at key stages including validation of the performance of the modelling in replicating experienced historical flood events and presentation of the design outcomes and impacts on properties and infrastructure. In future stages, ARTC will:

- Continue to work with landowners concerned with hydrology and flooding throughout the detailed design, construction and operational phases of the proposal
- Continue to work with directly impacted landowners affected by the alignment throughout the detailed design, construction and operational phases of the proposal
- Continue to work with local Councils and State government departments throughout the detailed design, construction and operational phases of the proposal.

# 1 Introduction

## 1.1 Inland Rail Program

Inland Rail is a once-in-a-generation Program connecting regional Australia to domestic and international markets, transforming the way we move freight around the country. It will complete the 'spine' of the national freight network between Melbourne and Brisbane via regional Victoria, New South Wales and Queensland.

This new 1,700 kilometres (km) line is the largest freight rail infrastructure project in Australia and is expected to commence operations in 2025.

## 1.2 North Star to NSW/QLD Border proposal

The Inland Rail section between North Star in New South Wales (NSW) and the NSW and Queensland (QLD) Border (known as the 'NS2B' proposal) will cross the Macintyre River and its floodplain which are a part of the Border Rivers catchment. The proposal alignment runs through Moree Plains local government area (LGA), Gwydir LGA and Goondiwindi LGA. The proposal rail alignment is shown in Figure 1.1.

Key features of the proposal include:

- Approximately 30 km of new, single line, standard gauge track (trains travelling in both directions share the same track)
- Upgrade to approximately 25 km of non-operational corridor and 5 km of new greenfield rail corridor to the NSW/QLD Border (Ch 30.6 km)
- Bridges to accommodate topographical variation, crossings of waterways and other infrastructure
- Reinforced concrete pipe culverts and reinforced concrete box culverts
- Rail crossings including level crossings, grade separations/rail or road overbridges, occupational/private crossings
- Removal of non-operational rail line up to southern side of Whalan Creek
- Roadworks including realignment and drainage structures on Bruxner Way.

For the purpose of the hydrology and flooding investigation the following was incorporated into the design:

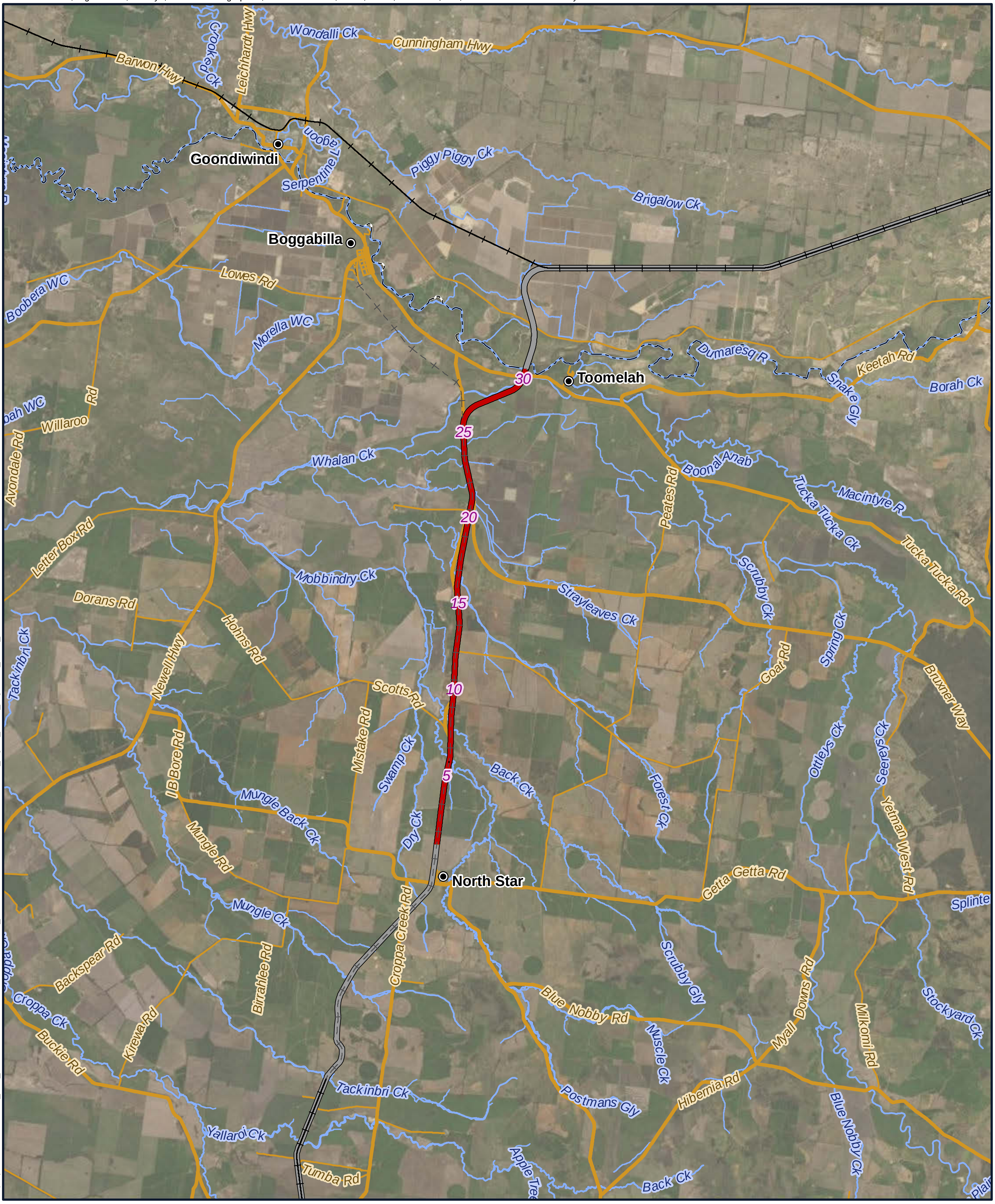
- An additional approximate 6 km of new, single line, standard gauge track within new greenfield corridor within the Border Rivers Floodplain to Ch 36.04 (B2G alignment).

## 1.3 Objectives of this report

This investigation has been undertaken to firstly identify high-risk watercourse crossings or floodplain locations that may be impacted by the proposal. Secondly a detailed quantitative assessment has been undertaken to better understand and quantify the existing flooding characteristics of each of the high-risk waterways in the vicinity of the proposal and to assess and mitigate any potential impacts associated with the proposal on the existing flooding regime of each waterway.

The key purpose of this report is to provide details of investigation undertaken including data collection and review, development and calibration of hydrology and hydraulic models, design event modelling, impact assessment of the proposal, development of mitigation measures and to provide comment on the performance of the proposal design. Consultation with stakeholders and the community has been progressively undertaken with feedback used to inform the development and calibration of the models and to refine the proposal design.

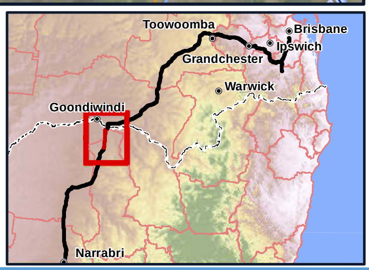
Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community




Map by: DTH Z:\GIS\GIS_270_NS2B\Tasks\270-IHY-201909261010_SurfaceWater\270-IHY-201909261010_Fig1.1_NS2B_alignment_FFJV_A4P.mxd Date: 7/02/2020 12:24

**Legend**

- 5 Chainage (km)
- Localities
- Existing rail (operational)
- Existing rail (non-operational)
- North Star to NSW/QLD border alignment
- Adjoining alignments
- Major roads
- Minor roads
- Watercourses




A4 scale: 1:300,000

0 1.5 3 4.5 6 7.5 km

Future Freight
Integrating Community, Environment and Engineering

Date: 10/01/2020 Version: 0

Coordinate System: GDA 1994 MGA Zone 56

**North Star to NSW/QLD border**

**Figure 1.1:**
**North Star to Border alignment**

Key objectives of the hydrology and flooding investigation were to:

- Consult with local authorities (Moree Plains Regional Council, Goondiwindi Regional Council and Gwydir Shire Council) regarding existing flood studies relevant to the proposal and consider these previous flood studies in the development of the proposal design
- Consult with landholders, stakeholders and government agencies to obtain flood data to assist in model development and calibration, and to discuss impacts associated with the Project
- Undertake detailed hydrologic and hydraulic modelling to establish the base (or Existing Case) flood conditions for the range of floods up to the 1% Annual Exceedance Probability (AEP) event as well as the 1 in 2,000 AEP, 1 in 10,000 AEP and Probable Maximum Flood (PMF) events
- Determine existing flood conditions including flood levels, flows and velocities
- Analyse the proposal design including the alignment design, drainage infrastructure and associated infrastructure works
- Assess the impacts of the proposal on neighbouring properties, infrastructure and the surrounding environment
- Identify and assess potential mitigation measures. The requirement for mitigation was based on the magnitude of impacts and how this aligned with the flood impact objectives.

# 2 Assessment methodology

Previous assessments have been carried out for the proposal including a feasibility assessment of the proposal including preliminary hydrology and hydraulic modelling. In the feasibility assessment limitations were identified regarding the ability of the hydrologic modelling to represent the flood flow volumes and the hydraulic model representation of levees and other drainage structures on the Macintyre River floodplain. It was concluded that the feasibility models required recalibration and updating to Australian Rainfall and Runoff 2016 (ARR 2016) standards.

The Department of Planning, Industry and Environment (DPIE) had developed hydrologic and hydraulic models that encompass the Macintyre River system. However, these were not available during the previous assessments. The DPIE models have undergone calibration with community engagement (including consideration of available historical data from the community and the LGAs) and therefore they were adopted as the basis for this current hydrology and flooding investigation.

The hydrology and flooding investigation involved the following activities:

The hydrology and flooding assessment of the proposal uses a quantitative approach to impact assessment. The assessment methodology was progressively refined as feedback from the community was considered and addressed. The refined methodology involved the following activities:

- Collation and review of available background information including existing hydrologic and hydraulic models, survey, rainfall and streamflow data, calibration information and anecdotal flood related data. This review established which datasets were suitable to use for the assessment of the proposal design.
- Determination of critical flooding mechanisms for waterways and drainage paths in the flood study area, i.e. regional flooding versus local catchment flooding
- Adoption of the DPIE hydrologic and hydraulic modelling as the basis of modelling for the proposal assessment for Border Rivers floodplain
  - The DPIE hydraulic model includes all constructed and approved structures on the floodplain and is the tool used by DPIE for assessment of proposed works on the floodplain
  - The provided hydraulic model is based on survey data which is a 10 m by 10 m gridded DEM derived from LiDAR survey datasets, including the Macintyre 2013 and Gwydir 2013 datasets. Where LiDAR was not available the dataset was supplemented with the Shuttle Radar Topographic Mission elevation data.
  - The grid spacing used in the DPIE hydraulic model is 40 m
  - The provided DPIE model represents existing floodplain levees as a mixture of height limited and height unlimited layers giving a representation of approved levees on the floodplain
- Update of the DPIE hydrologic model to include the 2011 historical event to support validation of the hydraulic sub-model performance
- Development of a hydraulic sub-model based on the DPIE hydraulic model focussed on the study area for the proposal to beyond Goondiwindi
  - The sub-model allowed the level of floodplain detail to be increased, improved representation of the proposal alignment and reduced hydraulic model simulation times. The grid spacing used in the hydraulic sub-model was 30 m.
  - For each of the calibration/validation events the model topography included the best representation possible of existing floodplain levees at the time of each historical flood event
- Joint calibration of the hydrologic model and hydraulic sub-models including:
  - Validation of the hydrologic model and hydraulic sub-model against the available recorded and anecdotal data for the 1976, 1996 and 2011 historical flood events

- Extensive community and stakeholder engagement to validate model performance, incorporate stakeholder and community feedback, leading to acceptance of modelling and calibration outcomes

- Design event modelling including:
  - Update of DPIE hydrologic models to include ARR 2016 design event hydrology. The range of flood event magnitudes assessed included the 20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and Probable Maximum Flood (PMF) events.
  - Preparation of Existing Case hydraulic sub-model to enable assessment of the proposal alignment and associated works. As part of the community and stakeholder engagement process, feedback identified that the levees represented in the DPIE hydraulic model as being of "unlimited height", which whilst appropriate for the DPIE assessment tool, did not represent the actual levee heights on the floodplain.
  - For design of the proposal alignment and mitigation of impacts, it was important that the hydraulic sub-model reflected the topographic reality of the floodplain. As new LiDAR was planned along the rail corridor, it was possible to expand the capture to include a significant portion of the floodplain and to obtain current levee heights on the floodplain. Use of this updated 2019 LiDAR dataset is consistent with the SEARs (Item 8.2 (a)) which requires the use of data of sufficient spatial coverage and accuracy to ensure the resultant models can accurately assess existing and proposed water flow characteristics. Therefore, two Existing Case hydraulic sub-model have been prepared, being:
    - DPIE levees Existing Case – for this scenario the majority of the hydraulic sub-model area was covered by LiDAR collected for the proposal between September 2014 and January 2015. The hydraulic sub-model was set up using these datasets combined with the DPIE representation of floodplain levees.
    - 2019 LiDAR (and levees) Existing Case – used the new LiDAR flown and processed November 2019 to provide a snapshot of current floodplain topography including current levee heights and floodplain features
  - Taking account of stakeholder and community feedback, the downstream boundary of the hydraulic sub-model was also extended a significant distance downstream of Goondiwindi. This extension provided flood modelling results around the township and extended the calibration footprint of the modelling and hence increased certainty in the hydraulic sub-model predictions.
  - Simulation of ARR 2016 design events in the hydraulic sub-model for both Existing Cases and comparison to previous studies to confirm drainage paths, waterways, and associated floodplain areas, and established the existing flood regime in the vicinity of the proposal

- Developed Case modelling including design assessment and refinement using the 1% AEP design event for both the DPIE levees hydraulic sub-model and the 2019 LiDAR hydraulic sub-model, including:
  - Inclusion of proposed rail alignment, drainage structures and associated works into the hydraulic sub-models and simulation of ARR 2016 design events
  - Assessment of impacts of proposal alignment against the flood impact objectives using the suite of design floods including consideration of change in flood levels, flow distributions, velocities and inundation periods
  - Determination of appropriate mitigation measures to manage potential impacts including refinement of location and dimensions of flood drainage structures under the proposed alignment. Iterations were undertaken using the hydraulic sub-model to achieve a design that met the flood impact objectives and addressed the SEARs requirements.
  - The performance of the proposal alignment design against the flood impact objectives has been documented in detail for the 2019 LiDAR hydraulic sub-model, this sub-model topography represents the current topography of the floodplain in which the proposal will be constructed.

- Developed Case modelling for the full range of design events (20% AEP to PMF) and assessment scenarios using the 2019 LiDAR hydraulic sub-model including consideration of:
  – Climate change
  – Blockage of drainage structures
  – Extreme events (1 in 2,000, 1 in 10,000 AEP and PMF events)
  – Flood hazard classifications
  – Emergency management planning and flood safety risk
  – Council and/or DPIE Floodplain Management Plans including the Border Rivers Valley Floodplain Management Plan requirements
  – Construction camps and borrow pits during construction phase
- Ongoing community and stakeholder engagement in accordance with the ARTC Flood Study Engagement Framework to confirm acceptance of the hydraulic sub-model and the proposal design against the flood impact objectives.

The hydrology and hydraulic impact assessment provide key inputs to the proposal design where the alignment is located within the modelled flood extents. Key dependencies for the proposal design include:

- Modelling of the Existing Case 1% AEP event to ascertain existing conditions and inform the flood immunity for the proposal alignment and to size drainage structures
- Modelling of 1 in 2,000 AEP event to provide inputs for bridge design and wider resilience assessment
- Modelling of rare flood events (1 in 10,000 AEP and PMF events) to assist in consideration of overtopping risk
- Modelling the full range of flood events to quantify potential impacts and inform mitigation measures
- Input to drainage design including scour protection design – water levels, flows and velocities from this assessment have been used to inform the design of scour protection
- Input to structure selection and design for culverts/bridges
- Geomorphology – the flows and velocities calculated from the hydrologic and flooding assessment have been used to inform the geomorphological assessment. Fluvial geomorphology has been evaluated in the Biodiversity and aquatic ecology assessments. The study includes general assessment of existing geomorphological aspects of targeted waterways within the flood study area. The aquatic ecology geomorphological assessment involves the assessment of waterways in accordance with the AUSRIVAS Physical Assessment Protocol and includes assessment of factors such as channel shape and modifications, bank shape and slope, bedform features, bed compaction and stability, sediment matrix and angularity, factors affecting bank stability, type and extent of bars and riparian zone structure and composition.

# 3 Existing environment

## 3.1 Waterways

There are several major waterways in the area of the proposal, as shown in Figure A1, with the key waterway being the Macintyre River and its two tributaries, the Dumaresq River and Macintyre Brook, which meet upstream of Boggabilla. At this confluence, flows in the Macintyre River split and break out south-westwards into Whalan Creek, an anabranch of the Macintyre River. A large portion of flood flows are conveyed by Whalan Creek. This network of waterways is referred to as the Border Rivers and the Border Rivers Valley Floodplain. There have been many major floods in the last 40 years including February 1976, which is considered the largest flood event that has been experienced in most areas of the floodplain. The Borders River Valley drains slowly due to the slow-moving nature of flood waters, a result of the typically flat terrain in the floodplain.

The proposal crosses several anabranches of the Macintyre River, including Whalan Creek, which convey significant portions of flood flows during moderate to major flood events. In addition, there are several smaller local creeks that cross the proposal alignment including Ottleys Creek, Strayleaves Creek, Forest Creek, Back Creek and Mobbindry Creek.

## 3.2 Floodplain infrastructure

Existing floodplain infrastructure in the vicinity of the proposal includes:

- Bruxner Way
- Tucka Tucka Road
- North Star Road
- Camurra-Boggabilla Railway (existing non-operational rail)
- Kildonan Road
- Eukabilla Road
- Queensland Rail Western Line
- Levees and dams from farming practices
- Newell Highway
- Goondiwindi Town Levee.

Appendix B includes mapping that presents the location of the existing infrastructure as well as photographs of significant drainage structures within the flood study area that were observed during site inspections.

Bruxner Way is a low-level road with minor drainage structures. Tucka Tucka Road is a low-level road with minor drainage structures including a bridge over Whalan Creek.

Details of the Goondiwindi Town Levee were supplied by Goondiwindi Regional Council from surveyed plans dated November 2016. The plans included chainage and long section detail with surveyed elevations that were included in the hydraulic model.

Design details of the Newell Highway upgrades were sourced for inclusion in the hydraulic model. Inconsistencies with the 2019 LiDAR, aerial imagery and ground levels were found. Therefore, due to time constraints, the Newell highway was included in the hydraulic model based on LiDAR rather than the provided design levels. A sensitivity on the height of the Newell Highway (as per top of road design with no culverts) was carried out with the hydraulic model to ensure the levels used to represent the road did not impact on the results at the proposal. The sensitivity found no difference in predicted peak water level at the alignment as a result of varying the height of the Newell Highway. It was therefore considered suitable to progress with LiDAR topography for the Newell Highway.

The non-operational Camurra-Boggabilla rail embankment is raised with limited hydraulic structures. The rail line runs in a northerly direction from North Star and then tracks west on the southern side of the Macintyre River towards Boggabilla. The rail embankment restricts flows during flood events and is overtopped under larger flood events. The existing rail embankment is in a state of significant disrepair and is elevated above the surrounding ground levels by approximately 0.5 m to 0.8 m, increasing up to 2 m near Whalan Creek. It has limited transverse drainage structures, many of which are in a degraded state.

Consistent with the DPIE regional model approach, key existing drainage structures are represented in the hydraulic sub-model as openings in the DEM of the rail or road. In existing conditions, these structures are generally minor, and the bridges have a low flood immunity. In large flood events these structures are overtopped and not considered critical to the existing flood regime.

# 4 Design requirements, standards and guidelines

## 4.1 Hydraulic design criteria

Table 4.1 outlines the hydraulic design criteria that have guided the proposal design. Detailed hydrologic and hydraulic modelling has been undertaken to meet these design criteria with a series of iterations undertaken to incorporate design refinement and stakeholder and community feedback. The resulting outcomes relative to these design criteria are detailed in Section 9.

**Table 4.1 Proposal hydraulic design criteria**

| Performance criteria | Requirement |
|---|---|
| Flood immunity | Rail line – 1% AEP flood immunity to formation level. |
| Hydraulic analysis and design | Hydrologic and hydraulic analysis and design to be undertaken based on Australian Rainfall and Runoff (ARR 2016) and State/local government guidelines.<br>ARR 2016 interim climate change guidelines are to be applied with an increase in rainfall intensity to be considered. No sea level change consideration required due to location outside tidal zone.<br>ARR 2016 blockage assessment guidelines are to be applied. |
| Scour protection of structures | All bridges and culverts should be designed to reduce the risk of scour with events up to 1% AEP event considered.<br>Mitigation to be achieved through providing appropriate scour protection or energy dissipation or by changing the drainage structure design. |
| Structural design | 1 in 2,000 AEP event to be modelled for bridge design purposes. |
| Extreme events | Damage resulting from overtopping to be minimised. |
| Flood flow distribution | Locate structures to ensure efficient conveyance and spread of floodwaters. |
| Sensitivity testing | Consider climate change and blockage in accordance with ARR 2016. Understand risks posed and proposal design sensitivity to climate change and blockage of structures.<br>Consider additional sensitivity options as identified throughout the proposal development and as a result of stakeholder engagement. |

## 4.2 Flood impact objectives

The impact of the proposal upon the existing flood regime was quantified and compared against flood impact objectives as detailed in Table 4.2. These objectives address the requirements of the SEARs and have been used to guide the proposal design. Acceptable impacts will ultimately be determined on a case by case basis with interaction with stakeholders/landholders through the community engagement process using these objectives as guidance. This will take into account flood sensitive receptors and land use within floodplain areas.

The resulting design outcomes relative to these flood impact objectives are outlined in are detailed in Section 9.2.

Table 4.2 **Flood impact objectives**

<table>
<tr><th>Parameter</th><th colspan="5">Objectives</th></tr>
<tr><td rowspan="3">Afflux[1]</td><td>Existing habitable and/or commercial and industrial buildings/premises (e.g. dwellings, schools, hospitals, shops)</td><td>Residential or commercial/industrial properties/lots where flooding does not impact dwellings/ buildings (e.g. yards, gardens)</td><td>Existing non-habitable structures (e.g. agricultural sheds, pump-houses)</td><td>Roadways</td><td>Agricultural and grazing land/forest areas and other non-agricultural land</td></tr>
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<tr><td>Flood flow distribution[1]</td><td colspan="5">Aim to minimise changes in natural flow patterns and minimise changes to flood flow distribution across floodplain areas.<br>Identify any changes and justify acceptability of changes through assessment of risk with a focus on land-use and flood sensitive receptors.</td></tr>
<tr><td>Velocities[1]</td><td colspan="5">Maintain existing velocities where practical.<br>Identify changes to velocities and impacts on external properties and waterway geomorphology.<br>Determine appropriate mitigation measures taking into account existing soil and geomorphological conditions.<br>Justify acceptability of changes through assessment of risk with a focus on land-use and flood sensitive receptors.</td></tr>
<tr><td>Hazard[1]</td><td colspan="5">Identify changes to hazard categories and any impacts on external properties.<br>Justify acceptability of changes through assessment of risk with a focus on land-use and flood sensitive receptors.</td></tr>
<tr><td>Extreme event risk management</td><td colspan="5">Consider risks posed to neighbouring properties for events larger than the 1% AEP event to ensure no unexpected or unacceptable impacts.</td></tr>
<tr><td>Climate change and blockage</td><td colspan="5">Consider risks posed climate change and blockage in accordance with ARR 2016.<br>Undertake assessment of impacts associated with proposal alignment for both scenarios.<br>Consider additional sensitivity options as identified throughout the proposal development and as a result of stakeholder engagement.</td></tr>
<tr><td>Emergency management[2]</td><td colspan="5">Consider the impacts the proposal may have upon existing community emergency management arrangements for flooding as well changes to flood safety risks on private and public land including roads and pathways.</td></tr>
<tr><td>Compliance with Floodplain Management Plans[2]</td><td colspan="5">Check to ensure consistency with:<br>▪ Moree Plains Development Control Plan<br>▪ Border Rivers Floodplain Management Plan</td></tr>
</table>

**Table notes:**

1 These flood impact objectives apply for events up to and including the 1% AEP event
2 These items specifically relate to SEARs requirements which are not addressed in this document

## 4.3 Project nomenclature for design events

The flood analysis adopts the latest approach to design flood terminology as detailed in ARR 2016.

Accordingly, all design events are quoted in terms of AEP using percentage probability. An extract of Figure 1.2.1 from Book 1 (shown in Table 4.3) details the relationship between Average Recurrence Interval (ARI) and AEP for a range of design events.

**Table 4.3 Event nomenclature (taken from ARR 2016 Book 1)**

| Exceedances per year | AEP (%) | AEP (1 in x) | ARI |
|---|---|---|---|
| 0.22 | 20.00 | 5 | 4.48 |
| 0.20 | 18.13 | 5.52 | 5.00 |
| 0.11 | 10.00 | 10 | 9.49 |
| 0.05 | 5.00 | 20 | 20 |
| 0.02 | 2.00 | 50 | 50 |
| 0.01 | 1.00 | 100 | 100 |
| 0.005 | 0.50 | 200 | 200 |
| 0.002 | 0.20 | 500 | 500 |
| 0.0005 | 0.05 | 2,000 | 2,000 |
| 0.0001 | 0.01 | 10,000 | 10,000 |

In line with ARR 2016 recommendations, the following terminology has been adopted for the simulated design events:

- 20% AEP
- 10% AEP
- 5% AEP
- 2% AEP
- 1% AEP
- 1 in 2,000 AEP
- 1 in 10,000 AEP
- Probable Maximum Flood (PMF).

## 4.4 Relevant standards and guidelines

The design standards applicable for the hydrologic and hydraulic analysis are listed below:

- AS7637:2014: Railway Infrastructure – Hydrology and Hydraulics
- Austroads (2013) Guide to Road Design Part 5: Drainage – General and Hydrology Considerations, Sydney
- Commonwealth of Australia (2016). Australian Rainfall and Runoff: A Guide to Flood Estimation. Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M, Testoni I, (Editors)
- Evaluating Scour at Bridges, Hydraulic Engineering Circular Number 18 (HEC-18), Fourth Edition, US Department of Transport – Federal Highway Administration, Virginia, USA, Richardson, EV and Davis, SR**:** 2001
- Hydraulic Design of Energy Dissipaters for Culverts and Channels, Hydraulic Engineering Circular Number 14 (HEC-14), Third Edition US Department of Transport – Federal Highway Administration, Virginia, USA, Thompson, PL & Kilgore, RT; 2006.

## 4.5 Sustainability

Sustainability has been considered across all aspects of the proposal including flooding. The flood impacts have been considered against climate change and Lan credit requirements. The assessment is documented in the Sustainability Report. A summary for flooding is provided in Section 9.6.

# 5 Data collection and review

The Border Rivers system feeding into the flood study area comprises the following main waterways:

- Macintyre River (including Whalan Creek)
- Macintyre Brook
- Dumaresq River
- Ottleys Creek
- Strayleaves Creek
- Forest Creek
- Back Creek
- Mobbindry Creek.

In addition, there are many other local creek and anabranch systems that feed into these main waterways. All watercourses are presented in Figure A1.

Available background information including existing hydrologic and hydraulic models, survey, streamflow data, available calibration information and anecdotal flood data has been gathered. Data was sourced from a wide range of stakeholders including:

- Local Authorities including Moree Plains LGA, Gwydir LGA and Goondiwindi LGA
- The Bureau of Meteorology (BoM) – rainfall and stream gauging data
- Department of Planning, Industry and Environment – stream gauging data and hydrologic and hydraulic modelling.

The following sections detail the existing information sourced and reviewed for use in the hydrologic and hydraulic assessment.

## 5.1 Previous studies

There have been many studies undertaken by the local governments and stakeholders for the area. These are summarised and discussed in detail below. The models developed for these studies are also documented and have been used for comparison of the models developed for this study.

The Macintyre River catchment lies within the Moree Plains LGA, Gwydir LGA and Goondiwindi LGA. Modelling and historical data from these Councils was provided for review. In addition, there are several previous studies of the Macintyre River catchment undertaken in earlier stages of Melbourne to Brisbane Inland Rail assessment, and other documents identified as potentially relevant to the proposal including:

- North Star to NSW/QLD Border – Hydrologic and Hydraulic Modelling – Illabo to Stockingbingal and North Star to Yelarbon, July 2016 (01-2700-PD-P00-DE-0010), SMEC, 2016
- Melbourne-Brisbane Inland Rail – 2016 Phase 1 Continuity Alignment Report North Star to Yelarbon (01-2700-PD-P00-DE-0008), WSP, 2016
- Melbourne-Brisbane Inland Rail - 2017 Phase 2 Preparatory Alignment Assessment Report North Star to Yelarbon (01-2700-PD-P00-DE-0011) WSP, 2017
- Draft Floodplain Management Plan for the Borders River Valley Floodplain, Department of Planning, Industry and Environment, 2018
- Toomelah Flood Risk Assessment, Water Technology, 2016
- Goondiwindi Environs Flooding Investigation, Cardno Lawson Treloar, 2007
- Moree and Environs Floodplain Risk Management Plan, Parsons Brinckerhoff, 2008
- Boggabilla Floodplain Risk Management Plan, BGE, 2015.

Key studies have been reviewed in detail and summarised below. A proposed approach for modelling the Macintyre River catchment was developed for the hydrologic and hydraulic modelling based on the information and data available.

**North Star to NSW/QLD Border – Hydrologic and hydraulic modelling – Illabo to Stockingbingal and North Star to Yelarbon (01-2700-PD-P00-DE-0010) – Phase 1 Hydrological works**

This study included development of an RORB hydrologic model and TUFLOW hydraulic model to assess impacts of the proposed rail alignment from North Star to Yelarbon. The modelling works undertaken for this assessment were subsequently updated in December 2016 and May 2017 to assess changes to the alignment.

**Melbourne-Brisbane Inland Rail – 2016 Phase 1 Continuity Alignment Report North Star to Yelarbon (01-2700-PD-P00-DE-0008) and Melbourne-Brisbane Inland Rail - 2017 Phase 2 Preparatory Alignment Assessment Report North Star to Yelarbon (01-2700-PD-P00-DE-0011)**

The Phase 1 Continuity Alignment Report documents the assessment of changes to the alignment, with updates to the models documented in 01-2700-PD-P00-DE-0010 North Star to NSW/QLD Border | Hydrologic & Hydraulic Modelling – Illabo to Stockingbingal and North Star to Yelarbon.

Key findings from review of these studies in relation to the hydrologic (RORB) and hydraulic models (TUFLOW) developed were:

- The results of the calibration of the hydrologic and hydraulic models demonstrated that the models replicate peak water levels well; however, they are poor at representing flood volume and flow rates. This may be a significant issue in the Macintyre River catchment where storage effects influence flood behaviour and runoff response. As a consequence, the Phase 1 hydrologic and hydraulic models will require recalibration.
- The RORB model uses single rainfall points rather than spatial variation of rainfall
- The design flood hydrology is based on ARR 1987 rather than ARR 2016 (Basis of Design requirement)
- The resulting hydraulic model can be generally considered to be non-compliant with the design requirements. Several areas have been identified for re-consideration before the modelling is adopted for further assessment, being:
  - A hydrologic model would need to be developed and calibrated to meet the requirements of the design requirements in terms of the ARR 2016 design hydrology
  - The hydraulic model would need to be re-calibrated to ensure consistency with available flood records

In addition, there were several common issues identified from studies of the catchment. These are reproduced below in accordance with reference report; Melbourne-Brisbane Inland Rail – 2016 Phase 1 Continuity Alignment Report North Star to Yelarbon (01-2700-PD-P00-DE-0008):

- Complex nature of waterway connections
- Whalan Creek is not a permanent watercourse at its upstream end, but relies on flood overflows from the Macintyre River
- Development of levees, channels, etc., has affected flood flow distribution across the floodplain
- The existing non-operational rail embankment affects flood behaviour
- The smaller waterways on the southern floodplain are susceptible to erosion and movement during flood events.

**Draft Floodplain Management Plan for the Borders River Valley Floodplain, 2018**

The Floodplain Management Plan for the Borders River Valley Floodplain is being finalised (at the time of this investigation). The plan provides a framework for coordinating and assessing development works on a whole of valley basis. The plan will have effect for ten years from commencement.

As part of the plan, hydrologic and hydraulic models (URBS, RAFTS and TUFLOW) have been established for the assessment of development impacts on flood characteristics within the floodplain. The hydrology uses previously established models from the Border Rivers Floodplain Hydraulic Analysis (Lawson and Treloar 1998). The URBS models were originally developed by the BoM for the Weir River and Macintyre Brook. The hydrologic models were not modified for the Draft Floodplain Management Plan, 2018. Details of the Lawson and Treloar, 2018 models are provided in Appendix 6 of the Draft Floodplain Management Plan for the Borders River Valley Floodplain, 2018 and are replicated below for information purposes.

The catchment delineation of the URBS models is summarised in Table 5.1.

**Table 5.1 URBS Models**

| Modelled catchment | Catchment area ($km^2$) |
|---|---|
| Dumaresq River | 9,093 |
| Macintyre River | 6,892 |
| Weir River | 4,760 |
| Macintyre Brook | 3,983 |
| Croppa Creek (including Back Creek and Mobbindry Creek) | 2,401 |
| Commoron Creek | 2,317 |
| Yarrill Creek | 2,070 |
| Ottleys Creek | 1,375 |

Major storages in the catchments including Pindari Dam, Glenlyon Dam and Coolmunda Dam were included in the hydrologic models with stage storage and discharge characteristics to provide for the appropriate routing functions.

The hydrologic models were calibrated to the 1976 and 1996 floods. The calibration focused on achieving a reasonable match between simulated recorded water level and hydrographs at the gauging stations. DPIE have identified constraints with calibrating to the 1976 flood event due to the uncertainty in floodplain conditions at the time and floodplain changes since 1976. As such, the 1996 model was weighted higher for calibration than the 1976 flood event. The purpose of the 1976 flood event modelling was to assess what a 1976 event would look like if it occurred in current floodplain conditions.

Table 5.2 and Table 5.3 present the calibration summary comparing modelled and recorded peak flood levels for the two calibration events. There was no available stream gauging information for Yarrill Creek, Commoron Creek and Ottleys Creek catchments.

**Table 5.2 1976 event calibration summary**

| Catchment | Gauging station | Recorded peak flood height (m) | Modelled peak flood height (m) |
|---|---|---|---|
| Macintyre Brook | Terraine<br>Inglewood CBM<br>Inglewood | 5.9<br>11.6<br>11.8 | 5.7<br>11.1<br>11.8 |
| Dumaresq River | Bonshaw Weir<br>Texas<br>Oaky Creek<br>Beebo | 7.9<br>10.3<br>5.4<br>5.0 | 7.8<br>10.4<br>5.3<br>5.0 |

| Catchment | Gauging station | Recorded peak flood height (m) | Modelled peak flood height (m) |
|---|---|---|---|
| Macintyre River | Pindari Dam TW<br>Ashford<br>Wallangra<br>Holdfast* | 7.6<br>9.5<br>8.6<br>8.9 | 7.6<br>9.7<br>8.6<br>9.4 |

**Table notes:**

1 The Holdfast gauge on the Macintyre River appears to have stopped while floodwaters were still rising and the peak level was not recorded

**Table 5.3 1996 event calibration summary**

| Catchment | Gauging station | Recorded peak flood height (m) | Modelled peak flood height (m) |
|---|---|---|---|
| Macintyre Brook | Inglewood<br>Booba Sands | 9.8<br>8.9 | 9.2<br>9.0 |
| Dumaresq River | Bonshaw Weir<br>Texas<br>Beebo<br>Mauro | 5.9<br>7.4<br>4.7<br>8.5 | 6.1<br>7.7<br>4.5<br>8.5 |
| Macintyre River | Ashford<br>Wallangra<br>Holdfast | 5.3<br>5.9<br>8.4 | 5.2<br>6.1<br>8.5 |
| Weir River | Walter Gunn Bridge | 4.7 | 4.8 |

The DPIE hydraulic model uses current conditions including existing and approved development in floodplain, with small (1996 flood event) and large (1976 flood event) historical rainfall events used to assess flood conditions and development impacts. Under the plan, development in the floodplain will require assessment using the DPIE hydraulic model to determine if the development meets nominated criteria in terms of changes to flood characteristics (i.e. changes in peak flood levels, changes in flowpaths, flow rates and velocities). A TUFLOW GPU hydraulic model has been developed.

The TUFLOW model covers an area of approximately 1.1 million hectares extending from approximately 50 km upstream of Boggabilla to 40 km downstream of Mungindi. The main watercourses within the model are the Macintyre River, Weir River, Boomi River and Barwon River.

The topography in the TUFLOW Model is defined using a high resolution digital elevation model (DEM). The DEM was created from a variety of LiDAR datasets including Macintyre 2013 and Gwydir 2013 datasets and supplemented to the north with Queensland LiDAR datasets. LiDAR was available for the majority of the modelled area. Where data was not available, Shuttle Radar Topography Mission 1-second (~30m) resolution elevation data was used.

The TUFLOW model grid size is 30 m. Topography modifiers were incorporated into the model to ensure that topographic features such as roads, rail and levee banks are correctly represented. There are no drainage structures included in the TUFLOW model (culverts/bridges). DPIE hydraulic roughness values are presented in Table 5.4.

**Table 5.4 DPIE hydraulic model roughness**

| Land use type | Roughness value |
|---|---|
| Waterway Channel | 0.03 |
| Farmland | 0.06 |
| Vegetation | 0.12 |

Boundary conditions have been incorporated in the DPIE TUFLOW model as follows:

- Inflows as flow versus time, extracted from the calibrated hydrologic models
- Downstream rating – normal flow boundary.

The DPIE hydraulic model was calibrated to 1996 and verified with 1976 (noting that the topographic conditions were difficult to replicate for the 1976 conditions). For the 1976 event topographic features (roads, rail, farm levees, farm channels etc., known not to be in place in 1976 were removed from the 1976 calibration hydraulic model.

Further discussion on the calibration results is provided in Section 7 of this report.

The following key findings can be drawn from the review of the Draft Floodplain Management Plan:

- The hydrologic and hydraulic models are calibrated to the 1996 event
- The 1976 flows are simulated with current topographic conditions for impact assessment (approximately 1% AEP event in Macintyre River)
- No design event analysis has been undertaken and historical event modelling is used for impact assessment of development on the floodplain.

DPIE is the custodian of the models and has provided the models to ARTC for review and use.

**Toomelah Flood Risk Assessment, Water Technology, September 2016**

A flood assessment was undertaken for the Toomelah Community in NSW. The township is located on the Macintyre River approximately 900 m upstream of the confluence of the Macintyre and Dumaresq Rivers. The TUFLOW hydraulic model was developed covering the township and surrounding areas. No hydrologic modelling was undertaken. The results were used to inform development of a Flood Emergency Management Plan (FEMP).

Key findings from the assessment are as follows:

- Flood depths were typically less than 1 m throughout the township in the 1976 flood event
- The peak level of the Probable Maximum Flood (PMF) at the existing Toomelah Community hall was estimated to be 230.52 m AHD
- A large breakout of flow occurs upstream of Toomelah community from the Macintyre River and flows in a south-westerly direction around the town (the anabranch)
- A significant increase in flows upstream of the Toomelah community results in only a proportionally small increase in peak water levels.

**Goondiwindi Environs Flooding Investigation, Cardno Lawson Treloar, 2007**

This study was carried out to assess the extent of flooding in the Goodiwindi Environs and the associated potential impact of development in the floodplain to inform risk management planning. The assessment utilised existing hydrology and developed a 2D hydraulic model in the SOBEK modelling software. The study predicted the 1976 flood event to be slightly less than a 1% AEP event, and proposed new levees and development apply a planning level of the 1% AEP with additional 300 mm freeboard.

## 5.2 Existing Case hydrologic modelling

For the Borders River catchment, one key suite of hydrologic models has been developed and adopted by most preceding studies. These are the URBS models from the study titled, Border Rivers Floodplain Hydraulic Analysis (Lawson and Treloar 1998). These models were sourced for use in the DPIE Border Rivers Floodplain Management Plan and have been provided by DPIE and adopted for this study (referred to in this report as the DPIE hydrologic models).

## 5.3 Existing Case hydraulic modelling

Several hydraulic models have been developed across the flood study area, these include:

- North Star to NSW/QLD Border, Hydrologic and Hydraulic Modelling – Illabo to Stockingbingal and North Star to Yelarbon, SMEC, July 2016 – TUFLOW model
- Draft Floodplain Management Plan for the Border Rivers Valley Floodplain, DPIE 2018 – TUFLOW model
- Toomelah Flood Risk Assessment Water Technology September 2016 – TUFLOW model
- Goondiwindi Environs Flooding Investigation, Cardno Lawson and Treloar, 2007 – SOBEK model
- Flood Study for Boggabilla, Lawson and Treloar, 2004 – SOBEK Model.

The most up-to-date hydraulic model with detail for topographic conditions is the DPIE Border Rivers Valley Floodplain model, which has been adopted for use in this investigation (referred to in this report as the DPIE hydraulic TUFLOW model). The other available models have been considered for comparison purposes of 1% AEP predicted flood levels and flows.

A review of the DPIE models has been undertaken for suitability in this study and the following is noted:

- Topographic modifiers were used to lower the main tributaries by 1 m to adjust the LiDAR data to represent bed level. Comparison to the current LiDAR of the flood study area, shows a 0.5 m difference between the bed levels of the datasets (areas outside of the watercourses are comparable). Therefore, for the current modelling a 0.5 m lowering has been applied where the more recent LiDAR dataset is available. The 1 m lowering has been maintained in the other locations as per the DPIE model.
- Farm levees have been applied as either vertical walls in the hydraulic model to above the extreme event water levels or approved heights where the development height is limited. Both datasets were provided by DPIE. Where vertical walls were applied, floodwaters do not overtop these levees in any events. This is assumed as conservative as there is an overall reduction in floodplain storage in the larger events. For the current assessment digitised levee lines cased on the current LiDAR will be applied in the model. The DPIE limited and unlimited heights have been used as a sensitivity.
- Topographic modifiers have been applied to raise the model topography to represent the crest levels of the roads. A section from Tucka Tucka Road was found to be blocking the anabranch affecting only smaller events. This section was removed in the current modelling as it does not represent what is occurring in reality.
- The 1976 calibration event flows have been factored up (20%) in the hydraulic model to achieve the calibration at the gauge. It is possible that the rainfall distribution may not have been picked up by the recorded gauges such that the rainfall was underestimated. Table 7.7 shows that while the calibration was reasonable between the recorded and simulated peaks, it was typically lower, suggesting the flows may be lower than those that occurred. DPIE have indicated that the 1976 hydrology will not be revisited due to the uncertainty in changes to catchment conditions between now and 1976.
- The DPIE 1996 model is factored up (60 per cent) to achieve calibration levels downstream of Goondiwindi.
- Due to the uncertainty in the flows it was determined that inclusion of another flood event would improve confidence in the DPIE hydrologic and hydraulic model performance. Modelling of the 2011 event has been included in the current modelling and is discussed in the following sections.

## 5.4 Survey data

The flood study area includes many existing roads, levees, the non-operational rail line and road crossings over the waterways. Road and rail embankments, levees and other key features have been represented in the supplied DPIE model. The raw data (excluding LiDAR) has not been provided by DPIE.

The DPIE model utilises a 10 m by 10 m gridded DEM derived from a variety of LiDAR survey datasets including Macintyre 2013 and Gwydir 2013 datasets. Where LiDAR was not available the dataset was supplemented with the Shuttle Radar Topography Mission 1-second (~30 m) resolution elevation data. The extents of the data sources for the DPIE model are shown in Figure A2. The majority of the sub-model area is covered by LiDAR data, and mostly covered by LiDAR collected for the proposal as shown in Figure A2.

Two sets of LiDAR data were collected for the proposal design, to supplement the DPIE data. The first was collected between September 2014 and January 2015. The second was collected in November 2019 to provide details of current topographic conditions. This dataset provides a recent capture of the floodplain conditions and floodplain features.

Where the proposal LiDAR merges with the DPIE LiDAR differences in the levels are typically within 100 mm with some isolated areas up to 300 mm (with the proposal dataset being lower). These areas of difference are outside of the main flow paths and do not appear to have an impact on peak water levels. Therefore, no adjustment to the DPIE LiDAR elevations was undertaken.

Ground survey at five sites was completed to validate the 2014/15 LiDAR data and provide additional information for validation of floodplain waterways bed elevations.

The survey results showed the 2014/15 LiDAR Ground TIN to be consistently higher than the ground survey verification sections by 3 mm to 146 mm which is in line with what would be expected for LiDAR data of this nature as explained below:

- LiDAR survey data often measures the top of any vegetation such as grass, bushes or trees where it cannot directly measure the ground and therefore is quite often higher in level than ground survey.
- The LiDAR 2015 data was specified with the following metadata:
  - Vertical = 0.15 m (68 per cent confidence level or 1 sigma)
- The LiDAR 2019 data was specified with the following metadata:
  - Vertical = 0.15 m (95 per cent confidence level or 2 sigma)

With a maximum mean difference of approximately 150 mm it is considered that the 2019 LiDAR data is appropriate for the purposes of this assessment.

## 5.5 Existing drainage structure data

Drainage structure geometry information was obtained from the following sources:

- Previous studies
- Site inspection
- Field and validation survey (refer Section 5.4).

Details of existing drainage structures are presented in Appendix B.

## 5.6 Stream gauge data

Stream gauges are used to provide a record of observed stream levels. These were originally manually recorded staff levels (typically recorded on a daily basis with more frequent records during flood events) with modern gauges providing a continuous automated record.

Although levels may be adequate for flood warning services, hydrologic investigations are usually more interested in streamflow. A rating curve is required to convert recorded levels into an equivalent stream discharge. The most reliable source of data for deriving a rating curve are actual in stream flow measurements taken during flood events. These are often difficult/dangerous to obtain during major flood events unless the gauge site is located near an appropriate structure spanning the waterway (e.g. a high-level bridge), and so are often only available for low to moderate flows. The rating must therefore be extrapolated to higher flows. This is often based on simple power-law best fit through the available data, however ideally the extrapolation is based on more reliable means, such as a hydraulic model calibrated to the reliable part of the rating curve.

Other factors can also influence the short- and long-term reliability of the rating curve. Changes to channel bed or roughness, either long-term or during a flood event, can change the hydraulic properties and hence the rating curve. Gauges are preferably located at a hydraulic control, either natural or artificial, (e.g. a weir), or where the bed material has low erodibility. The gauge location may also not produce a singular relationship between flow and level. This may occur in areas where there is significant floodplain storage, and hence the level is dependent on the duration and rate of change of the flow, or the gauge location may be affected by backwater from a downstream tributary.

Figure A3 presents the existing stream gauge stations available for historical events within the Border Rivers catchment. These stations are listed in Table 5.5.

Peak height records have been obtained from the BoM for use in developing a series of partial peak flood flows for input into the flood frequency analysis (FFA) at the Boggabilla and Goondiwindi stream gauges.

Continuous gauge recordings have been collected from the BoM Water Data Online website. This information has been used for the additional calibration event (2011) modelling.

**Table 5.5 Stream gauges used for calibration**

| Gauge | Location | Period | Catchment area (km$^2$) | Rating ratio |
|---|---|---|---|---|
| 416002 | Macintyre River at Boggabilla | 22 Apr 1982 – Current | 22,600 | 89.5% |
| 416012 | Macintyre River at Holdfast | 18 Oct 1972 – Current | 6,740 | 42.2% |
| 416020 | Ottleys Creek at Coolatai | 9 Nov 1978 – Current | 402 | 10.1% |
| 416307 | Dumaresq River at Bonshaw Weir | 30 Jun 1966 – 29 Aug 1974 | 7,280 | 20.2% |
| 416310 | Dumaresq River at Farnbro | 14 Sep 1962 – Current | 1,310 | 11.4% |
| 416011 | Dumaresq River at Roseneath | 14 Jun 1972 – Current | 5,550 | 9.1% |
| 416415 | Macintyre Brook at Booba Sands | 17 Feb 1987 – Current | 4,092 | 49.7% |
| 416201A | Macintyre River at Goondiwindi | 20 Sep 1917 - Current | 23,090 | 94% |

The total catchment area of the Macintyre River at Boggabilla is 22,600 km$^2$, with the upstream gauging stations accounting for 18,154 km$^2$, or just over 80 per cent of the contributing catchment, which means that there is a residual catchment area of 4,446 km$^2$ which is ungauged.

The rating ratio of the stream gauges is the ratio of the maximum measured flow to the maximum observed flow at the site. This index provides an indication of how well the site is rated and hence how much confidence can be placed in the high stage rating.

## 5.7 Rainfall data

Historical rainfall data in the form of daily rainfall and pluviograph records was required for the calibration of the URBS hydrologic model for the 2011 event. This information was sourced from the BoM, and from the SMEC 2016 RORB model. Data was obtained for the 2011 flood event.

Figure A3 presents the historical rainfall stations available within the Border Rivers catchment. These are listed in Table 5.6.

Continuous rainfall records are generally required for hydrologic model calibration. However, as the event-based data for 1976 and 1996 is already included in most of the URBS model files, additional continuous rainfall record was only required for the 2011 and 1996 (for Ottleys Creek) flood events. This list of rainfall stations is not exhaustive, the gauges selected were based on the quality of data available and suitability for the catchment model for the 2011 validation event.

**Table 5.6 Rainfall data used for calibration events**

| Gauging Station Number | Location | Period of operation | Type |
|---|---|---|---|
| **1976** | | | |
| 41022 | Dalveen | Mar 1887 – Current | Daily |
| 41060 | Leyburn | Mar 1959 – May 2006 | Daily |
| 41122 | Yelarbon | May 1923 – Feb 2011 | Daily |
| 41139 | Wyaga | Feb 1901 – Jan 2009 | Daily |
| 41175 | Applethorpe | Jul 1966 – Current | Daily |
| 56018 | Inverell Research Centre | May 1949 – Current | Continuous |
| 56217 | Guyra | May 1973 – May 1978 | Daily |
| **1996** | | | |
| 56111 | Danthonia TM | Aug 1958 – Aug 2018 | Daily |
| 56128 | Swan vale TM | Jan 1957 – Dec 2017 | Daily |
| 56123 | Paradise Stn TM | Jan 1954 – Mar 2012 | Daily |
| 56139 | Ben Lomond TM | Jan 1959 – Jul 2018 | Daily |
| 54159 | Bukkulla TM | Jan 1987 – Nov 2013 | Daily |
| 56165 | Elsmore TM | Sep 1964 – Dec 2012 | Daily |
| 41360 | New Bengalla TM | Aug 1928 – Aug 1996 | Daily |
| 541053 | Farnbro TM | Not available | Daily |
| 41495 | Terraine TM | | Daily |
| 541063 | Dalveen TM | | Daily |
| 41507 | New Kildonan TM | | Daily |
| 41519 | Booba Sands TM | | Daily |
| 41040 | Greenmount (Nav) | | Daily |
| 56008 | Deepwater | Mar 1889 – Current | Daily |
| 54012 | Coolatai Orana | Jun 1901 – Mar 2018 | Daily |
| 54032 | Coolatai Willunga | Aug 1903 – May 2018 | Daily |
| **2011** | | | |
| 41122 | Yelarbon | May 1923 – Feb 2011 | Daily |
| 41175 | Applethorpe | Jul 1966 – Current | Daily |
| 41097 | Inglewood_Forest | Feb 2,000 – May 2015 | Continuous |
| 41100 | Texas_Post_Office | Jan 1897 – Current | Daily |
| 41116 | Wallangarra_Po | Apr 1888 – Current | Daily |
| 41430 | Glenlyon_Dam | Aug 1974 – May 2018 | Daily |
| 41457 | Coolmunda_Dam | Oct 1976 – Current | Daily |
| 54012 | Coolatai Orana | Jun 1901 – Mar 2018 | Daily |
| 54032 | Coolatai Willunga | Aug 1903 – May 2018 | Daily |

## 5.8 Anecdotal flood data

Anecdotal flood data for the historical flood events has been collected from many sources including:

- Previous studies
- DPIE
- Landholders and stakeholders including Goondiwindi Regional Council, Gwydir Shire Council and Moree Plains Regional Council.

Anecdotal data includes information obtained from a wide range of sources and as such it is of varying levels of accuracy and reliability. The anecdotal data has been used to assess of the performance of the hydraulic model to replicate historical flood conditions.

## 5.9 Community consultation

Community consultation has been undertaken at key milestones in alignment with ARTC's Stakeholder Engagement Strategy. Flood impacts continue to be a significant issue raised by stakeholders, particularly relating to the crossing of the Macintyre River floodplain. Issues raised by key stakeholders have included:

- The existing landform (levee banks) not being reflected in hydraulic model
- The proposed design will change water flow paths and velocities
- The proposed design will increase flood levels in Goondiwindi
- The proposed design will increase flood levels in the Toomelah Community
- There is too much risk associated with the proposed alignment
- ARTC have not engaged with local flood specialists.

In response to these concerns, ARTC agreed to facilitate several technical flooding workshops to verify the calibration of the hydraulic model against historical events and seek endorsement the hydraulic model as a suitable design tool for the NS2B proposal. ARTC's consultation objectives for these workshops were to:

- Seek feedback and inputs on flooding conditions on the Macintyre River floodplain model from key stakeholders
- Verify the hydraulic model calibration against historical flood events reflecting all available information including community inputs
- Seek endorsement that the hydraulic model was a suitable to use as a design tool for the NS2B proposal
- Present ARTC's proposed mitigation measures to directly-affected landowners prior to EIS submission
- Proactively seek feedback from directly-affected landowners to incorporate into the EIS
- Continue to provide the community with additional information in relation to ARTC's design criteria as the design progresses
- Build confidence in the feasibility design of the proposal alignment.

ARTC proactively arranged the technical workshops in April 2019 where it was identified the existing landform, in particular the levee bank heights on the Macintyre River floodplain, were not accurately represented in the hydraulic model. It was agreed however, with enhancement to the model topography, that the Macintyre River floodplain hydraulic model was a suitable design tool for the NS2B proposal.

At the start of June 2019, a further workshop was convened with the local Councils, local flood specialists and the DPIE to work through an updated Macintyre River floodplain hydraulic model. At this session it was demonstrated how the previous feedback had been incorporated into the hydraulic model and the floodplain crossing solution design. It was again acknowledged, the hydraulic model is a suitable design tool for the NS2B proposal, however Inland Rail needed to build more confidence in the feasibility design.

In addition to the technical workshops, ARTC committed to continuing to engage with directly affected landowners and key stakeholders. Interaction with stakeholders and the community has included:

- Presentation at CCC meetings
- Presentation to Goondiwindi Regional Council, Gwydir Shire Council and Moree Shire Council
- Three technical flood model workshops involving:
  - Department of Planning, Industry and Environment
  - Goondiwindi Regional Council and Moree Plains Council
  - Local flood specialists (community recommended)
  - Directly affected landowners and interested community members.

Table 5.7 summarises stakeholder engagement activities that ARTC have completed in relation to the proposed crossing of the Macintyre River floodplain and issues identified by key stakeholders.

**Table 5.7 Summary of flood related stakeholder engagement activities**

| Timing and activity | Topics discussed | Issues raised/feedback received | ARTC responses/actions |
|---|---|---|---|
| Phase 1 | Alignment selection – Macintyre River crossing location | ▪ Community not consulted during crossing selection<br>▪ Concerns around flooding and crossing location<br>▪ Too much risk associated with crossing location<br>▪ Alignment should follow the existing Boggabilla rail track | ▪ During Phase 1, ARTC undertook six face-to-face meetings, a Toomelah Community LALC meeting and three Council meetings<br>▪ These sessions involved seeking information from the community to confirm the modelling findings<br>▪ The MCA Phase 1 route alignment strategy was made publicly available on NS2B ARTC website<br>▪ Route D1 was selected through the ARTC MCA process<br>▪ Option A was recognised as the preferred community alignment within the MCA<br>▪ ARTC are guided by the same flood immunity criteria regardless of which route is selected<br>▪ ARTC implemented an education campaign to help the community better understand the flood design criteria |
| Scoping of EIS | Preliminary Macintyre River floodplain crossing design | ▪ Community not consulted during crossing selection<br>▪ Concerns around flooding and crossing location<br>▪ Too much risk associated with crossing location<br>▪ Alignment should follow the existing Boggabilla rail track<br>▪ Concerns around the DPIE's model and data used to develop the NS2B flood model<br>▪ Impacts of flooding as a result of levee bank heights in the area<br>▪ Impacts of proposal on flow paths, velocities and peak water levels<br>▪ Impacts to farming operations due to flooding | ▪ ARTC undertook seven face-to-face meetings, three CCC meetings, three Council presentations, six community drop-in meetings and a Toomelah LALC meeting during the preliminary Macintyre River floodplain crossing design phase<br>▪ A technical flood workshop engaging three recommended local flood specialists<br>▪ Feedback received from technical flood workshop was incorporated into the flood model and preliminary design Apr – Jun 19<br>▪ Inland Rail run specialised engagement campaigns about the hydrology modelling<br>▪ Inland Rail will continue to work with landowners concerned with hydrology throughout the detailed design, construction and operational phases of the proposal<br>▪ ARTC will continue to work with directly impacted landowners affected by the alignment throughout the detailed design, construction and operational phases of the proposal |

| Timing and activity | Topics discussed | Issues raised/feedback received | ARTC responses/actions |
|---|---|---|---|
| | | ▪ Impact of proposal on in-flows to irrigators | ▪ Education program on flood immunity design criteria which has been used to develop the feasibility design<br>▪ MCA route alignment strategy made publicly available on NS2B ARTC website.<br>▪ Monthly e-newsletters implemented to further disseminate information around the MCA process and review, flood modelling updates, technical documents available on the NS2B |
| Feasibility Design | Macintyre River floodplain crossing solution | ▪ Raised concerns around the economic impact between option A and D1<br>▪ Economic opportunities lost due to Option D1 alignment<br>▪ Perceived flood impacts | ▪ ARTC undertook seven face-to-face meetings and design correspondence, a Toomelah community LALC meeting and three Council presentations.<br>▪ Two technical flood workshops<br>▪ Monthly e-newsletters implemented to further disseminate information around the MCA process and review, flood modelling updates, technical documents available on the NS2B |

In summary, ARTC have completed a comprehensive consultation package to provide the community with more information and certainty around the flood model and Macintyre River floodplain crossing solution. In addition to this, ARTC will:

- Continue to work with landowners concerned with hydrology and flooding throughout the detailed design, construction and operational phases of the proposal
- Continue to work with directly impacted landowners affected by the alignment throughout the detailed design, construction and operational phases of the proposal
- Continue to work with local Councils, DPIE and local flood specialists throughout the detailed design, construction and operational phases of the proposal.

## 5.10 Site inspection

A site inspection was undertaken on 9 to 10 April 2018. During the site inspection, proposed waterway crossings were inspected with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment and waterway environment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection. The site visit confirmed that the catchment conditions were consistent with the LiDAR and aerial imagery provided. Crossings inspected include:

- Macintyre River
- Whalan Creek
- Forest Creek
- Mobbindry Creek.

Existing drainage structures observed on site were used to validate the model with details presented in Appendix B.

## 5.11 Water quality

Water quality has been assessed in the North Star to NSW/QLD Border Surface Water Quality Report.

# 6 Development of models

## 6.1 Hydrologic models

The hydrologic models used for this assessment were sourced from DPIE. The following models were provided:

- Macintyre Brook – URBS
- Macintyre River – URBS
- Dumaresq River – URBS
- Weir River – URBS (not applicable for this study, as catchment located below Goondiwindi)
- Ottleys Creek – RAFTS.

These models were sourced by DPIE from the 1998 study titled, Border Rivers Floodplain Hydraulic Analysis (Lawson and Treloar 1998). The original model was developed without GIS interface for catchment delineation. Therefore, GIS delineation of sub-catchments is not available. The sub-catchment centroids have been created in GIS, to present the general location of the sub-catchments and are presented in Figure A3. Local catchment details including catchment delineation and catchment parameters are included in the drainage assessment.

Runoff from rainfall directly onto the DPIE hydraulic model (and therefore the sub-model) area was not included in the hydraulic model. The runoff generated from the hydraulic -model area would be small in comparison to the upstream catchment flows and more importantly will have left the model before peak flows from upstream enter the model domain. Therefore, local flows within the hydraulic model boundary were not considered relevant for this assessment. It is noted that local catchment flows, and local drainage structures have been assessed as a separate drainage analysis.

In addition, local catchment hydrologic models were developed for Strayleaves Creek, Forest Creek, Back Creek and Mobbindry Creek and their inflows included into the TUFLOW hydraulic model.

## 6.2 Hydraulic model

### 6.2.1 Border Rivers Valley Floodplain Model

The DPIE TUFLOW hydraulic model was sourced for use in this assessment. The following points outline the information supplied and used from the DPIE model:

- Base model
  - The DPIE TUFLOW model named TUFLOW_model_009 was supplied on 28 June 2018 by DPIE. Model updates for the limited and unlimited height levee structures were provided on 15 March 2019.
- Calibration
  - The June 2018 DPIE model with the March 2019 updates was used as the base model for the calibration of the historical flood events.
  - For the historical event scenarios, the current topographic features (levees) in the model were removed where the development was not constructed at that time as determined from community consultation and provided from DPIE. The 2015 LiDAR was also added to the model to improve topographic definition.

Model roughness as determined by the DPIE calibration process is presented in Table 6.1.

Table 6.1 DPIE Hydraulic model roughness

| Land use type | Value |
|---|---|
| Waterway | 0.03 |
| Floodplain | 0.06 |
| Vegetated floodplain | 0.12 |

These values are in agreement with the conditions observed on site, with farmland comprising a mix of grazing and crops, and the main river channel reasonably smooth. The vegetation roughness value is applicable for bushland areas and dense crops. Photograph 6.1 to Photograph 6.3 provide some examples observed on site.

**Photograph 6.1 Grazing land**

**Photograph 6.2 Macintyre River channel**

**Photograph 6.3  Vegetation**

### 6.2.2 Hydraulic sub-model

A localised hydraulic sub-model was created based on the regional DPIE TUFLOW hydraulic model. The sub-model allows for reduced simulation time and a finer scale model to be developed as the design progresses (DPIE model has a 40 m grid and significant simulation times).

The sub-model boundaries have been established to capture the extents of potential impacts. Generally, any increase to flood levels from a structure in the floodplain are expected to occur upstream of the structure. Therefore, the downstream boundary was not required to extend further than downstream of the Boggabilla stream gauge which was used for calibration purposes. However, following community feedback of concerns of potential impacts of the proposal on flood levels in Goondiwindi, the hydraulic model was extended to downstream of Goondiwindi and recalibrated to the Goondiwindi and Boggabilla Gauges. The model was extended a significant distance downstream to ensure there were no tailwater effects at Goondiwindi from the downstream boundary. The hydraulic sub-model extents are shown in Figure A4.

In developing the hydraulic sub-model, flows were extracted from the DPIE model and applied as inflow boundaries within the sub-model (in accordance with the Borders River Floodplain Management Plan procedures). A normal depth slope boundary of 0.001 was applied to the downstream boundary. A sensitivity test was undertaken on the downstream boundary using varying slope boundaries and comparison of flood levels at Goondiwindi to test the location of the boundaries. There was no resulting change of peak water levels at Goondiwindi.

Model runs including the calibration events were undertaken using a 30 m grid, to allow efficient run times. The Existing Case and the Developed Case have both been simulated with a 15 m grid for the 1% AEP event only. These results and differences from the 30 m grid model are presented in later sections.

When the hydraulic sub-model was established it was validated against the DPIE regional hydraulic model to ensure results were consistent. The hydraulic sub-model water levels were found to be within 10 mm of the DPIE regional hydraulic model and therefore, considered to suitably replicate the DPIE regional hydraulic model results.

Existing drainage structures observed on site were used to validate the model and are presented in Appendix B.

# 7 Joint calibration

## 7.1 Introduction

The hydraulic model is located in the lower section and downstream of the hydrologic models. The hydraulic model inflows therefore consist of total reach flows where the hydraulic model boundary intersects any major tributary (more than one upstream catchment.

Hydrologic models are based on simplistic empirical runoff routing equations using coefficients determined primarily by calibration to a specific point of interest. By contrast, hydraulic models are more physically based, providing a (relatively) realistic representation of the catchment geometry and solving equations of motion within the model domain. Some differences between the hydrologic and hydraulic routing must realistically be expected. Nevertheless, the hydraulic model should closely replicate the flow characteristics (attenuation, timing etc.) that in the hydrologic model have been validated by calibration to historical flood events.

The hydraulic model must also produce flood levels consistent with the flows. This can be confirmed by comparison with flood levels recorded during historical flood events, although the reliability is dependent upon the accuracy of the modelled flows, which are in turn dependent on the accuracy of the recorded rainfall. Further validation across a wide range of flows can be achieved by comparison of the modelled level-flow relationships at the stream gauge sites with the gauge ratings, which allows the level-flow relationship to be confirmed without necessarily having to exactly match a specific flow.

The TUFLOW hydraulic models have been validated using historical events. The primary objectives of the calibration process have been:

- To confirm hydraulic model roughness factors required to match level-flow relationships at the stream gauges, particularly those where the ratings are well defined by in-streamflow measurements
- To confirm that the flood routing through the TUFLOW hydraulic model reasonably matches the hydrologic model (TUFLOW physically represents storage and other catchment characteristics that are represented in hydrology software by empirical coefficients) and that the adopted roughness parameters do not adversely affect the timing or attenuation of the flood routing.

The historical events were selected to represent a range of magnitudes and duration. A summary of each event is outlined in the sections below.

## 7.2 Approach

The following process was undertaken for the calibration against the historical events:

- Determination of the available rainfall and stream gauging data for historical events
- Selection of appropriate historical events to use for the calibration (1976 and 1996 selected by DPIE based on available data and magnitude of the events, 2011 event selected based on recency of event and reasonable amount of data for calibration)
- Community consultation/site visit to outline the calibration process and seek for any anecdotal information on the events to support the calibration process
- Development of hydrologic and hydraulic models and simulation of 1976, 1996 and 2011 historical events, comparison of modelling results to stream gauging records, anecdotal flood level data and community information
- Presentation of calibration results to stakeholders and landholders for feedback
- Refinement of hydrologic and hydraulic models based on feedback
- Finalisation of calibration to historical events.

The following sections document the progression of this methodology and present the outcomes of the calibration.

## 7.3 Historical events

The Border Rivers floodplain has experienced many recent flood events. Table 7.1 provides a summary of the major floods at Boggabilla and Table 7.2 provides a summary of major floods at Goondiwindi.

**Table 7.1 Major historical flood events (Boggabilla)**

| Date | Peak water level (m) | Peak water level (m AHD) | Peak discharge ($m^3$/s) |
|---|---|---|---|
| Jan 2011 | 12.645 | 221.12 | 3,800 |
| Feb 1976 | 12.800 | 221.27 | 3,700 |
| Jan 1996 | 12.553 | 221.03 | 3,500 |
| Mar 1890 | 12.53 | 221.01 | 2,430 |
| Jan 1956 | 12.43 | 220.91 | 2,230 |
| Jul 1921 | 12.41 | 220.89 | 2,200 |
| Feb 1956 | 12.27 | 220.75 | 2.040 |
| Jul 1998 | 11.82 | 220.30 | 2,030 |
| Jul 1921 | 12.01 | 220.49 | 1,830 |

**Table note:**

1 Different rating curves were applied for pre and post changes to the Boggabilla gauge, with the URBS rating curve received from OEM applied for levels post 1991 and the WaterNSW rating curve applied for levels pre 1991.

**Table 7.2 Major historical flood events (Goondiwindi)**

| Date | Peak water level (m) | Peak water level (m AHD) | Peak discharge ($m^3$/s) |
|---|---|---|---|
| Jan 1996 | 10.62 | 218.20 | 1,767 |
| Jan 2011 | 10.62 | 218.20 | 1,767 |
| Feb 1976 | 10.50 | 218.08 | 1,560 |
| Jul 1998 | 10.43 | 218.01 | 1,586 |
| May 1983 | 10.40 | 217.98 | 1,557 |
| Dec 1970 | 10.34 | 217.92 | 1,528 |
| Jan 1956 | 10.27 | 217.85 | 1,506 |
| July 1984 | 10.25 | 217.83 | 1,424 |
| July 1950 | 10.13 | 217.71 | 1,440 |

The three highest floods on record at both the Boggabilla and Goondiwindi gauges have been considered in the calibration of the hydrologic models and hydraulic sub-model. At the Boggabilla gauge the 1976 has an estimated annual exceedance probability (AEP) of between 1 in 200 and 1 in 500, 1996 has an estimated AEP of between 1 in 30 to 1 in 50, and 2011 has an estimated AEP of between 1 in 60 to 1 in 75. AEPs at Goondiwindi gauge cannot be reliably derived as detailed in the FFA for both gauges in Section 8.2.3.2.

## 7.4 Hydrologic model calibration

The provided DPIE hydrologic models were calibrated to the 1996 and 1976 flood events. The adopted DPIE model parameters were not altered and were considered suitable for this assessment.

The DPIE hydrologic model was updated to include 2011 historical rainfall data. The 2011 event was included to provide a second recent event for calibration purposes as the catchment has changed significantly from 1976. In addition, the 2011 event provides further confidence in the ability of the hydraulic sub-model to replicate flooding characteristics in the flood study area in more recent conditions.

The previous RORB hydrologic model was used to source the 2011 rainfall data to input to the URBS models for assessment of the 2011 historical event. All other URBS parameters for the 2011 event have been derived during this assessment and are presented in Table 7.3.

- alpha = channel lag parameter
- beta = catchment lag parameter
- m = non-linearity parameter (0.8, in accordance with Australian Rainfall and Runoff guidelines).

**Table 7.3 Tributary adopted parameters**

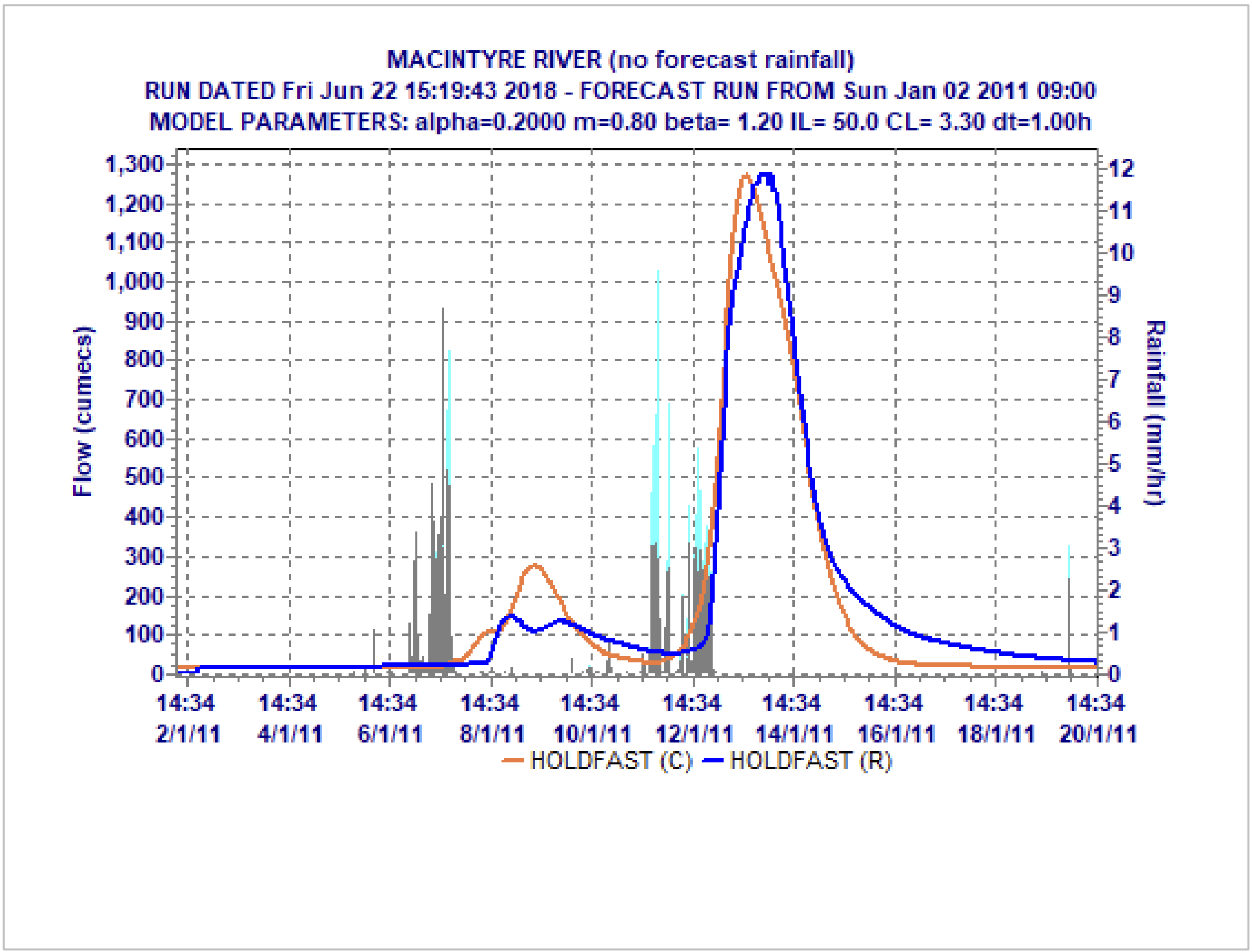
| Sub-catchment | Alpha | Beta | m |
|---|---|---|---|
| Macintyre Brook | 0.20 | 1.2 | 0.8 |
| Dumaresq River | 0.10 (2011, 1976)<br>0.20 1996) | 1.2 | 0.8 |
| Macintyre River | 0.20 | 1.2 | 0.8 |
| Ottleys Creek | 0.20 | 1.2 | 0.8 |
| Local catchments (Strayleaves, Forest, Back, and Mobbindry Creeks) | 0.20 | 1.2 | 0.8 |

Initial and continuing losses for the three historical rainfall events are presented in Table 7.4. The losses for the 1976 and 1996 events were provided from the DPIE model. For Ottleys Creek no calibration was undertaken for the 1976 rainfall event as there was no recorded streamflow data for that period within the catchment and in 2011 there was no event recorded for the catchment.

**Table 7.4 Initial and continuing loss parameters**

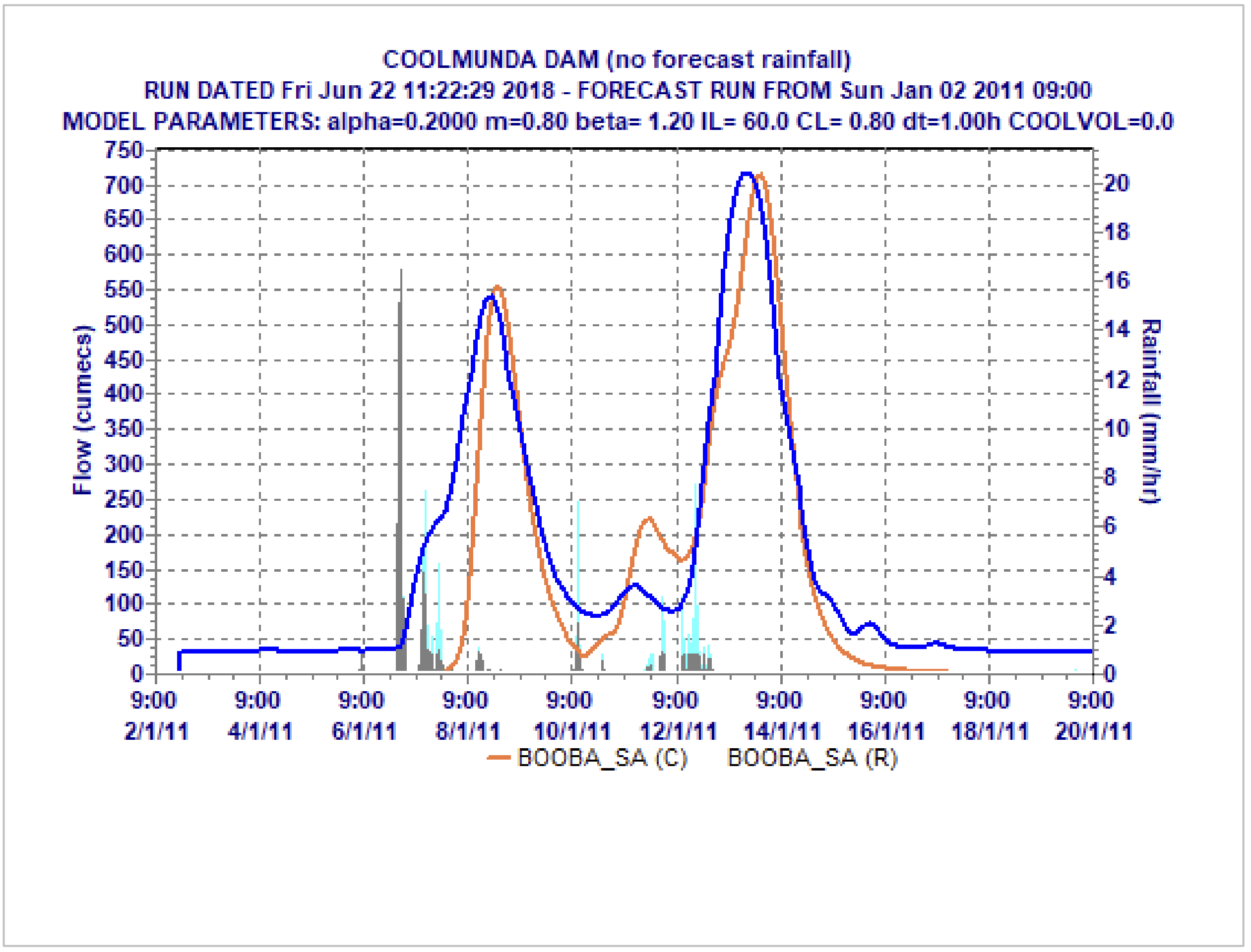
| Event | Sub-catchment | Initial loss (mm) | Continuing loss (mm/hour) |
|---|---|---|---|
| 1976 | Macintyre Brook<br>Dumaresq River<br>Macintyre River<br>Ottleys Creek<br>Local catchments | 0.0<br>42.9<br>36.5<br>n/a<br>36.5 | 2.50<br>4.34<br>2.32<br>n/a<br>2.32 |
| 1996 | Macintyre Brook<br>Dumaresq River<br>Macintyre River<br>Ottleys Creek<br>Local catchments | 25.0<br>40.0<br>26.2<br>100.0<br>26.2 | 2.00<br>0.94<br>0.85<br>0.85<br>0.85 |
| 2011 | Macintyre Brook<br>Dumaresq River<br>Macintyre River<br>Ottleys Creek<br>Local catchments | 60.0<br>47.0<br>50.0<br>n/a<br>50.0 | 0.80<br>0.50<br>3.30<br>n/a<br>3.30 |

Figure 7.1 to Figure 7.4 present the URBS model calibration results for the 2011 rainfall event for the Macintyre River, Dumaresq River and Macintyre Brook. The orange line in each figure is the hydrologic model discharge hydrograph and the blue line indicates recorded hydrograph. The grey bars represent rainfall removed by the applied losses and light blue bars show the residual rainfall applied to the hydrologic model.



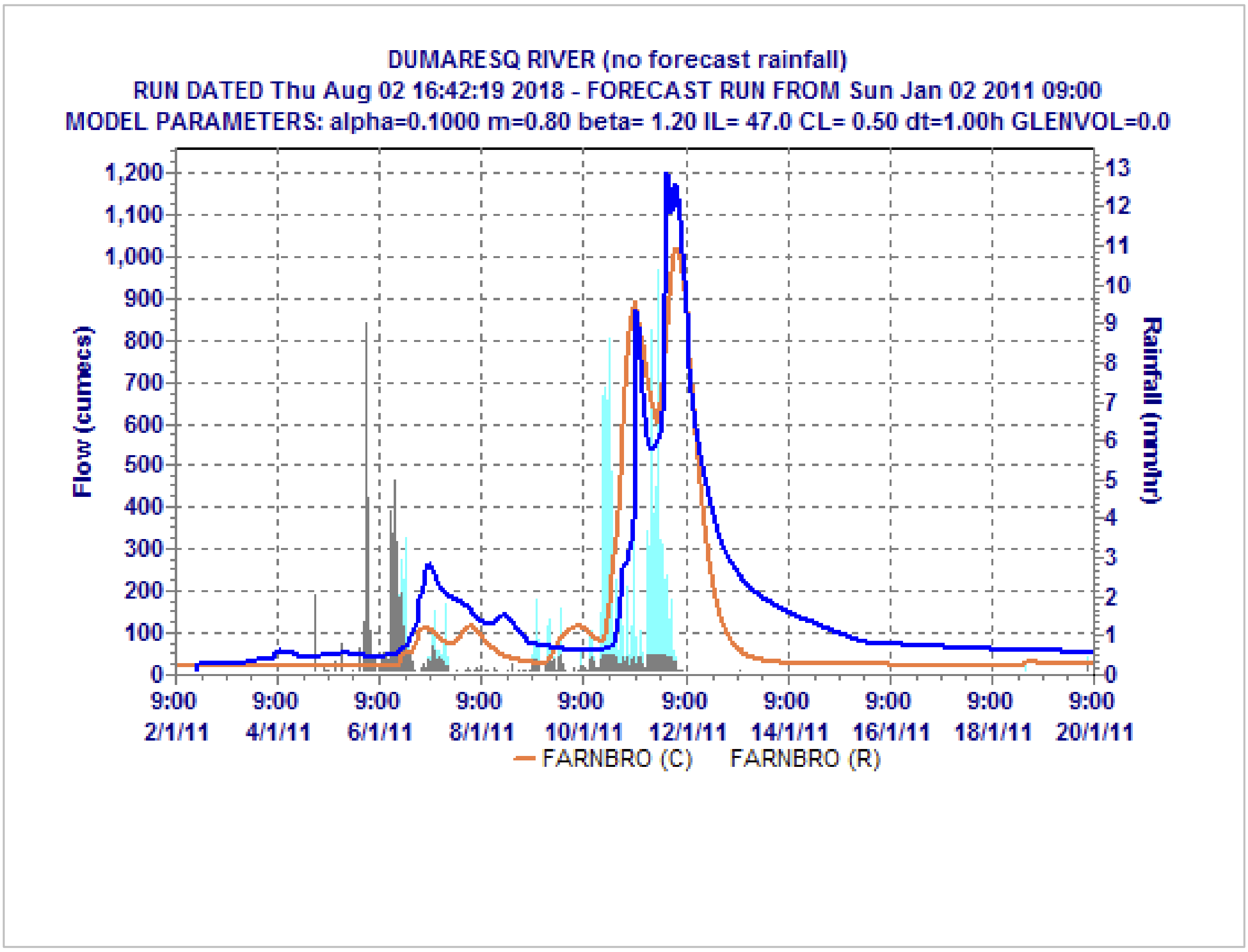

| Calculated | | | Recorded | | | Nate-Sutcliffe Ratio | Volume Ratio | Peak Ratio |
|---|---|---|---|---|---|---|---|---|
| Peak Flow (m3/s) | Volume (ML) | Time of Peak | Peak Flow (m3/s) | Volume (ML) | Time of Peak | | | |
| 1271 | 330716 | Jan 13 2011 16:00 | 1275 | 263657 | Jan 13 2011 23:00 | 0.9505 | 1.2543 | 0.997 |

**Figure 7.1** **Macintyre River 2011 calibration result (Holdfast - end of system)**



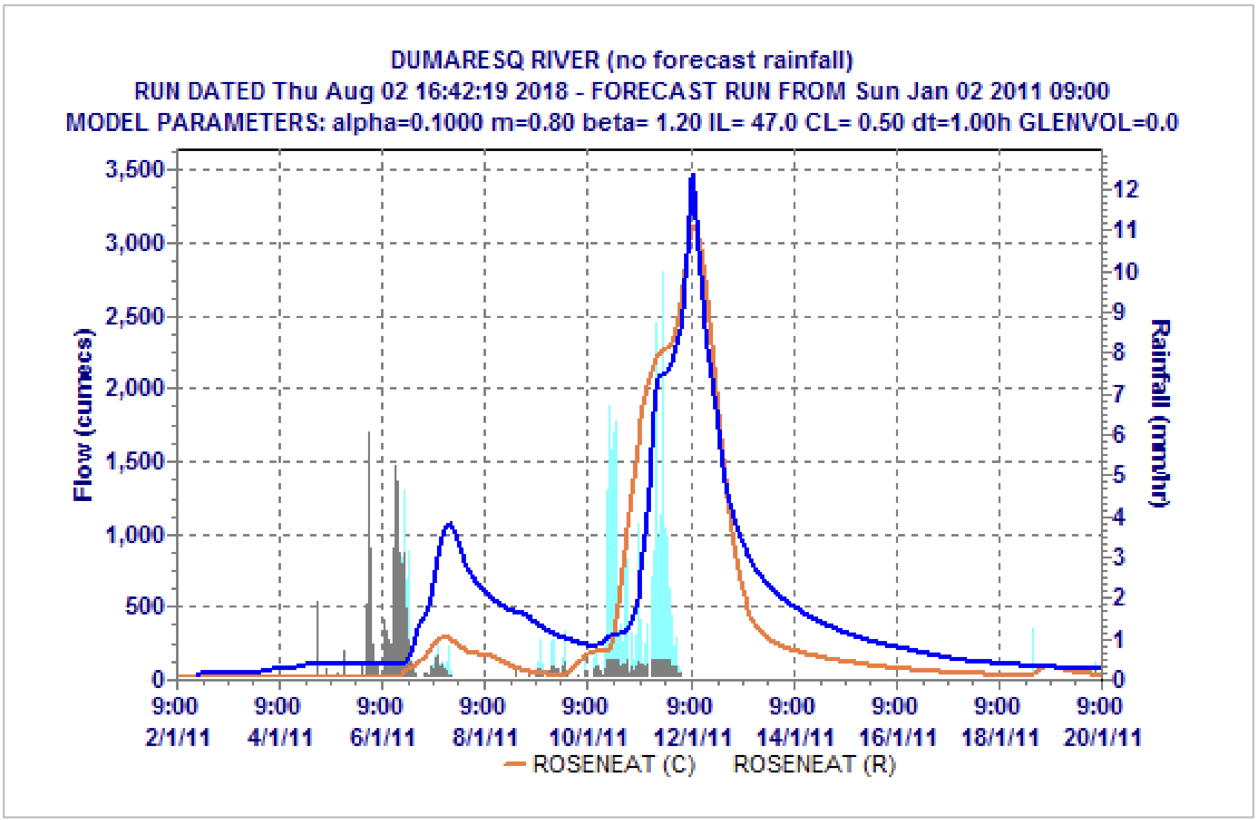

| Calculated | | | Recorded | | | Nate-Sutcliffe Ratio | Volume Ratio | Peak Ratio |
|---|---|---|---|---|---|---|---|---|
| Peak Flow (m3/s) | Volume (ML) | Time of Peak | Peak Flow (m3/s) | Volume (ML) | Time of Peak | | | |
| 715 | 169150 | Jan 13 2011 23:00 | 717 | 229356 | Jan 13 2011 16:00 | 0.7955 | 0.7375 | 0.9976 |

**Figure 7.2** **Macintyre Brook 2011 calibration result (Booba Sands - end of system)**




| Calculated | | | Recorded | | | Nate-Sutcliffe Ratio | Volume Ratio | Peak Ratio |
|---|---|---|---|---|---|---|---|---|
| Peak Flow (m3/s) | Volume (ML) | Time of Peak | Peak Flow (m3/s) | Volume (ML) | Time of Peak | | | |
| 1019 | 176290 | Jan 12 2011 04:00 | 1199 | 231153 | Jan 12 2011 00:00 | 0.7707 | 0.7627 | 0.8495 |

**Figure 7.3 Dumaresq River 2011 calibration result (Farnbro)**




| Calculated | | | Recorded | | | Nate-Sutcliffe Ratio | Volume Ratio | Peak Ratio |
|---|---|---|---|---|---|---|---|---|
| Peak Flow (m3/s) | Volume (ML) | Time of Peak | Peak Flow (m3/s) | Volume (ML) | Time of Peak | | | |
| 3123 | 539744 | Jan 12 2011 11:00 | 3471 | 718394 | Jan 12 2011 10:00 | 0.8005 | 0.7513 | 0.8998 |

**Figure 7.4 Dumaresq River 2011 calibration result (Roseneath)**

The hydrologic model calibration results indicate that the URBS models are replicating the 2011 event flows well at all sites considered.

Figure 7.5 shows the rainfall totals and temporal distributions of gross catchment rainfall upstream of the key sites for 2011 calibration model.

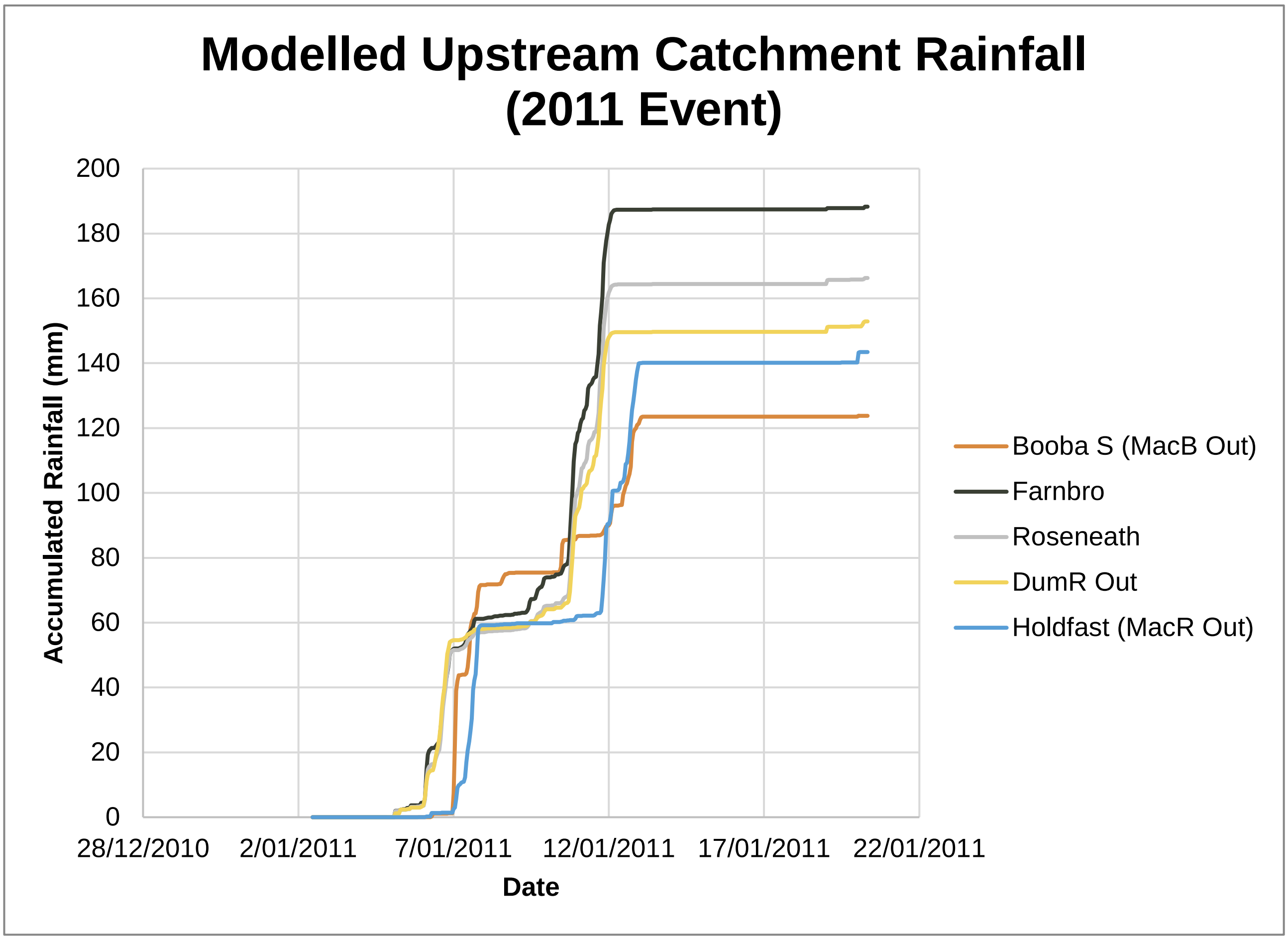
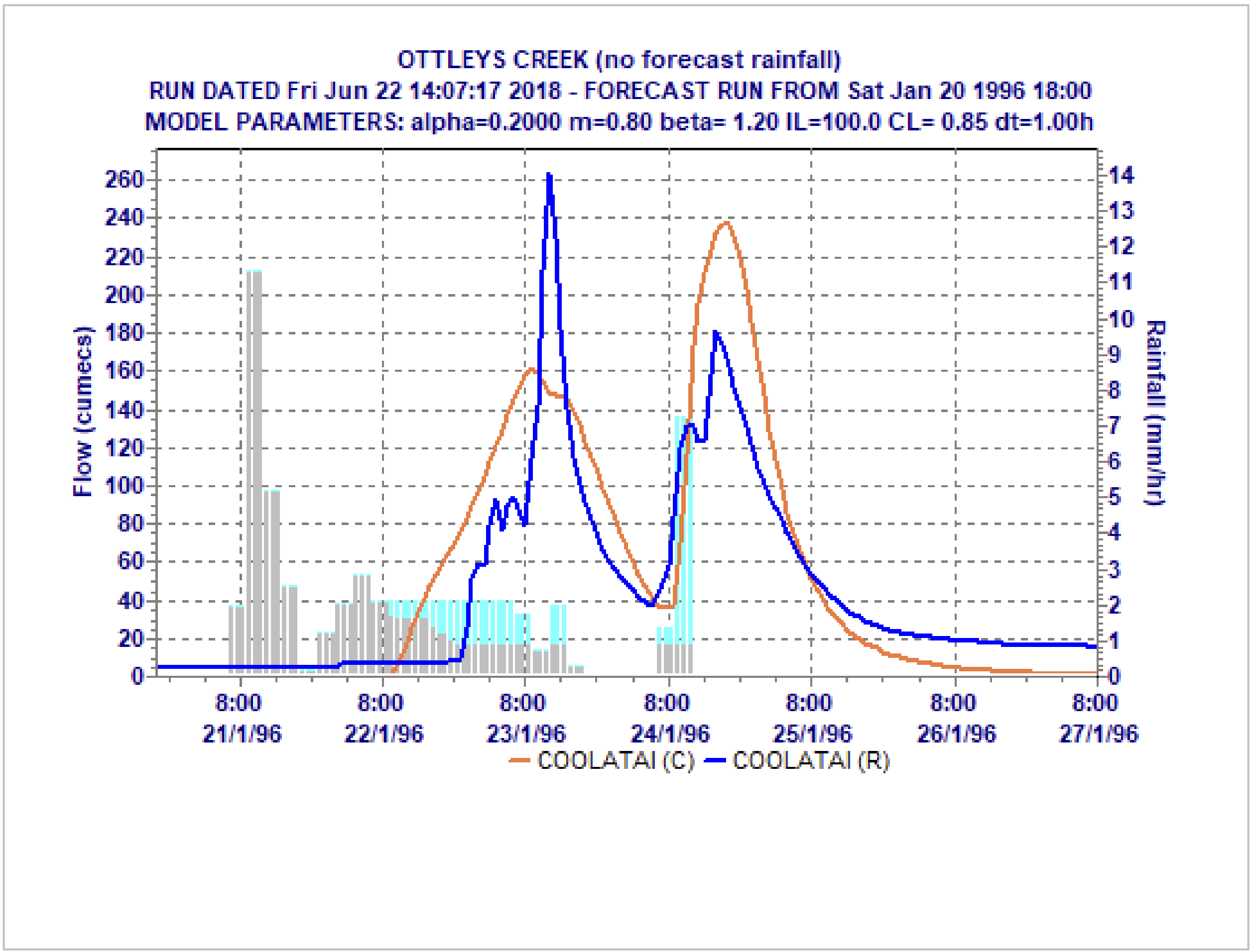



**Figure 7.5 Rainfall totals and temporal distributions for January 2011**

Ottleys Creek flows parallel to the Macintyre River in a northerly direction towards Boggabilla, joining the Macintyre River to the north-west of Holdfast. The Ottleys Creek model provided by DPIE was a RAFTS hydrological model. For consistency this model was converted to an URBS hydrological model.

The Ottleys Creek URBS model was developed using a GIS shapefile of the catchment boundary. The catchment area was divided into three approximately equal sub-areas (214.4 km$^2$) upstream of Coolatai and three approximately equal sub-areas (192.2 km$^2$) downstream of Coolatai.

The Ottleys Creek 1996 calibration model included both the Macintyre River URBS rainfall data and daily rainfall data at Coolatai (Orana) and Coolatai (Willunga). Figure 7.6 presents the results of the 1996 rainfall event in URBS.




| Calculated | | | Recorded | | | Nate-Sutcliffe Ratio | Volume Ratio | Peak Ratio |
|---|---|---|---|---|---|---|---|---|
| Peak Flow (m3/s) | Volume (ML) | Time of Peak | Peak Flow (m3/s) | Volume (ML) | Time of Peak | | | |
| 237 | 29716 | Jan 24 1996 17:00 | 263 | 25314 | Jan 23 1996 12:00 | 0.539 | 1.1739 | 0.9008 |

**Figure 7.6 Ottleys Creek 1996 calibration result**

It was not possible to achieve a good calibration at the Coolatai stream gauge for the 1996 flood event due to limited data. The hydrological model was tested for sensitivity to a range of model parameters and the calibration was not improved. It was not possible to replicate the recorded discharge for this event. It is noted the rainfall data was of limited temporal definition and is likely that the unrepresentative rainfall is the main issue with the calibration. As the flows are minor compared to the overall catchment flows, calibration parameters consistent with the other calibrated catchments were applied.

The model could not be verified to the 1976 event due to lack of stream gauge data, while the 2011 event was not a large event on the Ottleys Creek sub-catchment. Figure 7.7 shows that total recorded rainfall at Coolatai was only about 27 mm for the 2011 event as compared to the other sub-catchments which showed total recorded rainfall of about 104 mm. Correspondingly, peak recorded discharge was less than 250 ML/d (approximately 3 $m^3$/s).

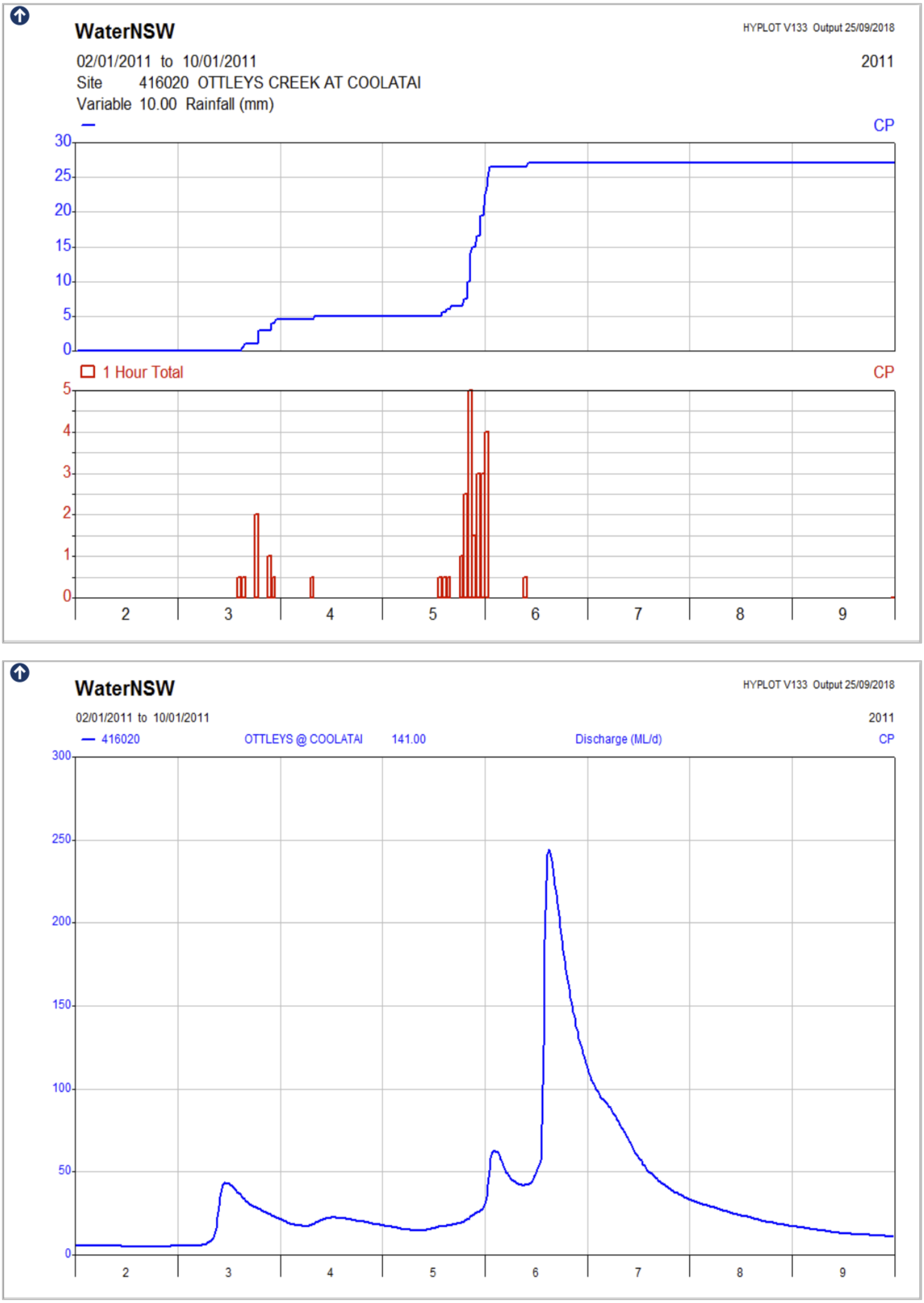



**Figure 7.7 Coolati rainfall 2011 event**

## 7.5 Hydraulic model calibration

The hydraulic sub-model was calibrated to the 1996 and 2011 flood event and verified against the 1976 flood event. The URBS hydrologic model flows were included in the hydraulic sub-model for the three historical events and simulations undertaken to assess the ability of the hydraulic sub-model replicate peak water levels recorded during the historical events.

The models provided by DPIE had been calibrated to the 1996 flood event and validated against the 1976 event. Therefore, the hydraulic calibration parameters set in the DPIE hydraulic model were adopted in the sub-model for these events and for the 2011 event. The 2011 event was tested in the hydraulic sub-model to test the ability of the hydraulic sub-model to replicate a recent flood event.

The topography included in the model was based on the following:

- 1976 – DPIE model topography (2015 LiDAR) with levees removed as per DPIE definition and consultation outcomes
- 1996 – DPIE model topography (2015 LiDAR) with levees removed as per DPIE definition and consultation outcomes
- 2011 – DPIE model topography (2015 LiDAR).

It is noted that the 1976 and 1996 flows were factored up in the DPIE hydraulic model, most likely to account for uncertainties in the rainfall distribution. DPIE has indicated that the factoring of flows has been applied to achieve calibration downstream of Goondiwindi and does not significantly impact the proposal area. The results presented are for the factored flows (20 per cent increase for 1976 and 60 per cent for 1996). This assessment has also tested the impact of unfactored 1976 and 1996 flows on the calibration of the hydraulic model, and is presented for comparison and discussed in Section 7.5.3. The 2011 flows in the hydraulic sub-model were not factored for this assessment. The addition of the 2011 historical event provides further confidence in the ability of the hydrologic and hydraulic models to replicate historical events.

### 7.5.1 Recorded data

#### 7.5.1.1 Boggabilla Gauge

The Boggabilla stream gauge was in place and operational for all three historical events. It is noted that the location of the stream gauge changed between the 1976 and 1996 flood events. The Boggabilla Weir, completed in 1991, rendered the existing gauge ineffective due to ponding behind the weir. The new gauge was established in October 1991 downstream of the weir (Goondiwindi Environs Flooding Investigation 2007). The current location of the stream gauge is shown on Figure A3. The previous and existing gauge locations are shown in Figure 7.8. The recorded gauge levels for the three historical events are shown in Table 7.5. The stream gauge records levels and the recorded flows are derived from a rating curve derived for the gauge location. The rating curve has changed over time with the changes to the gauge location and this is likely to have produced the higher flow corresponding to a lower level in 2011 as compared with 1976.

As discussed further in Section 8.2.3, the current rating (used for the 1996 and 2011 floods in Table 7.5) is based on four high-flow measurements recorded during the 1996 flood that included breakout flows into the Whalan Creek system upstream of Boggabilla. This is a key issue that must be taken into account when comparing rated flows for events prior to 1996.


**Figure 7.8** Historical and existing Boggabilla Gauge locations

**Table 7.5** Boggabilla Gauge recorded levels and derived flows

| Event | Recorded level (m AHD) | Rated flow ($m^3/s$) | Rated gauge flow (ML/D) |
|---|---|---|---|
| 1976 | 221.27 | 3,700 | 319,680 |
| 1996 | 221.03 | 3,486 | 301,190 |
| 2011 | 221.12 | 3,803 | 328,579 |

#### 7.5.1.2 Goondiwindi Gauge

The Goondiwingi Gauge (416201A) has been operational since September 1894. The gauge is located immediately downstream of the Gunsynd Way/McLean Street Bridge. The recorded levels and derived flows for the three historical flood events are shown in Table 7.6.

**Table 7.6** Goondiwindi Gauge recorded levels and derived flows

| Event | Recorded level (m AHD) | Rated flow ($m^3/s$) | Rated gauge flow (ML/D) |
|---|---|---|---|
| 1976 | 218.08 | 1,560 | 134,784 |
| 1996 | 218.19 | 1,767 | 152,669 |
| 2011 | 218.195 | 1,767 | 152,669 |

Further details of the gauges and their reliability are discussed in detail in Section 7.5.3.

### 7.5.2 Anecdotal data

Anecdotal information for the three historical events was obtained from many sources including:

- Previous studies – modelled and recorded flood heights, from landholders and local government
- DPIE – flood heights and aerial imagery from landholders and local government
- Stakeholder engagement including landholders – historical flood photography and knowledge.

### 7.5.3 Joint calibration outcomes

#### 7.5.3.1 Boggabilla gauge

The recorded and predicted flood levels and flows at the Boggabilla stream gauge are presented in Table 7.7. As discussed in Section 7.5.1 the current rating curve includes floodplain flows that break out into Whalan Creek and Morella Watercourse, i.e. represents flows across the floodplain upstream of Boggabilla. For comparison purposes flows have been extracted at two locations as presented in Figure A12 being Macintyre River 4 (US Boggabilla) to give the floodplain wide flow and Boggabilla 1 (DS Bogabilla) to give the flows at Boggabilla, as presented in Table 7.7.

**Table 7.7 Comparison of results at the Boggabilla stream gauge**

| Event | Recorded gauge data | | | TUFLOW rainfall | TUFLOW results | | |
|---|---|---|---|---|---|---|---|
| | Level (m AHD) | Flow US of Boggabilla | Flow DS of Boggabilla | | Level (m AHD) | Flow US of Boggabilla | Flow DS of Boggabilla |
| 1976 | 221.27 | n/a[1] | 3,700 $m^3/s$ 319,600 ML/d | Unfactored | 221.18 (-0.09m) | n/a[1] | 3,626 $m^3/s$ *318,300 ML/d* |
| | | | | Factored | 221.22 (-0.05m) | n/a[1] | 3,836 $m^3/s$ *331,400 ML/d* |
| 1996 | 221.03 | 3,486 $m^3/s$ 301,200 ML/d | 2,485 $m^3/s$ [2] 214,700 ML/d | Unfactored | 220.91 (-0.12m) | 3,175 $m^3/s$ *274,300 ML/d* | 2,542 $m^3/s$ *219,600 ML/d* |
| | | | | Factored | 221.11 (+0.08m) | 5,104 $m^3/s$ *441,000 ML/d* | 3,237 $m^3/s$ *279,700 ML/d* |
| 2011 | 221.12 | 3,803 $m^3/s$ 328,600 ML/d | n/a | Unfactored | 221.07 (-0.05m) | 4,449 $m^3/s$ *384,400 ML/d* | 3,057 $m^3/s$ *264,100 ML/d* |

**Table notes:**

1 1976 event rating curve only considered flows at Boggabilla and not the full floodplain
2 From flow measurement data

The predicted versus recorded levels and flows for 1976, 1996 and 2011 are presented in Figure 7.9 to Figure 7.14 respectively.

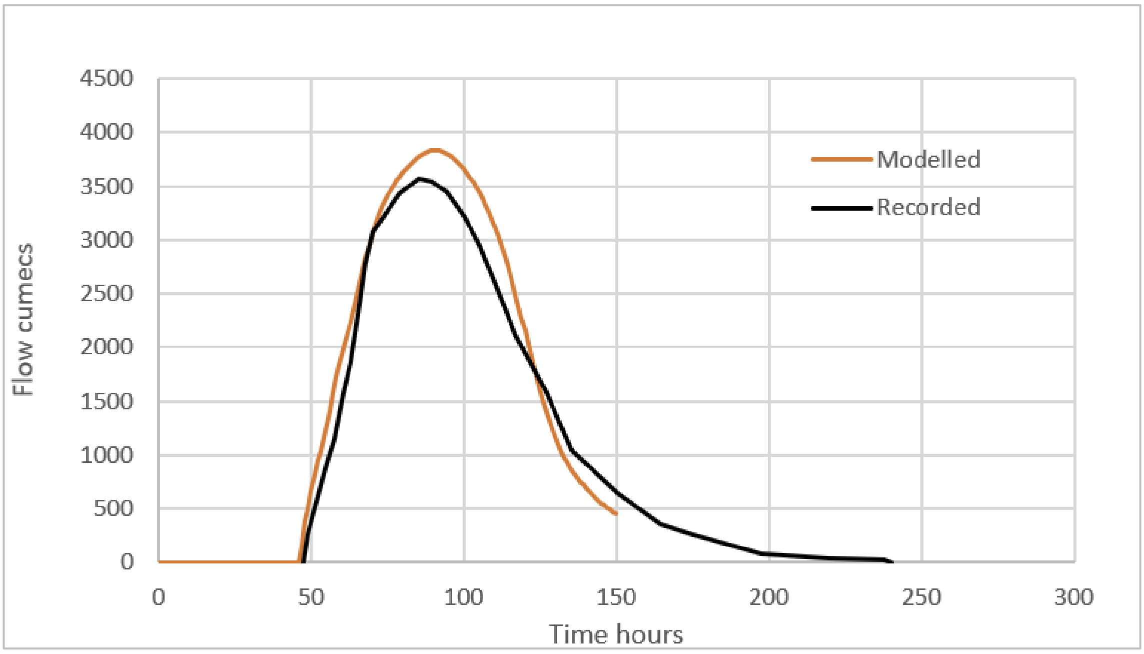



**Figure 7.9 Boggabilla Gauge – 1976 flows recorded and predicted**

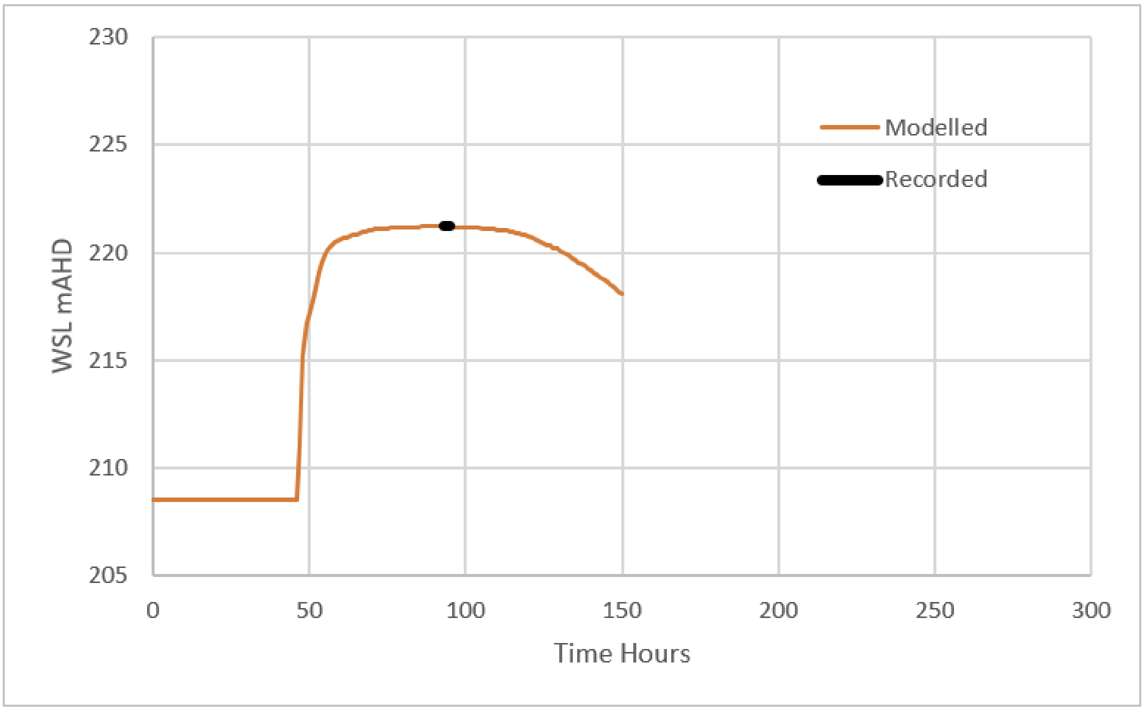



**Figure 7.10** **Boggabilla Gauge – 1976 levels recorded and predicted (time series not available for 1976 recorded level)**

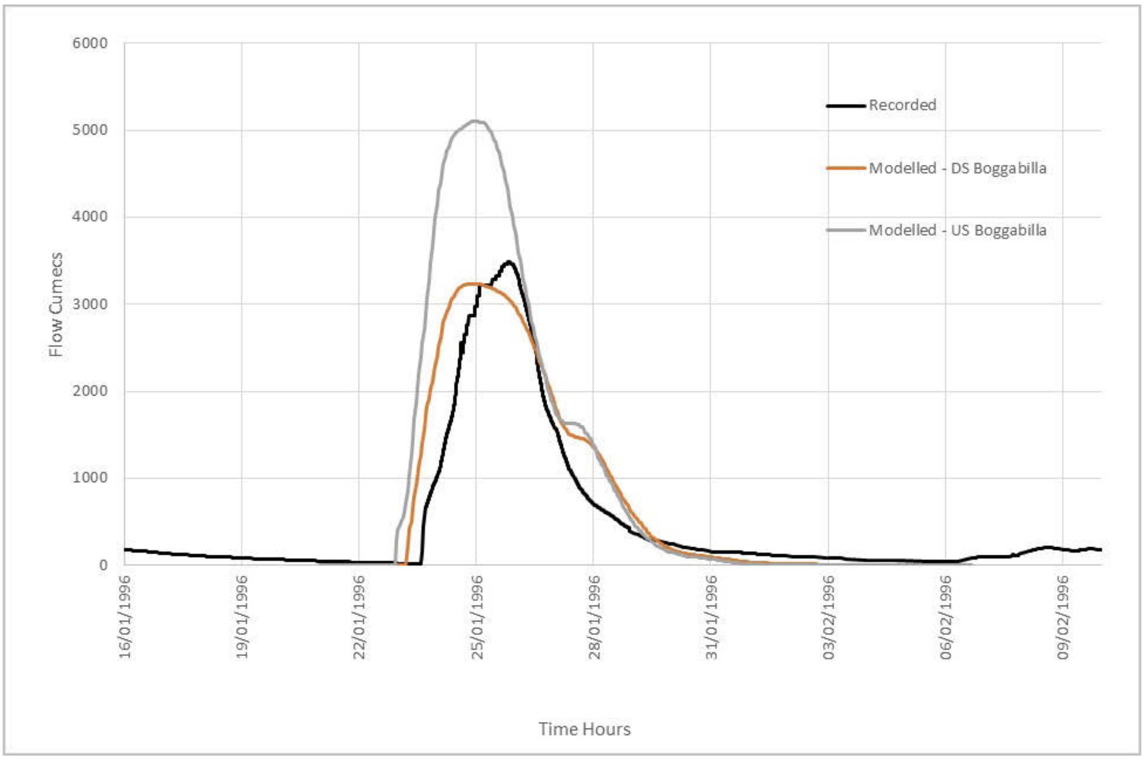



**Figure 7.11** **Boggabilla Gauge – 1996 flows recorded and predicted**

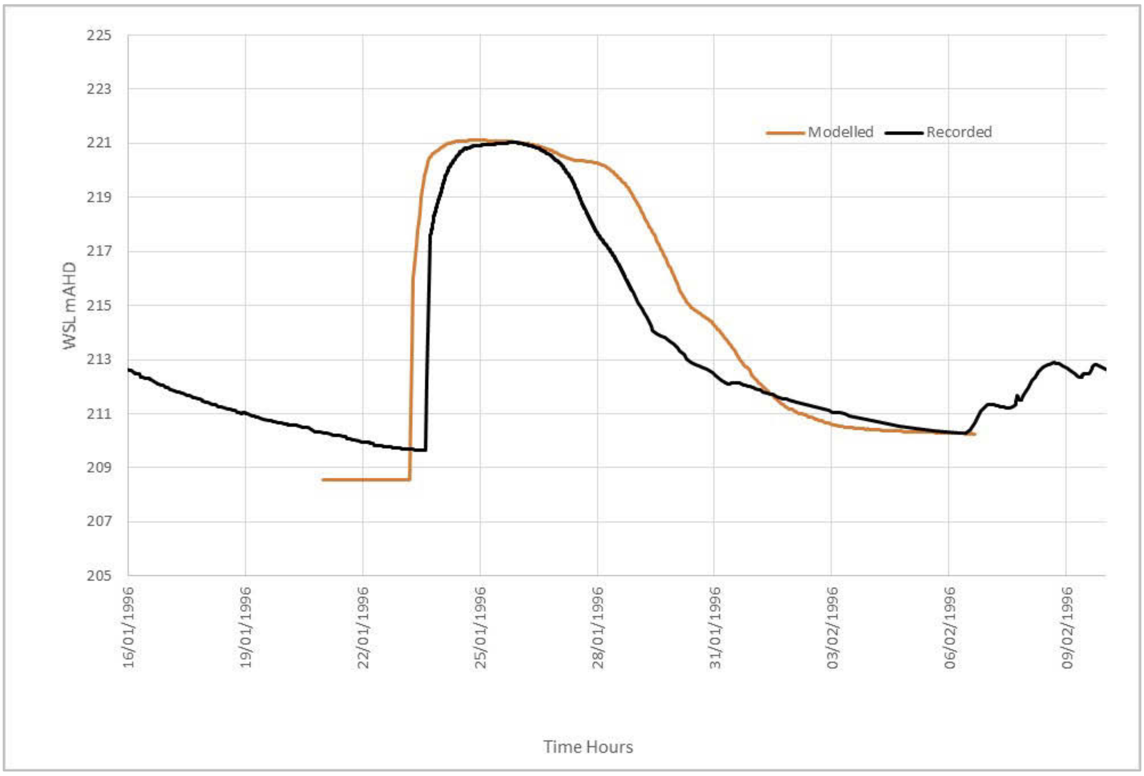



**Figure 7.12 Boggabilla Gauge – 1996 levels recorded and predicted**

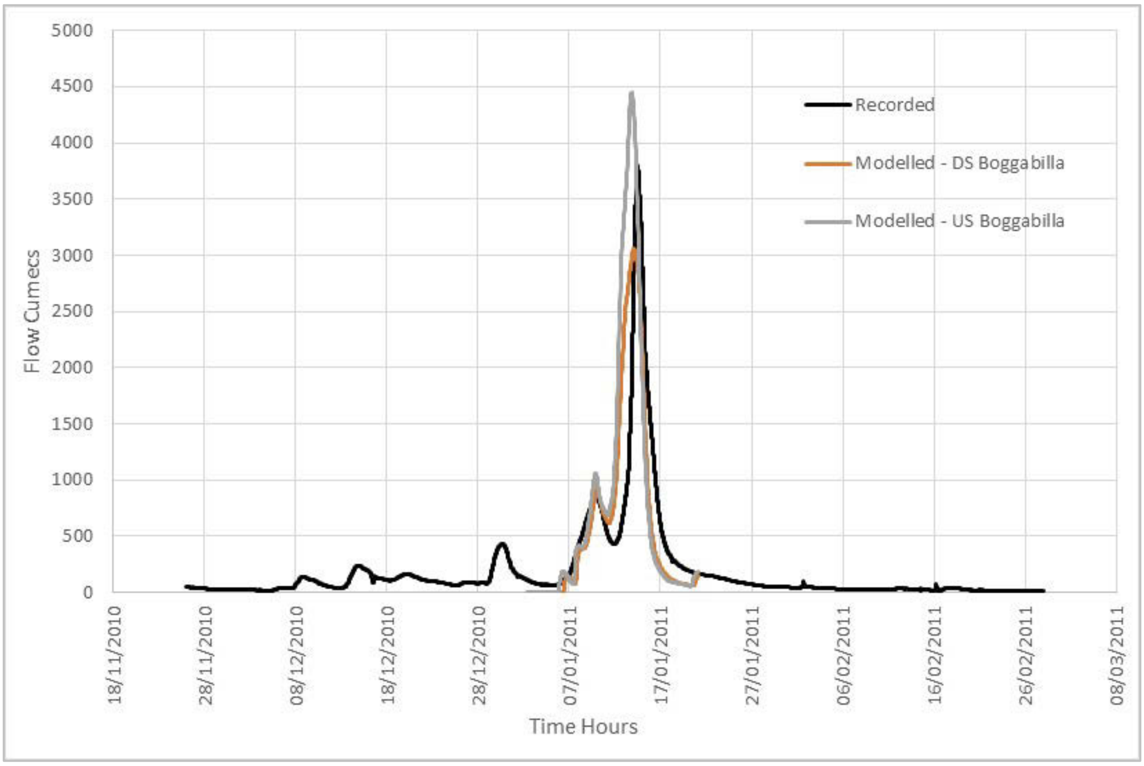



**Figure 7.13 Boggabilla Gauge – 2011 flows recorded and predicted**

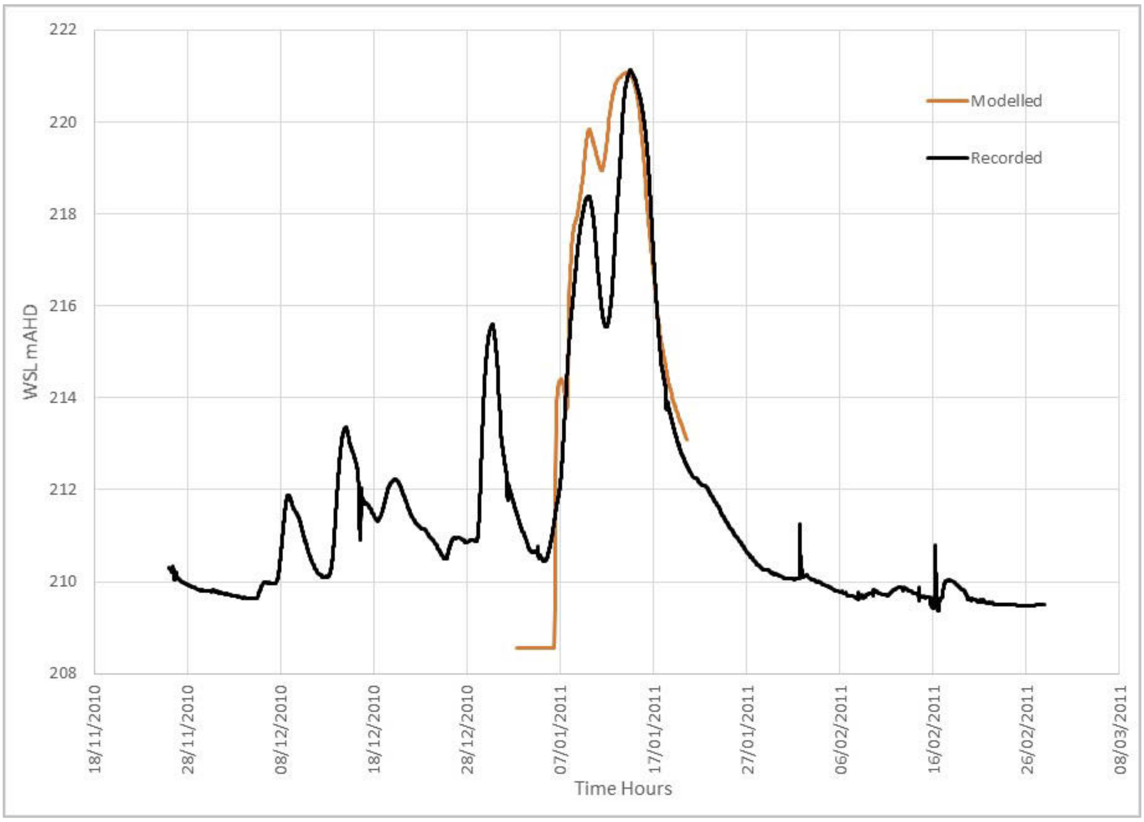



**Figure 7.14** **Boggabilla Gauge – 2011 levels recorded and predicted**

The TUFLOW hydraulic model results match the peak water level at the Boggabilla gauge to within 50 mm. The sensitivity analysis using unfactored and factored flows demonstrates that the peak water level is relatively insensitive to flow for events of this magnitude. The rated peak flow downstream of Boggabilla lies roughly midway between the unfactored and factored flows. The modelled flows upstream of Boggabilla are significantly higher than the flows predicted using the current rating. However, comparison of the rating curve projection above the highest gauge flow (the 1996 flood) suggests that it may significantly underestimate the flow.

The 1996 flood event theoretically provides good information as flows were recorded close to the flood peak. The unfactored TUFLOW results show relatively good agreement of the recorded flood level and flows upstream and downstream of Boggabilla. The factored TUFLOW hydraulic model results significantly overestimate the peak levels and flows.

The TUFLOW model provides a good match of the 2011 flood levels however, as with the 1976 event, the modelled peak flows upstream of the gauge are higher than the rated flows suggesting that there is significant sensitivity and uncertainty in the projection of the rating. This issue is also discussed in Section 8.2.3.2.

### 7.5.3.2 Goondiwindi gauge

The recorded and predicted flood levels and flows at the Goondiwindi stream gauge are presented in Table 7.8.

**Table 7.8** **Comparison of results at the Goondiwindi stream gauge**

| Event | Recorded level (m AHD) | TUFLOW modelled level (m AHD) | Rated gauge flow ($m^3/s$) | Rated gauge flow (ML/D) | TUFLOW modelled flow ($m^3/s$) | Modelled flow (ML/day) |
|---|---|---|---|---|---|---|
| 1976 | 218.08 | 218.42 (+0.34) | 1,560 | 134,784 | 2,072 | 179,021 |
| 1996 | 218.19 | 218.43 (+0.24) | 1,767 | 152,669 | 2,069 | 178,762 |
| 2011 | 218.195 | 218.42 (+0.23) | 1,767 | 152,669 | 1,995 | 172,368 |

The predicted versus recorded levels and flows for 1976, 1996 and 2011 are presented in Figure 7.15 to Figure 7.20 respectively.

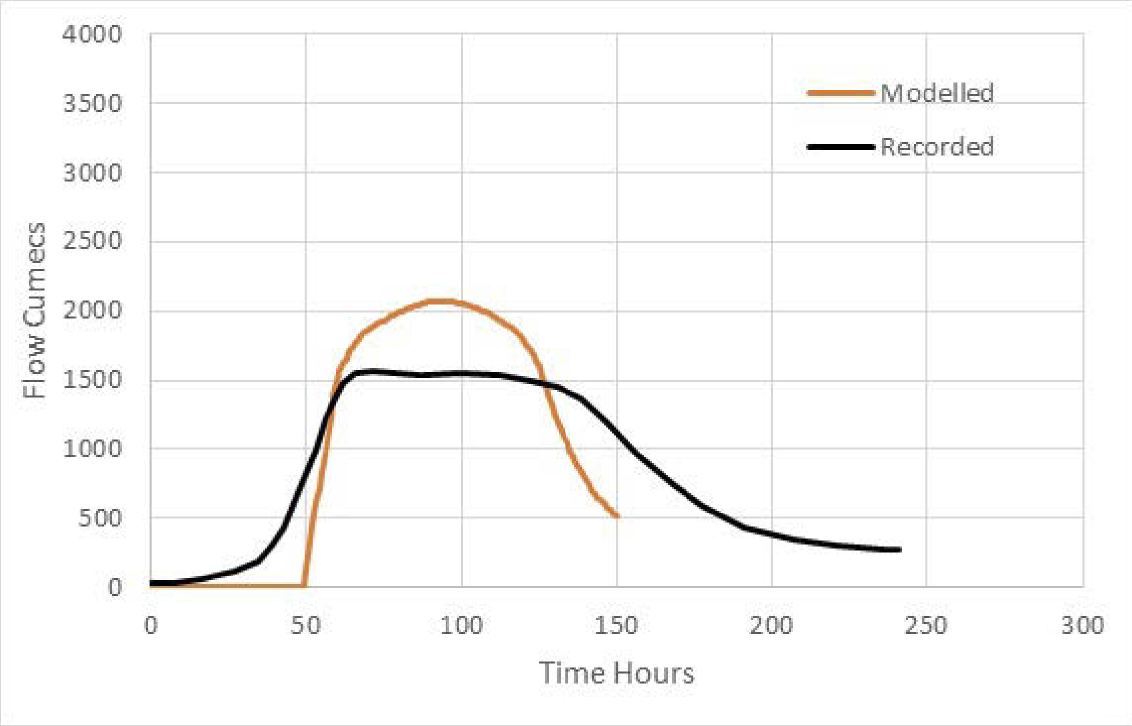



**Figure 7.15** **Goondiwindi Gauge – 1976 flows recorded and predicted**

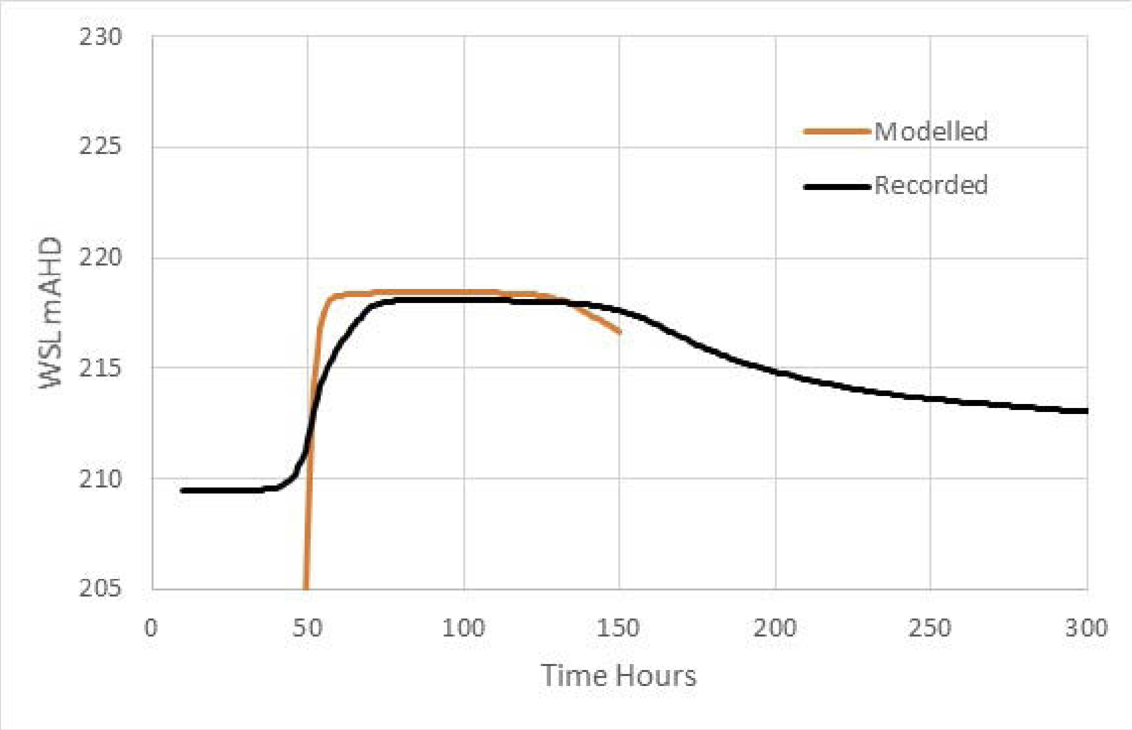



**Figure 7.16** **Goondiwindi Gauge – 1976 levels recorded and predicted**

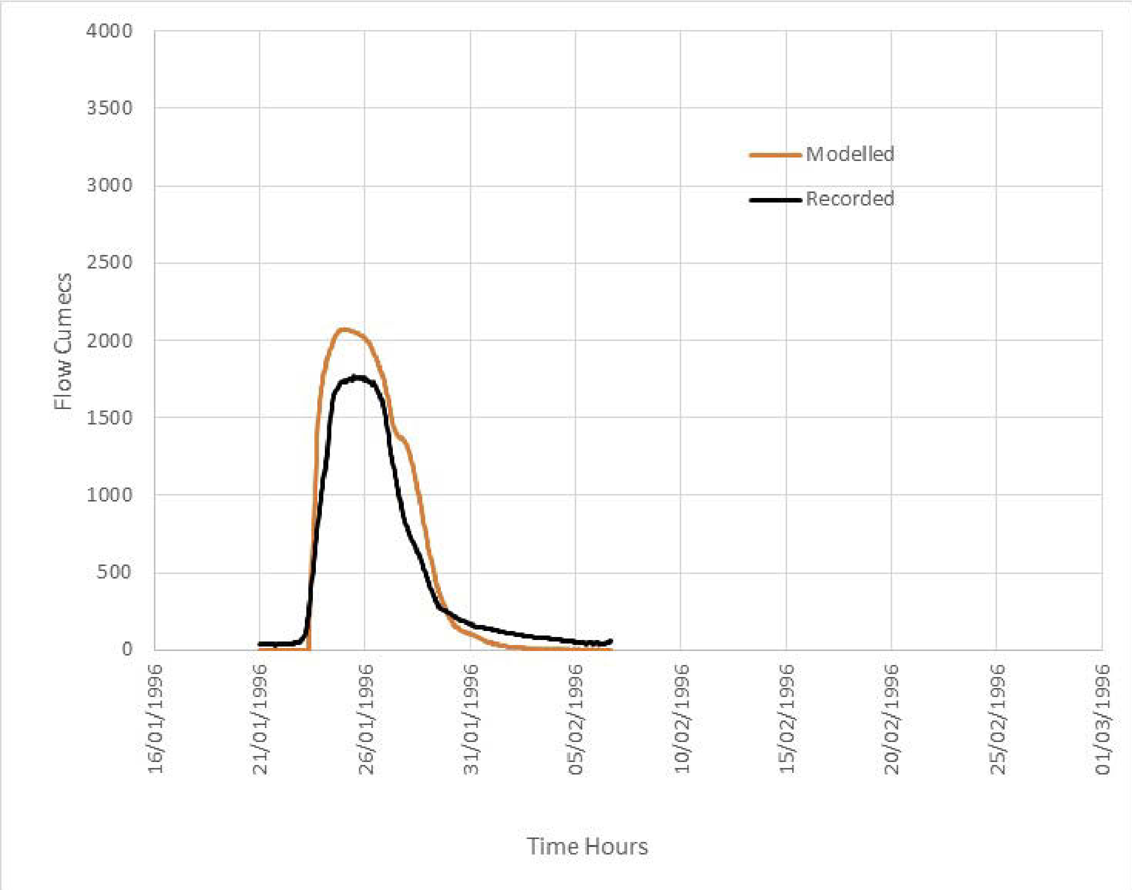



**Figure 7.17** Goondiwindi Gauge – 1996 flows recorded and predicted

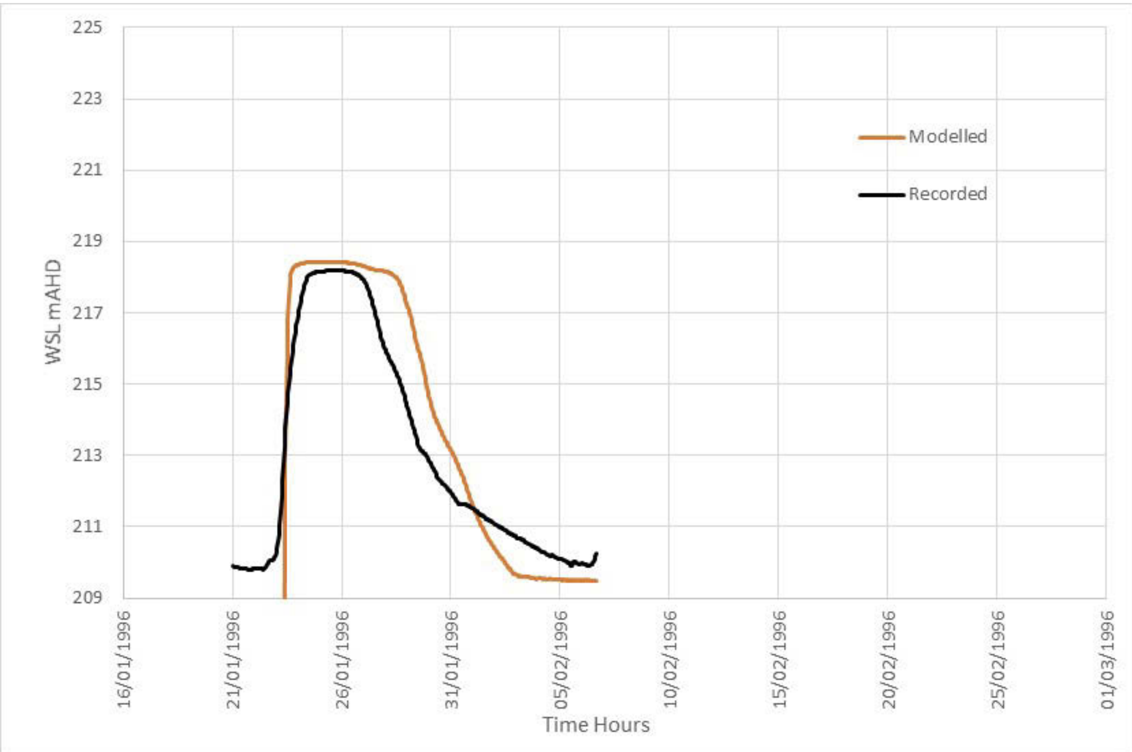



**Figure 7.18** Goondiwindi Gauge – 1996 levels recorded and predicted

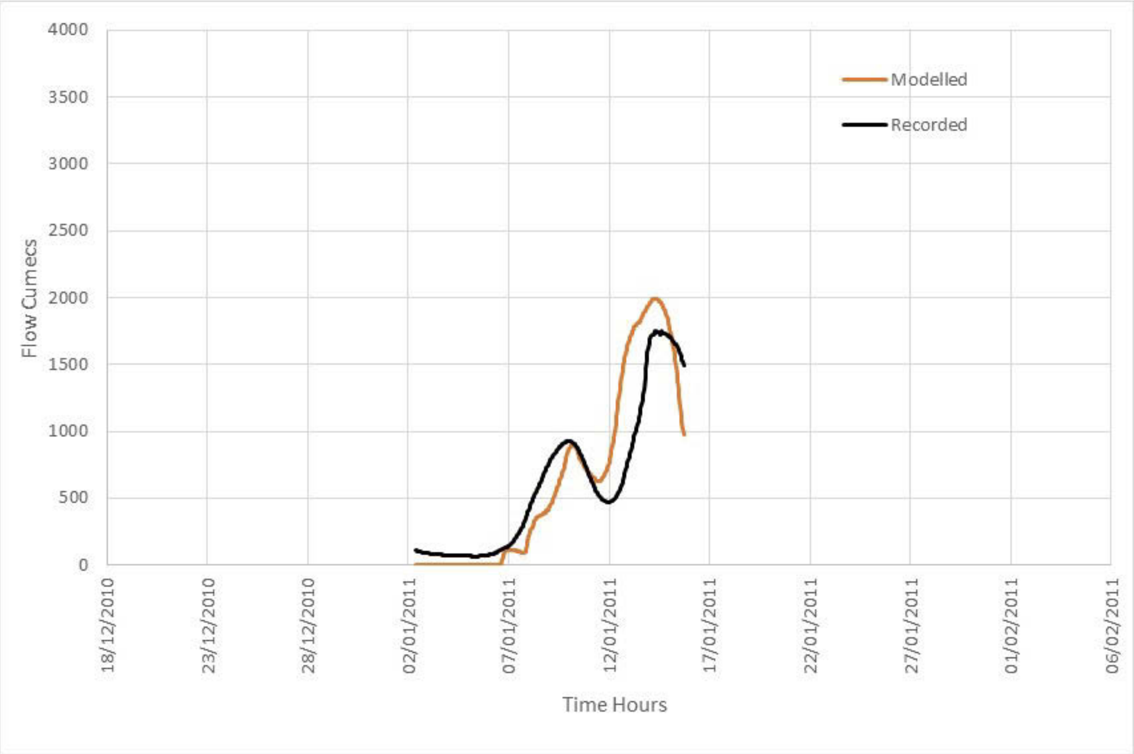



**Figure 7.19** **Goondiwindi Gauge – 2011 flows recorded and predicted**

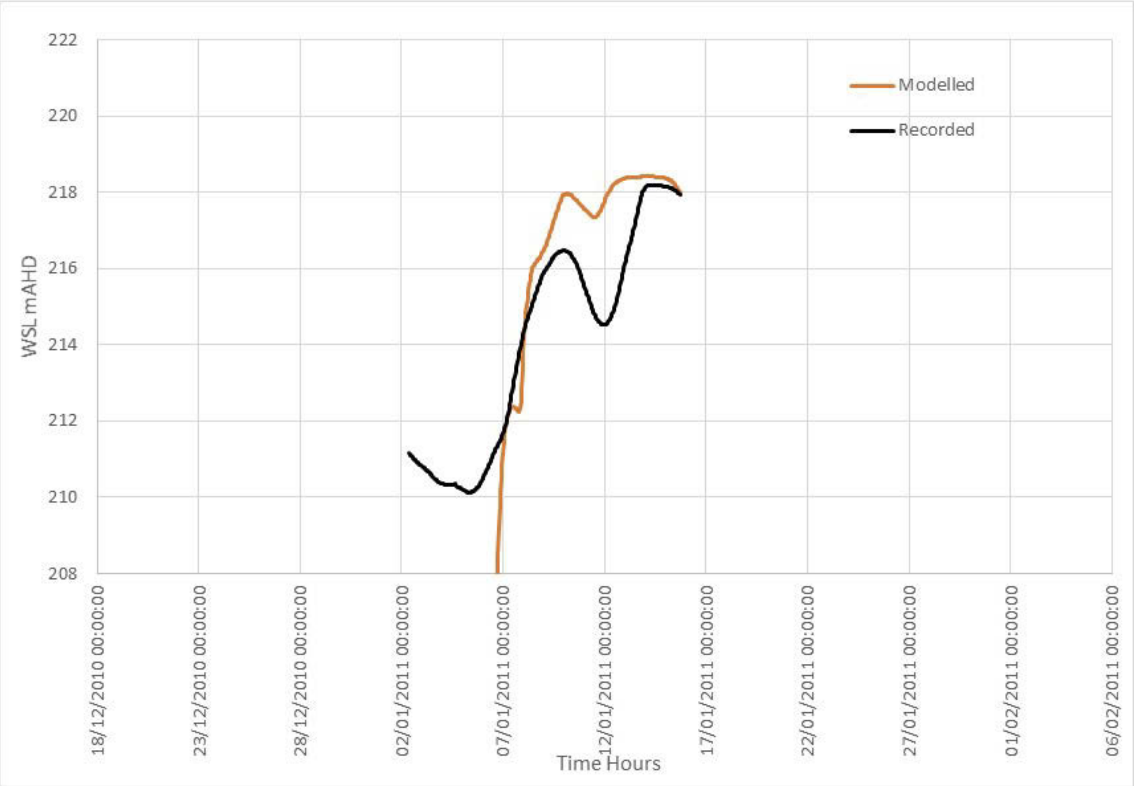



**Figure 7.20** **Goondiwindi Gauge – 2011 levels recorded and predicted**

The Goondiwindi stream gauge is located approximately 18 km downstream of the proposed alignment. The hydraulic sub-model was found to represent the peak levels well at the gauge within 0.34 m (1976 event) of the recorded level 0.24 m for the 1996 and 0.23 for the 2011 event. The flows were found to be within 13 per cent for the 2011 event, 17 per cent for the 1996 event and within 33 per cent for the 1976 event. This is based on the DPIE factoring of flows for 1976 and 1996, and no factoring of 2011 flows. It is noted that the flows are not recorded, but rather are derived from a rating curve and therefore do not have the same level of confidence as the recorded level data.

The predicted results show that the hydraulic sub-model is representing both the peak of flood events and the volume of the events well, with the shape of the predicting hydrograph matching closely with the shape of the recorded hydrograph. It is therefore considered that the performance of the hydraulic model against the stream gauges is acceptable.

#### 7.5.3.3 Historical flood level markers

##### 1976

There were 38 recorded flood marks provided by DPIE and extracted from the Goondiwindi Environs Study within the flood study area for the 1976 event. A comparison of the predicted flood levels to the recorded flood levels is presented in Figure A5-A to Figure A5-C. In general, the sub-model predicts levels within 0.3 m of the recorded flood levels. Where the model is outside of 0.3 m it is typically higher than the recorded levels. The exception to this is along Tucka Tucka Road where the levels are consistently low (approximately 600 mm). To raise the flood levels on Tucka Tucka road an increase on Bruxner Way would likely result, raising levels higher along the Bruxner Way, where the surveyed flood levels are currently well represented by the model. It is possible that the higher recorded levels on Tucka Tucka Road are a result of wave effects raising the debris marks above the actual peak flood levels. The recorded and predicted flood levels are presented in Table 7.9.

**Table 7.9 1976 recorded flood level comparison**

| Location | Source | Recorded level (m AHD) | TUFLOW modelled level (m AHD) | Difference (m) |
|---|---|---|---|---|
| 76-01 | DPIE – Border Rivers Floodplain management study | 214.60 | 214.42 | -0.18 |
| 76-02 | | 213.80 | 213.20 | -0.60 |
| 76-03 | | 210.50 | 210.16 | -0.34 |
| 76-04 | | 224.72 | 225.38 | +0.66 |
| 76-05 | | 218.10 | 218.47 | +0.36 |
| 76-06 | | 224.93 | 225.35 | +0.42 |
| 76-07 | | 220.12 | 220.24 | +0.12 |
| 76-08 | | 223.62 | 223.82 | +0.20 |
| 76-09 | | 224.96 | 225.17 | +0.21 |
| 76-10 | | 217.90 | 217.99 | +0.09 |
| 76-11 | | 224.72 | 224.82 | +0.09 |
| 76-12 | | 223.68 | 223.63 | -0.05 |
| 76-13 | | 223.31 | 223.18 | -0.13 |
| 76-14 | | 222.39 | 222.31 | -0.08 |
| 76-15 | | 222.32 | 222.23 | -0.09 |
| 76-16 | | 224.96 | 224.91 | -0.05 |
| 76-17 | | 219.60 | 219.24 | -0.36 |
| 76-18 | | 224.63 | 224.41 | -0.22 |
| 76-19 | | 226.51 | 225.99 | -0.52 |
| 76-20 | | 227.33 | 226.82 | -0.51 |
| 76-21 | | 224.26 | 223.66 | -0.60 |
| 76-22 | | 226.10 | 225.42 | -0.68 |
| 76-23 | | 226.92 | 226.18 | -0.75 |
| 76-24 | | 217.78 | 217.04 | -0.74 |
| 76-25 | | 218.85 | 218.45 | -0.40 |

| Location | Source | Recorded level (m AHD) | TUFLOW modelled level (m AHD) | Difference (m) |
|---|---|---|---|---|
| 76-26 | | 208.87 | 207.92 | -0.95 |
| 76-27 | | 207.42 | Dry | - |
| 76-28 | | 217.78 | 217.10 | -0.68 |
| 76-29 | | 217.05 | 217.06 | +0.01 |
| 76-30 | | 212.60 | 212.39 | -0.21 |
| 76-31 | | 213.00 | 212.37 | -0.63 |
| 76-32 | | 212.40 | 212.05 | -0.35 |
| 76-33 | | 215.88 | 216.04 | +0.16 |
| 76-34 | | 213.00 | 212.11 | -0.89 |
| 76-35 | | 210.56 | 210.59 | +0.03 |
| 76-36 | | 209.06 | Dry | - |
| 76-37 | Goondiwindi Environs Study | 219.88 | 220.06 | +0.18 |
| 76-38 | | 218.06 | 218.44 | +0.38 |
| **Mean** | | | | **0.36 m** |
| **Standard Deviation** | | | | **+/- 0.27 m** |

## 1976 Unfactored flows

The 1976 flows were simulated unfactored to predict the impact of factoring on the proposal area. The comparison of the unfactored peak water level and flows at both gauges are presented in Table 7.10 and Table 7.11 for both gauge locations.

**Table 7.10 Comparison of results at the Boggabilla Gauge for 1976 unfactored flows**

| Event | Recorded level (m AHD) | TUFLOW modelled level (m AHD) | Rated gauge flow ($m^3/s$) | TUFLOW modelled flow ($m^3/s$) |
|---|---|---|---|---|
| 1976 | 221.27 | 221.18 (-0.09m) | 3,700 | 3,628 |

**Table 7.11 Comparison of results at the Goondiwindi Gauge for 1976 unfactored flows**

| Event | Recorded level (m AHD) | TUFLOW modelled level (m AHD) | Rated gauge flow ($m^3/s$) | TUFLOW modelled flow ($m^3/s$) |
|---|---|---|---|---|
| 1976 | 218.08 | 218.41 (+0.33m) | 1,560 | 2,029 |

The recorded flood level comparison is presented in Table 7.12.

**Table 7.12 1976 recorded flood level comparison – unfactored flow**

| Location | Source | Recorded level (m AHD) | TUFLOW modelled level (m AHD) | Difference (m) |
|---|---|---|---|---|
| 76-01 | DPIE – Border Rivers Floodplain management study | 214.60 | 214.23 | -0.37 |
| 76-02 | | 213.80 | 212.98 | -0.82 |
| 76-03 | | 210.50 | Dry | - |
| 76-04 | | 224.72 | 225.22 | +0.50 |
| 76-05 | | 218.10 | 218.31 | +0.21 |
| 76-06 | | 224.93 | 225.20 | +0.27 |
| 76-07 | | 220.12 | 220.03 | -0.09 |
| 76-08 | | 223.62 | 223.68 | +0.06 |
| 76-09 | | 224.96 | 225.00 | +0.04 |

| Location | Source | Recorded level (m AHD) | TUFLOW modelled level (m AHD) | Difference (m) |
|---|---|---|---|---|
| 76-10 | | 217.90 | 217.83 | -0.07 |
| 76-11 | | 224.72 | 224.68 | -0.04 |
| 76-12 | | 223.68 | 223.44 | -0.24 |
| 76-13 | | 223.31 | 223.05 | -0.26 |
| 76-14 | | 222.39 | 222.24 | -0.15 |
| 76-15 | | 222.32 | 222.09 | -0.23 |
| 76-16 | | 224.96 | 224.75 | -0.21 |
| 76-17 | | 219.60 | 219.02 | -0.58 |
| 76-18 | | 224.63 | 224.12 | -0.52 |
| 76-19 | | 226.51 | 225.85 | -0.66 |
| 76-20 | | 227.33 | 226.66 | -0.67 |
| 76-21 | | 224.26 | 223.45 | -0.81 |
| 76-22 | | 226.10 | 225.28 | -0.82 |
| 76-23 | | 226.92 | 226.01 | -0.91 |
| 76-24 | | 217.78 | 216.77 | -1.01 |
| 76-25 | | 218.85 | 218.44 | -0.41 |
| 76-26 | | 208.87 | 207.92 | -0.95 |
| 76-27 | | 207.42 | Dry | - |
| 76-28 | | 217.78 | 216.97 | -0.81 |
| 76-29 | | 217.05 | 217.03 | -0.02 |
| 76-30 | | 212.60 | 212.23 | -0.37 |
| 76-31 | | 213.00 | 212.22 | -0.78 |
| 76-32 | | 212.40 | 211.81 | -0.59 |
| 76-33 | | 215.88 | 215.98 | +0.10 |
| 76-34 | | 213.00 | 211.97 | -1.03 |
| 76-35 | | 210.56 | 210.58 | +0.02 |
| 76-36 | | 209.06 | Dry | - |
| 76-37 | Goondiwindi Environs Study | 219.88 | 220.05 | +0.17 |
| 76-38 | | 218.06 | 218.43 | +0.37 |
| **Mean** | | | | **0.43 m** |
| **Standard Deviation** | | | | **+/- 0.33 m** |

The model predicts that removal of the factored flows (factored by +20 per cent) results in minor changes to peak water level at the gauges and a reduction of flood levels by approximately 140 mm across the hydraulic model area and a change in mean of 70 mm from +/-0.36 m factored and +/-0.43 m unfactored). There is little change in the standard deviation (0.27 factored and 0.33 unfactored). These results agree with DPIE comments that the factoring of flows has only minor impact to the proposal area.

### 1976 community consultation

As part of the community consultation for the hydraulic model calibration, 1976 flows were extracted from locations requested by landholders for comparison to their own recollections and collected data. The locations were spread across the floodplain capturing main tributary and breakout flows. Figure 7.21 presents the flow extraction locations. Table 7.13 presents the 1976 calibration event peak flows.

It was found that these flows were in agreement with landholders' recollections of flood flows during the 1976 flood event.

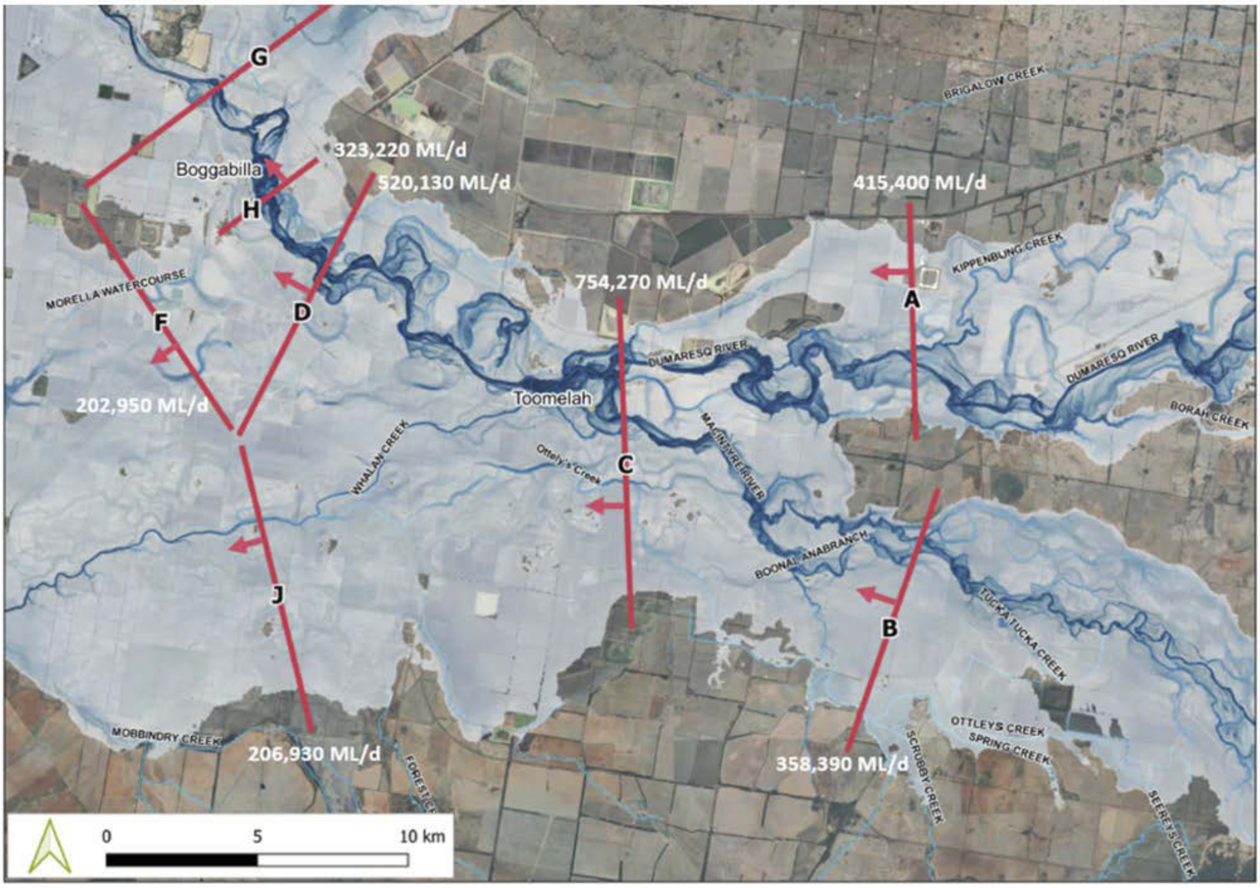



**Figure 7.21** 1976 event flow extraction locations for community feedback

**Table 7.13** 1976 event flows for community feedback

| Location | TUFLOW model flows ($m^3/s$) | TUFLOW model flows (ML/d) |
|---|---|---|
| A | 4,808 | 415,411 |
| B | 4,148 | 358,387 |
| C | 8,730 | 754,272 |
| D | 6,020 | 520,128 |
| F | 2,349 | 202,954 |
| G | 3,645 | 314,928 |
| H | 3,741 | 323,222 |
| J | 2,395 | 206,928 |

## 1996

There were eight recorded flood marks provided by DPIE and extracted from the Goondiwindi Environs Study for the 1996 flood event within the flood study area. The location of the recorded flood marks is shown in Figure A6-A to Figure A6-C. The recorded and predicted flood levels are presented in Table 7.14.

**Table 7.14 1996 recorded flood level comparison**

| Location | Source | Recorded level (m AHD) | TUFLOW modelled level (m AHD) | Difference (m) |
|---|---|---|---|---|
| 96-01 | DPIE – marks derived from high water marks on sign posts etc. | 220.95 | 221.01 | +0.05 |
| 96-02 | | 219.23 | 219.50 | +0.27 |
| 96-03 | | 218.73 | 218.88 | +0.15 |
| 96-04 | | 218.13 | 218.71 | +0.58 |
| 96-05 | Goondiwinidi Environs Study | 215.73 | 215.58 | -0.15 |
| 96-06 | | 221.71 | 222.18 | +0.47 |
| 96-07 | DPIE – Border Rivers Floodplain management study | 221.10 | 222.36 | +1.26 |
| 96-08 | Goondiwinidi Environs Study | 215.04 | 215.10 | +0.06 |
| **Mean** | | | | **0.37 m** |
| **Standard Deviation** | | | | **+/- 0.41 m** |

Aerial photography was provided by DPIE of the 1996 flood extent and is presented in Figure 7.22. This has been compared to the predicted 1996 flood extent results (shown as peak water surface levels) below (refer Figure 7.23).

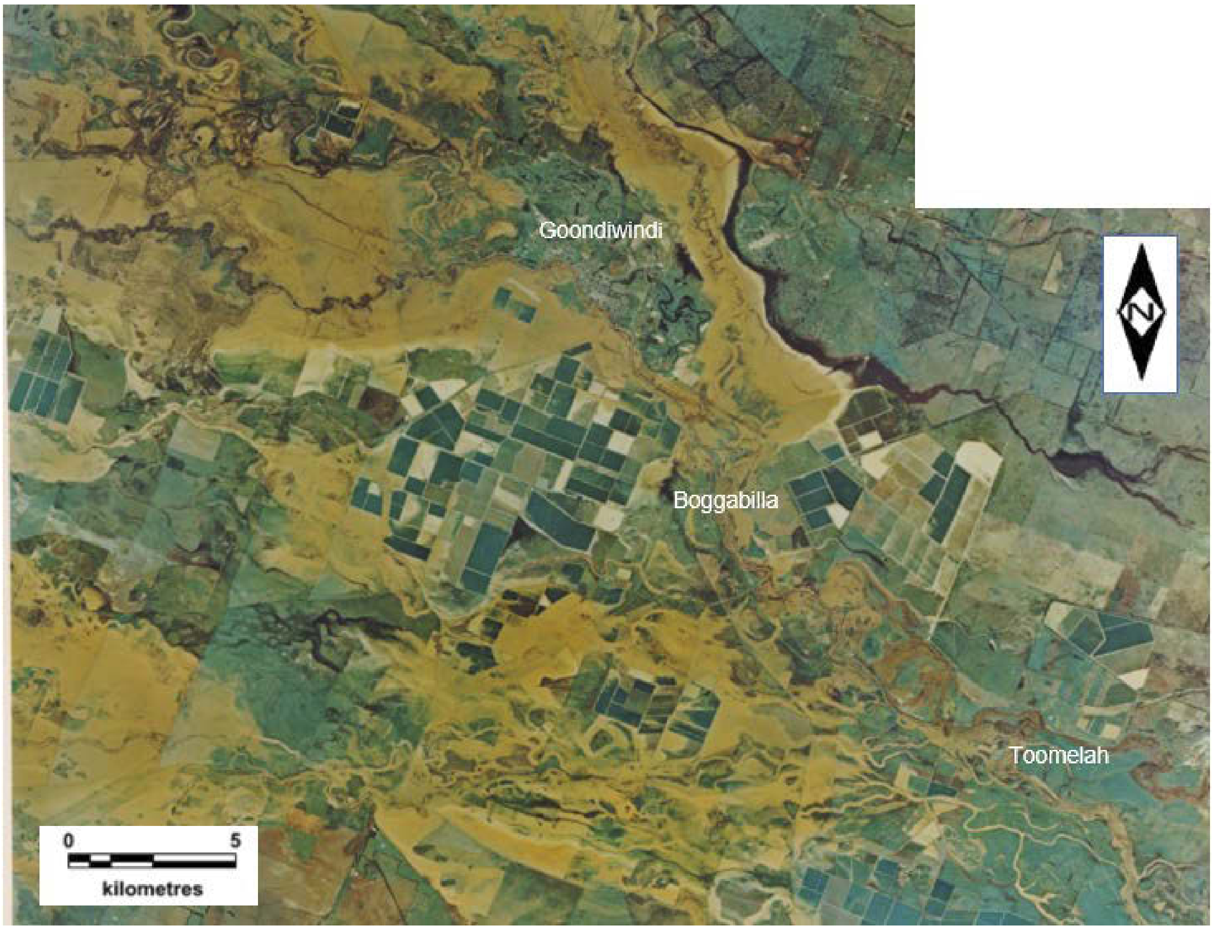



**Figure 7.22 1996 historical aerial flood photo**

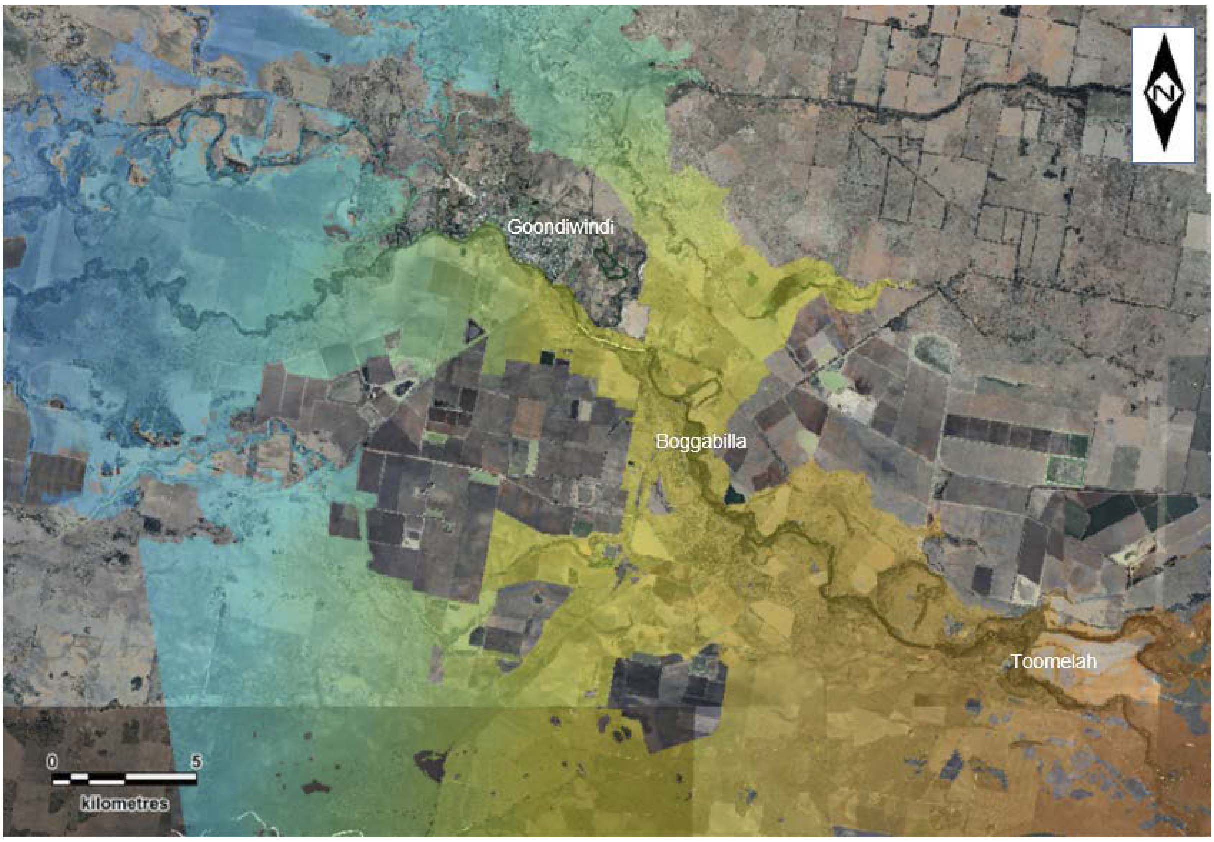



**Figure 7.23** **1996 predicted flood extent**

The predicted hydraulic sub-model flood levels generally compare well to the 1996 recorded flood heights with four of the eight points within 0.15 m of the recorded heights. It is possible that 96-03 recorded level (218.73 m AHD) may be a transcript error from the handwritten notes with 96-04 located 300 m downstream with a recorded height of 218.13 m AHD, it is possible that this level was also 218.13 m AHD (rather than 218.73 m AHD).

The recorded level 96-07 is 221.1 m AHD and located approximately 3 km upstream of the Boggabilla stream gauge which recorded a peak flood height of 221.03 m AHD. Hence the hydraulic sub-model was unable to match this level (predicted level 222.36 m AHD). It is likely there is an error in this recorded flood level.

The extent of inundation predicted by the hydraulic sub-model compares reasonably well with the aerial imagery taken during the 1996 flood event, with extents being slightly larger than the aerial image (refer Figure 7.23). As the time of capture of the image is not known it is possible that the photograph was not taken at the peak but rather during the rising or receding phase of the flood. This would account for the difference in flood extent.

## 1996 Unfactored flows

To understand the implication of factoring the 1996 event flows (60 per cent increase in flow), a sensitivity test was undertaken removing the factoring to test the change in flows to the flood study area. Comparison of gauge levels is presented in Table 7.15 and Table 7.16.

**Table 7.15** **Comparison of results at the Boggabilla Gauge for 1996 unfactored flows**

| Event | Recorded level (m AHD) | TUFLOW modelled level (m AHD) | Rated gauge flow ($m^3/s$) | TUFLOW modelled flow ($m^3/s$) |
|---|---|---|---|---|
| 1996 | 221.03 | 220.91 (-0.12m) | 3,486 | 2,542 |

Table 7.16 Comparison of results at the Goondiwindi Gauge for 1996 unfactored flows

| Event | Recorded level (m AHD) | TUFLOW modelled level (m AHD) | Rated gauge flow ($m^3/s$) | TUFLOW modelled flow ($m^3/s$) |
|---|---|---|---|---|
| 1996 | 218.19 | 218.38 (+0.19m) | 1,767 | 1,865 |

Removal of the factoring of the 1996 flows resulted in a reduction of flood level at the Boggabilla gauge of 200 mm and 50 mm at the Goondiwindi gauge. The recorded and predicted flood levels are presented in Table 7.17.

Table 7.17 1996 recorded flood level comparison unfactored flows

| Location | Source | Recorded level (m AHD) | TUFLOW modelled level (m AHD) | Difference (m) |
|---|---|---|---|---|
| 96-01 | DPIE – marks derived from high water marks on sign posts etc. | 220.95 | 220.77 | -0.18 |
| 96-02 | | 219.23 | 219.34 | +0.11 |
| 96-03 | | 218.73 | 218.75 | +0.02 |
| 96-04 | | 218.13 | 218.60 | +0.47 |
| 96-05 | Goondiwinidi Environs Study | 215.73 | 215.54 | -0.19 |
| 96-06 | | 221.71 | 221.80 | +0.09 |
| 96-07 | DPIE – Border Rivers Floodplain management study | 221.10 | 221.96 | +0.86 |
| 96-08 | Goondiwinidi Environs Study | 215.04 | 215.05 | +0.01 |
| **Mean** | | | | **0.24 m** |
| **Standard Deviation** | | | | **+/- 0.29 m** |

The unfactored 1996 flows are predicted to provide a closer match to the recorded flood levels with a reduction in the mean from +/-0.37 to +/-0.24.

The extent (shown as peak water surface level) of inundation predicted by the hydraulic sub-model is presented in Figure 7.24.

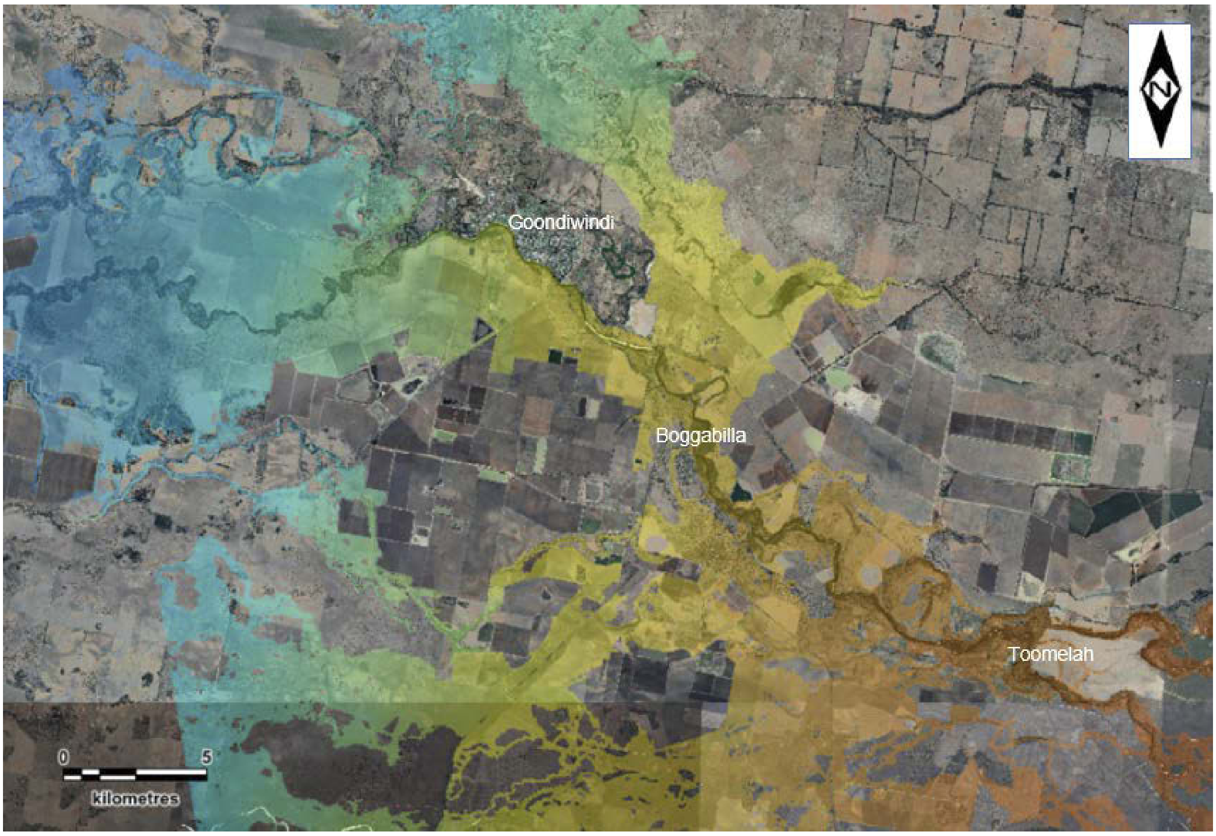



**Figure 7.24 1996 predicted flood extent, unfactored**

The modelling shows that the DPIE factoring of the flows for the 1996 event raises levels by approximately 50-200 mm across the study area. The comparison to the recorded flood levels shows the calibration is similar with the unfactored flows producing a closer match to the recorded flood levels, and recorded gauge levels. Therefore, without factoring the comparison to gauge level and recorded flood heights is considered improved in the flood study area. It is noted that the extent of inundation is smaller than the factored flows model and compares closer to the aerial image of the 1996 flood event.

Calibration of the model is considered acceptable with or without the factoring of flows. Therefore, the calibration of the sub model is not sensitive to the factoring applied to the 1976 and 1996 DPIE hydraulic model flows.

## 2011

For the 2011 flood event there were 52 historical flood marks available for comparison. These are summarised in Table 7.18 and presented in Figure A7-A to Figure A7-C.

**Table 7.18 2011 recorded flood level comparison**

| Name | Source | Recorded level (m AHD) | TUFLOW modelled level (m AHD) | Difference (m) | TUFLOW modelled level (15m grid) (m AHD) | Difference (15m grid) (m) |
|---|---|---|---|---|---|---|
| 11-01 | DPIE – marks derived from high water marks on sign posts etc. | 219.15 | 219.58 | +0.43 | 219.51 | +0.37 |
| 11-02 | | 220.98 | 221.17 | +0.20 | 221.11 | +0.13 |
| 11-03 | | 224.79 | 225.47 | +0.68 | 225.41 | +0.62 |
| 11-04 | | 219.23 | 219.45 | +0.21 | 219.42 | +0.18 |
| 11-05 | | 221.85 | 222.13 | +0.28 | 222.11 | +0.26 |
| 11-06 | | 218.28 | 219.42 | +1.14 | 219.46 | +1.19 |
| 11-07 | | 219.60 | 219.58 | -0.02 | 219.51 | -0.09 |
| 11-08 | | 219.57 | 219.80 | +0.23 | 219.81 | +0.24 |

| Name | Source | Recorded level (m AHD) | TUFLOW modelled level (m AHD) | Difference (m) | TUFLOW modelled level (15m grid) (m AHD) | Difference (15m grid) (m) |
|---|---|---|---|---|---|---|
| 11-09 | | 221.94 | 222.13 | +0.20 | 222.11 | +0.17 |
| 11-10 | | 217.87 | 218.13 | +0.26 | 218.15 | +0.27 |
| 11-11 | | 220.32 | 219.83 | -0.49 | 219.79 | -0.53 |
| 11-12 | | 220.41 | 220.21 | -0.20 | 220.19 | -0.22 |
| 11-13 | | 220.44 | 220.36 | -0.09 | 220.35 | -0.10 |
| 11-14 | | 220.79 | 220.77 | -0.02 | 220.82 | +0.03 |
| 11-15 | | 220.85 | 220.80 | -0.06 | 220.79 | -0.06 |
| 11-16 | | 221.05 | 221.15 | +0.10 | 221.09 | +0.04 |
| 11-17 | | 222.22 | 222.00 | -0.22 | 221.94 | -0.27 |
| 11-18 | | 221.12 | 221.15 | +0.03 | 221.10 | -0.02 |
| 11-19 | | 223.96 | 223.89 | -0.07 | 223.85 | -0.11 |
| 11-20 | | 220.79 | 220.77 | -0.02 | 220.82 | +0.03 |
| 11-21 | | 224.02 | 223.89 | -0.13 | 223.85 | -0.17 |
| 11-22 | | 224.06 | 223.93 | -0.13 | 223.89 | -0.17 |
| 11-23 | | 224.06 | 223.89 | -0.17 | 223.85 | -0.21 |
| 11-24 | | 224.06 | 223.93 | -0.13 | 223.90 | -0.16 |
| 11-25 | | 224.06 | 223.87 | -0.19 | 223.81 | -0.25 |
| 11-26 | DPIE – marks derived from high water marks on sign posts etc. | 220.73 | 220.23 | -0.50 | 220.48 | -0.25 |
| 11-27 | | 228.27 | 228.03 | -0.24 | 227.93 | -0.34 |
| 11-28 | | 225.73 | 225.44 | -0.29 | 225.41 | -0.32 |
| 11-29 | | 216.96 | 217.03 | +0.07 | 216.97 | +0.01 |
| 11-30 | | 216.80 | 216.86 | +0.06 | 216.80 | +0.01 |
| 11-31 | | 216.50 | 216.41 | -0.08 | 216.36 | -0.13 |
| 11-32 | | 216.34 | 216.06 | -0.28 | 216.06 | -0.29 |
| 11-33 | | 215.19 | 215.17 | -0.02 | 214.72 | -0.47 |
| 11-34 | | 214.89 | 214.47 | -0.41 | 213.74 | -1.14 |
| 11-35 | | 214.83 | Dry | - | Dry | - |
| 11-36 | | 217.53 | 217.48 | -0.05 | 217.52 | -0.01 |
| 11-37 | | 217.96 | 217.86 | -0.10 | 217.94 | -0.02 |
| 11-38 | | 218.55 | 218.38 | -0.17 | 218.50 | -0.05 |
| 11-39 | | 218.44 | 218.70 | +0.26 | 218.77 | +0.33 |
| 11-40 | | 213.52 | Dry | - | 213.34 | -0.19 |
| 11-41 | | 213.73 | Dry | - | 213.34 | -0.39 |
| 11-42 | | 215.87 | 215.40 | -0.47 | 215.36 | -0.50 |
| 11-43 | | 218.49 | 218.76 | +0.27 | 218.78 | +0.29 |
| 11-44 | | 218.69 | 219.15 | +0.45 | 219.15 | +0.46 |
| 11-45 | | 219.21 | 219.53 | +0.33 | 219.55 | +0.34 |
| 11-46 | | 219.35 | 219.60 | +0.26 | 219.61 | +0.27 |
| 11-47 | | 219.56 | 219.69 | +0.14 | 219.73 | +0.17 |
| 11-48 | | 219.98 | 220.00 | +0.03 | 219.97 | -0.01 |
| 11-49 | | 218.54 | 219.06 | +0.52 | 219.02 | +0.47 |

| Name | Source | Recorded level (m AHD) | TUFLOW modelled level (m AHD) | Difference (m) | TUFLOW modelled level (15m grid) (m AHD) | Difference (15m grid) (m) |
|---|---|---|---|---|---|---|
| 11-50 | | 218.46 | 218.18 | -0.28 | 218.02 | -0.44 |
| 11-51 | | 217.13 | 217.50 | +0.38 | 217.40 | +0.27 |
| 11-52 | | 216.18 | Dry | - | Dry | - |
| **Mean** | | | | **0.24 m** | **Mean** | **0.26 m** |
| **Standard Deviation** | | | | **+/- 0.21 m** | **Standard Deviation** | **+/- 0.24 m** |

Aerial photography was provided by DPIE of the 2011 flood extent and is presented in Figure 7.25. This has been compared to the predicted 2011 flood extent results below (refer Figure 7.26).

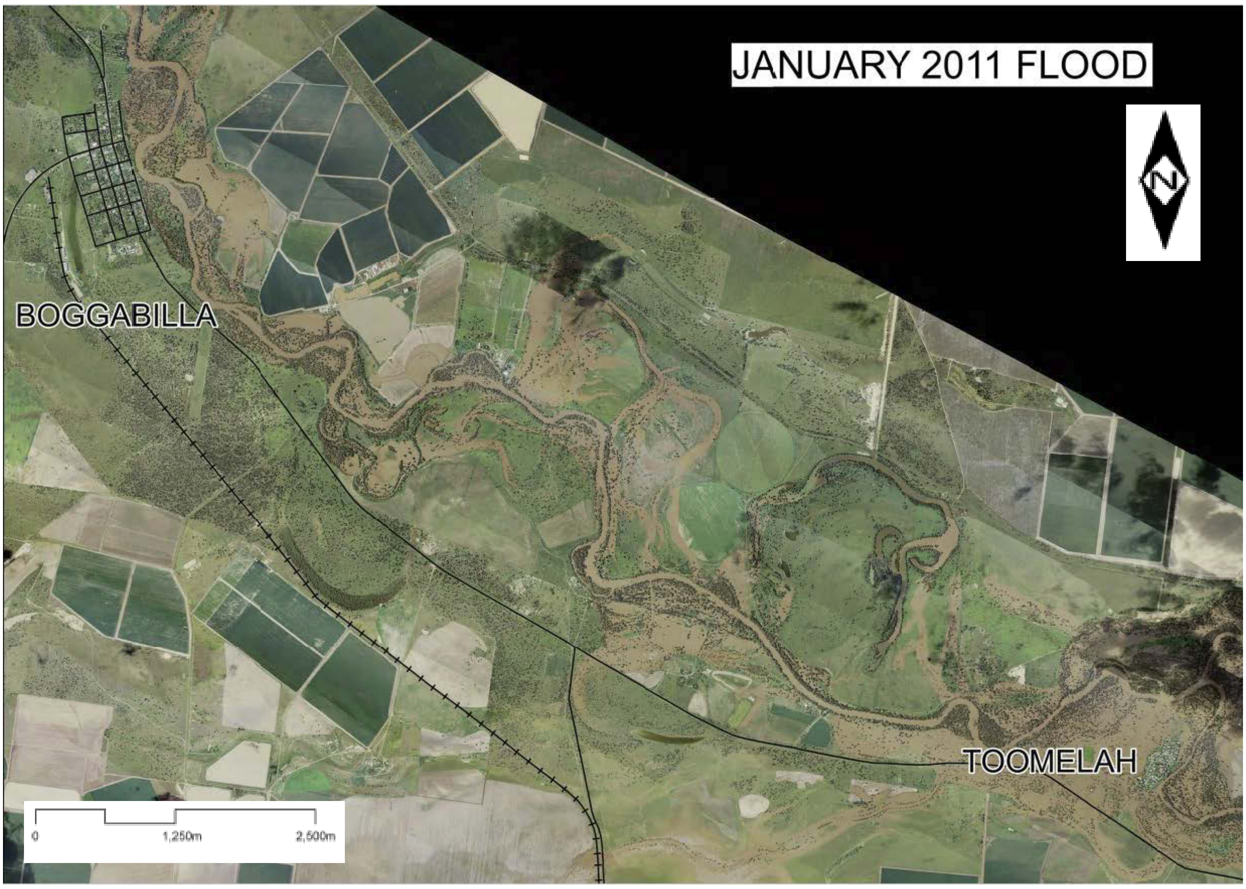



**Figure 7.25 2011 historical aerial flood photo**

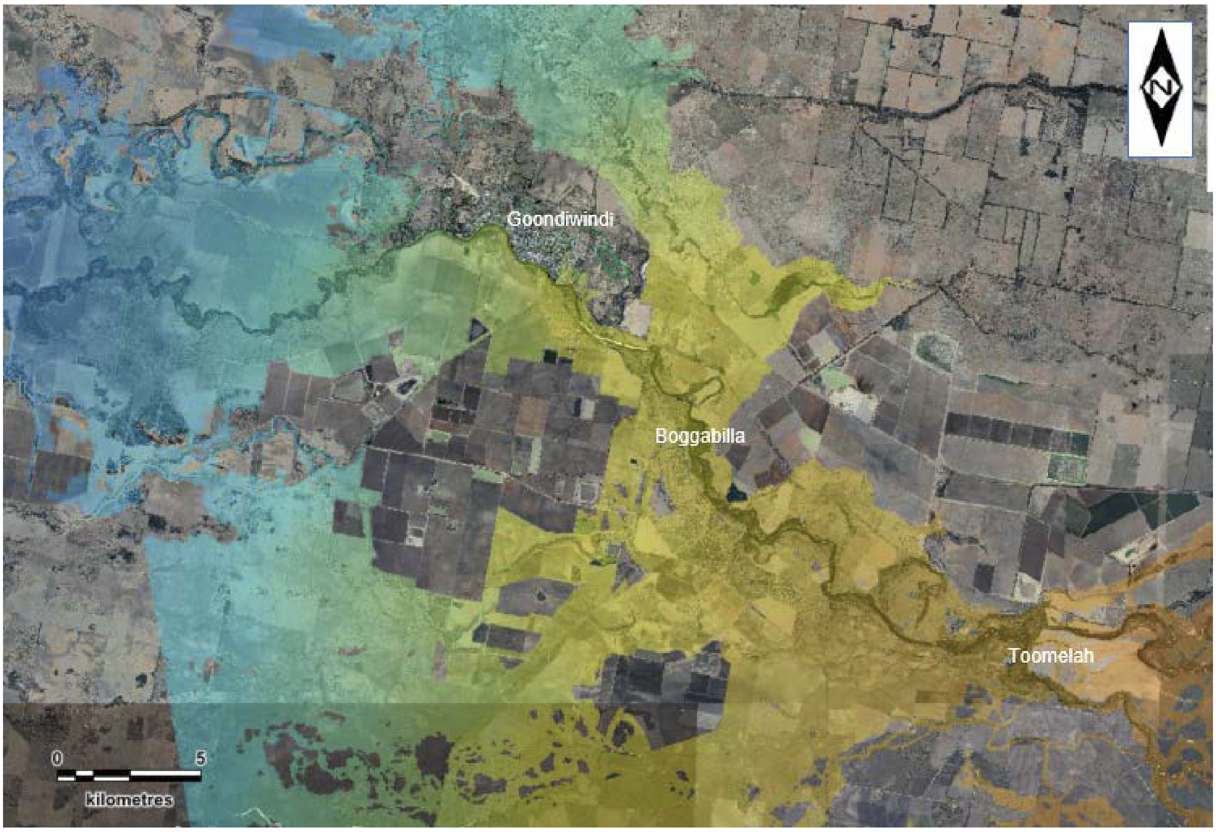
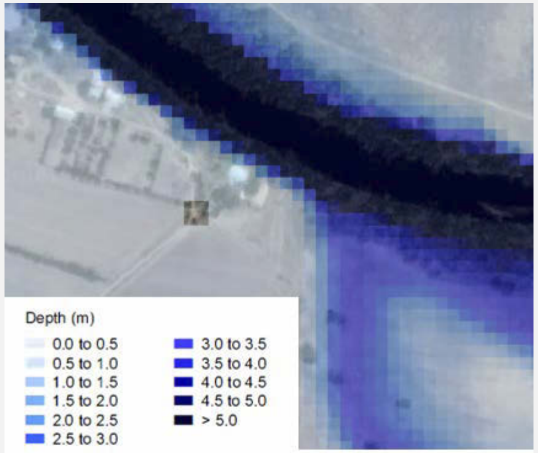
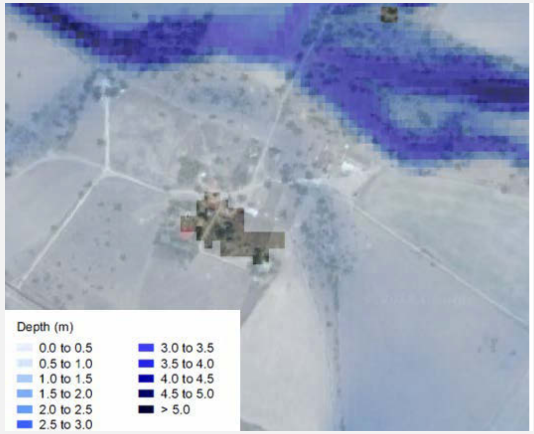
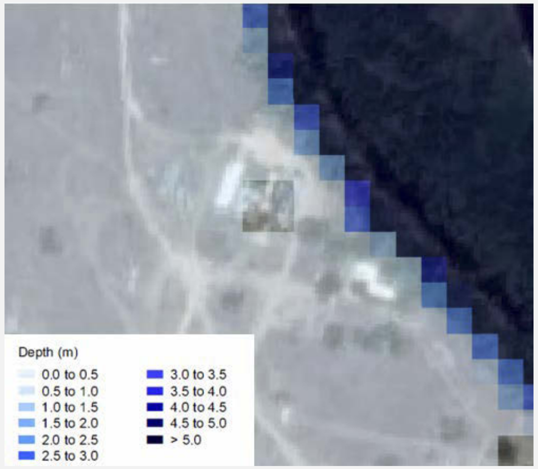
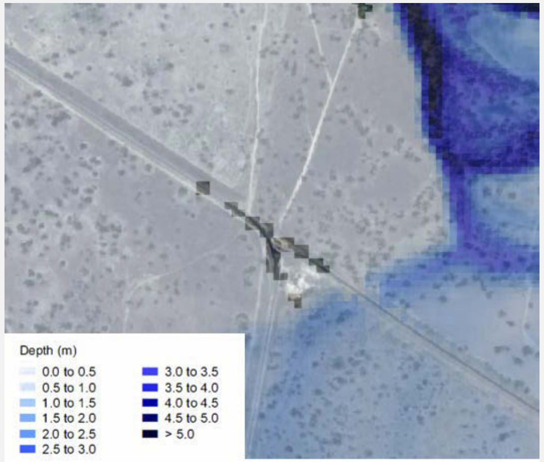



**Figure 7.26** **2011 predicted flood extent**

The hydraulic model results were compared with flood photos provided by landholders for the 2011 event to validate the model performance and are shown in Table 7.19.

**Table 7.19 2011 flood photos and model comparison**

| Description | Photo | Model results | Notes |
|---|---|---|---|
| Flood photo taken at Budleigh during 2011 flood, from house looking out to Macintyre River. |  |   | The model predicts water depths of 50 mm at the peak of the flood. It is noted that this photo may not have been taken at the peak of the flood. |
| Flood photo taken at Merawah post 2011 flood, at house, estimated water level depth at house, approximately 1.2 m |  |   | Model predicts depths of 1.14m at house in agreement with photo |

| Description | Photo | Model results | Notes |
| --- | --- | --- | --- |
| Flood photo taken at Malgarai in 2011 |  |  | The photos from flooding at Malgarai indicate wide spread shallow flooding in agreement with the model. The depths are approx. 0.2m. |
| Flood photo taken at Tucka Tucka Road, looking west (Malgarai) in 2011 |  |  | Shallow road flooding less than 100 mm predicted |

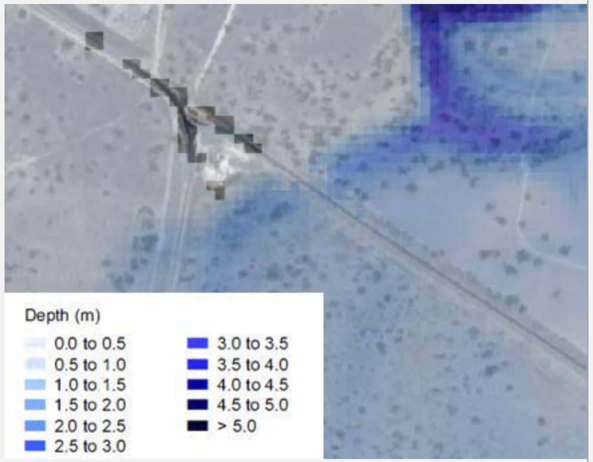
| Description | Photo | Model results | Notes |
|---|---|---|---|
| Flood photo taken at Tucka Tucka Road, Looking east (Malgarai) in 2011 |  |  | Shallow road flooding predicted at intersection (<0.1 m) deepening to <1 m east along the road |

Table 7.19 shows that the predicted hydraulic sub-model flood levels are a good match to the recorded flood heights across the flood study area.

The extent of inundation predicted from the hydraulic sub-model generally agrees with the aerial imagery. However, the predicted flood inundation extent appears larger in some areas than the extent in the aerial image. This may be due to the shallow areas not being visible in the aerial, or the photo may not have been taken at the peak of the flood event. As the time of capture of the image is not known it is possible that the photo was not taken at the peak but rather during the rising or receding phase of the flood.

A sensitivity test on grid size was undertaken for the 2011 event with the model simulated at a 15 m grid size to test the performance of the model at a finer grid size. The difference to the recorded levels is shown in Table 7.18. Typically, the reduced grid lowered levels by approximately 50 mm across the hydraulic model. This resulted in only minor changes to the calibration of the hydraulic sub-model. At the gauges the change in peak flood level was 50 mm (lower) at Boggabilla and 70 mm higher at Goondiwindi.

Overall, the hydraulic sub-model provides a good match to levels at the stream gauges, recorded flood heights across the flood study area and anecdotal flood photographs of flood inundation extents at properties upstream of the gauge and near the proposed alignment.

## 7.6 Calibration summary

Available data and previous studies for the Macintyre River floodplain were collected and reviewed to support the development and calibration of the hydrologic and hydraulic models for this assessment. The DPIE Border Rivers Floodplain hydrologic and hydraulic modelling has been identified as the most detailed and suitable models for the assessment of floodplain conditions and impacts of the proposed rail alignment.

The DPIE hydrologic models have been adopted for this assessment and updated to include the 2011 flood event in addition to the 1976 and 1996 historical events. A hydraulic sub-model was developed from the regional DPIE hydraulic model for the Macintyre River floodplain area.

The models were simulated for the three historical events and compared to the Boggabilla and Goondiwindi stream gauge data, recorded historical flood heights and flood photographs.

The following is concluded from the hydrologic and hydraulic calibration:

- The three historical events flood levels compare well to the recorded levels at the Boggabilla and Goondiwindi stream gauges
- Flows are within 20 per cent of the stream gauge recorded flows, with the exception of the 1976 event predicted flow at Goondiwindi (33 per cent). It is noted that the flows (estimated from the recorded levels using rating curves and are not recorded flows.
- For the 1976 event the hydraulic sub-model predicts flood levels that generally compare well with the recorded flood heights
- Simulating the 1976 event with unfactored flows results in minimal change to predicted peak flood levels
- For the 1996 event the hydraulic sub-model predicts flood levels that generally compare well with the recorded flood heights and aerial extents of flood inundation
- Simulating the 1996 event with unfactored flows results in a reduction in flood levels of approximately 50 to 200 mm across the model area and is predicated to result in a minor improvement to the hydraulic sub-model calibration
- For the 2011 event the hydraulic sub-model predicts flood levels that compare very well with the recorded flood heights
- The predicted 2011 flood inundation extent is comparable to the aerial photography of the flood extent, with the predicted extent being slightly larger. Given the representation of the flood levels at the gauge compared to recorded flood levels (within -0.05 m for Boggabilla and +0.23 for Goondiwindi), and the very good match of predicted levels to historical flood heights, it is likely the photography was not taken at the peak of the flood event.

- Simulating the 2011 model at a 15 m grid resulted in a lowering of water levels across the model and a slight improvement in the calibration to recorded flood levels. While 1996 and 1976 have not been modelled using this finer grid, it is likely that there would be similar outcomes with a minor reduction in flood levels across the floodplain area.

Based on the performance of the hydraulic sub-model to predict the flood gauge heights at the Boggabilla and Goondiwindi gauges for all three events and the good correlation between the historical flood photographs and recorded flood levels for the 1996 and 2011 flood event, the hydrologic and hydraulic models for this assessment are considered suitably calibrated to take forward to the next phase of this assessment.

# 8 Existing Case modelling

## 8.1 Hydrology

### 8.1.1 Approach

Hydrologic modelling has been undertaken using the ARR 2016 methodology. This methodology adopts a design event type approach, whereby a spatially uniform temporal pattern is applied across the whole catchment. The major difference from the previous ARR 1987 Design Event approach is that an ensemble of ten different temporal patterns are simulated for each duration and frequency rather than a single pattern. A comparison on predicted flows from ARR 1987 and ARR 2016 was undertaken and is documented in Appendix D.

The general procedure for conducting the design event assessment was:

- Obtaining rainfall Intensity-Frequency-Duration (IFD) relationships, temporal patterns, losses and other parameters pertinent to each catchment
- Simulation of the ensemble of design events for a range of durations for each AEP
- Application of Areal Reduction Factors (ARF) to account for catchment size (rainfall IFD is based on point intensities; ARF modifies this to provide areal average values)
- Determination of the design flows for each AEP. The median peak flow of the critical storm duration (the duration that causes the highest median peak flow) has been adopted. Since an ensemble of ten patterns is tested, the median value technically lies between the 5$^{th}$ and 6$^{th}$ ranked values, so the current practice is to conservatively take the 6$^{th}$
- Comparison of the resulting 2016 design event flow estimates with a FFA and modify the design parameters, where necessary, to achieve consistency
- Extraction of design hydrograph(s) for use in the hydraulic model.

### 8.1.2 Rainfall data

Rainfall IFD relationships for each sub-catchment within each hydrologic model were obtained from the BoM online Data Hub. Due to the size of the catchment area, IFDs were extracted at multiple locations. An example of this data is presented in Table 8.1 for the 24 hour duration.

**Table 8.1 24 hour rainfall depth (mm)**

| Catchment area | 24 hour duration rainfall depth (mm) | | |
|---|---|---|---|
| | 50% AEP event | 10% AEP event | 1% AEP event |
| Macintyre Brook to Booba Sands | 55 | 88 | 139 |
| Dumaresq River to Mauro | 53 | 84 | 133 |
| Macintyre River to Holdfast | 55 | 84 | 128 |
| Ottleys Creek to Junction | 60 | 96 | 151 |

For each event, the catchment-average rainfall depth was derived based on the duration and AEP for the upstream catchment of the location under consideration. The rainfall depth is sampled from catchment IFD curves from the Bureau of Meteorology to derive point rainfall intensities for each of the sub-areas of the URBS model. An ARF was applied to the rainfall intensities to account for the fact that rainfall is generally not equally extreme over all of the catchment area.

For Macintyre Brook, the 10 (5,000 km$^2$) Central Slopes temporal patterns were applied for the catchment to Booba Sands (~4,000 km$^2$).

### 8.1.3 Extreme rainfall events

Extreme rainfall events have been assessed. For extreme rainfall estimates (Probable Maximum Precipitation, PMP), the generalised techniques described by the Generalised Short Duration Method and Generalised Tropical Storm Method Revised (BoM 2003) were adopted. The techniques specified in Book VIII of ARR 2016~~,~~ have been used to interpolate design rainfall estimates between 1 in 2,000 AEP and the PMP (1 in 300,000 AEP).

Ten temporal patterns were adopted for 15 durations from 1 to 120 hours for 1 in 10,000 AEP, 1 in 100,000 AEP and the PMP.

### 8.1.4 Design rainfall losses

Rainfall losses are applied to a hydrologic model to represent rainfall that does not contribute to overland flow (i.e. infiltrates the ground or is lost to evaporation). The loss method adopted was the initial/continuing loss model, where the initial loss (in mm) represents initial catchment wetting where no runoff is produced, followed by a constant continuing loss rate (in mm/h) to account for infiltration/evaporation during the rainfall runoff process.

Design event IFD data and temporal patterns are based on 'bursts' rather than complete storms; that is, they represent the worst part of a rainfall event that may (or may not) be preceded or followed by additional rainfall. The initial losses applied to a design event may therefore be different from those applied to a full storm (e.g. a calibration event). The ARR 2016 design event methodology tries to address this issue by combining a constant initial loss depth with a variable pre-burst depth, a depth of rainfall assumed to occur sometime before the design burst[1]. The pre-burst depth is a function of event duration and frequency. Recommended loss and pre-burst depths are accessed from the online ARR Data Hub.

The initial loss and continuing loss rates were applied as constant values across each catchment area. The design rainfall losses used for each event are presented in Table 8.2.

The adopted losses for the hydrologic models were based on the recommendations in ARR 2016 Book 5, Chapter 3, Section 3.5. These are the recommended medium loss values for the Central Slopes Zone and were adjusted for this catchment using a combined hydrologic/hydraulic model approach with comparison of the levels at the gauge, and consideration of the calibration losses. It is noted that there was no comparable data available from the Border Rivers Floodplain Management Study (DPIE 2018) as there was no design event assessment undertaken for the DPIE study.

**Table 8.2 ARR 2016 Rainfall runoff losses**

| Catchment area | ARR Data Hub | | Adopted | |
|---|---|---|---|---|
| | Initial loss (mm) | Continuing loss (mm/hr) | Initial loss (mm) | Continuing loss (mm/hr) |
| Macintyre Brook | 28.0 | 1.0 | 25.0 | 0.5 |
| Dumaresq River | 28.0 | 6.5 | 47.0 | 2.5 |
| Macintyre River | 32.0 | 2.3 | 36.5 | 1.5 |
| Ottleys Creek | 62.0 | 0.0 | 60.0 | 1.5 |
| Back Creek | 53.0 | 0.0 | 53.0 | 1.5 |
| Forest Creek | 53.0 | 0.0 | 48.0 | 1.5 |
| Strayleaves Creek | 53.0 | 0.0 | 43.0 | 1.5 |
| Mobbindry Creek | 53.0 | 0.0 | 56.0 | 1.5 |

[1] Note that ARR 2016 advises that there is currently little research into the temporal pattern of pre-burst rainfall. The appropriate methodology for applying pre-burst rainfall is open to interpretation. If the pre-burst depth is less than the initial loss, it can be simply considered to reduce the initial loss by that amount. However, if the pre-burst depth exceeds the initial loss then different software packages treat the excess pre-burst rainfall in different ways.

### 8.1.5 Design hydrology model parameters

Design hydrologic model parameters are consistent with the 2011 flood event and are presented in Table 8.3.

**Table 8.3 Design event modelling adopted parameters**

| Sub-catchment | Alpha | Beta | m |
|---|---|---|---|
| Macintyre Brook | 0.20 | 1.2 | 0.8 |
| Dumaresq River | 0.10 | 1.2 | 0.8 |
| Macintyre River | 0.20 | 1.2 | 0.8 |
| Ottleys Creek | 0.20 | 1.2 | 0.8 |
| Local catchments (Strayleaves, Forest, Back, and Mobbindry Creeks) | 0.20 | 1.2 | 0.8 |

### 8.1.6 Climate change

The impacts of climate change (CC) were assessed for the Macintyre River floodplain for the 1% AEP design event to determine the sensitivity of the proposed alignment design to the potential increase in rainfall intensity. The assessment was undertaken in accordance with ARR 2016 guidelines.

The selected representative concentration pathway for the climate change analysis was 8.5 for a 2090 design horizon. The climate change analysis was undertaken by increasing rainfall intensities in the IFDs for the contributing catchments.

### 8.1.7 Comparison to ARR 1987

A sensitivity analysis was undertaken to compare the design flows derived from ARR 2016 methodology with design flows from ARR 1987 methodology. Documentation of the assessment is provided in Appendix E. The comparison found that ARR 2016 methodology provides higher flows than the ARR 1987 method in the Border Rivers catchment. Therefore, ARR 2016 provides more conservative levels and flows for assessment of the proposal alignment than ARR 1987 approaches. In Macintyre Brook, ARR 2016 flows were lower in the higher frequency events. This is predominantly related to the higher continuing loss (CL) reducing peak flows in the smaller events. This does not impact the large events (i.e. 1% AEP and larger) that are used for the assessment and design of the proposal alignment.

### 8.1.8 Flood frequency analysis – contributing catchments

A FFA was undertaken using historical stream gauge data sourced from BoM for each stream gauge within the contributing catchment as shown in Figure A3. Details of the gauges are provided in Table 8.4.

**Table 8.4 Gauge details**

| Gauge | Length of record | Location (catchment) | Comments |
|---|---|---|---|
| Booba Sands | 32 years (1987-current) | Macintyre Brook | All annual peaks were used. Maximum value is 1,160 $m^3/s$ (1988) |
| Farnbro | 57 years (1962-current) | Dumaresq River | All annual peaks were used. Maximum value is 1,600 $m^3/s$ (1976) |
| Roseneath | 47 years (1972-current) | Dumaresq River | All annual peaks were used. Maximum value is 5,687 $m^3/s$ (1976) |
| Holdfast | 47 years (1972-current) | Macintyre River | All annual peaks were used. Maximum value is 2,612 $m^3/s$ (1976) |
| Coolatai | 41 years (1978-current) | Ottleys Creek | All annual peaks were used. Maximum value is 562 $m^3/s$ (1994) |

The FFA 1% AEP flow estimates were compared against that determined by the hydrologic models. These are presented in Figure 8.1 to Figure 8.5. Figure 8.1 to Figure 8.5 present the results of the FFA as well as the hydrologic model flow estimates for the 1% AEP event and the historical calibration events. These figures show that the hydrologic model prediction of the 1% AEP flow is reasonable compared to the FFA.

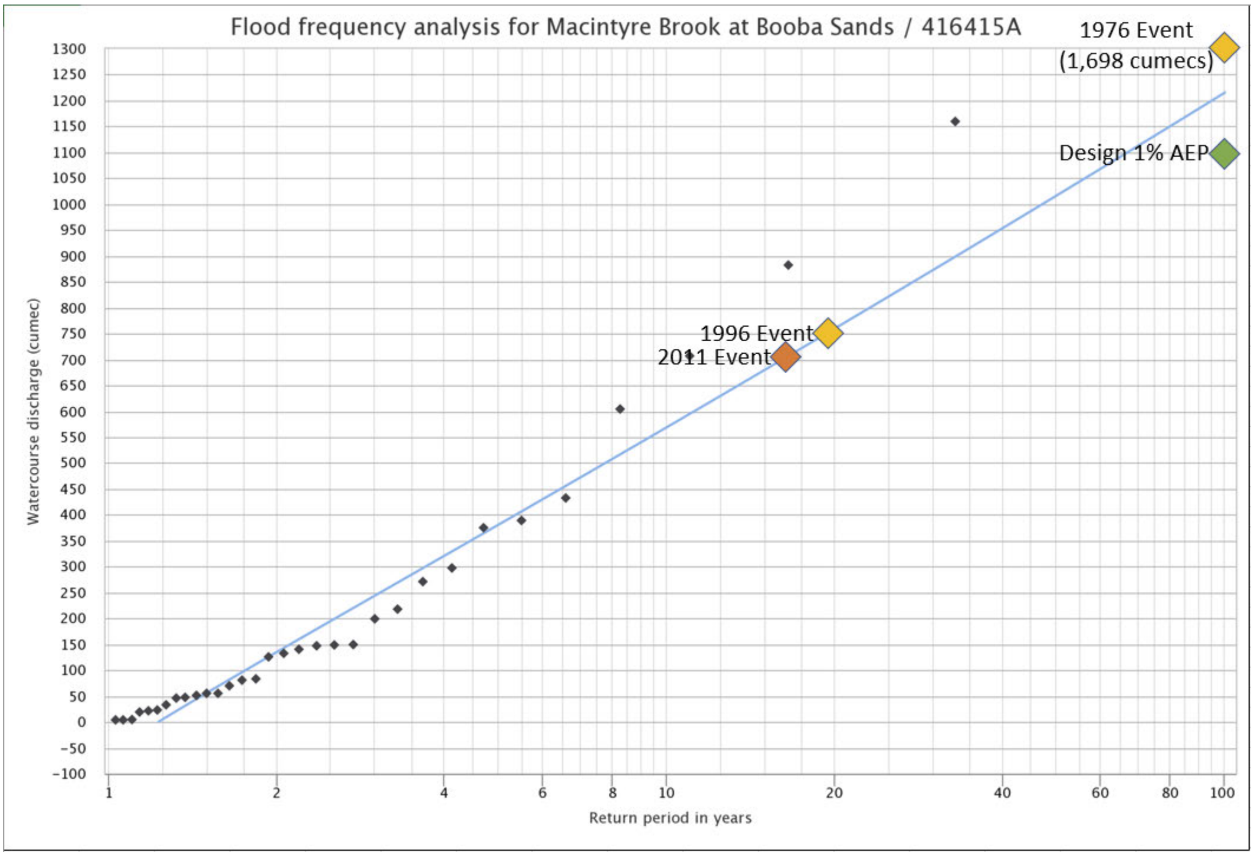



**Figure 8.1** Flood frequency analysis at Booba Sands (GEV)

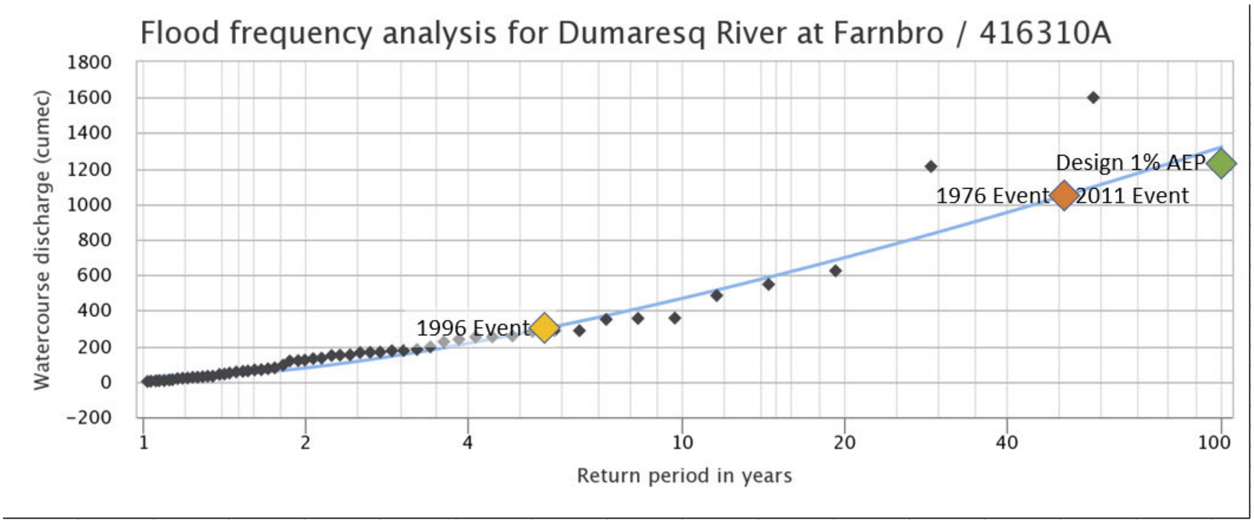



**Figure 8.2** Flood frequency analysis at Farnbro (GEV)

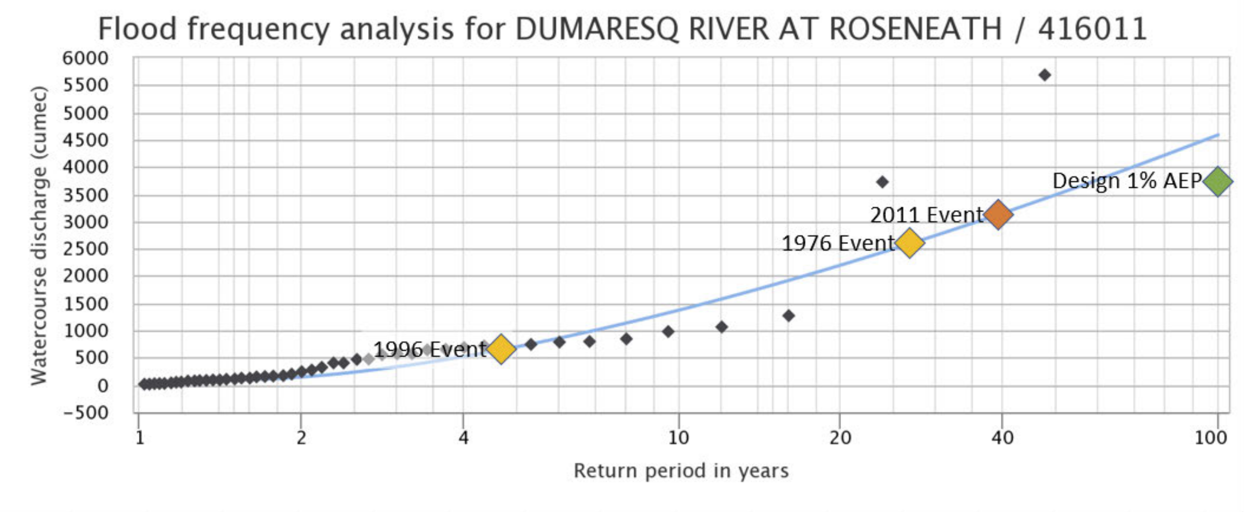



Figure 8.3 Flood frequency analysis at Roseneath (LP3)

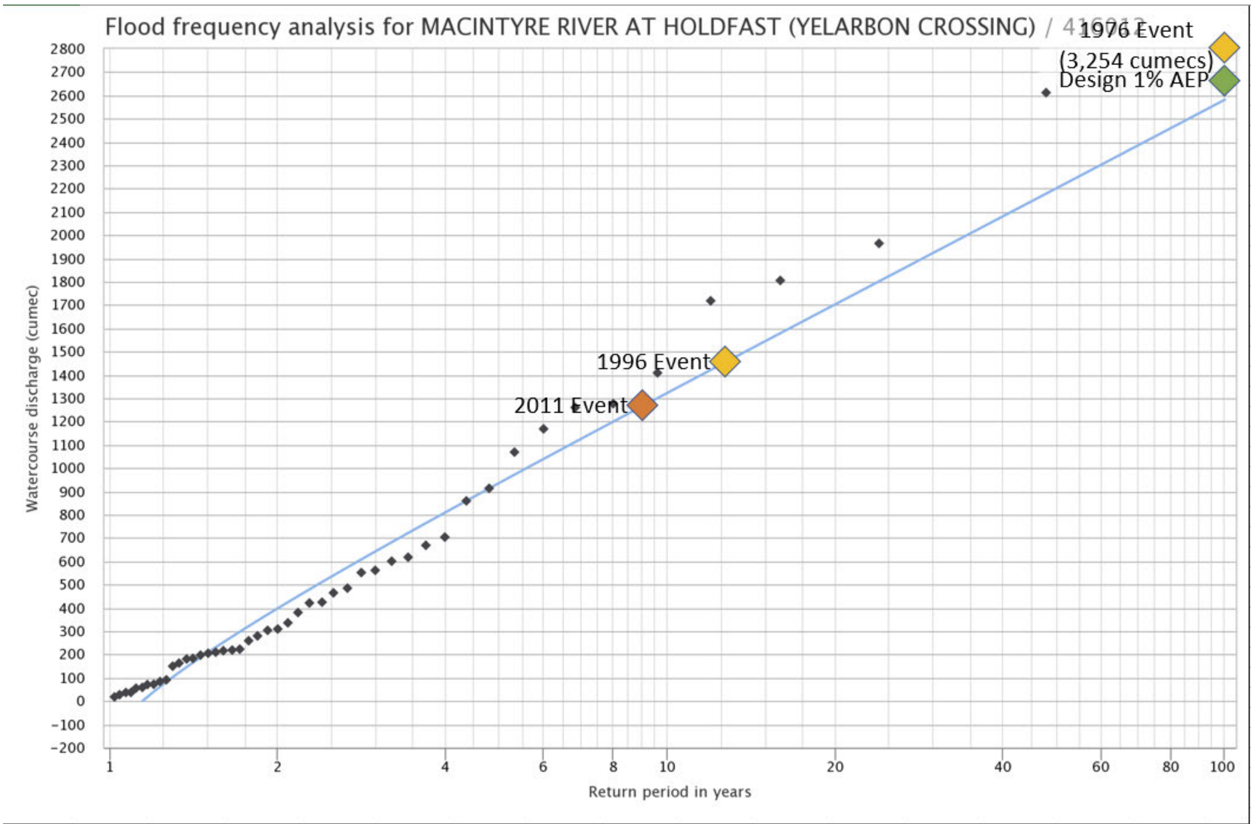



Figure 8.4 Flood frequency analysis at Holdfast (GEV)

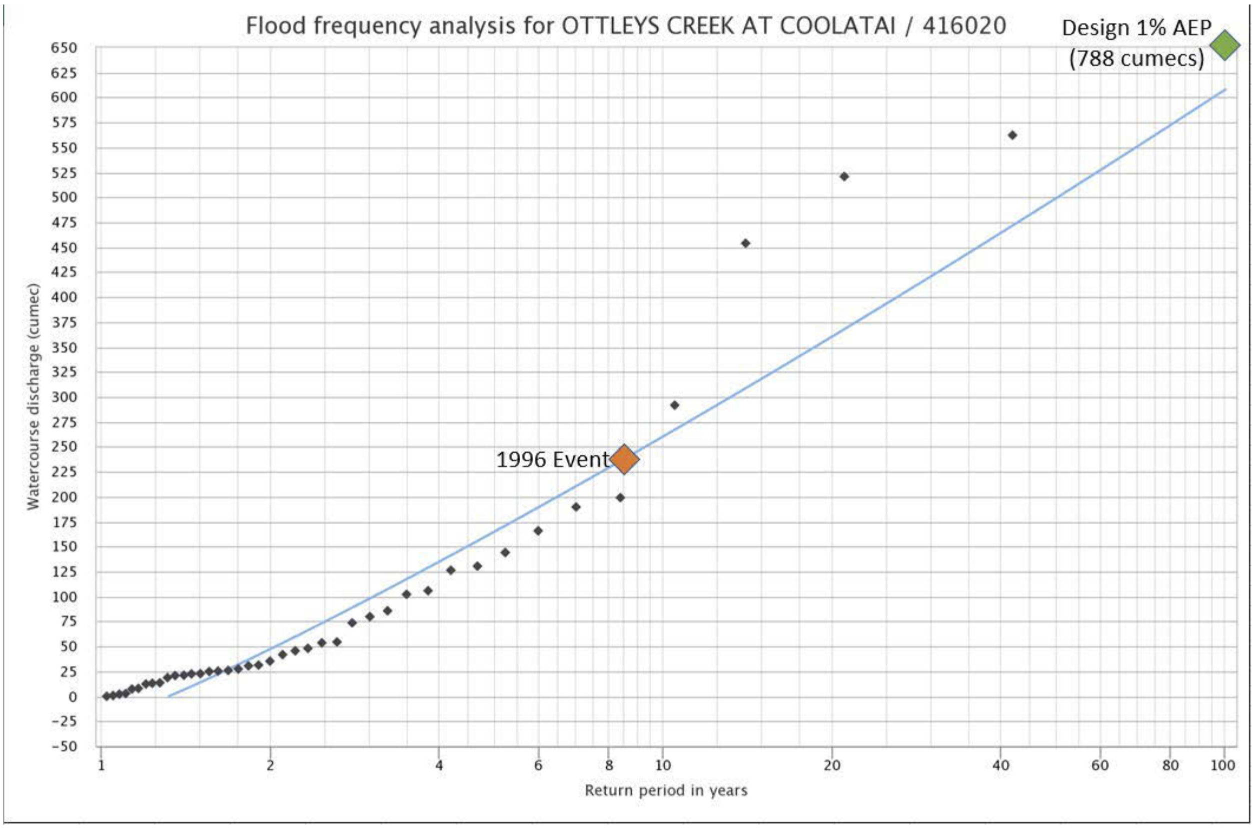



**Figure 8.5 Flood frequency analysis at Coolatai (GEV)**

## 8.2 Hydraulics

To establish an Existing Case hydraulic model, the 2011 historical model was updated to include the 2019 LiDAR data capture and digitisation of the levees to represent the current topographic conditions. Levees were manually digitised from upstream of the model to downstream of Goondiwindi. Digitisation was focused on key levees that impact the flood flows and are within the floodplain.

Design event flows were simulated in the hydraulic sub model for a range of AEP events: 20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF. The hydraulic sub-model has been developed to represent two scenarios, being the current state of development (Existing Case) and where the proposal has been constructed (Developed Case). The Existing Case hydraulic sub-model has been developed based on the 2011 calibrated sub-model.

The hydraulic sub-model has been reviewed for stability. The cumulative mass error is recorded as 0 per cent from the model log, indicating the model is not gaining or losing water through the simulation. The water levels and flows have been plotted for culverts (one dimensional structures) to check for any peak instabilities that may affect the results. There were no structures in the model demonstrating instabilities that may impact peak flood levels and flows. It should be noted when considering afflux predictions, it is common for “edge effects” to occur in the afflux grids. These result from wetting and drying anomalies that produce inconsistent peak water levels between existing and developed models and are usually below the model ground level (i.e. cell should be dry). These only occur in a few random cells at any location but can result in a larger change than expected. Where these occur in the hydraulic model results, they are individually sanity checked and disregarded. They may be visible in the afflux maps.

### 8.2.1 Existing Case topography

Preparation of Existing Case hydraulic sub-model to enable assessment of the proposal alignment and associated works. As part of the community and stakeholder engagement process, feedback identified that the levees represented in the DPIE hydraulic model as being of “unlimited height”, which whilst appropriate for the DPIE assessment tool, did not represent the actual levee heights on the floodplain. For design of the proposal alignment and mitigation of impacts, it was important that the hydraulic sub-model reflected the topographic reality of the floodplain. As new LiDAR was planned along the rail corridor, it was possible to expand the capture to include a significant portion of the floodplain and to obtain current levee heights on the floodplain. Therefore, two Existing Case hydraulic sub-model have been prepared, being:

- DPIE levees Existing Case – for this scenario the majority of the hydraulic sub-model area was covered by LiDAR collected for the proposal between September 2014 and January 2015. The hydraulic sub-model was set up using these datasets combined with the DPIE representation of floodplain levees.
- 2019 LiDAR (and levees) Existing Case – used the new LiDAR flown and processed November 2019 to provide a snapshot of current floodplain topography including current levee heights and floodplain features. To represent this, the hydraulic sub-model was set up using 2019 LiDAR including representation of existing levees on the floodplain. The levees were represented with z-lines in the hydraulic model. These z-lines were manually digitised using the LiDAR DEM and aerial photography. To ensure the ridges in the levees were picked up, elevation points along the z-lines were given the highest elevation within a buffer region of 30 m.

### 8.2.2 Critical duration assessment

A critical duration assessment was undertaken to determine which storm duration/s produced peak flood levels across the model domain and more specifically within the flood study area. To assess the critical storm duration the following methodology was adopted:

- Flows for the 1% AEP event were extracted from the hydrologic models for a range of durations from 540 to 5760 minutes for each of the ARR 2016 ten temporal patterns and simulated in the hydraulic sub-model
- Results from each storm duration and temporal pattern were mapped for the peak flood level for the 1% AEP event
- A critical duration assessment was undertaken at key locations across the model area to determine which duration produced the peak levels for the median ($6^{th}$ smallest) temporal pattern. The critical durations were determined to be 1080 m (07b, 08b), 1440 m (02b, 04b, 09b) and 2880 m (02b) for the 1% AEP event within the flood study area.

The same process was undertaken for the other design events with the critical durations for the other design events as presented below in Table 8.5.

**Table 8.5 Critical durations within the study corridor**

| Design event | Duration (minutes) |
|---|---|
| 20% AEP | 1080m_08b<br>1080m_10b<br>1440m_01b<br>1440m_10b<br>2880m_01b<br>2880m_10b<br>4320m_04b<br>4320m_05b<br>4320m_08b |

| Design event | Duration (minutes) |
|---|---|
| 10% AEP | 1080m_08b<br>1080m_10b<br>1440m_01b<br>1440m_02b<br>2880m_07b<br>2880m_10b<br>4320m_02b<br>4320m_04b |
| 5% AEP | 1080m_08b<br>1440m_01b<br>2880m_07b<br>2880m_10b<br>4320m_02b<br>4320m_07b |
| 2% AEP | 1080m_01b<br>1080m_08b<br>1440m_02b<br>1440m_04b<br>2880m_05b<br>2880m_10b<br>4320m_04b |
| 1% AEP | 1080m 07b<br>1080m 08b<br>1440m 02b<br>1440m 04b<br>1440m 09b<br>2880m 02b |
| 1 in 2,000 AEP | 2880m_01b<br>2880m_09b |
| 1 in 10,000 AEP | 1440m_09b<br>2160m_08b<br>2880m_05b<br>4320m_03b |
| PMF | 1440m 09b<br>2160m 08b<br>2880m 05b<br>4320m 03b |

### 8.2.3 Flood frequency analysis – Boggabilla and Goondiwindi gauges

A FFA of the Macintyre River stream gauge records at Boggabilla and Goondiwindi has been used to corroborate the magnitude of design flows used for assessment of the proposal within the hydraulic model area.

Flow estimates at the stream gauges are dependent on the reliability of the rating curves used to translate recorded water level to an equivalent flow. Discussion of the reliability of the gauge ratings and the impact on the FFA is provided in the sections below.

#### 8.2.3.1 Rating curves

Stream gauges physically measure water depth at a fixed point in the river over time. Rating curves are used to estimate stream flow from the measured level. Rating curves are ideally based on gauged flows (physically recorded during a flood event) where available. These data points may be extrapolated to higher levels/flows when necessary, ideally taking into consideration the geometry and flow properties of the channel.

Rating curves are usually assumed to provide a consistent relationship between level and flow. In reality, backwater and floodplain storage effects may result in different levels for the same flow depending on the rate of rise or fall of the flood level. Usually these effects are minor enough to be ignored.

Rating curves may be updated as more flow measurements become available, but also as flow conditions change. Channel properties such as shape and vegetation may be subject to rapid change during flood events or to long-term migration, with corresponding impact on the relationship between flow and level. Ratings with a fixed control such as a weir tend to be more consistent than those with erodible beds but may still be subject to downstream influences if/when the control is drowned. Care should therefore be taken when applying current rating curves to historical flood levels.

##### Boggabilla stream gauge rating

The Macintyre River gauge at Boggabilla has been in operation since 1894. Gauge details are summarised in Table 8.6.

**Table 8.6 Stream gauge details**

| Details | Boggabilla Gauge | Goondiwindi Gauge |
|---|---|---|
| River | Macintyre River | Macintyre River |
| Location | Boggabilla | Goondiwindi |
| Station Number | 416002 | 416201A |
| Operator | WaterNSW | QLD DNRME |
| Catchment Area | 22,600 km² | 23090 km² |
| Site Commenced | 0/10/1894 | 20/09/1917 |
| Gauge Datum | 208.478 m AHD | 207.577 m AHD |
| Control | Sand | Timber Weir |
| Maximum Gauged Stage | 12.537m (221.015 m AHD) | 9.95m (217.527 m AHD) |
| Highest Recorded Level | 12.80m (221.278 m AHD) | 10.618m (218.195 m AHD) |

The gauge rating is based on 603 gaugings recorded between 1924 and 19/11/2019. The gauge ratio between the highest gauged stage (level at which stream flow was physically recorded) and highest recorded flood level is 98% and is considered to be excellent, although it is noted that a significant proportion of high flows are carried out of channel over a wide floodplain. In high flow events (nominally above ~220 m AHD at the gauge), flow from Macintyre River breaks out into the Morella Watercourse and Whalan Creek systems upstream of the gauge location. The high-flow section of the current rating is strongly influenced by four flow measurements obtained during the 1996 flood, the highest three of which include an estimate of the breakout flows. The current rating should therefore be considered to give the total flow arriving upstream of Boggabilla, rather than the remaining flow in the Macintyre River at the actual gauge location downstream of Boggabilla.

The gauge rating has been updated on numerous occasions (WaterNSW website lists 79 historical tables; current table No. 153 dated from 14/01/2011). Additionally, it is understood that the Boggabilla gauge site has physically changed locations on several occasions, including in response to the construction of the Boggabilla Weir. The gauged flow points in Figure 8.6 show the recorded points for all locations with the current rating. No work to assess the rating quality or re-rate historical measurements has been undertaken.

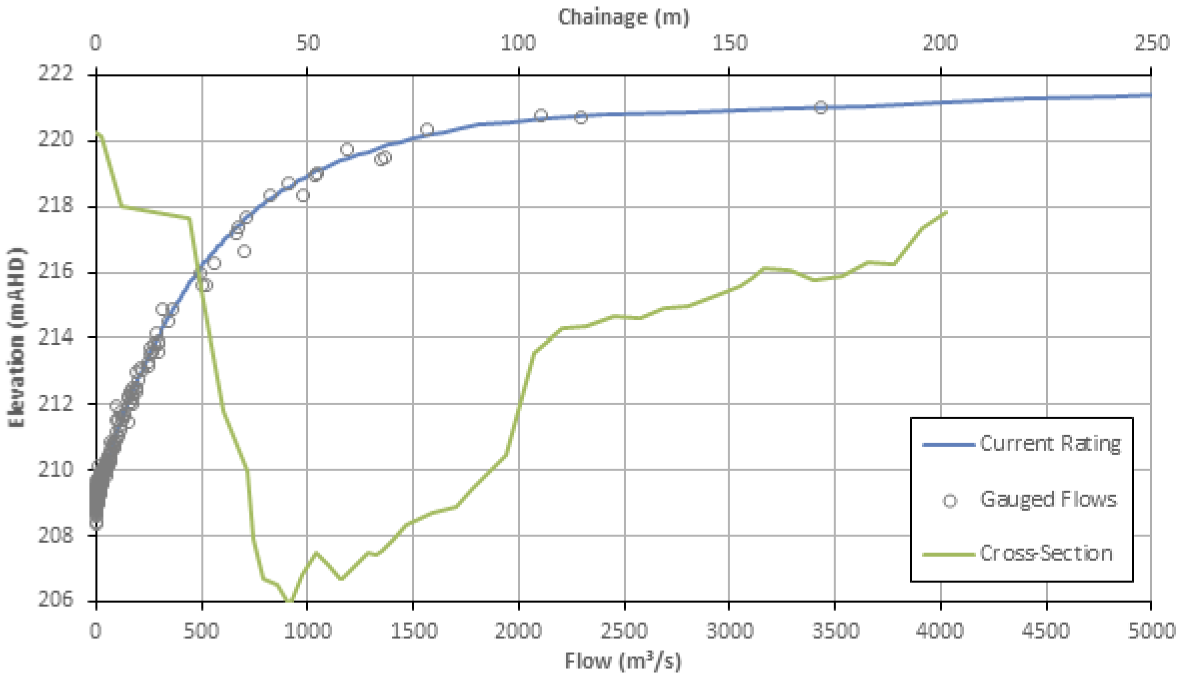



**Figure 8.6** **Boggabilla stream gauge rating**

Figure 8.7 shows the relationship between the Macintyre River flow at the gauge and the total flow upstream of Boggabilla for the recorded 1996 flow measurements and the TUFLOW hydraulic model peak flows for events of different magnitude. Although there is generally a good relatively good agreement between the TUFLOW model results and the gauged flows, examination of the middle two recorded flows identifies that the points have similar water levels and total flows, but the proportion of breakout flow varies significantly and counterintuitively decreases as the level and total flow increase. This could be attributed to either variability in the amount of breakout (potentially due to floodplain storage attenuation or downstream effects) or uncertainty in the flow estimates. This uncertainty/inconsistency in the flow measurements is carried into the rating. The rating is noted to become very sensitive at high flows, with minor differences (or errors) in level potentially representing a large change (or error) in the estimated flow.

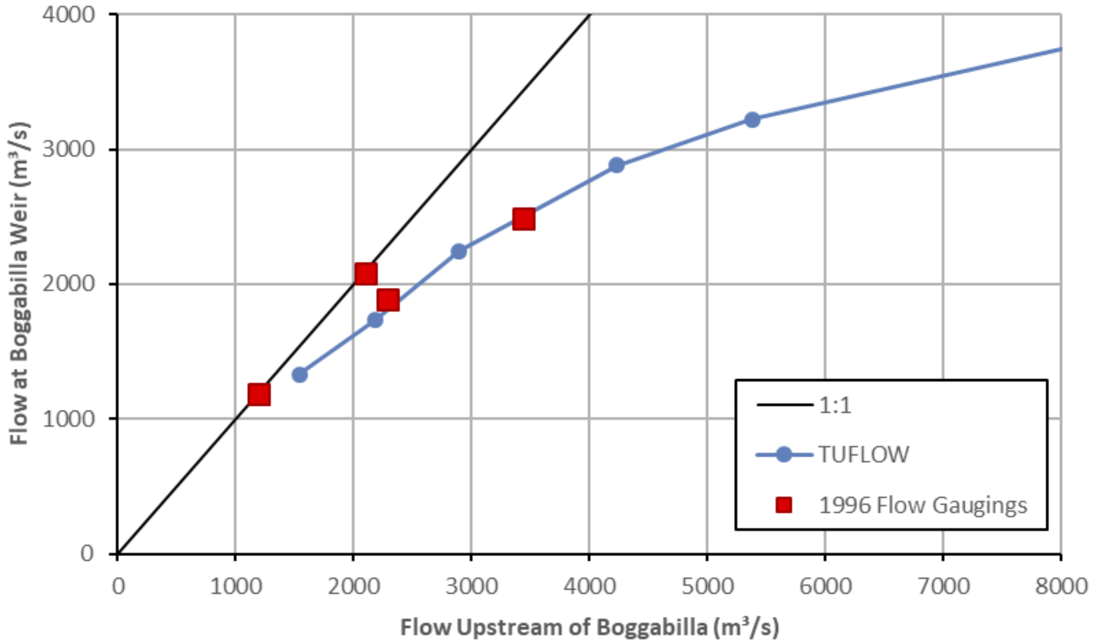



**Figure 8.7** **Relationship between flows upstream and downstream of Boggabilla**

### Goondiwindi stream gauge rating

The Macintyre River gauge at Goondiwindi commenced operation in 1917. The gauge is located approximately 6 km upstream from a timber weir, which acts as the low-flow control. During large flood events a significant proportion of the Macintyre River flow breaks out around the northern side of Goondiwindi upstream of the of the gauge site. This flow is not captured by the gauge rating. The proportion of this flow has potentially changed over time with the construction of various levees to protect Goondiwindi from floods.

The gauge rating is based on 331 gaugings taken between 1949 and 2018. The gauged level ratio of 94 per cent would ordinarily be considered as excellent. However, as shown in Figure 8.8, the level of the highest gauging is just below the surrounding floodplain level. The projection of the rating to higher levels/flows is therefore considered to be highly uncertain. Examination of the flood frequency analysis results discussed below and the TUFLOW hydraulic model calibration suggests that the rating does not reliably represent the flow conveyed in the floodplain.

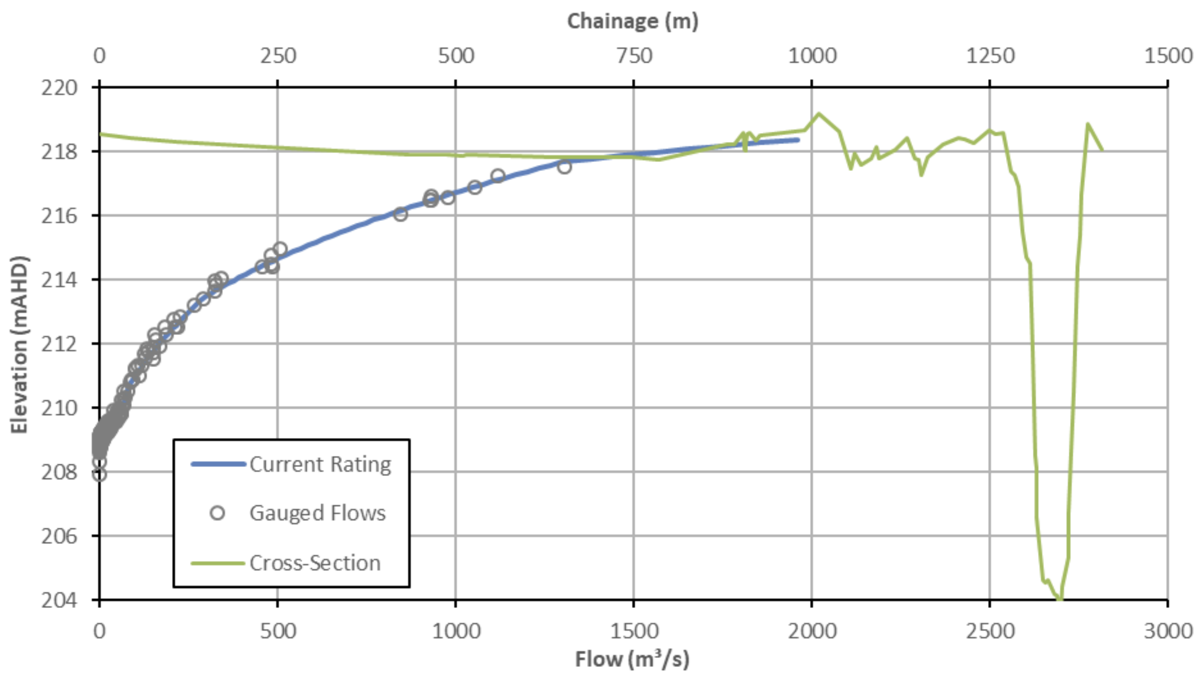



**Figure 8.8 Goondiwindi stream gauge rating**

## 8.2.3.2 Flood frequency analysis

Flood frequency analysis is the fitting of a probability relationship to historical data series. The data series is usually either the annual peak series (largest peak flood each year, ignoring other events that are potentially larger than the peak in other years) or a partial series consisting of the largest events irrespective of whether they occur in the same year. The resulting probability distributions correspond respectively to the AEP and ARI. The annual series is traditionally easier to assess.

The statistical analysis is typically based on the assumption that the data series fits a recognised probability distribution. The Log Pearson Type III (LP3) and Generalised Extreme Variable (GEV) probability distributions are commonly applied to annual peak flow series for Australian catchments. ARR (2016) does not advocate a specific distribution, and rather recommends testing different distributions and adopting the one that best fits the data.

The FFA has been conducted using the FLIKE statistical analysis software package. FLIKE uses Bayesian fitting techniques to determine the most likely probability curve to match the recorded data. The technique allows missing and censored data (typically low flows filtered to prevent excessive influence on projection of the high-flow curve) to be included as unknown values below a threshold.

## Boggabilla flood frequency analysis

Numerous previous studies have performed flood frequency analysis of the Boggabilla stream gauge. A summary of the estimated 1% AEP flows is summarised in Table 8.7. FFA results are dependent upon the adopted probability distribution, method used to obtain a best fit, and the magnitude of the flows estimated for each flood event. Historically there appears to have been significant uncertainty around the magnitude of the larger flood events. For example, the 1976 flood of record has been estimated to have a peak varying flow from 2,760 $m^3/s$ (LT 2007) and 5,500 $m^3/s$ (LT 2004).

Another significant complication is whether the flow lost from the system into Whalan Creek and other breakouts upstream of the gauge location during high flow events has been included. FFA should ideally be conducted on the total catchment flow, as 'lost' flow above a threshold would lead to discontinuities in the relationship (see discussion below). It is unknown whether the previous studies report total flow or flow at the gauge.

**Table 8.7 Boggabilla flood frequency analysis assessment comparison of results to previous studies**

| Study | Year of study | FFA 1% AEP flow ($m^3/s$) | Modelled 1% AEP water level (m AHD) |
|---|---|---|---|
| L&T | 2004 | 3,120 | 221.3 |
| L&T | 2007 | 2,912 | 221.2 |
| SMEC | 2016 | 3,336 | 221.2 |
| OEH | 2018 | 2,800 | - |
| FFJV | 2019 | 3,800[a] | 221.2 |

**Table note:**

a Includes Whalan Ck and associated overbank flows (extracted from reporting DS Boggabilla)

For the current assessment, the FFA has been conducted using the annual peak series. Comparison of peak flows and levels indicates that the flows are the total flows from the catchment inclusive of Morella Watercourse and Whalan Creek flows, although as previously noted the reliability of the gauge rating for high flows is low. The gauge has 117 years of available record with details presented in Table 8.8. FLIKE's Multiple-Grubs-Beck test recommended censoring of 39 low-flow records to minimise influence on the high flow projection, with sensitivity testing identifying that this had relatively minor influence on the final flow estimates. Analysis was conducted for both the LP3 and GEV distributions, with the LP3 considered to give a slightly better fit (this is consistent with experience in south-east Queensland and NSW).

**Table 8.8 Stream gauge record**

| Item | Boggabilla Gauge | Goondiwindi Gauge |
|---|---|---|
| Years of record | 117 | 76 |
| Censor threshold | 350 $m^3/s$ | 110 $m^3/s$ |
| Censored records | 39 | 8 |

Results of the FFA are compared with peak flows from the Design Event modelling in Figure 8.9. Flows at the Boggabilla Weir and total flow upstream of Boggabilla are presented to demonstrate the effect of the Morella Watercourse and Whalan Creek breakouts.

Below approximately 1,200 $m^3/s$ most of the flow is conveyed in the main channel. In the 20% AEP event, the breakout flow constitutes less than 15 per cent of the total flow. As also observed in Figure 8.7, the proportion of breakout flow increases significantly with flood magnitude and by the 1 in 2,000 AEP event, less than 40 per cent of the upstream flow is conveyed in the Macintyre River downstream of Boggabilla.

Despite the FFA results theoretically predicting the total (upstream) flows, a relatively good agreement between with downstream (excluding breakout) flows is observed up to around 5% AEP. This would suggest the Design Event flows are overestimated, however it is also important to consider the sensitivity and uncertainty in the proportion of breakout flow in the rating.

The Design Event flows and FFA agree relatively well at frequent events, where there is most confidence in the rating and statistical predictions of the FFA. If the flows extracted from the TUFLOW calibration runs are used to replace the rated flows using the same plotting position, noting that there is significant uncertainty in both the plotting position and the flow (i.e. unfactored and factored rainfall were used) then Figure 8.10 suggests that the rated flows for the 1976 and 2011 design events are underestimated and the Design Event flows are consistent with the observed historical event probabilities.

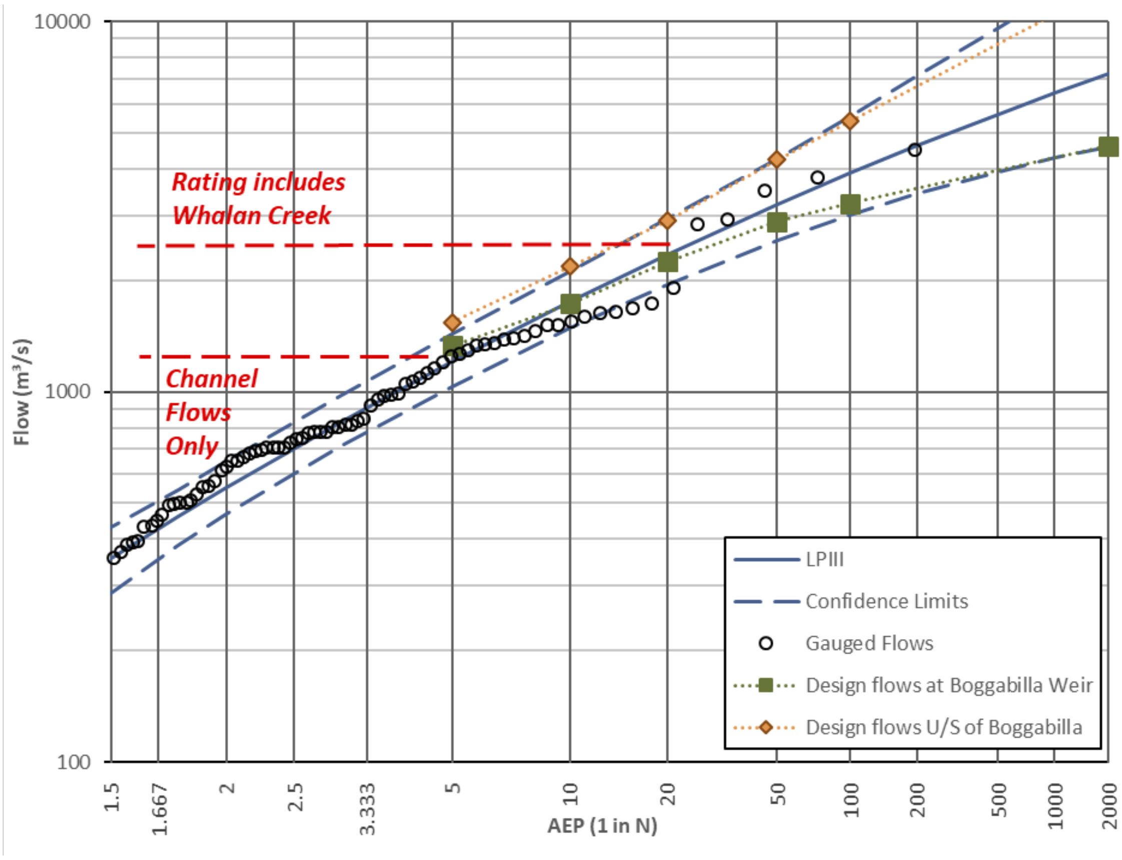



**Figure 8.9** **Boggabilla gauge flood frequency analysis results**

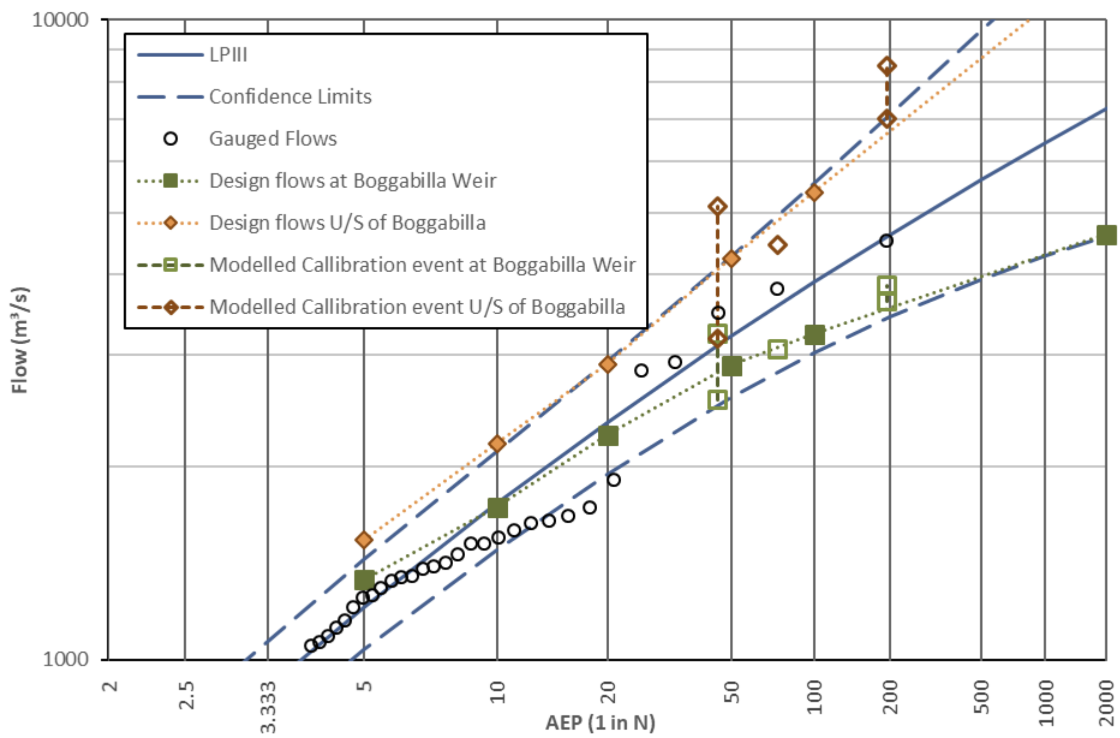



**Figure 8.10** **Probability assessment of TUFLOW model flow estimates for historical events**

## Goondiwindi flood frequency analysis

Although the Goondiwindi gauge has been operational since 1917, continuous stream gauge data is only available since 1943 giving 76 years of data (Goodiwindi Weir was constructed in 1941, so sourcing prior data would serve little practical point). An LP3 distribution fit to the annual peak data series exhibits a significant downward curvature (skew = -1.64). This is atypical of natural catchments in the area, and can be attributed to the breakout of higher flows around Goondiwindi upstream of the gauge site (as well as additional flows upstream of Boggabilla), which leads to 20 of the years (over 1/4 of the data set) having a rated flow between 1200 m³/s and 1800 m³/s.

The validity of fitting an LP3 (or any other) probability distribution to a streamflow record exhibiting these characteristics is questionable. Comparison of the FFA results with peak flows from the Design Event modelling in Figure 8.11 shows a reasonable match for the more frequent events (20% to 5% AEP). The divergence for larger events can likely be attributed to the uncertainty of the rating projection above the bank-full capacity and the ability to represent overbank flows discussed in above. The reported Design Event flows include all floodplain flows south of Goondiwindi.

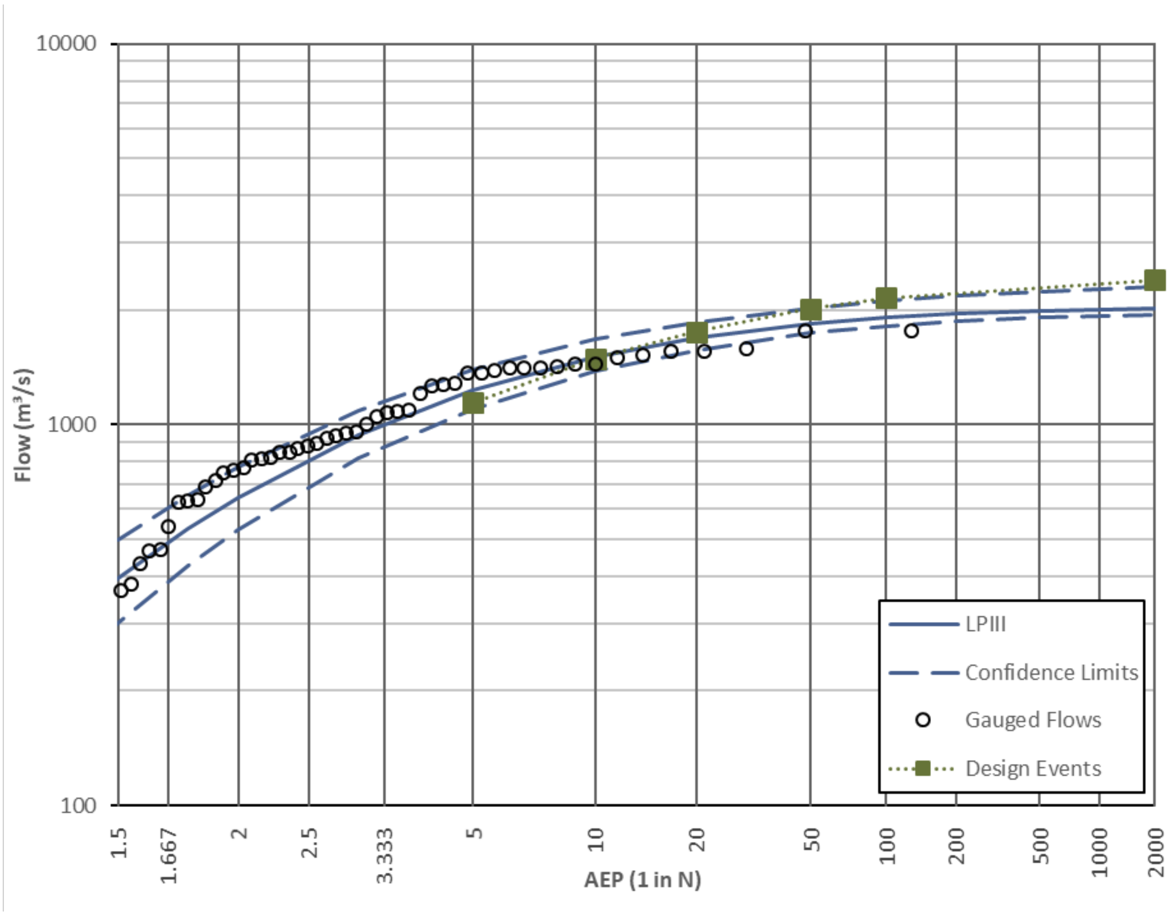



**Figure 8.11 Goondiwindi gauge flood frequency analysis results**

### 8.2.4 Design flows based on flood frequency analysis

Preliminary results from the Design Event Analysis predicted flows significantly higher than what was expected for the 1% AEP flood event, (3,800 $m^3/s$) based on the FFA assessment at the Boggabilla Gauge. This is due to the inherent assumption in Design Event Analysis that the entire catchment will experience rainfall of the same magnitude. In a catchment like the Border Rivers, there are several major catchments that meet upstream of the study corridor. In an actual rainfall event it is highly unlikely that all catchments will experience the same AEP flood event, which is seen by the results of the FFA analysis. To account for this phenomenon, a factor has been applied to the four major inflows, Macintyre River, Dumaresq River, Macintyre Brook and Ottleys Creek. This factor was selected through iterations to achieve reasonable agreement with the 1% AEP flows in accordance with the FFA, with an uniform factor of 0.7 adopted for all inflows. In the absence of a full joint probability assessment, this approach was considered appropriate for the level of design currently being undertaken. At Detailed Design the benefit of undertaking joint probability analysis should be considered. It is noted however as the base data (Boggabilla gauge) for reconciling flows will be the same, the assessment is not expected to produce significantly different flows. In addition, it is noted a large change in flows in the Macintyre River catchment results in a relatively small change in flood levels in the vicinity of the proposal alignment (Water Technology 2016).

Table 8.9 shows the FFA predicted flows and the factored modelled flows at the Boggabilla Gauge (DS Boggabilla) and for the full floodplain flow (US Boggabilla). With a 0.7 factor applied the flows are predicted to be higher than the flows derived from the FFA.

**Table 8.9 Factored design flows – Boggabilla Gauge rating**

| Design event | FFA predicted flows ($m^3/s$) | FFA predicted flows (ML/d) | TUFLOW model flows (factored) ($m^3/s$) DS Boggabilla | TUFLOW model flows (factored) (ML/d) DS Boggabilla | TUFLOW model flows (factored) ($m^3/s$) US Boggabilla | TUFLOW model flows (factored) (ML/d) US Boggabilla |
|---|---|---|---|---|---|---|
| 1% AEP | 3,800 | 328,320 | 3,294 | 284,602 | 5,379 | 464,746 |
| 2% AEP | 3,100 | 267,840 | 2,875 | 248,400 | 4,235 | 365,904 |
| 5% AEP | 2,300 | 198,720 | 2,219 | 191,722 | 2,895 | 250,128 |
| 10% AEP | 1,700 | 146,880 | 1,635 | 141,264 | 2,180 | 188,352 |
| 20% AEP | 1,300 | 112,320 | 1,289 | 111,370 | 1,539 | 132,970 |

### 8.2.5 Modelling outcomes

The Existing Case peak water levels are presented in for the range of modelled AEP events. Figure A8-A to Figure A9-G present the existing case results. Widespread inundation is predicted under the 1% AEP event on the Macintyre River floodplain, with depths of approximately 10 to 13 m in the Macintyre River, 6 m in Whalan Creek and up to 2 m on the floodplain area. Velocities approximately 0.5 m/s are predicted across the floodplain area under the 1% AEP event with higher velocities in the creek and river channels. Flow remains mainly in the creek and river channels up to the 10% AEP event and breakouts occur downstream of the Toomelah community between a 10% and 5% AEP event.

Within the flood study area under the 1% AEP event, over 14.5 km of the existing non-operational rail line is inundated, and Bruxner Way is also inundated for approximately 18 km. There are also local access roads to properties and Toomelah community that are cut by flood waters. Table 8.10 presents a summary of overtopping depths for key roads and the existing rail near the proposed alignment.

**Table 8.10 Existing Case – Overtopping depths of key infrastructure**

| Infrastructure | Location | Maximum overtopping depth (m) | | | | |
|---|---|---|---|---|---|---|
| | | 1% AEP | 2% AEP | 5% AEP | 10% AEP | 20% AEP |
| Kildonan Road | Intersection with proposed alignment | 0.36 | 0.05 | Dry | Dry | Dry |
| Tucka Tucka Road | Intersection with proposed alignment* | 0.15 | Dry | Dry | Dry | Dry |
| Bruxner Way | Whalan Creek Bridge | 3.48 | 3.36 | 3.21 | 3.06 | 2.54 |
| Bruxner Way | Near intersection with rail alignment | 0.83 | 0.69 | 0.48 | 0.15 | Dry |
| Bruxner Way | Strayleaves Creek | 1.57 | 1.56 | 1.49 | 1.39 | 1.29 |
| Boggabilla Rail | Strayleaves Creek | 1.40 | 1.38 | 1.32 | 1.24 | 1.14 |
| North Star Road | Intersection with Bruxner Way | Dry | Dry | Dry | Dry | Dry |
| North Star Road | Forest Creek | 0.92 | 0.8 | 0.77 | 0.70 | 0.61 |
| Boggabilla Rail | Forest Creek | 0.65 | 0.62 | 0.55 | 0.50 | 0.45 |
| Boggabilla Rail | Back Creek | 0.45 | 0.44 | 0.40 | 0.36 | 0.34 |
| North Star Road | Mobbindry Creek | 0.71 | 0.65 | 0.52 | 0.49 | 0.42 |
| Boggabilla Rail | Mobbindry Creek | 0.36 | 0.26 | 0.11 | 0.09 | 0.03 |

**Table note:**

* Tucka Tucka Road inundated to west and east of this location

# 9 Developed Case modelling

The Developed Case incorporates the proposal design into the Existing Case hydraulic model. The Developed Case model was run for the nominated design events and assessed against the hydraulic design criteria and flood impact objectives. Mitigation measures that have been incorporated into the Project design include:

- The proposal has been designed to achieve the hydraulic design criteria (refer Section 4.1), and key design criteria including:
  - 50-year design life for formation and embankment performance
  - Track drainage ensures that the performance of the formation and track is not affected by water
  - Earthworks designed to ensure that the rail formation is not overtopped during a 1% AEP flood event
  - Embankment cross section can sustain flood levels up to the 1% AEP
- Bridges are designed to withstand flood events up to and including the 1 in 2,000 AEP event
- Where possible, the proposal utilises existing rail corridors as much to avoid introducing a new linear infrastructure corridor across floodplains
- The proposal incorporates bridge and culvert structures to maintain existing flow paths and flood flow distributions
- Bridge and culvert structures have been located and sized to avoid increases in peak water levels, velocities and/or duration of inundation, and changes flow distribution in accordance with the flood impact objectives
- Progressive refinement of bridge extents and culvert banks (number of barrels and dimensions) has been undertaken as the proposal design has evolved. This refinement process has considered engineering requirements as well as progressive feedback from stakeholders to achieve acceptable outcomes that address the flood impact objectives.
- Scour and erosion protection measures have been incorporated into the design in areas determined to be at risk, such as around culvert headwalls, drainage discharge pathways and bridge abutments
- A climate change assessment has been incorporated into the design of cross drainage structures for the proposal in accordance with the Australian Rainfall and Runoff Guidelines (2016) for the 1% AEP design event to determine the sensitivity of the design, and associated impacts, to the potential increase in rainfall intensity
- Identification of flood sensitive receptors and engagement with stakeholders to determine acceptable design outcomes.

In some areas of the floodplain, both local catchment events and regional flooding events can occur. Therefore, sizing of drainage structures needed to consider both scenarios. The following approach was adopted:

- A local drainage assessment was undertaken to determine drainage structures required to convey runoff from local catchment areas
- The size of drainage structures required to convey flood flows associated with the regional flood event were determined
- The larger drainage structure size was adopted and included in the proposal design. The larger structure was included in the hydraulic sub-model to assess impacts associated with the proposed works.

The following sections outline how the proposal design addresses the hydraulic design criteria and flood impact objectives on each floodplain. For the hydraulic modelling the adjacent B2G proposal alignment has been included in the Developed Case to quantify cumulative impacts.

## 9.1 Drainage structures

The hydraulic design of the flood drainage structures was undertaken using the TUFLOW model (1d and 2d approach). On the Macintyre River floodplain, the proposal design includes:

- Flood drainage structures
  - 13 bridges
  - 26 reinforced concrete pipe (RCP) locations (multiple cells in places)
  - 6 reinforced concrete box culvert (RCBC) locations (multiple cells in places)
- Local drainage structures
  - 17 RCP locations (multiple cells in places)
  - 1 RCBC locations (multiple cells)
- Local Drainage Structures B2G Section:
  - 3 RCP locations (multiple cells in places)
  - 8 RCBC locations (multiple cells in places)
- Removal of non-operational rail line up to southern side of Whalan Creek
- Roadworks including drainage structures on Bruxner Way.

The locations of the structures are presented in Figure A11-A to Figure A11-F. The structures listed in Table 9.1 were assessed within the hydraulic model. It is noted that these structure details reflect how the structures are represented in the hydraulic sub-model and minor variations may occur between the modelled structures and the design structures (i.e. culvert lengths).

Bridges have been represented within the TUFLOW hydraulic model through use of layered flow constrictions. Each bridge within the model has had a flow constriction coefficient applied to represent obstruction of waterway area due to the piers.

Form loss was also applied to all proposed bridges. A form loss value of 0.2 was applied to Layer 1 (beneath the bridge deck) of the layered flow constrictions to represent the waterway opening area. This value is considered conservative, although it is noted that changing form loss would not have a significant impact in this floodplain where the floodwaters are slow moving. No additional blockage was applied to the waterway area. The bridge deck (Layer 2) was modelled as 100 per cent blocked, and above the bridge deck (Layer 3), 50 per cent blocked. It is recommended that following detailed design, these parameters be revisited.

**Table 9.1 Macintyre River floodplain – flood structure locations and details**

| Chainage (km) | Waterway | Structure type | No of culvert cells | Diameter/width of culvert or bridge length (m) | Culvert height (m) or soffit level (m AHD) | Culvert length (m) |
|---|---|---|---|---|---|---|
| 5.58 | Mobbindry Creek | RCP | 2 | 1.05 | - | 17 |
| 5.76 | | Bridge (BR01) | - | 109 | 243.3 | - |
| 6.08 | | RCP | 7 | 2.10 | - | 18 |
| 6.12 | | RCP | 7 | 2.10 | - | 16 |
| 6.23 | | Bridge (BR02) | - | 170 | 242.91 | - |
| 6.53 | | RCP | 6 | 2.10 | - | 17 |
| 6.58 | | RCP | 5 | 2.10 | - | 17 |
| 8.11 | Back Creek | Bridge (BR03) | - | 67 | 238.6 | - |
| 15.33 | Forest Creek | RCBC | 10 | 1.2 | 1.2 | 8 |
| 15.52 | | RCBC | 10 | 1.2 | 1.2 | 10 |

| Chainage (km) | Waterway | Structure type | No of culvert cells | Diameter/width of culvert or bridge length (m) | Culvert height (m) or soffit level (m AHD) | Culvert length (m) |
|---|---|---|---|---|---|---|
| 15.67 | | RCP | 10 | 1.2 | - | 13 |
| 15.83 | | RCP | 20 | 1.2 | - | 14 |
| 15.90 | | RCP | 20 | 1.2 | - | 14 |
| 15.98 | | RCP | 20 | 1.2 | - | 16 |
| 16.08 | | RCP | 20 | 1.2 | - | 15 |
| 16.29 | | Bridge (BR04) | - | 40 | 229 | - |
| 16.49 | | RCBC | 1 | 3 | 2.4 | 9 |
| 16.60 | | RCP | 8 | 1.2 | - | 17 |
| 16.83 | | RCP | 8 | 1.2 | - | 17 |
| 20.73 | Strayleaves Creek | Bridge (BR05) | - | 131 | 227.1 | - |
| 21.35 | | RCP | 3 | 1.35 | - | 28 |
| 21.97 | | RCP | 3 | 1.05 | - | 20 |
| 22.27 | | RCP | 3 | 1.2 | - | 13 |
| 22.86 | Whalan Creek | RCP | 10 | 1.2 | - | 25 |
| 23.22 | | RCP | 10 | 1.2 | - | 25 |
| 23.70 | | RCP | 10 | 1.2 | - | 25 |
| 23.80 | | RCP | 10 | 1.2 | - | 25 |
| 24.03 | | RCP | 8 | 1.05 | - | 26 |
| 24.2 | | RCP | 5 | 0.9 | - | 28 |
| 24.62 | | RCBC | 35 | 1.2 | 0.9 | 27 |
| 24.71 | | RCBC | 35 | 1.2 | 0.9 | 26 |
| 24.85 | | RCBC | 35 | 1.2 | 0.9 | 30 |
| 25.34 | | Bridge (BR06) | - | 131 | 227.77 | - |
| 25.8 | | Bridge (BR07) | - | 104 | 229.9 | - |
| 26.09 | | Bridge (BR08) | - | 156 | 230.4 | - |
| 27.06 | | RCP | 10 | 1.2 | - | 15 |
| 27.56 | | Bridge (BR09) | - | 116 | 227.7 | - |
| 28.03 | | Bridge (BR10) | - | 117 | 227.7 | - |
| 30.35 | Macintyre River | Bridge (BR11) | - | 1748 | 230 | - |
| 31.26 | | RCP | 10 | 1.8 | - | 32 |
| 31.32 | | RCP | 10 | 1.8 | - | 30 |
| 31.52 | | Bridge (BR12) | - | 144 | 227.46 | - |
| 31.87 | | RCP | 15 | 0.9 | - | 14 |
| 31.97 | | RCP | 15 | 0.9 | - | 15 |
| 32.55 | | Bridge (BR13) | - | 521 | 225.71 | - |

Local drainage structures are presented in Table 9.2. These structures were sized through the local drainage design and if they interacted with flood waters on the Macintyre River floodplain they were incorporated in the hydraulic model.

**Table 9.2 Macintyre River floodplain – local drainage structure locations and details**

| Chainage (km) | Structure type | No of cells | Diameter or width (m) | Height (m) | Culvert length (m) |
|---|---|---|---|---|---|
| 5.12 | RCP | 2 | 0.90 | - | 13 |
| 9.00 | RCP | 6 | 1.20 | - | 12 |
| 10.19 | RCP | 2 | 1.35 | - | 12 |
| 10.82 | RCP | 3 | 1.80 | - | 14 |
| 11.87 | RCP | 2 | 0.90 | - | 12 |
| 12.43 | RCP | 1 | 1.35 | - | 12 |
| 13.44 | RCP | 1 | 0.90 | - | 13 |
| 14.16 | RCP | 2 | 1.20 | - | 18 |
| 15.00 | RCP | 4 | 1.05 | - | 13 |
| 18.09 | RCP | 3 | 1.65 | - | 13 |
| 19.60 | RCP | 4 | 1.20 | - | 13 |
| 34.70 | RCBC | 19 | 3.00 | 1.50 | 18 |
| 35.03 | RCP | 20 | 0.90 | - | 16 |
| 35.08 | RCP | 25 | 0.90 | - | 17 |
| 35.21 | RCP | 20 | 0.90 | - | 18 |
| 35.88 | RCP | 7 | 0.90 | - | 15 |
| 35.91 | RCP | 7 | 0.90 | - | 15 |
| 36.04 | RCP | 6 | 0.90 | - | 14 |
| **Macintyre River Floodplain North (B2G)** | | | | | |
| 6.6 | RCBC | 3 | 1.5 | 1.2 | 17 |
| 8.39 | RCBC | 8 | 1.2 | 1.2 | 10 |
| 13 | RCBC | 13 | 2.4 | 1.2 | 10 |
| 17.89 | RCP | 2 | 1.05 | | 22 |
| 18.51 | RCP | 2 | 1.2 | | 20 |
| 18.87 | RCP | 4 | 1.2 | | 20 |
| 20 | RCBC | 6 | 1.2 | 1.2 | 11 |
| 22.42 | RCBC | 8 | 1.2 | 1.2 | 9 |
| 23.05 | RCBC | 6 | 1.2 | 1.2 | 10 |
| 23.53 | RCBC | 7 | 1.2 | 1.2 | 9 |
| 24.41 | RCBC | 3 | 1.2 | 1.2 | 9 |

### 9.1.1 Bruxner Way Design

At approximately rail alignment chainage 25.00 km, a Bruxner Way re-alignment is proposed to facilitate the crossing of the proposal alignment over Bruxner Way. The design for the road alignment considers the shallow flooding experienced in this area. Options assessment identified that a design with a lower road height and small cross-drainage structures minimised the impacts upstream of Bruxner Way where are a higher road with larger drainage structures could not provide the same benefit in a 1% AEP flood event. Therefore, Bruxner way realignment is proposed to convey the flood flow over the road during large rainfall events. This currently occurs on the existing Bruxner Way alignment during large rainfall events. The cross-drainage structures for Bruxner Way realignment are presented in Table 9.3.

Table 9.3 Bruxner Way realignment, cross-drainage structures

| Road chainage (km) | Culvert name | Road name | Structure type | Number of cells | Span (m) | Height (m) | Length (m) |
|---|---|---|---|---|---|---|---|
| 0.27 | C0.27_BW | Bruxner Way | RCBC | 3 | 1.20 | 0.45 | 21 |
| 1.37 | C1.37_BW | Bruxner Way | RCBC | 3 | 1.20 | 0.30 | 17 |
| 1.71 | C1.71_BW | Bruxner Way | RCBC | 1 | 1.20 | 0.30 | 15 |

It is noted that these structures are to convey local drainage only and have minimal benefit in a Macintyre River flood event. Therefore. these structures were not incorporated in the hydraulic model.

## 9.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case with the drainage structures and embankment areas included. Modelling of a range of events was undertaken (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events). The proposal design outcomes relative to the hydraulic design criteria are presented in the following sections.

### 9.2.1 Flood immunity and overtopping risk

Under the 1% AEP event there is no overtopping of the formation predicted across the Macintyre River floodplain. The risk of overtopping of the rail alignment has been assessed for the modelled extreme events with Table 9.4 presenting the overtopping locations by chainage and the depth of water above formation level and over the rail level.

Table 9.4 Extreme events – Overtopping depths and locations

| Chainage (km) | Depth of water above formation level (m) | | | Depth of water above top of rail (m) | | |
|---|---|---|---|---|---|---|
| | 1 in 2,000 AEP | 1 in 10,000 AEP | PMF | 1 in 2,000 AEP | 1 in 10,000 AEP | PMF |
| 7.20 – 8.10 | - | - | 1.0 | - | - | 0.3 |
| 15.00 – 17.00 | - | - | 0.9 | - | - | 0.2 |
| 18.70 – 20.80 | 1.4 | 2.0 | 2.7 | 0.7 | 1.3 | 2.0 |
| 20.80 – 25.50 | 0.5 | 1.2 | 2.0 | - | 0.5 | 1.3 |
| 28.00 – 28.50 | - | 0.1 | 1.2 | - | - | 0.5 |
| 28.50 – 31.00 | - | 0.3 | 1.4 | - | - | 0.7 |
| 31.00 – 34.00 | - | - | 1.2 | - | - | 0.5 |
| 34.00 – 39.50 | - | 0.2 | 2.0 | - | - | 1.3 |

The portion of the alignment that experiences the largest increase in peak water levels upstream of the proposal alignment is approximately between Chainages 20 km and 24 km. In this area there is a drop in water levels between the upstream and downstream sides of the alignment and overtopping occurring under the 10,000 AEP and larger events. During detailed design mitigation measures to refine the design and to address the risks to the embankment and rail infrastructure as well as downstream properties will be investigated further. If required this may include engineering solutions to increase the strength and resilience of the rail embankment in this specific location, thereby mitigating the flood risk impact to both the asset and the adjacent floodplains. It should be noted that the inclusion of the 2019 LiDAR has enabled accurate representation of existing levee heights and hence the impact of the existing levees being overtopped has been identified.

Ch 28.00 km to Ch 28.50 km is a location of the overtopping under the 1 in 2,000 AEP event that functions differently to Ch 20 km to 24 km. Figure 9.1 presents the water levels on both sides of the rail during the 1 in 2,000 AEP, 1 in 10,000 AEP, and PMF events. The results show that at the time of overtopping (and throughout the event) the water levels are predicted to be similar on both sides of the rail embankment. Under these rare events, the bridge structures and culverts allowing adequate passage of flow during the flood event. Therefore, at the time of overtopping a significant difference in water levels is not predicted and “damming” effects are not expected to occur. In addition, failure of the embankment during a flood event is not predicted to result in a dam failure type event as the water level on both sides of the embankment are predicted to be similar throughout the event.

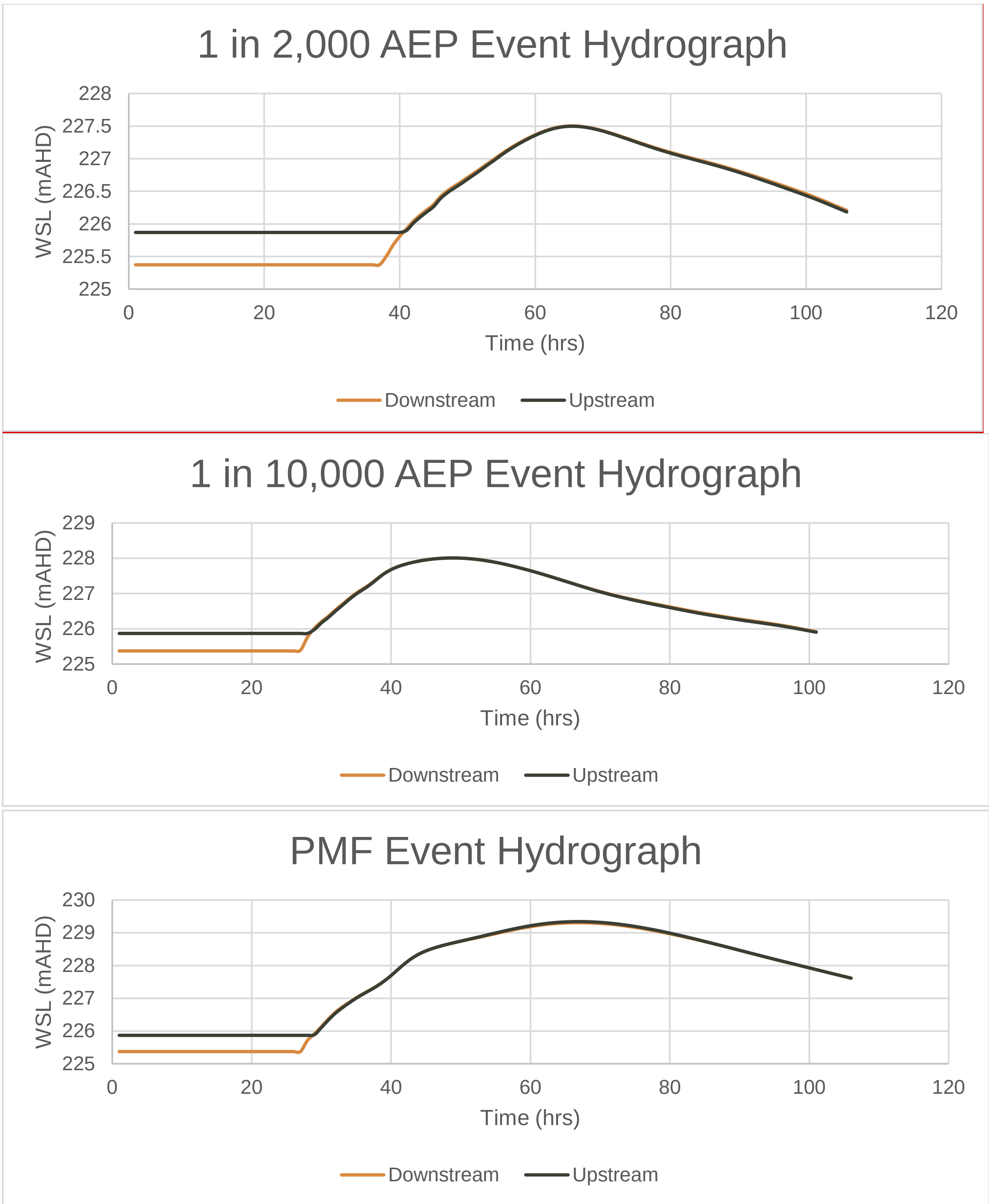



**Figure 9.1 Upstream and downstream water levels between Ch 28.00 km to Ch 28.50 km**

### 9.2.2 Structures results

Table 9.5 presents hydraulic model results at each structure for the 1% AEP event. Velocity has been extracted as the peak velocity from the flood grid.

**Table 9.5 Macintyre River floodplain – 1% AEP event major structure results**

| Chainage (km) | Waterway | Structure type | Upstream peak water level (m AHD) | Freeboard to Top of Rail (m) | Outlet velocity (m/s) | Peak discharge ($m^3/s$) |
|---|---|---|---|---|---|---|
| 5.76 | Mobbindry Creek | Bridge | 242.2 | 2.60 | 2.1 | 291 |
| 6.23 | | Bridge | 241.9 | 3.56 | 1.4 | 291 |
| 8.11 | Back Creek | Bridge | 238.3 | 2.00 | 1.5 | 145 |
| 15.33 | Forest Creek | RCBC | 227.3 | 1.63 | 1.3 | 4 |
| 15.52 | | RCBC | 227.1 | 1.86 | 2.1 | 9 |
| 15.67 | | RCP | 226.9 | 1.99 | 1.8 | 13 |
| 15.83 | | RCP | 226.8 | 2.11 | 2.1 | 28 |
| 15.90 | | RCP | 226.8 | 2.13 | 2.0 | 26 |
| 15.98 | | RCP | 226.8 | 2.18 | 1.9 | 22 |
| 16.08 | | RCP | 226.7 | 2.28 | 1.9 | 21 |
| 16.29 | | Bridge | 226.7 | 2.29 | 0.9 | 55 |
| 16.49 | | RCBC | 226.5 | 2.50 | 2.0 | 3 |
| 16.60 | | RCP | 226.4 | 2.52 | 2.0 | 13 |
| 16.83 | | RCP | 226.5 | 2.49 | 1.9 | 8 |
| 20.73 | Strayleaves Creek | Bridge | 225.1 | 4.66 | 1.0 | 117 |
| 21.35 | | RCP | 224.9 | 3.19 | 1.6 | 3 |
| 21.97 | | RCP | 224.7 | 2.38 | 1.3 | 2 |
| 22.27 | | RCP | 225.1 | 2.06 | 2.2 | 5 |
| 22.86 | Whalan Creek | RCP | 225.2 | 2.31 | 1.4 | 13 |
| 23.22 | | RCP | 225.2 | 2.38 | 1.3 | 28 |
| 23.70 | | RCP | 225.3 | 2.37 | 1.7 | 24 |
| 23.80 | | RCP | 225.3 | 2.46 | 1.3 | 23 |
| 24.03 | | RCP | 225.4 | 2.50 | 1.1 | 6 |
| 24.20 | | RCP | 224.4 | 3.47 | 1.6 | 4 |
| 24.62 | | RCBC | 225.2 | 2.81 | 0.8 | 23 |
| 24.71 | | RCBC | 225.3 | 2.80 | 1.0 | 29 |
| 24.85 | | RCBC | 225.3 | 3.37 | 1.2 | 35 |
| 25.34 | | Bridge | 225.4 | 5.96 | 0.9 | 107 |
| 25.80 | | Bridge | 225.5 | 5.48 | 0.8 | 47 |
| 26.09 | | Bridge | 225.6 | 4.67 | 0.4 | 47 |
| 27.06 | | RCP | 225.9 | 2.88 | 0.8 | 1 |
| 27.56 | | Bridge | 226.4 | 2.43 | 0.5 | 14 |
| 28.03 | | Bridge | 226.6 | 2.24 | 0.6 | 26 |
| 30.35 | | Bridge | 228.0 | 3.26 | 3.1 | 3,785 |
| 31.26 | | RCP | 227.5 | 5.68 | 1.4 | 24 |

| Chainage (km) | Waterway | Structure type | Upstream peak water level (m AHD) | Freeboard to Top of Rail (m) | Outlet velocity (m/s) | Peak discharge ($m^3/s$) |
|---|---|---|---|---|---|---|
| 31.32 | Macintyre River (NS2B) | RCP | 227.5 | 5.27 | 1.4 | 25 |
| 31.52 | | Bridge | 227.4 | 4.68 | 1.0 | 69 |
| 31.87 | | RCP | 227.1 | 2.61 | 1.1 | 4 |
| 31.97 | | RCP | 227.1 | 2.61 | 1.0 | 3 |
| 32.55 | | Bridge | 227.1 | 2.60 | 1.8 | 274 |

Scour protection requirements for culverts have been calculated based on the velocities predicted from the hydraulic modelling. The scour protection has been designed in accordance with Austroads Guide to Road Design Part 5B: Drainage (AGRD). Scour protection was specified where the culvert outlet velocities for the 1% AEP event exceeded the allowable soil velocities shown in Table 3.1 of AGRD as follows:

- Stable rock – 4.5 m/s
- Stones 150 mm diameter or larger – 3.5 m/s
- Gravel 100 mm or grass cover – 2.5 m/s
- Firm loam or stiff clay – 1.2 to 2 m/s
- Sandy or silty clay – 1.0 to 1.5 m/s.

The scour protection length and minimum rock size (d50) were determined from Figure 3.15 and Figure 3.17 in AGRD. Resulting length of scour protection required were determined through the drainage assessment. All required scour lengths were predicted to fit within the proposed rail footprint.

There was insufficient information available at this stage to provide a meaningful scour assessment at each bridge site. A conservative scour estimation based on the 1 in 2000 AEP event has been undertaken for pier substructure designs at each bridge site based on available information and will be refined during detailed design.

## 9.3 Flood impact objectives

The proposal design outcomes relative to the flood impact objectives are presented in the following sections.

### 9.3.1 Afflux

The afflux for the two modelled scenarios, DPIE levees and 2019 LiDAR levees, are presented in Figure A14-A and A14-B-1 to A14-B-3 respectively. Comparison of these figures shows that the impacts associated with the proposal alignment do not vary greatly between the two cases even with the two different topographic datasets are used. For the remainder of the impact assessment against the flood impact objectives the 2019 LiDAR levee scenario has been adopted as this reflects the current state of development on the floodplain.

From the 2019 LiDAR 1% AEP event the Developed Case has been predicted to result in afflux that generally comply with the flood impact objectives. The flood impact objectives are used as a guideline and therefore impacts will be assessed in consultation with individual's, stakeholders and landholders. This includes any impacts on agricultural land.

Flood sensitive receptors on the floodplain are identified on Figure A10-A and Figure A10-B.

For the 1% AEP event there is no afflux above 10 mm predicted at identified habitable dwellings. At non habitable dwellings there is no afflux above 50 mm in the 1% AEP event predicted. Of these only two are above 10 mm and consist of:

- One shed (ID1) 50 mm afflux with existing depth of 174 mm
- One pump with (ID149) 14 mm afflux with existing depth 5.4 m.

The township of Toomelah is within 3 km of the proposed alignment with the afflux predicted to be approximately 25 mm immediately upstream of the rail and reducing to less than 10 mm at the Toomelah township. To the south of Whalan Creek, the afflux is predicted to be up to 40 mm immediately upstream of the rail dissipating to less than 10 mm within 2.1 km.

Under the 1% AEP event in the floodplain there are a number of localised occurrences where the afflux is predicted to be greater than 0.2 m but less than 0.4 m, which still complies with the flood impact objectives. The afflux is typically a result of localised build up on the upstream side of the formation. Between Ch 20.0 km and Ch 24.0 km the afflux occurs from the constriction of flow between the formation and upstream farm levees. These locations are:

- Ch 5.60 km – with an increase of 230 mm dissipating to less than 200 mm within 30 m of the rail embankment, over an area of 0.002 $km^2$
- Ch 6.0 km – with an increase of 240 mm dissipating to less than 200 mm within 30 m of the rail embankment, over an area of 0.003 $km^2$
- Ch 8.4 km – with an increase of 240 mm dissipating to less than 200 mm within 30 m of the rail embankment, over an area of 0.005 $km^2$
- Ch 15.80 km (upstream of the Forest Creek crossing) – with an increase of 280 mm dissipating to 200 mm within 80 m of the rail embankment, over an area of 0.02 $km^2$
- Ch 20.80 km to 22.30 km – with an increase of 360 mm dissipating to less than 200 mm within 200 m of the rail embankment, over an area of 0.24 $km^2$
- Ch 22.40 km – with an increase of 350 mm dissipating to less than 200 mm within 140 m of the rail embankment, over an area of 0.04 $km^2$
- Ch 25.00 km – with an increase of 280 mm dissipating to less than 200 mm within 180 m of the rail embankment, over an area of 0.06 $km^2$.

At two floodplain locations afflux is predicted above 0.4 m:

- Ch 6.4 km – with an increase of 470 mm dissipating to less than 400 mm within 30 m and less than 200 mm within 100 m of the rail embankment, over an area of 0.02 $km^2$. Afflux is a result of water levels increasing upstream of the formation
- Ch 23.90 km – with an increase of 570 mm dissipating to less than 200 mm within 85 m of the rail embankment, over an area of area 0.025 $km^2$. It is noted that the afflux above 400 mm is located in one model cell only (30 m x 30 m). The afflux at this location is a result of water levels increasing on the upstream side of the formation.

Both of these locations are localised and dissipate to below 200 mm within 100 m of the rail embankment.

For events smaller than the 1% AEP event the changes in peak water levels reduce as the magnitude of the flood reduces and the flow is mostly contained to the creek and river channels. Table C1, Appendix C, presents the afflux at each flood sensitive receptor for all design events.

The afflux on roads across the floodplain has been assessed at a number of road inspection locations shown on Figure A13. The afflux on local roads on the floodplain all comply with the flood impact objectives with less than an increase of 100 mm except at two locations, Bruxner Way (Bruxner Wy 3) and along North Star Road on Mobbindry Creek, to the north (Access Road 3) and south of North Star 1. Under the 1% AEP event, there is a localised increase in peak water levels from 90 mm to 405 mm (+315 mm) over a 50 to 100 m section of the Bruxner Way. Bruxner Way is inundated to the north and south of this location by over 1 m of flood waters with access not feasible by road. In this location there is a culvert bank (3/1.35 m RCPs) under the rail line. At North Star Road there is an increase of approximately up to 300 mm. This location is predicted to be up to 550 mm deep in the existing 1% AEP event and not trafficable. Further refinement of the drainage structures in these locations should be assessed in detailed design stage. Further discussion on the impacts on roads in terms of inundation periods is provided in Section 9.3.2. The peak 1% AEP levels at roads are presented in Table 9.6.

Table 9.6 Afflux for roads

| Road name | Inspection location | Afflux (mm) |
|---|---|---|
| Local Access Roads | Access Rd 1 | 0 |
| | Access Rd 2 | +93 |
| | Access Rd 3 | +302 |
| | Access Rd 4 | +85 |
| | Access Rd 5 | -10 |
| | Access Rd 6 | -18 |
| | Access Rd 7 | -31 |
| | Access Rd 8 | 0 |
| | Access Rd 9 | -1 |
| | Access Rd 10 | +2 |
| | Access Rd 11 | +1 |
| | Access Rd 12 | +2 |
| | Access Rd 13 | +1 |
| | Access Rd 14 | +3 |
| | Access Rd 15 | +1 |
| | Access Rd 16 | +1 |
| | Access Rd 17 | +2 |
| | Access Rd 19 | +1 |
| | Cemetry Rd | 0 |
| | Gunsynd Wy | 0 |
| | Kentucky Ln | 0 |
| | Oakhurst Rd 1 | -6 |
| | Oakhurst Rd 2 | -4 |
| | Oakhurst Rd 3 | +1 |
| | Mungindi Goondiwindi Bdg Rd | 0 |
| | Scotts Rd | 0 |
| | Tucka Tucka Rd 1 | +5 |
| | Tucka Tucka Rd 2 | +15 |
| | Tucka Tucka Rd 3 | +2 |
| Bruxner Way | Bruxner Wy 1 | 0 |
| | Bruxner Wy 2 | +20 |
| | Bruxner Wy 3 | +315 |
| | Bruxner Wy 4 | +45 |
| | Bruxner Wy 5 Developed | +162 |
| | Bruxner Wy 5 Existing | +102 |
| | Bruxner Wy 6 | +64 |
| | Bruxner Wy 7 | +12 |
| | Bruxner Wy 8 | +8 |
| | Bruxner Wy 9 | +6 |
| | Bruxner Wy 10 | +5 |
| | Bruxner Wy 11 | 0 |

| Road name | Inspection location | Afflux (mm) |
|---|---|---|
| North Star | N Star 1 | -25 |
| | N Star 2 | 0 |
| | N Star 3 | -60 |
| | N Star 4 | +1 |
| Newell Highway | Newell Hwy 1 | -1 |
| | Newell Hwy 2 | +3 |
| | Newell Hwy 3 | 0 |
| | Newell Hwy 4 | 0 |
| | Newell Hwy 5 | 0 |

### 9.3.2 Change in duration of inundation

The change to duration of inundation across the hydraulic sub-model is presented in Figure A15-C1-1 and A15-C1-2. On the Macintyre River floodplain Changes to time of duration of inundation are typically within 15 minutes. Localised increases up to 10 hours predicted on the upstream side of the proposal alignment in a 1% AEP event. Downstream of the proposal alignment decreases of up to 2 hours are predicted in the 1% AEP event. Downstream of the alignment in Forest Creek an increase of up to 10 hours is predicted where the flow through the structures is reduced compared to existing and results in a reduction in velocity downstream and the increased inundation time. In the 1% AEP existing flood event the time of duration is predicted to be 50 to 100 hours. Therefore, these changes are considered minor. There is minimal change predicted for the southern tributary crossings.

The time of submergence for the Existing Case and the change in duration of inundation due to the proposal, at the road inspection locations are presented in Table 9.7 for the 1% AEP event. The model predicts that increases in duration of inundation are minimal with most locations predicting a change less than 1 per cent. There is a large localised change predicted at Bruxner Way 3. This is the same location discussed in Section 9.3.1. With the proposal alignment in place, this location is inundated for approximately and additional 33 hours under the 1% AEP event. This localised impact affects only 50 to 100 m of the Bruxner Way, with the road to the north and south already inundated. Immediately to the north and south of this location the period of inundation for Bruxner Way is approximately 85 hours and 95 hrs respectively.

The change in duration of inundation for a range of events, up to the 1% AEP event, is detailed in Table C3, Appendix C.

**Table 9.7 Time of Submergence at road inspection locations**

| Inspection location | Existing Case<br>1% AEP ToS (hrs) | 1% AEP ToS Difference (hrs) |
|---|---|---|
| Access Rd 1 | 80.61 | +0.63 |
| Access Rd 2 | 43.74 | -0.20 |
| Access Rd 3 | 82.82 | -9.23 |
| Access Rd 4 | 0 | +10.96 |
| Access Rd 5 | 63.74 | 0 |
| Access Rd 6 | 57.74 | +0.04 |
| Access Rd 7 | 59.99 | +0.09 |
| Access Rd 8 | 47.38 | +0.05 |
| Access Rd 9 | 37.67 | +0.02 |
| Access Rd 10 | 43.59 | +0.17 |
| Access Rd 11 | 49.49 | +0.05 |

| Inspection location | Existing Case 1% AEP ToS (hrs) | 1% AEP ToS Difference (hrs) |
|---|---|---|
| Access Rd 12 | 33.10 | +0.12 |
| Access Rd 13 | 26.72 | +0.08 |
| Access Rd 14 | 31.34 | +0.33 |
| Access Rd 15 | 59.65 | 0 |
| Access Rd 16 | 50.99 | +0.05 |
| Access Rd 17 | 24.73 | +0.12 |
| Access Rd 19 | 62.28 | -0.01 |
| Cemetry Rd | 49.38 | +0.03 |
| Gunsynd Wy | 64.86 | -0.01 |
| Kentucky Ln | 62.99 | -0.01 |
| Oakhurst Rd 1 | 84.31 | -0.01 |
| Oakhurst Rd 2 | 0 | 0 |
| Oakhurst Rd 3 | 56.41 | -0.04 |
| Mungindi Goondiwindi Bdg Rd | 52.86 | -0.01 |
| Scotts Rd | 7.86 | -0.03 |
| Tucka Tucka Rd 1 | 23.29 | +0.29 |
| Tucka Tucka Rd 2 | 67.30 | +0.05 |
| Tucka Tucka Rd 3 | 35.77 | +0.14 |
| Bruxner Wy 1 | 34.94 | 0 |
| Bruxner Wy 2 | 2.98 | +0.78 |
| Bruxner Wy 3 | 40.00 | +33.00 |
| Bruxner Wy 4 | 52.05 | +0.03 |
| Bruxner Wy 5 developed | 57.38 | -3.95 |
| Bruxner Wy 5 existing | 59.59 | +2.92 |
| Bruxner Wy 6 | 56.02 | -3.24 |
| Bruxner Wy 7 | 40.55 | +0.61 |
| Bruxner Wy 8 | 64.17 | 0 |
| Bruxner Wy 9 | 54.26 | -0.01 |
| Bruxner Wy 10 | 55.15 | 0 |
| Bruxner Wy 11 | 13.68 | +0.12 |
| N Star 1 | 45.37 | +2.74 |
| N Star 2 | 41.20 | +0.03 |
| N Star 3 | 32.60 | +3.05 |
| N Star 4 | 54.09 | -0.15 |
| Newell Hwy 1 | 45.67 | +0.06 |
| Newell Hwy 2 | 37.99 | +0.10 |
| Newell Hwy 3 | 45.24 | +0.09 |
| Newell Hwy 4 | 50.50 | +0.03 |
| Newell Hwy 5 | 59.80 | 0 |

Average Annual Time of Submergence (AAToS) is a measurement of the estimated time per year of submergence of a roadway due to flooding. The AAToS has been determined for each road inspection location for the Existing and Developed Cases and with the outcomes detailed in Table 9.8. The locations that predicted to experience a change in AAToS of greater than 0.5 hrs/yr are on the southern tributaries of Strayleaves Creek (Bruxner Way 2 and Bruxner Way 3), Forest Creek (N Star 3) and Mobbindry Creek (N Star 1). N Star 1 and 3 are both downstream of the project alignment and experience a drop in the 1% AEP peak water levels with an increase in the time of inundation. Bruxner Wy 3 is discussed above in Section 9.3.2.

**Table 9.8 Average Annual Time of Submergence at road inspection locations**

| Location | AAToS Existing Case (hrs/yr) | AAToS Developed Case (hrs/yr) | Difference (hrs/yr) |
|---|---|---|---|
| Access Rd 1 | 65.60 | 65.92 | +0.32 |
| Access Rd 2 | 1.43 | 1.37 | -0.06 |
| Access Rd 3 | 58.24 | 56.96 | -1.28 |
| Access Rd 4 | 0.31 | 0.42 | +0.11 |
| Access Rd 5 | 7.90 | 7.89 | 0 |
| Access Rd 6 | 5.69 | 5.69 | 0 |
| Access Rd 7 | 6.57 | 6.58 | +0.01 |
| Access Rd 8 | 2.36 | 2.37 | +0.01 |
| Access Rd 9 | 1.21 | 1.21 | 0 |
| Access Rd 10 | 1.44 | 1.59 | +0.16 |
| Access Rd 11 | 2.87 | 2.89 | +0.02 |
| Access Rd 12 | 0.96 | 0.97 | +0.01 |
| Access Rd 13 | 0.68 | 0.69 | +0.01 |
| Access Rd 14 | 0.63 | 0.64 | 0 |
| Access Rd 15 | 4.74 | 4.73 | -0.02 |
| Access Rd 16 | 3.29 | 3.29 | 0 |
| Access Rd 17 | 0.54 | 0.54 | 0 |
| Access Rd 19 | 36.21 | 36.14 | -0.06 |
| Cemetry Rd | 6.39 | 6.39 | 0 |
| Gunsynd Wy | 45.95 | 45.95 | 0 |
| Kentucky Ln | 37.36 | 37.35 | -0.01 |
| Oakhurst Rd 1 | 60.52 | 60.45 | -0.08 |
| Oakhurst Rd 2 | 0.19 | 0.23 | +0.04 |
| Oakhurst Rd 3 | 36.51 | 36.55 | +0.05 |
| Mungindi Goondiwindi Bdg Rd | 20.12 | 20.10 | -0.02 |
| Scotts Rd | 0.34 | 0.34 | 0 |
| Tucka Tucka Rd 1 | 0.56 | 0.56 | 0 |
| Tucka Tucka Rd 2 | 21.16 | 21.39 | +0.23 |
| Tucka Tucka Rd 3 | 1.05 | 1.05 | 0 |
| Bruxner Wy 1 | 44.26 | 44.26 | 0 |
| Bruxner Wy 2 | 0.26 | 1.38 | +1.12 |
| Bruxner Wy 3 | 3.23 | 6.26 | +3.03 |
| Bruxner Wy 4 | 1.72 | 1.72 | 0 |
| Bruxner Wy 5 Developed | 4.98 | 3.29 | -1.69 |

| Location | AAToS Existing Case (hrs/yr) | AAToS Developed Case (hrs/yr) | Difference (hrs/yr) |
|---|---|---|---|
| Bruxner Wy 5 Existing | 9.14 | 7.72 | -1.43 |
| Bruxner Wy 6 | 4.72 | 2.96 | -1.76 |
| Bruxner Wy 7 | 1.61 | 1.64 | +0.03 |
| Bruxner Wy 8 | 7.41 | 7.36 | -0.05 |
| Bruxner Wy 9 | 3.65 | 3.64 | 0 |
| Bruxner Wy 10 | 4.62 | 4.62 | 0 |
| Bruxner Wy 11 | 0.43 | 0.44 | 0 |
| N Star 1 | 27.44 | 29.55 | +2.11 |
| N Star 2 | 27.42 | 27.43 | +0.01 |
| N Star 3 | 14.84 | 20.70 | +5.86 |
| N Star 4 | 19.05 | 19.08 | +0.02 |
| Newell Hwy 1 | 1.60 | 1.60 | 0 |
| Newell Hwy 2 | 1.24 | 1.25 | +0.01 |
| Newell Hwy 3 | 1.45 | 1.46 | 0 |
| Newell Hwy 4 | 3.54 | 3.55 | +0.01 |
| Newell Hwy 5 | 7.22 | 7.22 | 0 |

### 9.3.3 Flood flow distribution

The Macintyre River floodplain is complex with many braided flowpaths and channels. To assess potential changes to the flow distribution due to the inclusion of the proposal, flows have been extracted from the hydraulic sub-model at a number of locations across the floodplain shown in Figure A12, for the Existing and Developed Cases.

The flow is calculated across the length of each line and measures the flow across the width of the floodplain (for the longer flow lines) or the main flowpath of key waterways (generally for smaller flow lines). Table 9.9 presents the comparison of flows for the 1% AEP event and shows that there are minimal changes between the Existing and Developed Cases.

**Table 9.9 1% AEP event – Flow comparison**

| Flow comparison location | Existing Case Flow ($m^3/s$) | Developed Case Flow ($m^3/s$) | Change (%) |
|---|---|---|---|
| Boggabilla 1 | 3214 | 3214 | 0 |
| Boggabilla 2 | 3201 | 3201 | 0 |
| Brigalow Ck | 1107 | 1107 | 0 |
| Bruxner Way | 127 | 122 | -4.1 |
| Dumaresq Rvr 1 | 3742 | 3742 | 0 |
| Dumaresq Rvr 2 | 3203 | 3203 | 0 |
| Forest Ck | 191 | 194 | +1.5 |
| Goondiwindi | 2037 | 2038 | <0.1 |
| Mac River 1 | 2119 | 2119 | 0 |
| Mac River 2 | 2141 | 2141 | 0 |
| Mac River 3 | 5362 | 5362 | 0 |
| Mac River 4 | 5351 | 5351 | 0 |
| Mac River 5 | 2911 | 2911 | 0 |

| Flow comparison location | Existing Case Flow (m³/s) | Developed Case Flow (m³/s) | Change (%) |
|---|---|---|---|
| Mac River 6 | 3190 | 3190 | 0 |
| Mac River 7 | 4253 | 4258 | +0.1 |
| Mac River 8 | 3245 | 3245 | 0 |
| Mac River 9 | 3188 | 3188 | 0 |
| Mobbindry Ck | 285 | 291 | +2.0 |
| Morella 1 | 290 | 291 | +0.3 |
| Morella 2 | 1023 | 1028 | +0.4 |
| Morella 3 | 417 | 419 | +0.5 |
| Newell Hwy | 529 | 531 | +0.3 |
| Ottleys Ck | 53 | 53 | 0 |
| Rainbow Lgn | 740 | 742 | +0.3 |
| Telephone Lgn | 120 | 120 | 0 |
| Turkey Lgn | 353 | 354 | +0.2 |
| Whalan Ck 1 | 1042 | 1036 | -0.6 |
| Whalan Ck 2 | 989 | 989 | 0 |
| Whalan Ck 3 | 1353 | 1346 | -0.5 |
| Whalan Ck 4 | 331 | 330 | -0.3 |

The modelling generally predicts minimal change between Existing and Developed Case peak flows. Table 9.9 shows one location with a greater than 2 per cent change in flows in the 1% AEP event, Bruxner Way (with minus 4 per cent). There are also minimal changes in Existing and Developed Case flows for the 2%, 5%, 10% and 20% AEP events, as shown in Table C3, Appendix C.

### 9.3.4 Change in velocities

Figure A15-B-1 and Figure A15-B-2 present the changes in peak velocities associated with the proposal design for the 1% AEP event. In general, the changes are minor (less than 0.1 m/s), with most changes in velocities experienced immediately adjacent to the proposal alignment.

The flood modelling has shown that the proposal design results in minimal changes to peak water levels, velocities and flood flow distribution across the floodplain and in each of the waterways. This means that the proposal design minimises potential changes to the geomorphological conditions in the waterways and as such the risk of change to geomorphological conditions in each of the waterways is low.

Peak water levels, flows and velocities from the hydrology and flooding investigation have been used to inform the scour protection design. The scour protection has been designed in accordance with Austroads Guide to Road Design (AGRD) Part 5B: Drainage. Scour protection was specified where the outlet velocities for the 1% AEP event exceed the allowable soil velocities for the particular soil type for each location, with the soil type identified from published soil mapping.

### 9.3.5 Hazard assessment

#### 9.3.5.1 Road Hazard

Figure A15-E-1 and Figure A15-E-2 present the Developed Case velocity x depth flood hazard. The model predicts the 1% AEP Developed Case velocity x depth to be typically less than 0.3$m^2$/s on the floodplain, 0.3 to 0.6 $m^2$/s across waterway flowpaths (out of bank flow) and greater than 1$m^2$/s in the waterway channels. The afflux and flood hazard (velocity-depth) for the 1% AEP event have been evaluated at key locations on public roads and private access roads as shown in Table 9.10.

Table 9.10 Change in flood hazard (v*d) for roads

| Location ID | Existing Case flood hazard ($m^2/s$) | Developed Case flood hazard ($m^2/s$) | Change in peak flood hazard ($m^2/s$) | Existing peak flood depth (m) | Afflux (mm) |
|---|---|---|---|---|---|
| Access Rd 1 | 0.30 | 0.30 | 0.00 | 0.96 | 0 |
| Access Rd 2 | 0.17 | 0.17 | 0.00 | 0.63 | +93 |
| Access Rd 3 | 0.05 | 0.06 | 0.01 | 0.24 | +302 |
| Access Rd 4 | 0.01 | 0.04 | 0.02 | 0.03 | +85 |
| Access Rd 5 | 0.52 | 0.51 | -0.01 | 0.69 | -10 |
| Access Rd 6 | 0.16 | 0.16 | -0.01 | 0.57 | -18 |
| Access Rd 7 | 0.29 | 0.26 | -0.03 | 0.80 | -31 |
| Access Rd 8 | 0.14 | 0.14 | 0.00 | 0.36 | 0 |
| Access Rd 9 | 0.14 | 0.13 | 0.00 | 0.36 | -1 |
| Access Rd 10 | 0.12 | 0.12 | 0.00 | 0.64 | +2 |
| Access Rd 11 | 0.14 | 0.14 | 0.00 | 0.73 | +1 |
| Access Rd 12 | 0.11 | 0.11 | 0.00 | 0.40 | +2 |
| Access Rd 13 | 0.07 | 0.07 | 0.00 | 0.25 | +1 |
| Access Rd 14 | 0.10 | 0.10 | 0.00 | 0.12 | +3 |
| Access Rd 15 | 0.21 | 0.21 | 0.00 | 1.01 | +1 |
| Access Rd 16 | 0.21 | 0.21 | 0.00 | 0.94 | +1 |
| Access Rd 17 | 0.18 | 0.18 | 0.00 | 0.75 | +2 |
| Access Rd 19 | 0.08 | 0.08 | 0.00 | 0.36 | +1 |
| Cemetry Rd | 0.29 | 0.29 | 0.00 | 0.56 | 0 |
| Gunsynd Wy | 0.58 | 0.58 | 0.00 | 1.12 | 0 |
| Kentucky Ln | 0.22 | 0.22 | 0.00 | 0.49 | 0 |
| Oakhurst Rd 1 | 0.28 | 0.27 | -0.01 | 0.31 | -6 |
| Oakhurst Rd 2 | 0.00 | 0.00 | 0.00 | 0.00 | -4 |
| Oakhurst Rd 3 | 0.17 | 0.17 | 0.00 | 0.39 | +1 |
| Mungindi Goondiwindi Bdg Rd | 0.02 | 0.02 | 0.00 | 0.32 | 0 |
| Scotts Rd | 0.06 | 0.06 | 0.00 | 0.17 | 0 |
| Tucka Tucka Rd 1 | 0.06 | 0.07 | 0.00 | 0.24 | +5 |
| Tucka Tucka Rd 2 | 1.62 | 1.62 | 0.01 | 2.14 | +15 |
| Tucka Tucka Rd 3 | 0.11 | 0.11 | 0.00 | 0.38 | +2 |
| Bruxner Wy 1 | 0.10 | 0.10 | 0.00 | 0.14 | 0 |
| Bruxner Wy 2 | 0.01 | 0.01 | 0.00 | 0.08 | +20 |
| Bruxner Wy 3 | 0.00 | 0.05 | 0.05 | 0.00 | +315 |
| Bruxner Wy 4 | 0.19 | 0.18 | -0.01 | 0.77 | +45 |
| Bruxner Wy 5 developed | 0.29 | 0.31 | 0.02 | 0.91 | +162 |
| Bruxner Wy 5 existing | 0.68 | 0.86 | 0.18 | 1.08 | +102 |
| Bruxner Wy 6 | 0.30 | 0.39 | 0.10 | 0.81 | +64 |
| Bruxner Wy 7 | 0.04 | 0.04 | 0.00 | 0.29 | +12 |
| Bruxner Wy 8 | 0.61 | 0.62 | 0.01 | 1.32 | +8 |
| Bruxner Wy 9 | 0.42 | 0.43 | 0.01 | 1.03 | +6 |
| Bruxner Wy 10 | 0.54 | 0.54 | 0.00 | 1.04 | +5 |

| Location ID | Existing Case flood hazard ($m^2/s$) | Developed Case flood hazard ($m^2/s$) | Change in peak flood hazard ($m^2/s$) | Existing peak flood depth (m) | Afflux (mm) |
|---|---|---|---|---|---|
| Bruxner Wy 11 | 0.02 | 0.02 | 0.00 | 0.07 | 0 |
| N Star 1 | 0.85 | 1.31 | 0.46 | 0.86 | -25 |
| N Star 2 | 0.62 | 0.62 | 0.00 | 0.91 | 0 |
| N Star 3 | 0.32 | 0.24 | -0.08 | 0.83 | -60 |
| N Star 4 | 0.02 | 0.02 | 0.00 | 0.41 | +1 |
| Newell Hwy 1 | 0.15 | 0.15 | 0.00 | 0.46 | -1 |
| Newell Hwy 2 | 0.72 | 0.72 | 0.00 | 0.80 | +3 |
| Newell Hwy 3 | 0.14 | 0.14 | 0.00 | 0.30 | 0 |
| Newell Hwy 4 | 0.40 | 0.40 | 0.00 | 0.37 | 0 |
| Newell Hwy 5 | 0.25 | 0.25 | 0.00 | 0.42 | 0 |

The model predicts that changes to flood hazard (velocity x depth) in a 1% AEP event is minor at identified public and private roads. Several locations experience minor increases in peak 1% AEP event peak water levels; however, the trafficability of these roads has not been negatively impacted (i.e. these locations were not predicted to be trafficable in the Existing Case or are not trafficable at locations on the road close by and therefore do not adversely impact on traffic movement during flood events.

### 9.3.5.2 Floodplain hazard

The Australian Disaster Resilience Handbook Flood Hazard Guideline 7-3 (2017) produced by the Australian Institute for Disaster Resilience (AIDR) provide guidelines for the categorisation of flood hazard as shown in Figure 9.2. Using these guidelines flood hazard mapping has been prepared for the Existing and Developed Cases with the outcomes presented on Figure A8-E and Figure A15-D-1 to Figure A15-D-2 respectively.

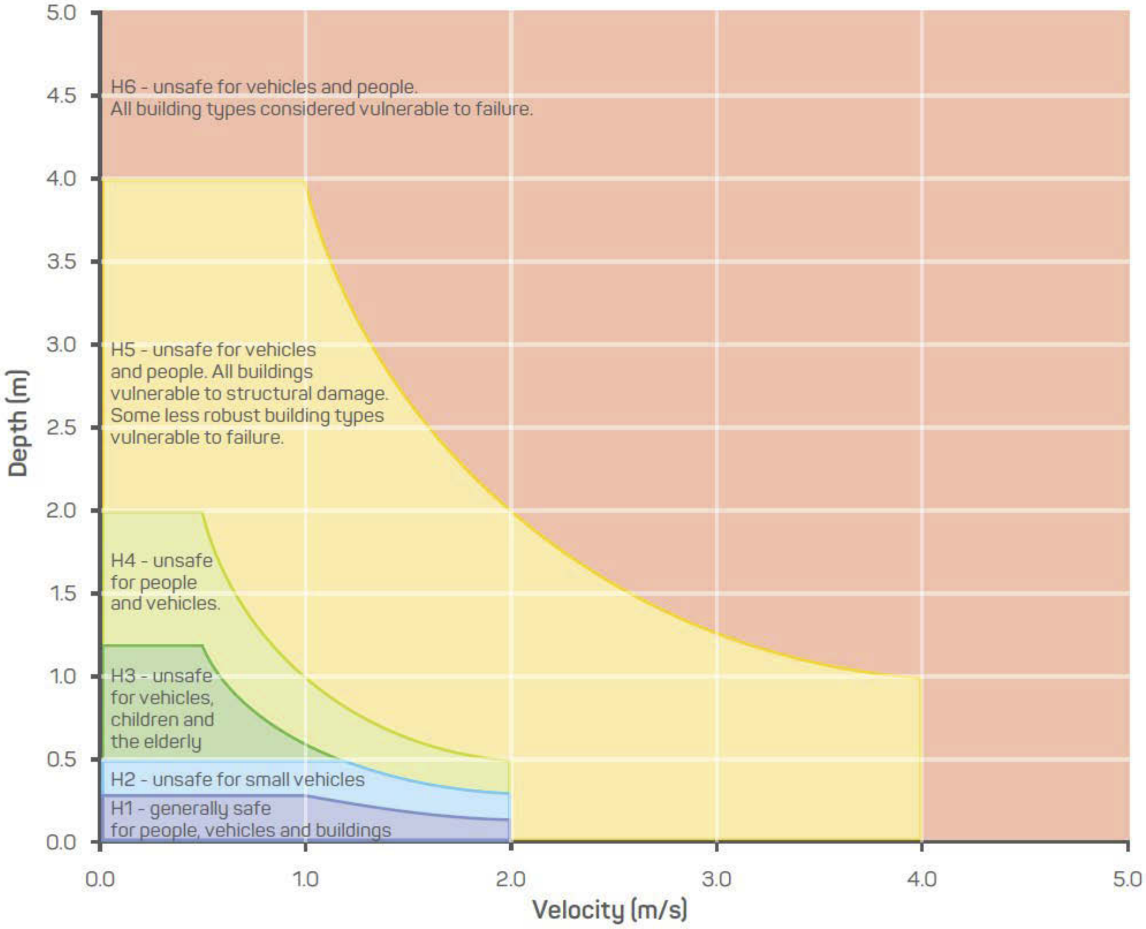



**Figure 9.2** **Flood hazard classification, Australian Disaster Resilience Handbook – Guideline 7-3 (AIDR 2017)**

As can be seen from the results the lower hazard classifications (H1 to H3) generally apply across the majority of the floodplain area with the higher (H5) classifications occurring in the creek and river channels were the flow is higher. The highest classification (H6) applies along the deeper waterways, in particular on the Macintyre River and Whalan Creek, due to higher flood depths and velocities than on the floodplain areas.

The model predicts that the developed case typically does not impact on the hazard classifications across the floodplain. This is due to the fact that the changes to the peak water levels, flood flow distribution and velocities are all minimised. Across the floodplain, hazard categories generally remain the same as existing with the development in place, for the 1% AEP event (H1 to H3). Some impact is experienced along the proposal alignment. Upstream of the formation where afflux is predicted in the 1% AEP event, there are localised areas that shift into the next higher hazard category (H2 to H3).

### 9.3.6 Extreme event risk management

Several design events larger than the 1% AEP event, including the 1 in 2,000 AEP, 1 in 10,000 AEP and PMF, have been modelled to assess the performance of the proposal alignment and to review impacts on the flooding regime. Figures A16-E, A16-F and A16-G present the afflux for the 1 in 2,000 AEP, 10,000 AEP and PMF events respectively.

Table 9.11 outlines the changes in peak water levels at flood sensitive receptors for the assessed extreme events where the change in peak water levels exceeds 50 mm under one of the modelled events.

The Existing Case flood depth is also presented for each event in Table 9.11. As can be seen the Existing Case flood depth at many locations is already high and the incremental increase in peak water levels associated with the proposal design is unlikely to have a detrimental impact.

There are limited locations where the change in peak water levels is elevated under the extreme events (i.e. FSRs 10, 12, 23 and 44) and a detailed review of modelling results was undertaken in these locations. This review determined the following:

- FSR 12 (House) is protected by local levee around the house. Existing Case peak water levels for the 2,000 AEP event and larger events overtop the local levee with depths of 1.4m and deeper. The additional depth due to the proposal alignment may not make a material difference to the flood impacts under these extreme events.
- FSRs 10 (House), 23 (Shed) and 44 (Shed) are all located between existing floodplain levees located on the eastern side of Bruxner Way and the proposal alignment located in the western side of Bruxner Way. Modelling of the extreme events identifies that overtopping of the floodplain levees occurs under these large events and this leads to significant flood water behind the proposal embankment, approximately between Chainages 20 km and 24 km, which impacts these FSRs.

During detailed design these outcomes will be discussed in detail with landholders and a range of alternative mitigation measures will be further investigated including refined drainage structures, property solutions, scour and embankment protection, etc. Formal third party agreements will be negotiated with landholders that takes account of these impacts and the adopted mitigation measures.

**Table 9.11 Extreme event impacts at flood sensitive receptors**

| Flood sensitive receptor number | Description | 1 in 2,000 AEP event | | 1in 10,000 AEP event | | PMF event | |
|---|---|---|---|---|---|---|---|
| | | Change in peak water level (mm) | Existing case flood depth (m) | Change in peak water level (mm) | Existing case flood depth (m) | Change in peak water level (mm) | Existing case flood depth (m) |
| 1 | Sheds | +210 | 1.13 | +230 | 1.46 | +330 | 2.60 |
| 2 | House | +10 | 1.84 | +40 | 2.10 | +60 | 3.06 |
| 3 | House | +20 | 1.08 | +40 | 1.35 | +70 | 2.33 |
| 6 | Sheds | +10 | 1.01 | +40 | 1.27 | +60 | 2.24 |
| 7 | Sheds | +10 | 1.34 | +40 | 1.61 | +60 | 2.59 |
| 10 | House | +1,820 | 0.14 | +1,900 | 0.71 | +1,250 | 2.02 |
| 11 | House | +10 | 0.89 | +30 | 1.81 | +60 | 0.65 |
| 12 | House | +440 | 1.41 | +640 | 1.67 | +730 | 2.62 |
| 23 | Sheds | +1,350 | 0.05 | +1,440 | 0.66 | +1,040 | 2.10 |
| 24 | Sheds | 0 | 0 | +60 | 1.01 | +40 | 2.14 |
| 27 | Toomelah Community | +10 | 1.06 | +50 | 1.40 | +70 | 2.65 |
| 33 | House | +10 | 0.90 | +30 | 1.14 | +50 | 2.04 |
| 37 | Sheds | +10 | 0.07 | +20 | 0.15 | +50 | 1.05 |
| 38 | House | +10 | 0.83 | +30 | 1.07 | +60 | 2.00 |
| 39 | House | +80 | 0.84 | +230 | 1.18 | +210 | 2.32 |
| 40 | House | +10 | 0.47 | +20 | 0.69 | +50 | 1.58 |
| 42 | Sheds | +10 | 1.41 | +30 | 1.66 | +60 | 2.61 |
| 43 | Shed | +50 | 1.38 | +150 | 1.79 | +120 | 3.06 |
| 44 | Shed | +1,530 | 0.30 | +1,730 | 0.77 | +1,270 | 2.08 |
| 149 | Pump | +40 | 6.31 | +70 | 6.65 | +130 | 7.76 |
| 150 | Pump | 0 | 2.63 | +20 | 2.88 | +50 | 3.84 |

### 9.3.7 Climate change assessment

The climate change guidelines set out in ARR 2016 have been followed and used to assess the potential impact of increased rainfall upon peak water levels in the proposal area.

The impacts of climate change were assessed for the 1% AEP design event to determine the sensitivity of the proposed alignment design to the potential increase in rainfall intensity. Two scenarios were considered:

1. Impact of the proposal with climate change conditions
2. Impact of climate change conditions on the floodplain.

The Representative Concentration Pathway (RCP) 8.5 climate change scenario has been adopted for the proposal. The climate change analysis was undertaken by increasing rainfall intensities in the IFDs for the contributing catchments. For the proposal, representative concentration pathway 8.5 corresponds to an increase in temperature of 3.7 degrees Celsius in 2090 and an increase in rainfall intensity of 23 per cent which was obtained from the ARR 2016 Data Hub.

The predicted flow resulting from a 23 per cent increase in rainfall is 3,500 $m^3/s$ in the Macintyre River at Boggabilla (where the flows are controlled by the topography) for the 1% AEP event (compared to 3,215 $m^3/s$ in existing climate conditions). In the upper sections of the hydraulic model in the Dumeresq and Macintyre Rivers, the flows are predicted to increase by approximately 25 per cent as a result of the increase in rainfall in the 1% AEP event.

#### 9.3.7.1 Impact of the proposal with climate change conditions

The impact of the proposal with 1% AEP representative concentration pathway 8.5 climate change scenario is presented in Figure A20-1 and Figure A20-2. The afflux is calculated from the difference between the Existing Case and the Developed Case with 23 per cent increase to rainfall intensity applied to both cases.

The afflux associated with the proposal design under Climate Change representative concentration pathway 8.5 conditions is predicted to be up to 200 mm, in localised areas within the vicinity of the alignment. This is similar to the impact of the proposal design under existing climate condition cases. The proposal alignment is not predicted to be overtopped as a result of the 23 per cent increase in rainfall intensity with peak water levels predicted to remain below formation level.

Table 9.12 presents the proposal design performance with representative concentration pathway 8.5 climate change conditions. With 2090 horizon climate change allowance included in the 1% AEP there is an increase peak water levels in both the Existing and Developed Cases which leads to higher changes in peak water levels at two of the identified flood sensitive receptors, as shown in Table 9.12.

**Table 9.12 1% AEP event with RCP 8.5 conditions – Afflux at flood sensitive receptors**

| Location | Description | 1% AEP Event afflux (mm) |
|---|---|---|
| 1 | Sheds | +120 |
| 10 | House | +390 |

#### 9.3.7.2 Impact of climate change conditions on the floodplain

The afflux for the 1% AEP representative concentration pathway 8.5 climate change scenario is presented in Figure A21-1 to Figure A21-2. The afflux is calculated from the difference between the Developed Case with 23 per cent increase to rainfall intensity minus the Developed Case with existing climate conditions. The model predicts that with an increase in rainfall intensity of 23 per cent across the catchment that peak water levels increase by up to 0.4 m in the vicinity of the proposal alignment.

### 9.3.8 Blockage

The hydraulic design has included an assessment regarding the blockage of culverts. Blockage potential has been assessed in accordance with the guidelines in ARR 2016. The blockage assessment resulted in no blockage factor being applied to bridges and a blockage factor of 25 per cent being applied to culverts. A minimum culvert size of 900 mm diameter was also adopted to reduce potential for blockage and for ease of maintenance.

ARR 2016 guidelines are focused on blockage of small bridges and culverts. The floodplain bridges proposed for the proposal alignment are all multi-span large bridges and ARR 2016 notes that there are limited instances of multiple span bridges being observed with blockages similar to those seen at single span bridges or culverts.

A community concern is the potential impacts on flood conditions should the proposed culverts become blocked with debris. The primary concern is that the blockage of culverts is likely to drive flood levels higher, particularly upstream of the culverts, and divert more flow through residences, across access roads and other infrastructure. A sensitivity analysis was undertaken with 0 per cent and 50 per cent blockage. The results are presented in Figure A17-1 to Figure A18-2 for the 0 per cent and 50 per cent blockage respectively.

The model predicts that in both the 0 per cent and 50 per cent blocked cases the predicted changes in peak water levels meet the design criteria for the 1% AEP event. Varying the level of blockage did not significantly change the impact on flood sensitive receptors and the flood impact objectives are still met.

During detailed design the blockage factors will be reviewed in line with the final design and local catchment conditions. This may result in a varied and/or lower blockage factors being applied along the proposal alignment. It may also take into account risk assessments associated with blockage, and/or risk mitigation where required.

## 9.4 Sensitivity analysis

### 9.4.1 Manning's n

The 1% AEP event was simulated to test the sensitivity of the model to change in Manning n (roughness). A general decrease of roughness (i.e. sparser vegetation) of 20 per cent was tested. The resulting roughness values are presented in Table 9.13.

**Table 9.13 Manning's n roughness sensitivity**

| Land use type | Value – Design | Manning's n – 20% decrease |
|---|---|---|
| Waterway | 0.03 | 0.02 |
| Floodplain | 0.06 | 0.05 |
| Vegetated floodplain | 0.12 | 0.10 |

Figure A19-1 to Figure A19-2 present the afflux for a 20 per cent decrease in roughness. Reducing the roughness by 20 per cent in a 1% AEP event results in a reduction of peak flood levels across the floodplain of 100 to 300 mm. Reducing the roughness values did not significantly change the impact on flood sensitive receptors and the flood impact objectives are still met.

### 9.4.2 Reduction in grid size – 15 m grid model

The 1% AEP event was simulated to test the sensitivity of the hydraulic sub-model to a reduction in grid size from 30 m to 15 m. The hydraulic sub-model predicted that peak flood levels were lowered as are result of the grid change by approximately 50 mm across the model area and by 150 mm in the proposal corridor.

Using the 15 m grid, Figure A24 presents the afflux for the Developed Case as compared to the 15 m grid Existing Case. The predicted afflux is similar to that estimated for the 30 m grid and therefore still meets the flood impact objectives. The changes in peak water levels at flood sensitive receptors are presented in Table 9.14. It is noted that with the 15 m grid, the afflux at the Toomelah Community is predicted to be 12 mm (as compared to 10 mm with the 30 m grid).

With the reduced grid size, the flow distribution between the Existing Case and Developed Case for the 1% AEP event is still similar as shown in Table 9.14.

**Table 9.14 1% AEP Event – Flow comparison (15m grid)**

| Location | Existing Case 1% AEP Flow ($m^3/s$) | Design Case 1% AEP Flow ($m^3/s$) | Change (%) |
|---|---|---|---|
| Boggabilla 1 | 3392 | 3393 | 0.02 |
| Boggabilla 2 | 3376 | 3377 | 0.02 |
| Brigalow Ck | 1005 | 1006 | 0.07 |
| Bruxner Hwy | 119 | 115 | -3.75 |
| Dumaresq Rvr 1 | 3691 | 3691 | 0 |
| Dumaresq Rvr 2 | 3294 | 3294 | -0.01 |
| Forest Ck | 167 | 169 | 1.19 |
| Goondiwindi | 2326 | 2326 | 0.01 |
| Mac River 1 | 2030 | 2030 | 0 |
| Mac River 2 | 2061 | 2061 | 0 |
| Mac River 3 | 3820 | 3816 | -0.10 |
| Mac River 4 | 5347 | 5346 | -0.03 |
| Mac River 5 | 3154 | 3153 | -0.04 |
| Mac River 6 | 3327 | 3327 | -0.01 |
| Mac River 7 | 4357 | 4362 | 0.11 |
| Mac River 8 | 3411 | 3412 | 0.02 |
| Mac River 9 | 3357 | 3358 | 0.02 |
| Mobbindry Ck | 275 | 280 | 1.80 |
| Morella 1 | 278 | 279 | 0.27 |
| Morella 2 | 947 | 951 | 0.44 |
| Morella 3 | 368 | 370 | 0.53 |
| Newell Hwy | 522 | 524 | 0.36 |
| Ottleys Ck | 65 | 65 | 0 |
| Rainbow Lgn | 650 | 656 | 0.88 |
| Telephone Lgn | 77 | 77 | 0.11 |
| Turkey Lgn | 370 | 371 | 0.26 |
| Whalan Ck 1 | 928 | 919 | -0.89 |
| Whalan Ck 2 | 912 | 905 | -0.86 |
| Whalan Ck 3 | 1242 | 1234 | -0.68 |
| Whalan Ck 4 | 313 | 313 | -0.02 |

Reducing the grid size to 15 m did not significantly change the impact on flood sensitive receptors and the flood impact objectives are still met.

### 9.4.3 Velocity sensitivity assessment

In addition to reducing the grid size of the model to 15 m to test the sensitivity of the results to changes in grid size, an assessment of the velocity through structures was undertaken to test the sensitivity of the predicted velocities to the model grid size.

The design of scour protection was undertaken utilising the velocities extracted from the culverts from the 30 m grid model as part of the drainage assessment and was not designed in the TUFLOW hydraulic model. This assessment is provided only as a sensitivity check on velocity with varying hydraulic model grid sizes.

The velocity sensitivity assessment was undertaken to meet a request from DPIE to demonstrate the velocities from the 30 m grid are reliable for designing scour protection. DPIE also required that a floodplain section be tested where there may be potential dispersive soils.

The section of the alignment from Ch 22.03 km to Ch 24.88 km was selected. A cutdown model covering this section was developed with flows extracted from the overall model for the upstream boundary and normal depth boundaries applied downstream. The cutdown model was simulated with a 30 m grid and compared with the overall model results to ensure that the model was replicating the levels sufficiently. The cutdown model was then reduced to a 5 m grid size. It was not feasible to reduce the grid size further for the assessment across the floodplain.

Table 9.15 shows the velocity predicted in the 1% AEP event from the 30 m grid model and the 5 m grid model predicted in the culvert and downstream, for the 5m model for existing roughness and higher roughness to represent scour protection. These values were extracted from the 2d model at the proposal boundary (assessment corridor) for the 5 m grid. The results are presented for the Developed Case with no scour mitigation (n=0.06) and with the roughness increased on the downstream side of the rail corridor to represent scour protection (rock protection). The roughness was increased from n=0.06, the selected floodplain roughness to n=0.10, to represent the roughness for rock lined surface for this scenario. It is noted that this assessment was undertaken during the design process and results are not reflective of the final design in this section of the alignment, including topography that is based on the OEH Levees, and 2015 LiDAR.

**Table 9.15 Comparison of velocities**

| Culvert Chainage (km) | Developed Case velocity in culverts | Developed Case velocity in culverts | Existing Case velocity at d/s proposal boundary | Developed Case velocity at d/s proposal boundary | Developed Case velocity at d/s proposal boundary |
|---|---|---|---|---|---|
| | (30 m, n=0.06) | (5 m, n = 0.06) | (5 m, n = 0.06) | (5 m, n = 0.06) | (5 m n = 0.1) |
| | (m/s) | (m/s) | (m/s) | (m/s) | (m/s) |
| 21.97 | 1.76 | 0.2 | 0.1 | 0.2 | 0.2 |
| 22.27 | 2.24 | 0.4 | 0.1 | 0.1 | 0.1 |
| 22.86 | 1.45 | 0.7 | 0.3 | 0.1 | 0.1 |
| 23.22 | 1.89 | 1.3 | 0.3 | 0.5 | 0.4 |
| 23.70 | 1.64 | 0.6 | 0.3 | 0.3 | 0.3 |
| 23.80 | 1.64 | 0.6 | 0.2 | 0.2 | 0.2 |
| 24.03 | 0.54 | 0.1 | 0.3 | 0.1 | 0.1 |
| 24.20 | 1.72 | 1.5 | 0.2 | 0.3 | 0.3 |
| 24.62 | 1.25 | 0.9 | 0.3 | 0.3 | 0.3 |
| 24.71 | 1.22 | 0.9 | 0.3 | 0.3 | 0.3 |
| 24.85 | 1.23 | 0.8 | 0.4 | 0.4 | 0.4 |

The model predicts that a reduction in the grid size to 5 m results in lower predicted velocities compared with those extracted from the 30 m grid model. This is a result of the number of cells in the model (36 in the 5m grid, compared to one cell for the 30m grid), allowing more defined flow paths, and finer scale calculations and movement of flow from one cell (or six cells) to the next. In addition, the velocities predicted from the 5 m grid model at the boundary are generally less than 0.5 m/s in both the existing no scour protection and with scour protection, and less than 0.4 m/s with the increased roughness, which is the allowable velocity for bare soil as per the maximum permissible velocities from the Border Rivers FPMP. (Table 1.1 BRVFMP 2018). It is also noted that the actual scour protection design was based on velocities extracted from the 30 m grid model which are consistently higher than the 5 m grid model and are therefore likely to be conservative, these higher values were used to design scour protection to achieve the allowable exit velocity at the project boundary.

The type and length of downstream works has been assessed in the drainage design. 14m is longest length scour protection calculated at the cross drainage structures. The available length from toe of embankment to project boundary at this location is approximately 20 m. The minimum length from toe of embankment to project boundary for the NS2B alignment is approximately 15 m. The assessment has demonstrated that there is adequate available width for scour protection design based on the feasibility design. It should be noted that given grid spacing of the current hydraulic modelling further refinement taking into account site specific soil data for the culvert locations will need to be undertaken during detailed design. This will include site specific geotechnical investigations as required to provide soil information. Using the updated data, the scour protection design will be reassessed during detailed design. Preliminary assessment at a finer grid resolution (5m) predicted that culvert velocities are reduced with more detailed modelling. Therefore, the available area from the toe of formation to the project boundary is expected to remain sufficient for reducing velocity at the project boundary, following reassessment during detailed design and incorporation of site specific soil data. For detailed design each structure location will be documented with length of scour protection required and available with within the project boundary

### 9.4.4 DPIE Existing Case – existing rail line removed completely

As a requirement of the Border Rivers Catchment Management Plan (the Plan), a sensitivity scenario was run with the Existing Case modified with the existing non-operational rail (Camurra-Boggabilla Railway) removed completely. This was carried out to assess the cumulative impact of the rail infrastructure over time on the floodplain. The two cases were assessed as:

- Existing case – no existing rail – non-operational rail removed
- Developed case – proposal design plus non-operational rail from north of Whalan Creek to Boggabilla.

The changes in peak water levels at sensitive receptors are presented in Table 9.16. The peak water levels for the 1% AEP DPIE Existing Case are typically 20 to 50 mm lower upstream and 20 to 50 mm higher downstream of the removed non-operational existing rail when compared to the 1% AEP Existing Case. The afflux for the 1% AEP event for the Developed Case compared to the DPIE Existing Case can be seen in Figure A22. Changes in peak water levels within the corridor typically range from 20 mm to 250 mm.

**Table 9.16 1% AEP Event – Afflux at flood sensitive receptors with DPIE Existing Case**

| Location | Description | 1% AEP Event Afflux (mm) |
|---|---|---|
| 15 | Home | +11 |
| 16 | Home | +13 |
| 97 | House | +26 |
| 105 | House | +31 |
| 144 | House | +14 |
| 154 | House | +15 |
| 190 | House | +12 |
| 207 | House | +14 |

| Location | Description | 1% AEP Event Afflux (mm) |
|---|---|---|
| 228 | House | +23 |
| 255 | House | +12 |
| 256 | House | +12 |
| 257 | House | +12 |
| 258 | House | +12 |
| 263 | House | +22 |
| 264 | House | +20 |
| 265 | House | +22 |
| 269 | House | +10 |

### 9.4.5 Removal of section of existing rail line

A sensitivity test with the removal of the existing non-operational Camurra-Boggabilla rail embankment where the proposed alignment is not within the existing rail corridor was undertaken. For this assessment the existing rail was removed from the topography in the hydraulic model from north of Whalan Creek to Boggabilla in the Developed Case. The section of the existing rail that was removed in this case was approximately 500 mm high, 25 m wide and 3 km long. Figure A23 shows the section removed and the resulting changes in peak flood levels. The hydraulic model predicts that removing the existing non-operational rail line results in increases in peak water levels of approximately 35 mm downstream of the rail line and decreases in peak water levels by approximately 100 mm upstream of the alignment in the vicinity of the removed rail. With removal of the existing rail line, three houses are impacted above 10 mm. The changes in peak water levels at sensitive receptors are presented in Table 9.17.

**Table 9.17 1% AEP Event – Afflux at flood sensitive receptors with existing section of rail removed**

| Location | Description | 1% AEP Event Afflux (mm) |
|---|---|---|
| 22 | Homes | +48 |
| 160 | House | +12 |
| 215 | House | +11 |

### 9.4.6 Adjustment of peak flows in Macintyre River tributaries

A sensitivity assessment of peak flows in the tributaries was carried out to determine the impact of changes to dam characteristics on the tributaries. It is noted that the hydrologic models already assume dam full for all scenarios, which is conservative. To provide a further conservative assessment the dams were removed from the hydrologic models and simulated to provide revised peak flows. The impact on 1% AEP flows was predicted to be small (<6% increase to 1% AEP peak flows) on the Macintyre River and Macintyre Brook and larger (approximately a 26% to 1% AEP peak flows) on the Dumaresq River from removing Glenlyon Dam. The hydraulic model was simulated with the increased flows from the “No dams” scenario to test the performance of the proposal design. Figure A25 shows the predicted 1% AEP Afflux. With the dams removed the design formation is not predicted to be overtopped in a 1% AEP event.

Increases in impacts are predicted upstream of the formation in the 1% AEP event as a result of the increased flows. This impact varies across the floodplain. From Ch 0.0 to Ch 21.0 there is no change to predicted afflux as a result of removing the dams. From Ch 21.0 to Ch 25.5 km an additional + 30 mm to + 50 mm is predicted. From Ch 25.5 km north less than +10 mm is predicted as a result of removing the dams. Adjusting the peak flows in tributaries, did not significantly change the impact on flood sensitive receptors.

### 9.4.7 DPIE levee assessment

The levee development on the Macintyre River floodplain that is incorporated in the hydraulic model was developed from the 2019 LiDAR levels. This data has been used to assess the impact of the proposal design.

The provided DPIE model represented levees as height limited and height unlimited layers to provide a representation of constructed and approved levees on the floodplain area. Whilst this provided a representation of the current levees on the floodplain there were some inconsistencies between the approved levee development heights and the levee heights captured by the 2019 LiDAR data. Through discussions with DPIE it was agreed that the levees based on the 2019 LiDAR would be used to assess the proposal. The DPIE levee case has been used as a sensitivity test of the proposal.

To test the performance of the proposal with the DPIE levees, the model was simulated with the digitised levees with locations and heights extracted from the LiDAR data removed and replaced with the DPIE levee heights. Figure A14-A presents the predicted afflux in the 1% AEP event based on the DPIE levees. Comparison of DPIE Levees against LiDAR Levees (Figure A14-B) shows that the impacts associated with the proposal alignment do not vary greatly between the two cases.

### 9.4.8 1976 Flow

To assess the impact on the floodplain from the proposal if 1976 flood flows were to occur under current topographic conditions (2019 LiDAR), the 1976 flow was simulated with the Existing Case and Developed Case models. The 1976 flows have been modelled to test the performance of the proposal design under varying scenarios (in the same way as the 1 in 2,000 AEP and other larger events have been considered). The results of the 1976 flows are presented in Table 9.18. The afflux is presented in Figure A-26.

During detailed design these outcomes will be discussed in detail with landholders and a range of alternative mitigation measures will be further investigated including refined drainage structures, property specific solutions, scour and embankment protection, etc. Formal third party agreements will be negotiated with landholders that takes account of these impacts and the adopted mitigation measures.

**Table 9.18 1976 flow Event – Afflux at flood sensitive receptors – 1976 event flows**

| Flood sensitive receptor number | Description | Change in peak water level (mm) | Existing case flood depth (m) |
|---|---|---|---|
| 1 | Sheds | +140 | 0.63 |
| 8 | House | +30 | 0.62 |
| 9 | Sheds | +30 | 0.87 |
| 10 | House | +870 | 0 |
| 12 | House | +320 | 1.05 |
| 27 | Toomelah Community | +10 | 0.60 |
| 41 | Airport | +20 | 0.28 |
| 44 | Shed | +620 | 0 |
| 59 | House | +10 | 0.36 |
| 60 | Shed | +10 | 0.73 |
| 67 | House | +10 | 0.96 |
| 68 | House | +10 | 0.22 |
| 69 | House | +10 | 1.05 |
| 70 | House | +10 | 0.99 |
| 71 | House | +10 | 0.69 |
| 73 | House | +10 | 1.76 |
| 74 | Shed | +10 | 0.88 |

| Flood sensitive receptor number | Description | Change in peak water level (mm) | Existing case flood depth (m) |
|---|---|---|---|
| 75 | Shed | +10 | 1.84 |
| 87 | House | +10 | 0.76 |
| 90 | Shed | +10 | 1.38 |
| 103 | House | +10 | 0.61 |
| 104 | Shed | +10 | 0.57 |
| 149 | Pump | +20 | 5.86 |

When compared to the design flood levels the existing case with 1976 flows event approximates between a 1% AEP and a 1 in 2000 AEP flood event. The 1976 flows produce levels approximately 500 mm higher in the existing case than the 1% AEP event in the vicinity of the proposal.

With 1976 flow the model predicts a larger volume of floodwater is conveyed from east to west to the south of Whalan Creek in the 1% AEP design event. This results in increased afflux in this section that is already constrained by farm levees. This is evident at sensitive receptors 10 and 12 where +870 and +320 mm of afflux is predicted respectively in the 1976 flow event with current conditions.

## 9.5 Construction phase – Camp and laydown facilities flood assessment

For the construction phases of the proposal, one construction camp including laydown facilities has been identified as required for the North Star to Border alignment. This proposed location of the camp is adjacent to the North Star township within the North Star sporting club grounds. The proposed assessment area for the camp is within the existing case 1% AEP flood event of Mobbindry Creek as shown in Figure A27.

The existing 1% AEP flood depths in this area are approximately 0.6 m and velocities range from 1.5 m/s in the creek channel to 0.5 m/s in overbank areas.

To assess the potential impact of the temporary construction of the camp facilities within the floodplain, the camp has been assessed by filling an area within the floodplain in the hydraulic model to represent the space required by the camp (or bunding the perimeter). It is noted that this is considered the worst-case scenario where the floodplain is completely blocked by the temporary works and is not expected to occur. The actual location and layout of the camp will be determined in detailed design. This assessment was carried out to demonstrate that a solution is attainable within the floodplain at the proposed location. Refinement and full mitigation assessment will occur in future design stages.

An area of approximately 8.2 hectares was represented as fill above the 1% AEP event and positioned adjacent to the North Star township. To prevent increased water levels within the town a small levee was included within the floodplain along the town boundary of approximately 100 mm. A small channel approximately 300 mm deep and one grid cell width was included along the floodplain side of the fill to direct flows to the north to replicate existing flow patterns. The hydraulic model was updated to reflect the design and simulated for the 20% and 1% AEP events. Figure A27 and Figure A28 present the camp area and associated channel and bunding, and resulting afflux predicted from the design for the two AEP events.

Impacts are presented in Table 9.19 and Table 9.20 for the 1% AEP and the 20% AEP respectively. The model predicts that blocking flow through the camp and laydown areas prevents flow to the north-west and reduces the flood extent, providing flood relief to North Star in a 1% AEP event. Blocking the flow through the proposed camp area results in increases in upstream peak water levels of up to 300 mm. Impacts reduce to less than 10 mm downstream of Getta Getta Road in the 1% AEP event. No houses are predicted to experience increased peak water levels. The North Star Sporting Club facilities are predicted to experience increases up to 200 mm in the 1% AEP event, due to its close proximity to the camp.

In the 20% AEP event the hydraulic model predicts that blocking the flow through the proposed camp and laydown areas results in increases in upstream peak water levels of up to 230 mm. At Getta Getta Road impacts are predicted to be less than 10 mm. No houses are predicted to experience increased peak water levels. The North Star Sporting Club facilities are predicted to experience increases up to 80 mm in the 20% AEP event, due to its close proximity to the camp.

**Table 9.19 1% AEP Afflux from proposed camp and laydowns**

| Location | Description | 1% AEP Event Afflux (mm) |
|---|---|---|
| 28 | North Star Sporting Club | +160 |

**Table 9.20 20% AEP Afflux from proposed camp and laydowns**

| Location | Description | 20% AEP Event Afflux (mm) |
|---|---|---|
| 28 | North Star Sporting Club | +80 |

## 9.6 Sustainability

The predicted flood impacts have been assessed against the Lan Credits for Level 1. The following provides a summary of the findings against each of the criteria.

### Design measures to minimise risk

Extreme rainfall events, including the 1 in 2,000 AEP, 1 in 10,000 AEP, and the Probable Maximum Flood events have been considered for the proposal.

The formation level of the proposal alignment is driven by several factors including achieving flood immunity and meeting geometric requirements (e.g. allowing for grade separations). Therefore, the freeboard (the height between the flood level and the crest of the formation) achieved varies along the alignment with the 1% AEP event flood immunity achieved with a freeboard greater than 300 mm across the Macintyre River floodplain. Overtopping of the rail formation by location for the extreme events has been identified and is summarised in Section 9.2.1.

Hydrologic and hydraulic modelling was undertaken to compare the Existing Case, reflecting the existing conditions, and the Developed Case, reflecting the proposal during operation, to inform and assess the potential impacts of the proposal design upon the existing flood regime. The modelling has demonstrated that:

- As a result of the Developed Case, there are no impacts greater than 10 mm predicted on habitable dwellings on the floodplain including at the Toomelah Community
- Increases in peak water levels at identified non-habitable dwellings are predicted to be less than 100 mm
- Tucka Tucka Road and Bruxner Way are non-trafficable in the Existing Case 1% AEP flood event. Both roads are not predicted to be non-trafficable for any longer than currently occurs due to the Developed Case.
- No significant changes to peak flood flow distributions are predicted as a result of the Developed Case
- Under the representative concentration pathway 8.5 climate change scenario 1% AEP event peak flood levels are predicted to increase by 0.4 m with no overtopping of the rail formation
- There is generally little change to the predicted impacts on sensitive receptors as a result of varying the applied culvert blockage allowance between 0 per cent and 50 per cent.

### 0.1 $m^3/s$ increase in maximum discharge to downstream receivers for 1% AEP

For the majority of locations within the floodplain, there is no change to the flow between the Existing Case and the Developed Case. It is noted that this requirement does not account for flow magnitude, which is a limitation of the threshold in large catchments.

### 0.1 m increase in afflux to upstream receivers for 1% AEP

The afflux upstream (to the east of the proposal alignment) for the 1% AEP does not exceed 0.1 m for any flood sensitive receptors (identified as sensitive receivers on the figure).

### Climate change considerations

The flood assessment for the proposal has been undertaken in accordance with Australian Rainfall and Runoff 2016 climate change. The assessment considered peak water levels for sensitive receivers through consideration of the Representative Concentration Pathway 8.5 which was recommended as an adaptation option as a result of the climate change risk assessment. Changes in peak water levels for sensitive receivers under the climate change scenario are presented in Section 9.3.7.

In addition to climate change influence on the 1% AEP, increased frequency or severity of flooding events, the proposal has considered the Development Case for the 1 in 2,000 year AEP and 1 in 10,000 year AEP as well as the Probable Maximum Flood (PMF).

Changes in the afflux and discharge rates for these events are shown in Figure A21-1 to Figure A22-2, for climate change and Figure A16-E to Figure A16-G for extreme events.

## 9.7 Hydrology and flooding – independent peer review

An independent peer review of the hydrology and flood assessment documented in this Report and undertaken by Neil Collins from BMT Global. This review was undertaken in accordance with the EIS Guidelines for Independent Reviewers. Findings from this review are provided in Appendix E.

# 10 Limitations

This assessment is based on the TUFLOW model developed by DPIE for the Border Rivers Floodplain Management Plan. At the time of undertaking this assessment the DPIE model calibration was ongoing. While any further changes to the DPIE model are expected to be minor, a review of the final model should be undertaken when available and the impact of changes to that model on this calibration considered. It is noted that the additional of the 2011 calibration event and the design flow analysis provides some further confidence in the ability of the hydraulic model to replicate flows independent of further refinements to the DPIE model.

FFJV has prepared this report in accordance with the usual diligence and thoroughness of the consulting profession with reference to current standards, procedures and practices.

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

This report was prepared for the exclusive use of the proposal. FFJV accepts no liability or responsibility whatsoever for, any use of, or reliance upon, this report by any third party.

This report was prepared based on information available at the time of writing. The models detailed in this report are based on LiDAR survey taken generally in 2014/15 and 2019 (as detailed in Section 5.4). Therefore, any development or topographical change occurring within the catchment after the surveys taken is not included in this investigation, unless specifically specified.

There are a number of limitations that apply to the modelling to date, some of which include:

- Stakeholder engagement will continue during detailed design, construction and operation. As such proposed impacts and structural solutions still need to be confirmed with relevant stakeholders. Modelling may need to be updated as a result of any ongoing stakeholder engagement.

ARR 2016 outlines several fundamental themes which are also particularly relevant to this investigation:

- All models are coarse simplifications of very complex processes. No model can therefore be perfect, and no model can represent all of the important processes accurately.
- Model accuracy and reliability will always be limited by the accuracy of the terrain and other input data
- Model accuracy and reliability will always be limited by the reliability/uncertainty of the inflow data
- No model is ‘correct’ therefore the results require interpretation
- A model developed for a specific purpose is probably unsuitable for another purpose without modification, adjustment, and recalibration. The responsibility must always remain with the modeller to determine whether the model is suitable for a given problem.
- Recognition that no two flood events behave in exactly the same manner
- Design floods are a best estimate of an “average” flood for their probability of occurrence.

It is noted that ARR 2019 has recently been released as an update to the ARR 2016 guidelines. Although there is limited difference in methodology between these versions it is recommended that in the next phase ARR 2019 guidelines are adopted.

The interpretation of results and other presentations in this report should be done with an appreciation of any limitations in their accuracy, as noted above.

Unless otherwise stated, presentations in this report are based on peak values of water surface level, flow, depth and velocity. Therefore, using water levels as an example, the peak level does not occur everywhere at the same time and, therefore, the values presented are based on taking the maximum value which occurred at each computational point in the model during the entire flood event. Hence, a presentation of peak water levels does not represent an instantaneous point in time, but rather an envelope of the maximum values that occurred at each computational point over the duration of the flood event.

Digitisation of the levees based on 2019 LiDAR data capture was undertaken to represent the current topographic conditions. Levees were manually digitised from upstream of the model to downstream of Goondiwindi to ensure the heights were captured in the hydraulic model. Digitisation was focused on key levees that impact the flood flows and are within the floodplain. It is noted that the area for capture is significant and that detail was extracted at a high level to allow efficient development of the data set. Some levees may not be included in the digitisation, but all levees are included in the 2019 LiDAR that is incorporated as a dataset in the model. The digitised levee lines are draped over the 2019 lidar to "force" the elevations in the model grid to ensure no gaps occur in the levees within the model topography based on the 30 m grid cell. To ensure the highest point was included along the levee a buffer was added to the line to capture the high points where the manually digitised line lies off the crest. For the 30 m grid scale of this model this is considered suitable to provide a representation of the topographic features. There may be some resulting inconsistencies in the elevations at each point and the elevation from the lidar at that location as a result of this process. These have been spot checked and found to be minor.

# 11 Conclusions

The key objective of the Hydrology and Flooding Technical Report is to provide information on the data investigation, hydrology and hydraulic calibration, design event modelling and provide comment on the performance on the proposal design. This report outlines the methodology followed, the outcomes of this investigation and the assessment of the proposal design.

There are several major waterways in the area of the proposal with the key waterway being the Macintyre River and its two tributaries, the Dumaresq River and Macintyre Brook, which meet upstream of Boggabilla. Detailed hydrologic and hydraulic assessments have been undertaken due to the catchment size and substantial floodplain flows associated with each of these watercourses. The most recent modelling of the system prior to this assessment was the DPIE Border Rivers modelling.

The DPIE models were utilised as a basis for the hydrology and hydraulic assessment of the proposal. DPIE used the 1976 and 1996 events to calibrate the hydrologic and hydraulic models. To confirm the reliability of the hydrologic and hydraulic models the 2011 event was added as an additional calibration event. The hydrologic models were found to represent flows across the floodplain well when compared to the recorded information for the 2011 event.

A hydraulic sub-model was developed covering the floodplain area down to Goondiwindi. The hydraulic sub-model reliably predicted the flood gauge heights at the Boggabilla and Goondiwindi stream gauges for all three historical events. Good correlation was achieved between the hydraulic sub-model results and historical flood photos and recorded flood levels for the 1976 and 2011 flood event. Based on this performance, the hydrologic and hydraulic models were considered suitably calibrated to use to assess the potential impacts associated with the proposal.

Design event hydrology was developed from the calibrated hydrologic models using ARR 2016 flood flow estimation techniques. The hydraulic sub-model was run for a suite of design events from the 20% AEP event to the PMF. The flows and levels were predicted by the hydrologic and hydraulic models were compared to the results of a FFA of the Boggabilla stream gauge, as well as results from previous flood studies, and were found to be consistent. The design validation of the 1% AEP event indicated that the hydrologic and hydraulic models were adequately representing the 1% AEP design event.

A design model was developed based on the calibration model and updated to include 2019 LiDAR and definition for the current floodplain conditions and features (i.e. levees). Modelling of the current state of development (Existing Case) was undertaken and details of the existing flood regime were determined for the modelled design events. The works associated with the proposal were incorporated into the hydraulic model to form the Developed Case. Assessment of the potential impacts upon the existing flood regime was undertaken and refinement of the proposal design was undertaken to mitigate impacts.

Consultation with stakeholders, including landholders, was undertaken at key stages including validation of the performance of the modelling in replicating experienced historical flood events and presentation of the design outcomes and impacts on properties and infrastructure.

The proposal design has been guided and refined using hydraulic design criteria and flood impact objectives. The resulting design outcomes relative to the hydraulic design criteria are detailed in Table 11.1.

**Table 11.1 Proposal hydraulic design criteria outcomes**

| Performance criteria | Design outcomes |
|---|---|
| Flood immunity | Rail line – 1% AEP flood immunity with freeboard greater than 300 mm to formation level has been achieved. |
| Hydraulic analysis and design | Hydrologic and hydraulic analysis and design has been undertaken using Australian Rainfall and Runoff (ARR 2016) and state/local government guidelines.<br>The proposal design includes significant rail drainage structures under the proposal alignment to convey flood flows on floodplains and minimise impacts under the full range of design events, being:<br>▪ 13 rail bridges<br>▪ Six (6) rail reinforced concrete box culvert (RCBC) banks<br>▪ 26 rail reinforced concrete pipe culvert (RCP) banks<br>▪ Local drainage structures:<br>– One (1) rail reinforced concrete box culvert (RCBC) banks<br>– 17 rail reinforced concrete pipe culvert (RCP) banks |
| Scour protection of structures | Culvert scour protection has been designed in accordance with Austroads Guide to Road Design Part 5B: Drainage (AGRD). Scour protection was specified where the culvert outlet velocities for the 1% AEP event exceeded the allowable soil velocities shown in Table 3.1 of AGRD. Required lengths of scour protection have been determined and are predicted to fit within the proposed rail disturbance footprint.<br>A conservative scour estimation has been undertaken at each bridge site based on available information and will be refined during detailed design. |
| Structural design | 1 in 2,000 AEP event has been modelled with details used for bridge design purposes. |
| Extreme events | Overtopping of the proposal alignment under extreme events occurs at limited locations being:<br>▪ Ch 18.70-20.80 and 28.00-28.50 in the 1 in 2000 AEP<br>▪ Ch 18.70-20.80, 20.80-25.50, and 28.00-28.50 in the 1 in 10,000 AEP<br>▪ Ch 7.20-8.10, 15.00-17.00,18.70-20.80, 20.80-25.50, 28.00-28.50, 28.50-31.00, 31.00-34.00, and 34.00-39.50 in the PMF |
| Flood flow distribution | Structures have been located along the proposal alignment to maintain existing flood conveyance and spread of floodwaters. |
| Sensitivity testing | The risk to the proposal design from climate change and blockage has been assessed in accordance with Australian Rainfall and Runoff 2016. Key outcomes are:<br>▪ The proposal design maintains 1% AEP flood immunity under 2090 climate change conditions<br>▪ Based on ARR 2016, a blockage factor of 25 per cent has been applied to culverts and no blockage factor has been applied to bridges<br>▪ Varying the level of blockage to culverts between 0 per cent and 50 per cent does not impact upon the proposal design. |

Flood impact objectives, have been established and used to guide the proposal design including mitigation of impacts through refinement of the hydraulic design, including adjustment of the numbers, dimensions and location of major drainage structures. Table 11.2 summarises how the proposal design performs against each of the flood impact objectives.

**Table 11.2 Flood impact objectives and outcomes**

| Parameter | Objectives and outcomes | | | | |
|---|---|---|---|---|---|
| Afflux | Existing habitable and/or commercial and industrial buildings/ premises (e.g. dwellings, schools, hospitals, shops) | Residential or commercial/industrial properties/lots where flooding does not impact dwellings/ buildings (e.g. yards, gardens) | Existing non-habitable structures (e.g. agricultural sheds, pump-houses) | Roadways | Agricultural and grazing land/forest areas and other non-agricultural land |
| | ≤ 10 mm | ≤ 50 mm | ≤ 100 mm | ≤ 100 mm | ≤ 200 mm with localised areas up to 400 mm |
| | **Objective:** Changes in peak water levels are to be assessed against the above proposed limits.<br>**Outcome:** Generally, the Project design meets the above limits with number of small localised areas along the proposal alignment where these increases of up to 400 mm occur. These areas very small in extent with increases dissipating within 30 m to 200 m of the alignment. There are two locations where the change in peak water levels exceed 400 mm. In both locations the impact reduces to less than 200 mm within 100m or less of the rail embankment, with the impact limited to an area 0.025 $km^2$ or less. No flood sensitive receptors are impacted by the changes in peak water levels under the 1% AEP event. | | | | |
| Change in duration of inundation | **Objective:** Identify changes to time of inundation through determination of TOS. For roads, determine AATOS and consider impacts on accessibility during flood events.<br>**Outcome:** There are minor localised changes in the duration of inundation (ToS) upstream and downstream of the proposal alignment. These changes in inundation duration do not affect flood sensitive receptors and compared to the duration of the flood events on the Macintyre River floodplain these changes are minor. The modelling results at a number of local roads have been inspected with the depth of water, TOS and AAToS assessed. With the exception of localised areas on Bruxner Way and North Star Road, there is no adverse impact on existing roads. The localised areas on Bruxner Way and North Star Road are isolated during flood events by flood waters to the north and south for long durations and with over 1 m of flood water. The localised increase in this location is therefore also considered not to be an adverse impact. | | | | |
| Flood flow distribution | **Objective:** Aim to minimise changes in natural flow patterns and minimise changes to flood flow distribution across floodplain areas. Identify any changes and justify acceptability of changes through assessment of risk with a focus on land-use and flood sensitive receptors.<br>**Outcome:** The Project has minimal impacts on flood flows and floodplain conveyance/storage with significant floodplain structures included to maintain the existing flood regime. | | | | |
| Velocities | **Objective:** Maintain existing velocities where practical. Identify changes to velocities and impacts on external properties and waterway geomorphology. Determine appropriate scour mitigation measures taking into account existing soil and geomorphological conditions.<br>**Outcome:** In general, changes in velocities are minor, with most changes in velocities experienced immediately adjacent to the proposal alignment and no flood sensitive receptors impacted.<br>The proposal results in minimal changes to peak water levels, velocities and flood flow distribution across the floodplain and in each of the waterways. This means that the proposal design minimises potential changes to the geomorphological conditions in the waterways and as such the risk of change to geomorphological conditions in each of the waterways is low.<br>Scour protection has been specified where the outlet velocities for the 1% AEP event exceed the allowable soil velocities for the particular soil type for each location, which was identified from published soil mapping. | | | | |
| Hazard | **Objective:** Identify changes to hazard categories and any impacts on external properties. Justify acceptability of changes through assessment of risk with a focus on land-use and flood sensitive receptors.<br>**Outcome:** There are no significant changes to hazard classifications across the floodplain as a result of the proposal alignment works. | | | | |

| Parameter | Objectives and outcomes |
|---|---|
| Extreme event risk management | **Objective:** Consider the risks posed to neighbouring properties for events larger than the 1% AEP event to ensure no unexpected or unacceptable impacts.<br>**Outcome:** A review of impacts under the 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events has been undertaken with the existing flood depths and increase in peak water levels at flood sensitive receptors identified on each floodplain. Overall, considering the high flood depths that occur, particularly under the PMF event, the changes in peak water levels would be unlikely to exacerbate flood conditions during extreme events. There are three locations, one house and two sheds, where water levels increase significantly under the extreme events.<br>During detailed design predicted outcomes will be discussed in detail with landholders and a range of alternative mitigation measures will be further investigated including refined drainage structures, property solutions, scour and embankment protection, etc. Formal third party agreements will be negotiated with landholders that takes account of these impacts and the adopted mitigation measures. |
| Sensitivity testing | **Objective:** Consider risks posed by climate change and blockage in accordance with ARR 2016. Undertake assessment of impacts associated with proposal alignment for both scenarios.<br>**Outcomes:**<br>Climate change – climate change has been assessed in accordance with ARR 2016 requirements with the representative concentration pathway 8.5 (2090 horizon) scenario adopted giving an increase in rainfall intensity of 23 per cent across the catchment areas. The impacts resulting from changes in peak water levels under the 1% AEP event with climate change are generally similar to those seen under the 1% AEP event.<br>Blockage – Blockage of drainage structures has been assessed in accordance with ARR 2016 requirements. The blockage assessment resulted in no blockage factor being applied to bridges and a blockage factor of 25 per cent being applied to culverts. Two blockage sensitivity scenarios were tested with both 0 per cent and 50 per cent blockage of all culverts assessed. The resulting changes in peak water levels associated with the Project alignment are still localised and do not impact on any flood sensitive receptors. |

The hydrologic and flooding assessment undertaken has demonstrated that the proposal is predicted to result in impacts on the existing flooding regime that generally comply with the flood impact objectives. Best practice flood risk management, including sensitivity testing, has been applied in developing the proposal design to minimise risk to life, property, infrastructure, the community and environment.

ARTC have completed a comprehensive consultation package to provide the community with detailed information and certainty around the flood model and Macintyre floodplain crossing solution. In future stages, ARTC will:

- Continue to work with landowners concerned with hydrology and flooding throughout the detailed design, construction and operational phases of the proposal
- Continue to work with directly impacted landowners affected by the alignment throughout the detailed design, construction and operational phases of the proposal
- Continue to work with local Councils, DPIE and local flood specialists throughout the detailed design, construction and operational phases of the proposal.

# 12 References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