APPENDIX Q1

Hydrology and Flooding Technical Report—Volume I

INLAND RAIL—BORDER TO GOWRIE ENVIRONMENTAL IMPACT STATEMENT

IR_1534

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

Inland Rail Border to Gowrie EIS

Appendix Q1 - Hydrology and Flooding Technical Report – Volume I

Australian Rail Track Corporation

Reference: 3100

Document Number:
2-0001-310-EAP-10-RP-0213

Contents

Appendices

Figures

Tables

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
ARF	Areal Reduction Factor
ARR 2016	Australian Rainfall and Runoff Guidelines – 2016 Edition
ARTC	Australian Rail Track Corporation
B2G	The Border to Gowrie Project
BoM	Bureau of Meteorology
CC	Climate Change
CG	The QLD Coordinator-General
DCDB	Digital Cadastral Database
DEA	Design Event Approach
DEM	Digital Elevation Model
Developed Case	Hydraulic modelling case with Project in place
Disturbance footprint	The Project disturbance footprint includes the rail corridor and other permanent works associated with the Project (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).
DNRME	QLD Department of Natural Resources, Mines and Energy
D/S	Downstream
DTMR	Department of Transport and Main Roads
EIS	Environmental Impact Statement
Existing Case	Hydraulic modelling case pre-Project (i.e. existing conditions)
FFA	Flood Frequency Analysis
FFJV	Future Freight Joint Venture
G2H	Gowrie to Helidon
GIS	Geographic Information System
GRC	Goondiwindi Regional Council
IFD	Intensity-Frequency-Duration
km	kilometres
LAS	Industry-standard Binary Format for Storing Airborne LiDAR
LGA	Local Government Authority
LiDAR	Light Detection and Ranging
m	metres
m AHD	metres above AHD
NS2B	North Star to Border
QGIS	Quantum Geographic Information System
QLD	Queensland
QR	Queensland Rail
QRT	Quantile Regression Technique

Term or acronym	Description
RAATM	Requirements Analysis Allocation Traceability Matrix
RCBC	Reinforced Concrete Box Culvert
RCP	Reinforced Concrete Pipe
RCPs	Representative Concentration Pathways
RFFE	Regional Flood Frequency Estimation
SILO	Scientific Information for Land Owners
SRTM	Shuttle Radar Topography Mission
TOF	Top of Formation
ToR	Terms of Reference set for the Project by the CG
ToS	Time of Submergence
TRC	Toowoomba Regional Council
The Project	The Border to Gowrie project
U/S	Upstream

Executive summary

Inland Rail is a once-in-a-generation Project 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 km line is the largest freight rail infrastructure project in Australia and is expected to commence operations in 2026.

The Inland Rail New South Wales (NSW)/Queensland (QLD) Border to Gowrie (B2G) Project (the 'Project') provides a connection between the northern end of the North Star to Border (NS2B) project and the western end of the Gowrie to Helidon (G2H) project. The Project is proposed to cross several major rivers, creeks and streams. The Project alignment travels through Goondiwindi Local Government Area (LGA) and Toowoomba LGA.

There are several major waterways within the Project study area, with the key waterways being Gowrie Creek, Condamine River, Macintyre Brook and Macintyre River. Other significant creek crossings include Pariagara Creek, Cattle Creek, Native Dog Creek, Bringalily Creek, Nicol Creek, Back Creek and Westbrook Creek.

The purpose of this assessment is to better understand and quantify the existing flooding characteristics of the floodplains that the Project crosses and to assess and mitigate any potential impacts associated with the Project alignment on the existing flooding regime of each waterway. The key objective of the report is to provide information on the data investigation, hydrologic and hydraulic calibration, design event modelling and provide comment on the performance on the 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 each waterway. These models were calibrated against multiple historical events and validated through stakeholder and community feedback.

Details regarding each model including its development, calibration and spatial extent are provided in the individual catchment chapters within this report. Hydrologic modelling for the Project was undertaken using the methodology consistent with Australian Rainfall and Runoff guidelines (ARR 2016). To support the flood impact assessment two cases were modelled for each floodplain: An Existing Case representing current floodplain conditions (or 'baseline' conditions), and a Developed Case representing floodplain conditions that include the Project. The Developed Case models were 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 each floodplain and, in particular, upon identified flood sensitive receptors.

The refinement of the Project design was guided using hydraulic design criteria and flood impact objectives that were developed for the Project based on industry practice and engineering judgement. 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 Project is predicted to result in impacts on the existing flooding regime that generally comply with the flood impact objectives and that the Project design meets the hydraulic design criteria.

A comprehensive consultation exercise was undertaken to provide the community with detailed information and certainty around the flood modelling and the Project 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 Project
- Directly impacted landowners affected by the alignment throughout the detailed design, construction and operational phases of the Project
- Local Councils and State government departments throughout the detailed design, construction and operational phases of the Project.

Flood maps were prepared to visualise communicate anticipated flood impacts for the range of modelled flood events. The flood maps are presented in Volume II of this report (Appendix Q2: Hydrology and Flooding Technical Report - Figures).

1 Introduction

1.1 Inland Rail Programme

Inland Rail is a once-in-a-generation Programme 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 km line is the largest freight rail infrastructure project in Australia and is expected to commence operations in 2026.

1.2 Border to Gowrie alignment

The New South Wales (NSW)/Queensland (QLD) Border to Gowrie Project (known as the 'B2G' Project) provides a connection between the northern end of the North Star to Border (NS2B) Project and the western end of the Gowrie to Helidon (G2H) Project. The Project is proposed to cross several major rivers, creeks and streams.

The Project alignment runs through Goondiwindi Local Government Area (LGA) and Toowoomba LGA. The Project alignment including the Project footprint is shown in Figure A1 in Appendix A.

Key features of the Project include:

- 216.2 km of new single-track dual gauge railway (trains travelling in both directions share the same track)
- 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 and fauna crossing structures.

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 Project alignment. 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 Project alignment and to assess and mitigate any potential impacts associated with the Project alignment 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 Project alignment, development of mitigation measures and to provide comment on the performance of the Project 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 Project design.

Key objectives of the hydrology and flooding investigation were to:

- Consult with local authorities regarding existing flood studies relevant to the design and consider these previous flood studies in the design
- Consult with stakeholders and government agencies to obtain flood data to assist in model development and calibration
- Undertake detailed hydrologic and hydraulic modelling for each major catchment to establish the Base Case (or Existing Case) flood conditions for the range of floods up to 1% Annual Exceedance Probability (AEP) as well as the 1 in 2,000 and 1 in 10,000 AEP and the Probable Maximum Flood (PMF) events

- Determine existing flood conditions including flood levels, flows and velocities
- Analyse the Project design including the alignment design, drainage infrastructure and associated infrastructure works
- Assess the impacts of the Project design 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

The hydrology and flooding investigation 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 Project design.
- Determination of critical flooding mechanisms for waterways and drainage paths in vicinity of the Project alignment, i.e. regional flooding versus local catchment flooding.
- Development of tailored hydrologic and hydraulic models for key waterways. Individual modelling approaches, including justification for the selection of each approach are outlined in the individual modelling sections within this report.
- Validation of the hydrologic and hydraulic models against recorded data for historical flood events.
- Community and stakeholder engagement to validate model performance and gain acceptance of modelling and calibration outcomes. Anecdotal flood event information such as flood photography, recorded flood markers and personal observations from landholders were sourced to validate the calibration of the hydrologic and hydraulic models.
- Update of hydrologic models to include Australian Rainfall and Runoff 2016 (ARR 2016) design events. ARR 2016, being the current version of this guideline during the formative stages of Inland Rail and this Project, was adopted as a guiding document for flooding aspects of this assessment to ensure consistency in assessment across the Inland Rail Program.
- Simulation of ARR 2016 design events for the Existing Case and comparison to previous studies to confirm drainage paths, waterways, and associated floodplain areas, and establish the existing flood regime in the vicinity of the Project.
- Inclusion of Project alignment and drainage structures (Developed Case) in the hydraulic models and simulation of ARR 2016 design events including the 20%, 10%, 5%, 2%, 1% events, extreme events including the 1 in 2,000 and 1 in 10,000 AEP events and the Probable Maximum Flood (PMF).
- Tropical cyclone-induced rainfall events are captured in the Australian Rainfall and Runoff (ARR 2016) Data Hub, and historic rainfall and stream gauge data used in the hydrologic assessments.
- Assessment of impacts of Project alignment using the suite of design flood events 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 drainage structures under the Project alignment. Iterations were undertaken in the hydraulic models to achieve a design that addresses the flood impact objectives.
- Sensitivity analysis on the design for factors including climate change and blockage risk.

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

- Modelling of the Existing Case 1% AEP event to ascertain existing conditions and inform the flood immunity for the Project 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 and bridges.

3 Existing environment

3.1 Waterways

There are several major waterways within the B2G impact assessment area, with the key waterways being the Macintyre River, Macintyre Brook, the Condamine River and Gowrie Creek. Other major creek crossings include Pariagara Creek, Cattle Creek, Native Dog Creek, Bringalily Creek, Nicol Creek, Back Creek and Westbrook Creek.

The Border Rivers and Darling Downs floodplains have experienced many floods in recent years including the 1956, 1976 and more recently the 2011 flood event. The floodplains are 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 Condamine River floodplain between Millmerran and Brookstead in particular houses a large number of significant waters storages (ring tanks).

3.2 Floodplain infrastructure

The floodplains located within the B2G impact assessment area include several major infrastructure assets near the Project alignment that could potentially influence local flooding behaviour, including:

- Gowrie Creek floodplain:
  - Warrego Highway
  - Kingsthorpe-Haden Road
  - Draper Road
  - Leesons Road
  - Gowrie Junction Road
- Westbrook and Dry Creeks floodplain:
  - Toowoomba Wellcamp Airport
  - Toowoomba-Cecil Plains Road
  - Brimblecombe Road
- Condamine River floodplain:
  - Gore Highway
  - Town of Pampas
  - Queensland Rail – Wyreema to Millmerran Line
  - Pampas-Horrane Road
  - Millmerran-Leyburn Road
  - Doug Hall Poultry at Yandilla
  - Several stream gauges including Pampas (DNRME), Yarramalong Weir (Sunwater), Centenary Bridge (Bureau of Meteorology (BoM)) etc.
- Back Creek floodplain:
  - Commodore Mine
  - Millmerran Power Station
  - Millmerran-Inglewood Road

  - Kooroongarra Road
- Macintyre Brook floodplain:
  - Cunningham Highway
  - Town of Inglewood
  - Inglewood-Texas Road
  - Texas-Yelarbon Road
  - Desert Creek Road
  - Bybera Road
  - Cremascos Road
  - Town of Yelarbon
  - Yelarbon-Keetah Road
  - Yelarbon flood levee
  - Kildonan Road
- Levees, dams, ring tanks and pump houses from farming practices.

4 Design requirements, standards and guidelines

4.1 Hydraulic design criteria

Table 4.1 outlines the hydraulic design criteria that have guided the Project design. Detailed hydrologic and hydraulic modelling was undertaken to meet these design criteria with a series of iterations undertaken to incorporate design refinement and stakeholder and community feedback. The resulting design outcomes relative to these design criteria are detailed in the impact assessment section of each chapter.

Table 4.1 Project hydraulic design criteria

Performance criteria	Requirement
Flood immunity	Rail line – 1 % AEP flood immunity with 300 mm freeboard 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. 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. 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. 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 Project design sensitivity to climate change and blockage of structures.

4.2 Flood impact objectives

The impact of the Project 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 Terms of reference for an environmental impact statement: Inland Rail – Border to Gowrie project (November 2018) (ToR) and were used to guide the Project 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 consider flood sensitive receptors and land use within the floodplain. Flood sensitive receptors have been identified from aerial and satellite imagery and ground-truthed where possible during site visits. In certain cases, such as the Condamine River floodplain flood sensitive receptors were confirmed with affected landowners. Flood sensitive receptors include dwellings, sheds, commercial properties such as petrol filling stations and shops, silos, hospitals, roads, rail lines, airports etc. In the B2G impact assessment area a total of 545 flood sensitive receptors were identified (as shown on the flood maps presented in Volume II of this report).

The resulting design outcomes relative to these flood impact objectives are detailed in the Impact Assessment section of each chapter.

Table 4.2 Flood impact objectives

Parameter	Objectives					
Change in peak water levels[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. Rail lines.	Agricultural (cropping) areas	Agricultural (grazing land/forest) areas and other non-agricultural land.
	≤ 10 mm.	≤ 50 mm.	≤ 100 mm.	≤ 100 mm.	≤ 100 mm with localised areas up to 400 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 Project. It should be noted that in many locations the presence of existing buildings or infrastructure limits the change in peak water levels.					
Change in time of submergence[1]	▪ Identify changes to duration of inundation through determination of Time of Submergence (ToS)[2] ▪ For roads, determine the Average Annual Time of Submergence (AAToS) (if applicable) and consider impacts on accessibility during flood events ▪ 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 ▪ 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 ▪ Identify changes to velocities and impacts on external properties ▪ Determine appropriate scour mitigation measures considering existing soil conditions ▪ 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.					
Sensitivity testing	▪ Consider risks posed climate change and blockage in accordance with ARR 2016 ▪ Undertake assessment of impacts associated with Project alignment for both scenarios.					

Table note:

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

4.3 Project nomenclature for design events

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

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

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

Exceedances per year (EY)	AEP (%)	AEP (1 in x)	Average Recurrence Interval (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

Source: ARR 2016 Book 1

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
- Australian Rainfall and Runoff: A Guide to Flood Estimation, (2016), Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M, Testoni I, (Editors), Commonwealth of Australia
- Austroads (2013) Guide to Road Design Part 5: Drainage – General and Hydrology Considerations, Sydney
- Queensland Department of Transport and Main Roads (2013) Bridge Scour Manual, http://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Bridge-scour-manual
- Evaluating Scour at Bridges, Hydraulic Engineering Circular Number 18 (HEC-18), Fifth Edition, US Department of Transport – Federal Highway Administration, Virginia, USA, Richardson, EV and Davis, SR: 2012
- 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.

5 Local catchment drainage

5.1 Catchment classification

The Project alignment crosses several existing flowpaths in the different catchment areas that contribute flows to the cross-drainage structures. To determine the appropriate hydrologic methods for the local drainage design, the existing catchments were categorised based on the contributing catchment areas. Table 5.1 presents the drainage catchment classification criteria and number of catchments relating to each classification.

Table 5.1 Drainage catchment classification

Catchment size	Drainage catchment classification	Number of catchments
Less than or equal to 10 km^2	Minor	165
Greater than 10 km^2 and less than or equal to 100 km^2	Moderate	5
Greater than 100 km^2	Major	8

The major and moderate floodplains are addressed in sections 6 to 19.

5.1.1 Minor catchments

The 1% and 0.05% AEP catchment flows for the minor catchments were generated in accordance with ARR 2016 using ILSAX within the 12D Drainage Network Editor.

Ten temporal patterns were run for each storm duration and the median temporal patterns from each duration were compared to determine the peak runoff for each catchment.

The losses adopted within ILSAX for the local catchment flows were taken from the calibrated regional hydrologic models along the Project alignment.

As no calibration data was available to compare against the local catchment flows, the 1% AEP flows generated from ILSAX was compared against the traditional Rational Method. The flows generated using ILSAX compare closely with the flows generated from the traditional Rational Method and are within a tolerance of -8 to 15%.

The Rational Method is no longer compliant with ARR 2016; however, it is still considered to give a reasonable approximation of local catchment flows and therefore the parameters and resultant ILSAX flows were adopted for the design.

5.1.2 Moderate catchments

Nicol Creek is one of three moderate waterways which cross the Project alignment before flowing into Canning Creek. The other waterways are Native Dog Creek and Cattle Creek. Due to their proximity to one another, all four waterways were modelled in a unified Canning Creek model. Two other moderate waterways that cross the Project alignment includes tributaries of the Macintyre Brook at Bybera Road and Cremascos Road.

Hydrology and hydraulic models were developed for moderate catchments, and reported in sections 6 to 19.

5.1.3 Major catchments

Major catchments > 100 km^2 that are traversed by the B2G Project alignment include Gowrie Creek, the Westbook/Dry Creeks system, the Condamine River, Back Creek, Bringalily Creek, Pariagara Creek, Macintyre Brook and the Macintyre River.

Hydrology and hydraulic models were developed for major catchments, and reported in sections 6 to 19.

5.2 Hydraulic design

Cross drainage structures are provided where the rail intercepts existing flowpaths. The type of structures adopted depends on a range of factors including as the natural topography, rail formation levels, design flows and soil type.

The cross drainage design was undertaken in accordance with the Project hydraulic design criteria set out in Table 4.1. Cross drainage structures outside the regional floodplains were sized based on the flows generated from the local drainage catchments. Cross drainage structures that have a well-defined local catchment boundary and are located within or near the regional floodplains were assessed for both the local catchment flows and regional floodplain conditions to determine the governing design conditions.

5.2.1 Minor catchments

Cross drainage structures located within minor catchments where the upstream flow path is primarily 1-Dimensional (1D) were assessed as per the following methodology:

- Culverts were initially sized and optimised using 12D Dynamic Culvert
- The resultant afflux was assessed in TUFLOW and the culvert designs were adjusted as required to meet the afflux criteria. Further details of the impact assessment are detailed in Section 5.2.3.
- Final culvert designs were analysed back in 12D Dynamic Culvert to determine final design water levels and velocities at the culverts which are detailed in Appendix E.

5.2.2 Moderate and major catchments

Cross drainage structures located within moderate or major catchments were assessed within flood models. The respective modelling methodologies are outlines in the respective sections within this report (refer Table 6.1).

5.2.3 Impact assessment

For each of the local catchment crossings the impact of the Project upon the existing flood regime was quantified and compared against flood impact objectives as detailed in Table 4.2. These objectives have been used to guide the Project 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 takes into account flood sensitive receptors and land use.

The land use across the Project area has been classified as agricultural cropping/grazing/pastureland or forest areas based on aerial imagery. Sensitive agricultural land may be identified during further consultation with landowners which will need to be considered in Detailed Design. The hydraulic impacts in the local catchments are considered ‘localised’ in comparison to regional flood impacts due to the shorter time of flood inundation and smaller flood extents. The afflux and change in time of inundation at each structure is tabulated in Appendix E. The predicted impacts all comply with the flood impact objectives.

5.2.4 Sensitivity analysis

5.2.4.1 Blockage

A blockage assessment for the 1% AEP event was undertaken in accordance with ARR 2016 Book 6 Chapter 6 Blockage of Hydraulic Structures.

A 25% blockage was adopted during feasibility design for all structures along the Project alignment.

The blockage factor was applied by reducing the culvert opening by 25% within the 12D Dynamic Culvert Editor and was applied in TUFLOW within the culvert network layer.

5.2.4.2 Climate change assessment

The impacts of climate change were assessed in accordance with ARR 2016 Book 1 Chapter 6 for the local drainage catchments for the 1% AEP design event to determine the sensitivity of the design to the potential increase in rainfall intensity.

The selected representative concentration pathway (RCP) for the climate change analysis was 8.5 which represents a high emissions scenario. For B2G, RCP 8.5 corresponds to an increase in temperature of 4.2°C in 2090 and an increase in rainfall intensity of 23% which was obtained from the ARR 2016 Data Hub.

The climate change analysis was undertaken in 12D Dynamic Culvert by increasing the rainfall intensities within the Intensity Frequency Duration (IFDs) for the local catchments.

The climate change RCP 8.5 scenario increases the resultant 1% AEP water levels at the local drainage structures by a maximum of 0.71 m along the alignment. Under this scenario, the rail formation is overtopped at four culvert locations and an additional 22 culverts have less than 300 mm freeboard.

6 Hydrologic and hydraulic model summary

The hydrologic and hydraulic models that were developed for major and moderate floodplains that are traversed by the B2G Project alignment are summarised in Table 6.1. The hydraulic model locations and extents are shown in Figure A2 in Appendix A.

Details regarding each model including its development, calibration and spatial extent are provided in the individual catchment chapters within this report.

Table 6.1 Project hydrologic and hydraulic model summary

Waterway	Catchment classification	Hydrologic modelling software	Hydraulic modelling software	Report section
Gowrie Creek	Major	RAFTS	TUFLOW	Section 7
Westbrook and Dry Creeks	Major	RAFTS	TUFLOW	Section 8
Condamine River	Major	URBS	TUFLOW	Section 9
Back Creek	Major	URBS	TUFLOW	Section 10
Nicol Creek	Moderate	URBS	TUFLOW	Section 11
Bringalily Creek	Major	URBS	TUFLOW	Section 12
Native Dog Creek	Moderate	URBS	TUFLOW	Section 13
Cattle Creek	Moderate	URBS	TUFLOW	Section 14
Pariagara Creek	Major	URBS	TUFLOW	Section 15
Macintyre Brook Yelarbon to Inglewood	Major	URBS	TUFLOW	Section 16
Macintyre Brook at Bybera Road	Moderate	URBS	TUFLOW	Section 17
Macintyre Brook at Cremascos Road	Moderate	URBS	TUFLOW	Section 18
Macintyre River	Major	URBS	TUFLOW	Section 19

The hydrologic and hydraulic models were reviewed and verified including checks for stability, health and robustness. Within the Gowrie Creek and Westbrook/Dry Creek models some minor modelling noise occur in isolated instances, e.g. during the PMF event around the downstream modelling boundaries. These minor instabilities do not however affect the results. It is recommended that during detailed design the hydraulic modelling domains for Gowrie Creek and Westbrook/Dry Creek are extended.

7 Gowrie Creek

The Project alignment does not cross Gowrie Creek, but instead skirts around the 1% AEP floodplain extents, before connecting into the Gowrie to Helidon section of Inland Rail at Draper Road. Under the 1% AEP event, around the Leesons Road/Draper Road junction, the Existing Case peak depth of water is approximately 5 m in the Gowrie Creek channel, and up to around 1 m on the floodplain at the Project alignment. The floodplain inundation under the 1% AEP event varies between 200 m and 500 m wide. Several small, un-named flow paths draining the area to the north of the Warrego Highway cross the Project alignment.

The location of the Project rail alignment in relation to Gowrie Creek is shown in Figure A-1a in Volume II – Appendix A.

7.1 Data collection and review – Gowrie Creek

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:

- Toowoomba Regional Council (TRC) – existing flood studies and stream gauging data
- The Bureau of Meteorology (BoM) – rainfall and stream gauging data
- Department of Natural Resources, Mines and Energy (DNRME) – stream gauging data
- Queensland Rail (QR) – existing infrastructure details
- Department of Transport and Main Roads (DTMR) – existing infrastructure details.

7.1.1 Previous studies

A number of previous hydrology and hydraulic studies were sourced as part of this assessment. A review of each study was undertaken to determine suitability for use on the Project as documented in the following sections.

Technical Report on the Oakey Flood of 10-11 January 2011, BMT WBM 2011 (BMT WBM, 2011)

This study provided information on flooding in relation to the Oakey flood event of 10 to 11 January 2011.

Gowrie Creek Flood Risk and Management Study Volume 1, TRC 2013 (TRC, 2013a)

This model covered the upper reaches of the Gowrie Creek catchment within Toowoomba city including East Creek and West Creek. The model was calibrated for the 2010 and 2011 flood events. The design flood hydrology was based upon Australian Rainfall and Runoff (1987) and so design flood estimates were not consistent with Australian Rainfall and Runoff 2016.

Gowrie Creek Flood Risk and Management Peer Review, TRC November 2013 (TRC, 2013b)

This study provided a technical review of flood modelling work undertaken as part of the TRC (2013a) study by a Peer Review Panel.

Work Package 4, Historical study for Kingsthorpe and Gowrie Junction Final Report, DHI/WRM 2014

This study focused on a small reach of Gowrie Creek between Kingsthorpe and Gowrie Junction and only considered the flood behaviour of the January 2011 flood event. The study involved the development of a coupled 1D/2D MIKE FLOOD hydraulic model.

Work Package 8, 2D Flood study for Cotswold Hills (Gowrie Creek Catchment) Final Report, DHI/WRM 2014

This study focused on small tributaries of Gowrie Creek in the vicinity of Cotswold Hills Township and did not explicitly cover flooding in Gowrie Creek itself. The study involved the development of a hydrologic model (RAFTS) and a coupled 1D/2D MIKE FLOOD hydraulic model. The design flood hydrology was based upon Australian Rainfall and Runoff (1987) and so design flood estimates were not consistent with Australian Rainfall and Runoff 2016.

Oakey Flood Study Final Report, TRC 2014 (TRC, 2014c)

A hydrologic and hydraulic model was developed in RAFTS and MIKE FLOOD for the Oakey catchment as part of this study. Gowrie Creek was modelled as one of the Oakey Creek tributaries; however, the area of interest for the Project was not included. The model was calibrated to the 10 January 2011 flood event using historical observed flood marks and flows from Cranley and Oakey stream gauges.

7.1.2 Survey

ARTC provided LiDAR data from 2015 as 1 m grid DEM tiles. Using GIS software, a Digital Elevation Model (DEM) was generated with a 1 m grid resolution for use in the Project based on the 2015 dataset. This was used for modelling within the disturbance footprint and up to the full extent of the 2015 LiDAR where relevant.

Additional LiDAR data extents were required to appropriately model downstream boundary conditions and facilitate calibration against streamflow gauges. In areas that were not covered by the LiDAR provided by ARTC, LiDAR tiles were sourced from Geoscience Australia. The DEM datasets utilised for modelling were based on surveys flown between 2009 and 2015, with preference given to the most recent data available.

To inform the hydrologic modelling, Shuttle Radar Topography Mission (SRTM) data was used for catchment delineation where no LiDAR data could be sourced.

Additional survey was undertaken during November 2018 of the QR line located to the north of the proposed alignment. This data was included in this assessment.

The survey data sources and DEM developed for Gowrie Creek are shown in Figure A-1b in Volume II – Appendix A.

7.1.3 Aerial imagery

Aerial imagery captured in 2015 was provided by ARTC and was used to identify and confirm topographic and vegetative characteristics of the study area. Additional imagery outside the study area was sourced from QGIS imagery in an open source format.

7.1.4 Existing drainage structure data

Structure geometry information contained within the previous hydraulic models was used in this assessment. Two culverts at Stankes Road and Burkes Road that could influence local flows were identified from the aerial imagery. The details of the culverts at these locations were assumed and used in the hydraulic model.

7.1.5 Stream gauge data

Stream gauges are used to provide a record of observed stream levels. These were originally manually recorded staff levels (typically recorded daily 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 streamflow. The most reliable source of data for deriving a rating curve are actual instream 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 or other infrastructure.

There are two gauges within Gowrie Creek, being Oakey and Cranley. The location of these gauges is presented in Figure A-1c in Volume II – Appendix A and the gauge details are outlined in Table 7.1.

Table 7.1 Stream gauges within the Gowrie Creek catchment

Station name	Station number	Owner	Record length (years)	Start of record
Oakey	422332B	DNRME	27	1992
Cranley	422332A	DNRME	49	1969

7.1.6 Rainfall data

Twenty-seven daily rainfall and fourteen pluviograph rainfall gauging stations exist within 30 km of the centre of Gowrie Creek catchment (as shown in Table 7.2 and Table 7.3, respectively).

Figure A-2a in Volume II – Appendix A shows the location of daily rainfall gauging and pluviograph stations.

Table 7.2 Summary of daily rainfall gauging stations used for calibration

Station name	Station number	Easting	Northing	Owner
Helidon Post Office	040096	152.12	-27.55	BoM
Pechey Forestry	040170	152.05	-27.30	BoM
Fordsdale	040395	152.12	-27.72	BoM
Mt Whitestone	040397	152.16	-27.67	BoM
West Haldon	040424	152.08	-27.75	BoM
Perseverance Dam	040480	152.12	-27.29	BoM
Withcott	040672	152.02	-27.55	BoM
Cressbrook Dam	040808	152.20	-27.26	BoM
Helidon TM	040829	152.11	-27.54	BoM
Deverton Sawpit Gully Road	040883	152.05	-27.69	BoM
Cambooya Post Office	041011	151.87	-27.71	BoM

Station name	Station number	Easting	Northing	Owner
Doctors Creek	041024	151.85	-27.21	BoM
Greenmount Post Office	041040	151.90	-27.78	BoM
Haden Post Office	041042	151.88	-27.22	BoM
Jondaryan Post Office	041053	151.59	-27.37	BoM
Mount Irving	041072	151.60	-27.48	BoM
Pittsworth	041082	151.63	-27.72	BoM
Mt Kynoch	041096	151.95	-27.51	BoM
Springside	041166	151.60	-27.68	BoM
Aubigny Purrawunda	041170	151.64	-27.54	BoM
Rosalie Plains	041212	151.68	-27.21	BoM
Oakey Aero	041359	151.74	-27.40	BoM
Moyola	041369	151.88	-27.52	BoM
Tamba	041510	151.95	-27.47	BoM
Cooby Creek Dam	041512	151.92	-27.38	BoM
Toowoomba Airport	041529	151.91	-27.54	BoM
Middle Ridge	041553	151.96	-27.60	BoM

Table 7.3 Summary of pluviograph rainfall gauging stations used for calibration

Station name	Station number	Easting	Northing	Owner
27 December 2010 event				
Toowoomba Airport	041529	151.91	-27.54	TRC (2013a)
Middle Ridge	041553	151.96	-27.60	TRC (2013a)
Gabbinbar Res	NA	151.95	-27.61	TRC (2013a)
Eastern Valley	NA	151.97	-27.58	TRC (2013a)
Picnic Point	NA	151.98	-27.57	TRC (2013a)
Alderley Street	NA	151.94	-27.58	TRC (2013a)
SPS 42 Prince Henry	NA	151.99	-27.55	TRC (2013a)
Prescott and Goggs Street	NA	151.94	-27.56	TRC (2013a)
Wetalla STP	NA	151.93	-27.51	TRC (2013a)
USQ	NA	151.93	-27.60	TRC (2013a)
Oakey at Gowrie Creek	422332	151.74	-27.47	TRC (2013a)
10 January 2011 event				
Toowoomba Airport	041529	151.91	-27.54	TRC (2013a)
USQ	NA	151.93	-27.60	TRC (2013a)
Oakey at Gowrie Creek	422332	151.74	-27.47	TRC (2013a)

7.1.7 Anecdotal and observed flood data

Observed flood markers were surveyed after the January 2011 event by TRC, presented in TRC (2014c) study. A total of 11 flood markers were available within the extent of the hydraulic model as shown in Figure A-1d in Volume II – Appendix A.

These observed levels typical consisted of debris marks observed on the ground, buildings, fences and poles. The flood markers were used for hydraulic model validation in this assessment. The accuracy and reliability of debris mark data is inferior to streamflow gauge records (+/- 200 to 300 mm for flood markers as opposed to +/- 100 mm for gauges).

7.1.8 Site inspection

A site inspection was undertaken during February 2018. During the site inspection, all major waterway crossings were visited and inspected with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

7.2 Development of hydrologic model – Gowrie Creek

7.2.1 Model setup

A RAFTS hydrologic model was developed for the Gowrie Creek catchment using catchment details and parameters from previous studies. A review of the two existing Gowrie Creek hydrologic models was undertaken and a summary is provided in Table 7.4.

Table 7.4 Existing Gowrie Creek hydrologic models review summary

Study	Software used	Area of interest	Calibration and validation information	Information used in this assessment
Toowoomba Regional Council, Gowrie Creek Flood Risk and Management Study Volume 1.	RAFTS	Covers the U/S extents of Gowrie Creek catchment within Toowoomba City.	Validated to 10 January 2011, 17 December 2010 and 27 December 2010 events.	▪ Delineated catchments and hydrologic parameters ▪ Historical rainfall data for the 27 December 2010 event.
2D Flood study for Cotswold Hills (Gowrie Creek Catchment) Final Report.	RAFTS	Covers Cotswold Hill Catchment. The focus of Cotswold Hill study was local flooding in the tributary catchments of Gowrie Creek. This catchment is located south of the Preferred Design Alignment. This model does not incorporate Gowrie Creek itself.	Validated with the Rational Method and calibrated to the 10 January 2011 event.	▪ Delineated catchments and hydrologic model parameters.
Oakey Flood Study Report Final Report.	RAFTS	Covers the Oakey Creek Catchment. Gowrie Creek catchment was modelled as one of the Oakey Creek tributaries but the area of interest for this assessment was D/S of Gowrie Creek at Oakey.	Validated to 10 January 2011.	▪ Loss values and estimated flows at Oakey gauging station.

7.2.2 Sub-catchments

The delineation of sub-catchments for the upstream catchment of Gowrie Creek and the Cotswold Hills area was adopted from the previous studies. The remaining catchment of Gowrie Creek was delineated into sub-catchments based on topographic data.

The hydrologic model setup including extent and sub-catchment map is presented in Figure A-1c in Volume II – Appendix A.

7.2.3 Fraction impervious

The catchment roughness (PERN) and percentage impervious were based on land use layer from the Queensland Globe database and aerial imagery. The Gowrie Creek catchment can broadly be divided into two areas based on land use as shown in Figure A-1c in Volume II – Appendix A, referred to as Area 1 and Area 2 for this assessment. Area 1 is mostly urbanised and located in the upstream part of the catchment whilst Area 2 consists mainly of rural floodplain located in the central and downstream reaches of the catchment.

Fraction impervious values of all sub-catchments within Area 1 were defined based on the TRC model (TRC 2013a). Within Area 2, fraction impervious values for the Cotswold Hills sub-catchments were based on the second TRC model (TRC 2014b). For the remaining Area 2 sub-catchments, fraction impervious values were estimated based on land use data from Queensland Globe (accessed April 2018) and GIS data provided by TRC.

7.2.4 Routing parameters

Routing between sub-catchments was modelled using the 'channel routing link' approach. The hydrograph lag time between sub-catchment nodes was adjusted as part of the model calibration. Initial estimates of the hydrograph lag were based on approximate flow distances between sub-catchment nodes and the average flow velocities based on catchment slope. The upstream catchment routing lag times were estimated based on the results of the existing HEC-RAS model.

7.3 Development of hydraulic model – Gowrie Creek

7.3.1 Model setup

The Gowrie Creek hydraulic model was developed in the TUFLOW HPC software package using a 5 m grid spacing. The hydraulic model setup, including extent and adopted land use, is presented in Figure A-1d in Volume II – Appendix A. The 2D model topography was modified to adequately represent the drainage flowpaths and the existing road/rail crowns.

7.3.2 Hydraulic structures

Structure geometry information contained within the previous hydraulic models was used in this assessment. Two culverts at Stankes Road and Burkes Road that could influence the local flows were identified from the aerial imagery. The details of the culverts at these locations were assumed and used in the hydraulic model. In total, 24 culverts and four bridges identified within the hydraulic model domain as summarised in Table 7.5.

Table 7.5 Identified existing structures within the hydraulic model extent

Structure modelling ID	Type	Infrastructure	U/S invert (m AHD)	D/S invert (m AHD)	Diameter/width (m)	Height (m)	Number of cells
COT-02	RCP	Road	542.46	542.06	1.2	-	3
KIN-GOJ-25	RCBC	QR line	467.40	467.02	2.8	3.0	3
KIN-GOJ-26	RCBC	QR line	472.60	472.20	3.0	2.1	2
KIN-GOJ-32	RCBC	QR line	481.40	481.05	1.8	1.8	6
KIN-GOJ-23	RCP	QR line	465.90	465.10	0.5	-	1
KIN-GOJ-24	RCP	QR line	469.00	466.70	1.2	-	1
KIN-GOJ-29	RCBC	Road	487.20	486.80	1.2	0.3	4
KIN-35a	RCBC	QR line	502.18	501.40	1.2	0.7	4

Structure modelling ID	Type	Infrastructure	U/S invert (m AHD)	D/S invert (m AHD)	Diameter/width (m)	Height (m)	Number of cells
KIN-35b	RCP	QR line	501.85	501.40	0.6	-	2
KIN-GOJ-36	RCBC	QR line	504.90	504.70	1.5	1.5	1
KIN-35c	RCBC	Road	500.70	500.40	1.2	1.2	1
KIN_26e	RCP	Road	472.75	472.50	0.5	-	1
KIN-GOJ-27	RCBC	QR line	477.50	477.30	1.2	0.6	2
COT-01	RCP	Road	564.41	562.60	1.8	-	1
KIN22-Draper	RCP	QR line	458.70	458.30	1.0	-	6
KIN-GOJ-21	RCBC	QR line	455.00	454.80	3.0	2.1	5
KIN-GOJ-19	RCBC	Road	447.33	447.08	2.1	1.1	3
KIN-GOJ-18	RCBC	QR line	448.40	448.10	2.1	1.2	2
KIN-GOJ-3	RCBC	QR line	443.30	443.10	1.2	0.8	2
KIN-GOJ-2	RCBC	QR line	442.80	442.60	1.2	0.9	5
KIN-GOJ-34	RCBC	QR line	497.50	497.10	1.2	0.3	1
KIN-GOJ-28	RCBC	Road	481.50	481.20	1.2	0.7	1
Stankes_Road	RCBC	Road	491.50	491.20	0.5	0.5	1
Burkes_Road	RCBC	Road	497.70	497.40	0.5	0.5	1

Table 7.6 Identified existing bridges within the hydraulic model extent

Bridge design ID	Bridge length (m)	Obvert level (m AHD)	Deck depth (m)
KIN-GOJ-33	22	492.60	0.60
KIN-GOJ-31-old Homebush	38	478.50	0.70
KIN_GOJ_20	64	454.25	1.75
KIN_GOJ_1	43	435.40	1.10

7.3.3 Roughness

The hydraulic roughness is reflective of the nature of development and ground cover that exist within the hydraulic model extent. The distribution of roughness categories adopted for this assessment was based on the information supplied in the TRC models (TRC 2014a and TRC 2014b), land use layer from the Queensland Globe database, TRC GIS data, aerial imagery and confirmed during the site inspection.

Specific roughness values applied to the hydraulic model are detailed in Table 7.7. Figure A-1e in Volume II – Appendix A shows the spatial breakdown of land use in the 2D model domain.

Table 7.7 Manning's n values

Land use	Manning's n
Floodplain	0.050
Roads	0.025
Developed area	0.083
Vegetated waterways	0.050
Waterways	0.033
Dense vegetation	0.100

7.3.4 Boundary conditions

The Gowrie Creek hydrologic model outputs were applied as inflows into the hydraulic model. Total inflows from catchments upstream of the hydraulic model extent were applied at the upstream model boundary and local inflows from areas within the TUFLOW hydraulic model were applied throughout the model.

Internal inflow boundaries were applied as SA polygons with the flow applied to the lowest point of each SA polygon. The proposed Toowoomba Tunnel Portal West is located within sub-catchments C7 and C17. The tunnel portal constitutes the lowest point of the SA polygons in the Developed Case. To avoid applying inflow to the tunnel portal in the Developed Case, SA polygons for C7 and C17 were adjusted. Similarly, since the lowest point of sub-catchments GOW1.15 and GOW24.01 lie downstream of the proposed alignment, extra internal inflow boundaries were added, and the total flow was divided accordingly (refer Figure A-1d in Volume II – Appendix A).

A normal depth boundary condition was applied at the downstream boundary.

7.4 Joint calibration

The hydrologic model was calibrated against the following historical events:

- 27 December 2010
- 10 January 2011.

Rainfall data collected in the upper catchment suggests that the 2011 event was between a 1% and 1 in 500 AEP event magnitude flood. The 2010 event was estimated to be approximately a 5% AEP event magnitude. Near the Project alignment, the 2011 event is the largest on record.

Daily rainfall and pluviograph data were available at several rainfall stations for these two historical events. Observed streamflow gauge data at two gauging stations (Oakey and Cranley stream gauges) were available for the 27 December 2010 event. However, for the 10 January 2011 event observed flow records at Oakey gauge were not considered reliable near the peak of the flood event as detailed in Section 7.4.1.2.

7.4.1 Hydrologic model calibration

7.4.1.1 December 2010 calibration event

Adopted initial loss values for the pervious area were minimal as the catchment received significant rainfall in the two weeks preceding the event. The loss parameters that were used in calibration are outlined in Table 7.8. Note that the modelling parameters were spilt into two areas to be consistent with past modelling.

Table 7.8 Rainfall losses used for 27 December 2010 calibration

Location	Area type	Initial loss (mm)	Continuing loss (mm/hr)
Area 1	Old urban impervious	8.0	4.0
	New urban impervious	1.5	0.0
	Pervious US	20.0	4.0
Area 2	New urban impervious	1.5	0.0
	Pervious DS	15.0	2.5

Figure 1 and Figure 2 present plots of observed and modelled flow hydrographs for the 2010 event at Cranley and Oakey streamflow gauges respectively. There is a good fit between observed and modelled flow hydrographs at both stations. The total runoff Volume and timing of the peak correspond reasonably well with observed peaks. The difference in peak flow is summarised in Table 7.9.

There is a good match between observed and modelled results with the difference between peak flows being less than 6%.

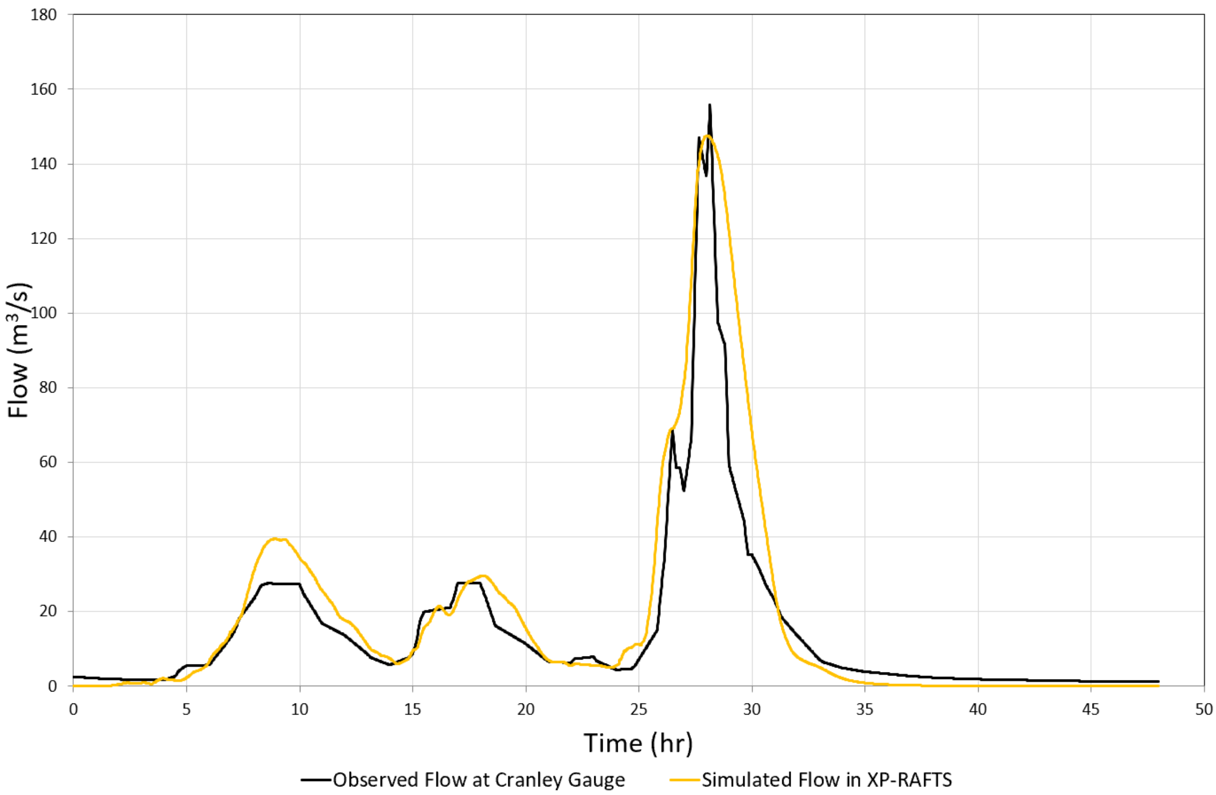

Figure 1 Comparison of gauged and modelled hydrographs at Cranley gauge – 2010 event

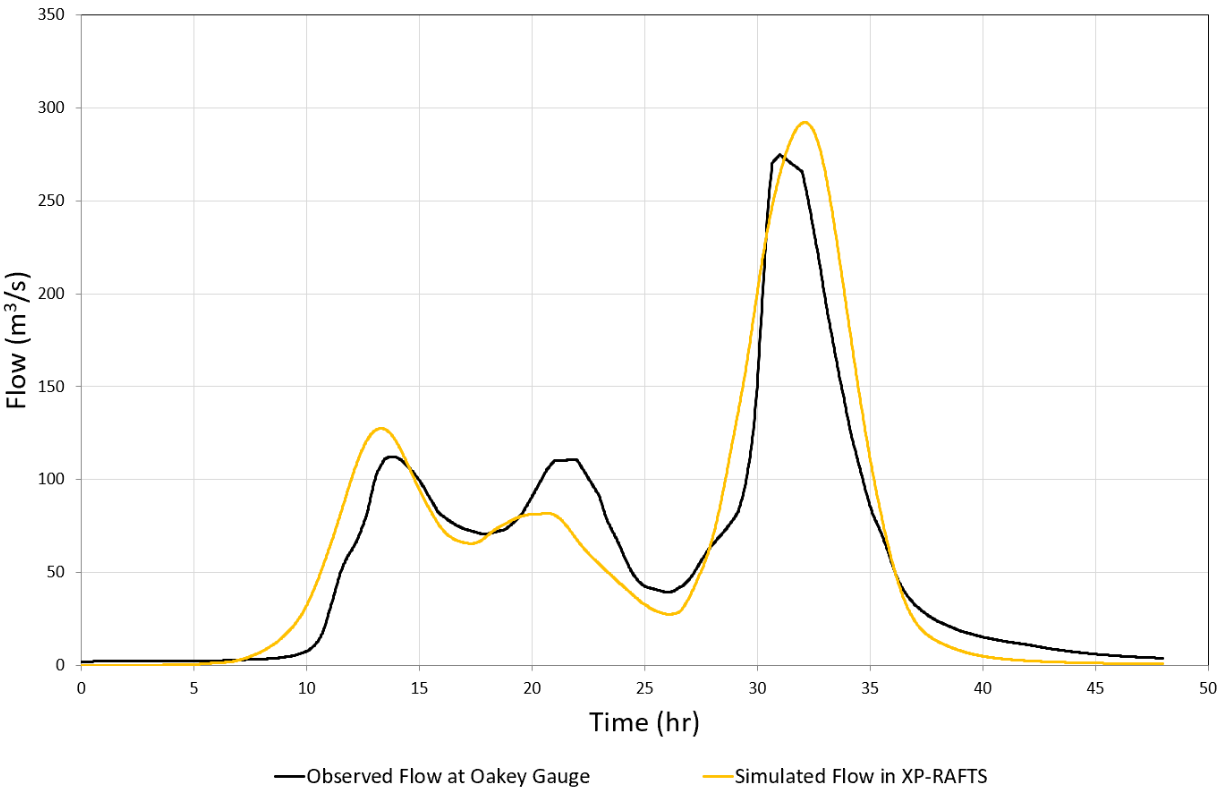

Figure 2 Comparison of gauged and modelled hydrographs at Oakey gauge – 2010 event

Table 7.9 **Comparison of gauge and modelling peak flows for 2010 calibration event**

Gauge	Observed peak flow (m^3/s)	Modelled peak flow (m^3/s)	% Difference between observed and modelled
Oakey	275	292	+5.8%
Cranley	156	148	-5.4%

7.4.1.2 January 2011 calibration event

The loss parameters that were used in the 2011 calibration are outlined in Table 7.10. A larger initial loss was used for this event in the hydrologic model in comparison to previous studies as in this current investigation, the January 2011 event was modelled as a three-day event whilst in previous studies it was modelled as a shorter event.

The routing parameter (K) was modified to match the peak flow values and the continuing losses were varied between acceptable values to best match the recession limb, the Volume of the hydrograph and secondary peaks. The parameter K was further refined based on the results of hydraulic modelling. Adopted initial loss values for the pervious area are small as the catchment received significant rainfall in the two weeks preceding the event.

Table 7.10 **Rainfall losses used for January 2011 calibration event**

Location	Area type	Initial loss (mm)	Continuing loss (mm/hr)
Area 1	Old urban Impervious	8.0	4.0
	New urban Impervious	1.5	0.0
	Pervious US	10.0	2.5
Area 2	New urban Impervious	1.5	0.0
	Pervious DS	10.0	2.5

Figure 3 shows the plot of modelled hydrograph against observed flow at Cranley gauge for the 2011 event. The three peaks match relatively well with regards to timing. The difference between modelled and observed major peak flow is less than 1% (Table 7.11). This indicates the model has predicted the peak and timing of observed event very well.

Table 7.11 **Comparison of Gauge and modelling peak flows at Cranley stream gauges for 2011 calibration event**

Gauge	Observed peak flow (m^3/s)	Modelled peak flow (m^3/s)	% difference between observed and modelled
Cranley	609	605	-0.7%

Figure 4 presents a plot of the modelled hydrograph against observed flow at the Oakey gauge for the January 2011 event. The two smaller peaks were modelled well in terms of timing and this indicated that the routing parameters used in the model are reasonable. The rising limb of major peak and timing is modelled well against the observed data. However, modelled major peak flow is significantly larger than observed peak flow. The recession of modelled hydrograph also matches well with the observed data.

It appears that there was a problem in the rating curve of the gauge. The agreement with the smaller peaks suggests that it is a high stage rating issue. The previous study also identified uncertainties in gauging rating curve (refer Section 7.1.1).

The hydrologic model calibration provides sufficient confidence that the hydrologic model can be used for the design event simulations and design option assessment.

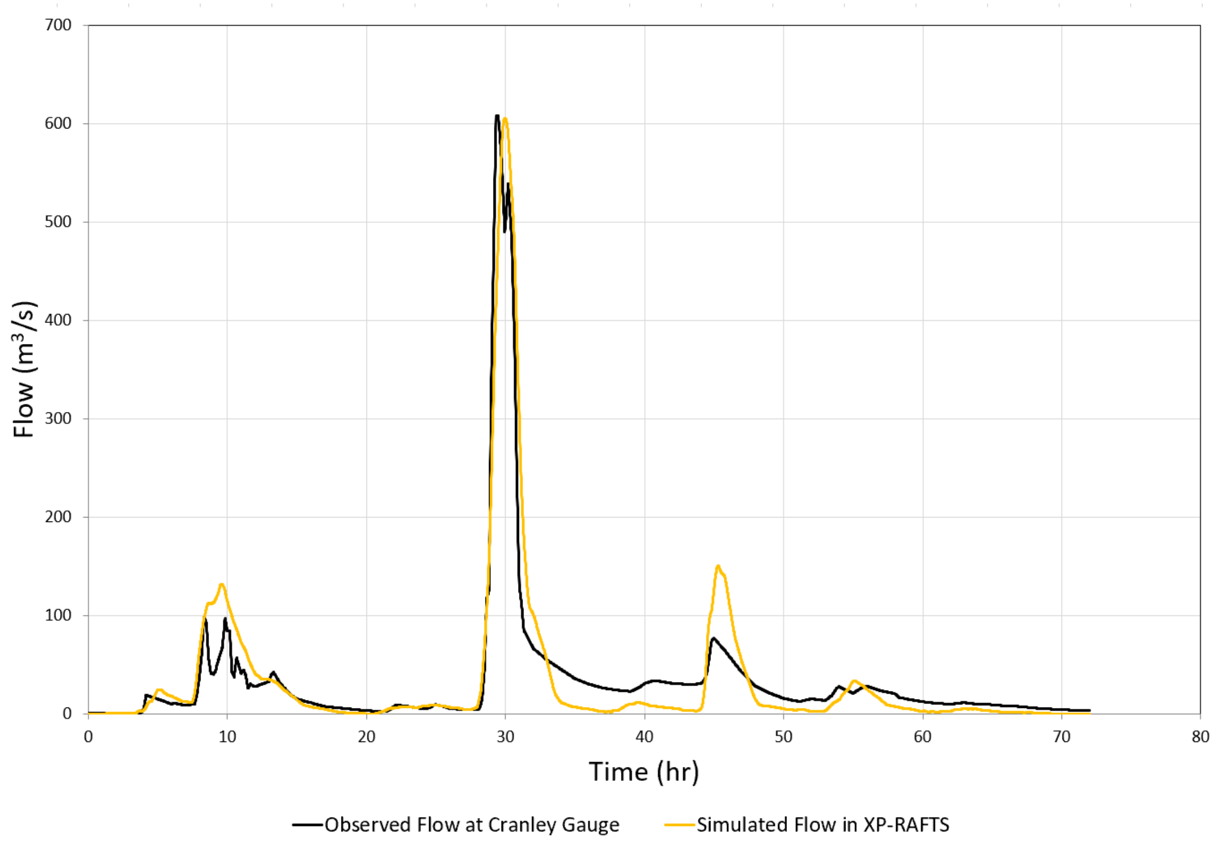

Figure 3 Comparison of gauged and modelled hydrographs at Cranley gauge – 2011 event

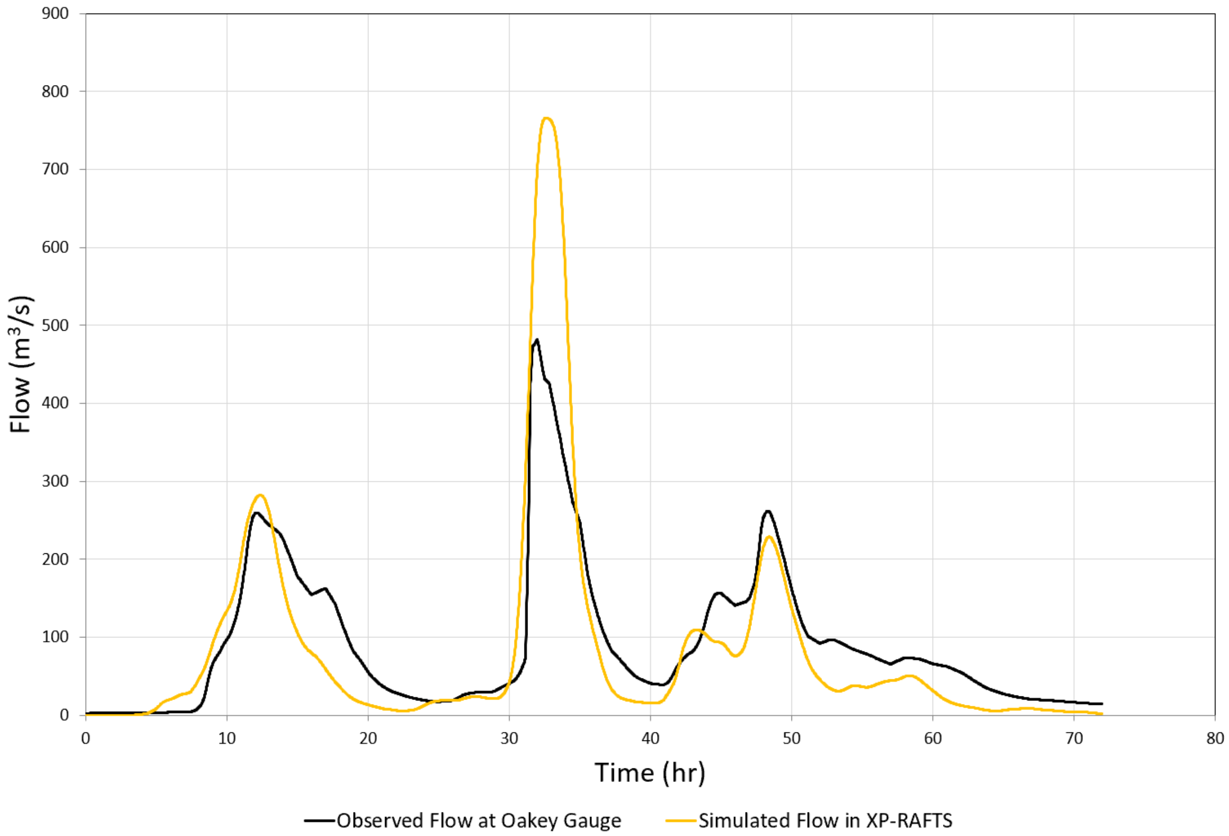

Figure 4 Comparison of gauged and modelled hydrographs at Oakey gauge – 2011 event

7.4.2 Hydraulic model calibration

7.4.2.1 Validation against observed flood markers

No flood markers were available for the 2010 event and therefore the calibration was limited to matching the hydrologic model outcomes to the stream gauges as discussed in Section 7.4.1.1.

Observed flood markers were surveyed after the January 2011 event by TRC as presented in the TRC (TRC 2014c) study. A total of 11 flood markers were available within the extent of the hydraulic model as shown in Figure A-1d in Volume II – Appendix A.

The 2011 observed levels typically consisted of debris marks observed on the ground, buildings, fences and poles. The flood markers were used for hydraulic model validation in this assessment. The accuracy and reliability of debris mark data is considered to be inferior to streamflow gauge records (+/- 200-300 mm for flood markers as opposed to +/- 100 mm for gauges).

To validate the developed hydraulic model, the hydraulic model was run for the 10 January 2011 flood event using the simulated flows from the RAFTS model. Table 7.12 provides a comparison of the model flood levels and observed levels for debris marks.

These results suggest a reasonable match between the simulated and observed (10 January 2011 event) flood debris mark levels. The hydraulic model does not consistently under or over-estimate the flood levels.

Table 7.12 Comparison between observed and model flood levels for January 2011 event

Location ID	Observed flood level at flood marks (m AHD)	Simulated water level in hydraulic model (m AHD)	Difference (m)
1	503.07	503.12	+0.06
2	501.28	501.19	-0.09[1]
3	480.17	480.33	+0.15
4	482.61	482.91	+0.30
5	482.80	482.54	-0.25[1]
6	492.91	492.97	+0.06
7	492.53	492.65	+0.12
8	519.02	518.84	-0.18[1]
9	518.75	518.73	-0.02[1]
10	450.17	450.16	-0.01[1]
11	455.88	455.58	-0.30[1]

Table note:

1 Nearest wet location reported

A flood inundation extent map for the January 2011 flood event is presented in Volume II – Appendix A, Figure A-2a.

7.5 Flood frequency analysis

A flood frequency analysis (FFA) was undertaken for the two streamflow gauges, Cranley (422332A) and Oakey (422332B), using the FLIKE software package. The FFA was based on the maximum historical instantaneous flow for each year of available record, also known as the annual series. The annual series of each gauge was fitted against different probability models to find the distribution model that achieved the best fit to the records.

As presented in Table 7.1, Cranley and Oakey gauges have 49 and 27 years of recorded flow respectively. The following sections provide further details regarding the FFA process and results.

7.5.1 Cranley stream gauge

The Cranley gauge location has not changed since installation. Generally, the Generalised Extreme Value (GEV) and Log Pearson III (LP3) distribution are recommended for streamflow analysis. FLIKE (version 5.0.251.0) software was used to fit LP3 probability distribution with Bayesian inference method for estimation of distribution parameters. Figure 6 shows that the LP3 distribution fits reasonably well to the Cranley gauge annual series. All the observed peak flow records are within the 90% limits of the LP3 distribution and close to the expected probability line, except for the January 2011 record.

The January 2011 event was a record-breaking flood, being of the order of three times the size of the second highest flood. TRC (2013b) reported that the Cranley gauge had malfunctioned during the 10 January 2011 event after an approximate flow of 330 m^3/s at 2pm was recorded. TRC (2013b) developed a hydrologic model for the January 2011 event that modelled 645 m^3/s of peak flow at the Cranley gauge. A review study undertaken in 2013 (TRC, 2013b) estimated a peak flow of 560 m^3/s at the Cranley gauge and provided a range of 293 m^3/s to 399 m^3/s peak flow for a 1% AEP event.

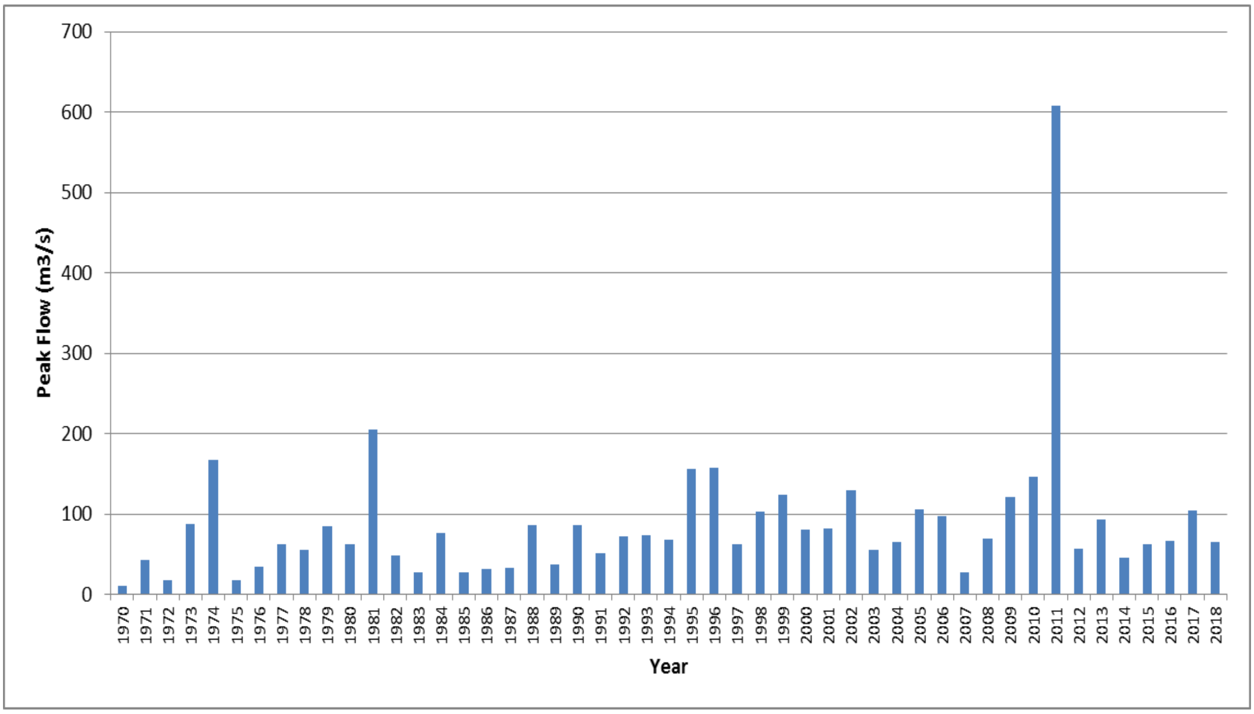

Figure 5 The annual series used in flood frequency analysis for Cranley gauge

Based on data from DNRME's website, the reported peak flow for the January 2011 event at Cranley gauge is 609 m^3/s, which is between the ranges recommended by previous studies. Therefore, a value of 609 m^3/s was used in the Cranley annual series shown in Figure 5.

The estimation of probability quantiles limits for the LP3 model for a 0.5 EY to a 1% AEP event is presented in Table 7.13. The estimated 1% AEP peak flow is 370 m^3/s, which is within the range as specified in the TRC (2013b) study.

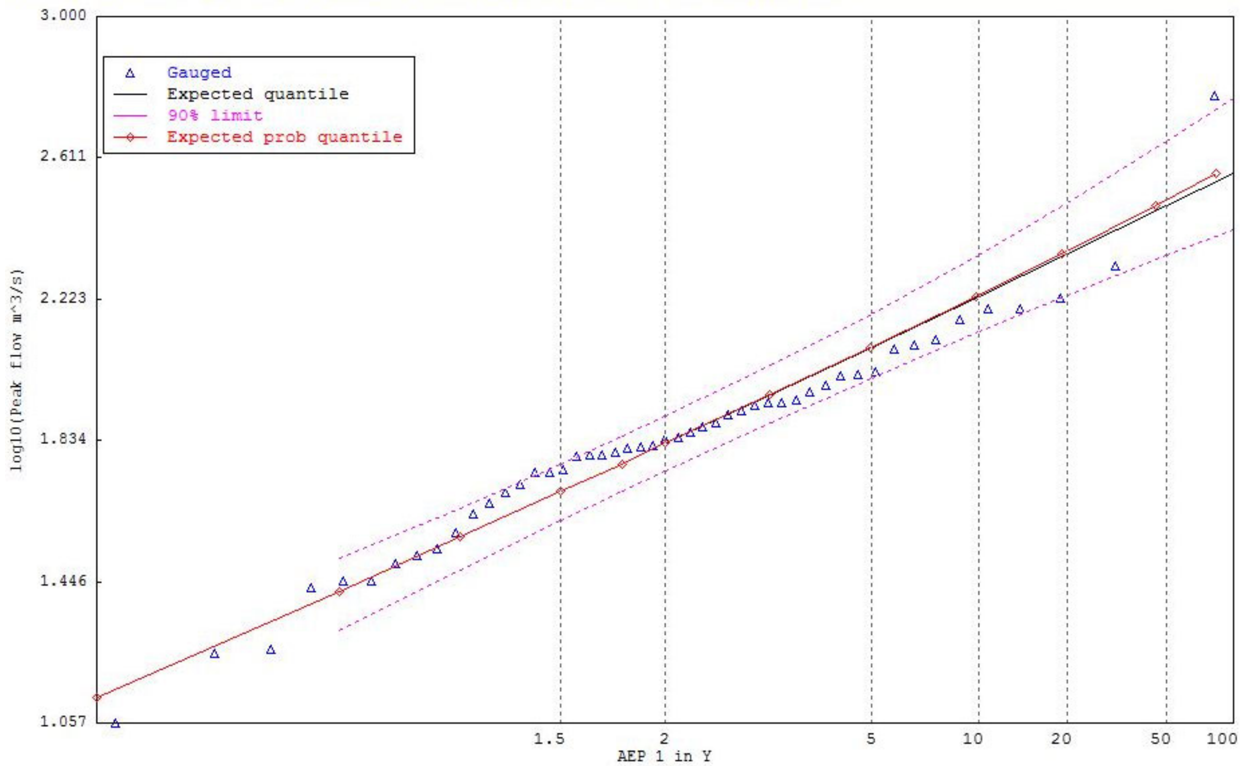

Figure 6 Probability model distribution – LP3 model – Cranley gauge – plot scale log-normal

Table 7.13 Flood frequency analysis results for Cranley gauge based on LP3 model

AEP	Expected probability quantile (m³/s)	90% probability limit (m³/s)	
0.22 EY	67	56	80
20%	123	101	153
10%	170	136	220
5%	222	172	307
2%	301	221	456
1%	370	260	598

7.5.2 Oakey gauge

7.5.2.1 Original gauged data

The annual series at Oakey gauge was fitted against various distribution models to determine a good fit. FLIKE (version 5.0.251.0) software was used to fit LP3 probability distribution with Bayesian inference method for estimation of distribution parameters. The annual series as shown in Figure 7 was used in the LP3 model. Figure 8 shows that the LP3 distribution fits reasonably well with the Oakey gauge annual series. All the observed peak flow records are within the 90% limit of LP3 distribution and close to the expected probability line. The estimation of probability quantiles limits of LP3 model is presented in Table 7.14.

According to TRC (2014c), DNRME had confirmed that there were issues with rating curves at the Oakey gauge. A technical report to the Queensland Floods Commission of Inquiry (BMT WBM, 2011) noted that Oakey gauged data for the January 2011 event is unvalidated (TRC, 2014c). Comparing the reported Oakey gauged peak flow (482 m³/s) for the January 2011 event with Cranley (610 m³/s) suggests that revision of gauge flow might be required as detailed in the next section.

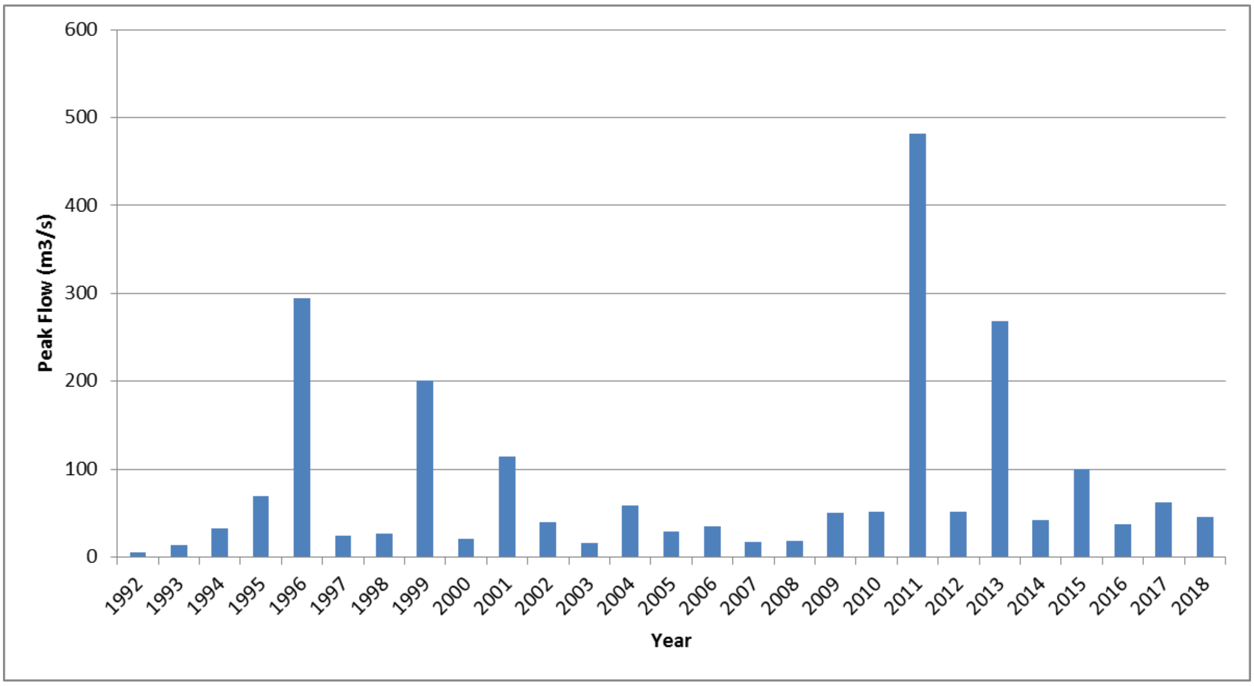

Figure 7 **The annual series used in flood frequency analysis for Oakey gauge**

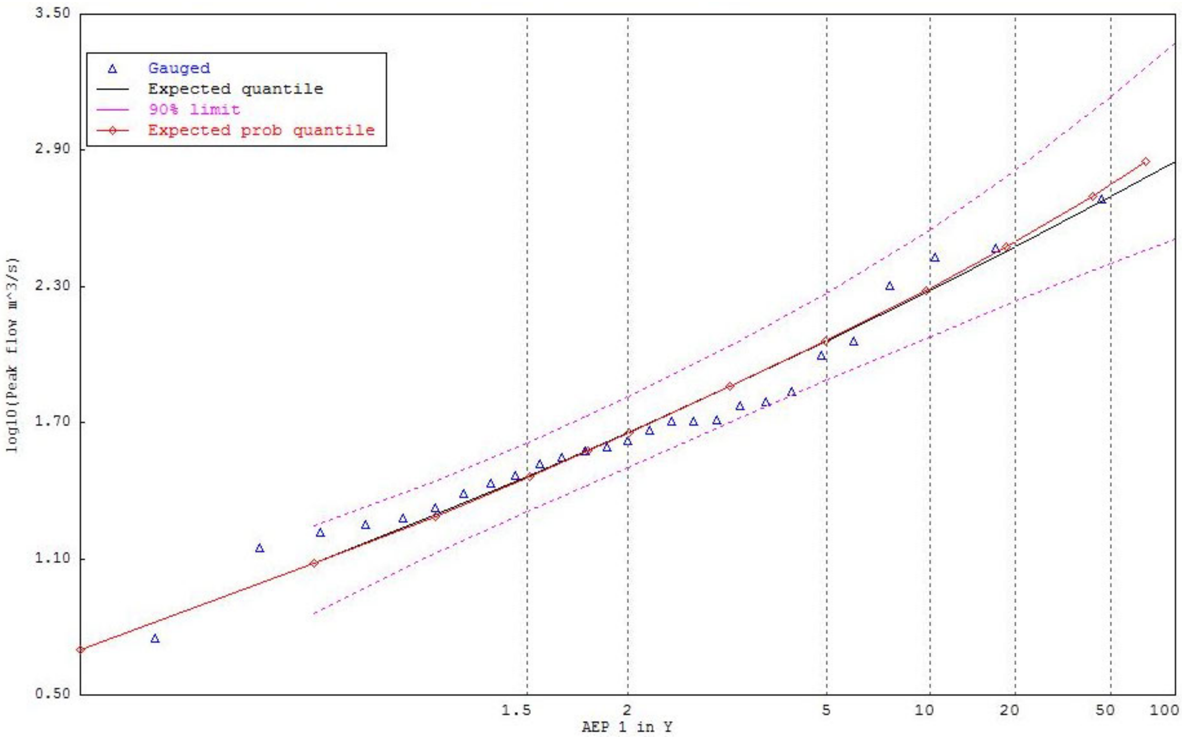

Figure 8 **Probability model distribution – LP3 model – Oakey gauge – plot scale log-normal**

Table 7.14 **Flood frequency analysis results for Oakey gauge based on LP3 model**

AEP	Expected probability quantile (m^3/s)	90% probability limit (m^3/s)	
0.22 EY	46	32	65
20%	115	77	186
10%	192	120	353
5%	298	172	644
2%	497	253	1,375
1%	705	323	2,382

7.5.2.2 Revised gauge data

As mentioned previously, there was an issue with the estimation of peak flow during the January 2011 event at Oakey gauge. The January 2011 event had three peak flows as shown in Table 7.15. The Volume modelled in the hydrologic model for these three peaks was compared with the Volume observed at the Oakey gauge for the three peaks, as summarised in Table 7.15.

The modelled volumes in RAFTS for the two smaller peaks (peak 1 and peak 3) are smaller than the observed Volume based on the rating of the gauge. However, the modelled Volume is 33% higher than the recorded Volume for the main peak (peak 2). This comparison shows that the rating of the gauge for the main peak (peak 2) is low and the flood peak should be higher than the reported 482 m^3/s.

Table 7.15 Comparison between modelled and observed Volume at Oakey gauge for three peaks observed during January 2011 event

Peak flow time (hr)	Modelled Volume in hydrologic model (ML)	Recorded Volume at Oakey gauge (ML)	Difference between observed and modelled Volume (%)
Peak 1 – 0 to 23	337,567	413,817	-18%
Peak 2 – 23 to 40	610,451	458,135	+33%
Peak 3 – 40 to 72	361,467	558,273	-6%

It appears that the Oakey gauge did not record peak of flow hydrograph accurately during the 2011 flood event. Therefore, the peak historical instantaneous flow for the 2011 event was revised from 482 m^3/s to 620 m^3/s by TRC (2014c) based on the results of FFA analysis. The FLIKE (version 5.0.251.0) software package was used to fit a LP3 probability distribution with Bayesian inference method for estimation of distribution parameters (refer Figure 9).

The results of the FFA (LP3 distribution) show an increase in the estimated 1% AEP flow from 705 m^3/s to 780 m^3/s. Therefore, a range of 705 m^3/s to 780 m^3/s is suggested as an acceptable range for the January 2011 event at Oakey gauge. Table 7.16 presents the FFA results for the revised flow at the Oakey gauge.

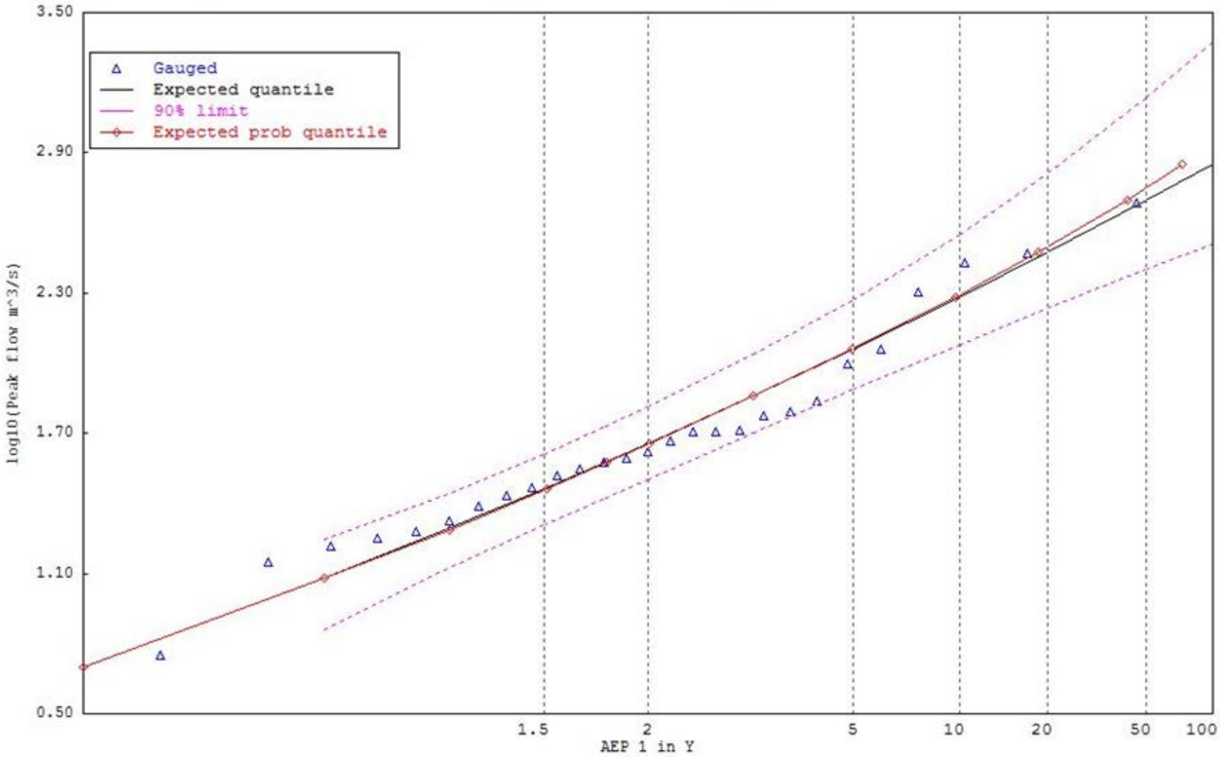

Figure 9 Revised probability model distribution – LP3 model – Oakey gauge – plot scale log-normal

Table 7.16 Revised flood frequency analysis results for Oakey gauge based on LP3 model

AEP	Expected probability quantile (m^3/s)	90% probability limit (m^3/s)	
0.22 EY	45	32	66
20%	117	78	193
10%	200	123	376
5%	316	179	706
2%	540	269	1,556
1%	780	348	2,755

7.6 Hydrologic modelling – Gowrie Creek

Hydrologic modelling for the Gowrie Creek catchment was undertaken using the methodology consistent with ARR 2016.

The design rainfall was estimated based on the 2016 Intensity-Frequency-Duration (IFD) data available on the BoM Data Hub. The temporal pattern ensembles, pre-burst losses, storm losses and areal reduction factors were directly downloaded via the RAFTS software from the ARR 2016 Data Hub.

For rare storm events, the critical duration at key locations within the Gowrie Creek catchment was found to be between 60 minutes and 180 minutes. Therefore, design rainfall depths for 1 in 10,000 AEP and PMP events were estimated from BoM's Generalised Short Duration Method for the Estimation of the Probable Maximum Precipitation (PMP) in Australia (BoM, 2003).

7.6.1 Rainfall data

IFD relationships for each sub-catchment within each hydrologic model were obtained from the Bureau of Meteorology Data Hub. Table 7.17 shows the change in catchment design rainfall depth between the 2013 and 2016 IFD tables (note that these trends are not necessarily consistent for different durations or across the entire catchment).

Table 7.17 Change in design rainfall depth from 2013 to 2016 IFD tables in Gowrie catchment

Duration (hour)	50% AEP (mm)	10% AEP (mm)	1% AEP (mm)
1	35.2 → 31.2 (-11.4%)	47.3 → 51.3(8.5%)	69.4 → 79.1 (14%)
2	44.0 → 37.7 (-14.3%)	59.0 → 61.4 (4.1%)	86.2 → 95.6 (10.9%)
3	49.2 → 41.6 (-15.4%)	66.0 → 67.2 (1.8%)	96.6 → 105.0 (8.7%)
6	59.0 → 49.2 (-16.6%)	79.2 → 78.1 (-1.4%)	116.4 → 121 (4.0%)
12	71.9 → 58.9 (-18.1%)	97.6 → 92.6 (-5.1%)	145.2 → 143.0 (-1.5%)
24	90.5 → 71.6 (-20.9 %)	124.8 → 114.0 (-8.7%)	187.7 → 176.0 (-6.2%)

IFDs are varied across the Gowrie Creek catchment. Table 7.18 shows the variation in 1% AEP rainfall intensity across different parts of catchment for various storm durations.

Table 7.18 Comparison of IFD rainfall intensity for 1% AEP in Gowrie catchment

Duration (hours)	1% AEP rainfall intensity (mm/hr)			
	IFD Area 1	IFD Area 2	IFD Area 3	IFD Area 4
1	81.2	79.5	78.5	78.3
2	49.2	48.1	47.3	47.1
3	36.0	35.2	34.5	34.4
6	20.9	20.4	19.9	19.8
12	12.4	12.1	11.7	11.6
24	7.8	7.51	7.14	6.95

7.6.2 Design rainfall losses

Rainfall losses, (rainfall that does not contribute to runoff), were applied using a standard Initial Loss/Continuing Loss model. Initial losses occur at the start of a rainfall event and are analogous to an initial wetting of the catchment that must occur before the runoff can form, such as interception and the filling of puddles and minor other depression storages within the catchment. Continuing losses represent an ongoing loss of water from the catchment, such as infiltration.

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. The pre-burst depth is a function of event duration and frequency. Recommended loss and pre-burst depths are accessed from the online ARR 2016 Data Hub.

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.

Rainfall loss parameters used in the previous studies were reviewed. In (TRC 2013a) study, impervious areas were divided into Old urban and New urban with different loss values. In (TRC 2014b) study, 40% of the urban area (low-density urban area) is considered as impervious.

As described in Section 7.2.3, in this current study, Gowrie catchment was divided into two areas as shown in Figure A-1c in Volume II – Appendix A and imperviousness was defined as follow:

- Area 1:
  - Old urban impervious: Refers to areas of the catchment that were developed for over 20 years and generally have no defined or formalised overland flow path. Initial and continuous rainfall losses for this area type were defined based on (TRC 2013a) study.
  - New urban impervious: Refers to all impervious areas except for the old urban impervious. Initial and continuous rainfall losses for this area type were defined based on TRC 2013a.
  - Pervious U/S: Refers to all pervious area. Initial and continuous rainfall losses for this area type were defined based on TRC 2013a.
- Area 2:
  - New urban impervious: Refers to all impervious area types within the Area 2. Initial and continuous rainfall losses for this area type were defined based on the TRC 2013b study.
  - Pervious D/S: Refers to all pervious area types within the Area 2. Initial and continuous rainfall losses for this area type was defined based on TRC 2013b and ARR 2016 Data Hub where applicable.

A range of applicable losses were used as specified in Table 7.19. It should be noted that these losses were used in the previous studies.

Table 7.19 Range of rainfall losses used in previous studies for 5% AEP to 1% AEP design storms

Location	Area type	Initial loss range (mm)	Continuing loss range (mm/hr)
Area 1	Old urban Impervious	8.0	4.0
	New urban Impervious	1.5	0
	Pervious U/S	37.0	2.5 to 6.0
Area 2	New urban Impervious	0 to 1.5	0
	Pervious D/S	15.0 to 40.0	1.0 to 2.5

Gowrie Creek design rainfall losses were selected as a result of an improved correlation between estimated flow from FFA results and the RAFTS model for 1% AEP event at the two streamflow gauging stations, Oakey and Cranley.

For the 1 in 10,000 AEP and PMF events zero initial and 1.6 mm/hr continuous rainfall loss rates were assumed for all areas. Comparison between FFA and the hydrologic model results was not undertaken for the 1 in 10,000 AEP event as the number of recorded peak flows at Oakey and Cranley gauges were not sufficient to estimate a rare event peak flow. Adopted design rainfall losses are summarised in Table 7.20.

Table 7.20 Adopted design rainfall losses for Gowrie Creek for 1% AEP design event

Location	Area type	Initial loss (mm)	Continuing loss (mm/hr)
Area 1	Old urban Impervious	8.0	4.0
	New urban Impervious	1.0	0
	Pervious US	37.0	4.0
Area 2	New urban Impervious	1.0	0
	Pervious DS	15.0	2.5

7.6.3 Routing parameters

The parameter ‘K’ is a storage constant expressing the ratio between storage and flow and is usually expressed in hours. It may also be viewed as the lag or travel time through each reach. The dimensionless parameter ‘X’ is indicative of the relative importance of inflow and outflow to storage.

Routing parameters as adopted for this assessment were mainly based on the calibration parameters. The selected value for X is 0.25, which is a typical value for natural streams. The K values selected were lag time in hours for each routing link based on assumed flow velocity. The routing parameter K was slightly adjusted to account for large flow for a 1% AEP event. The TUFLOW hydraulic model was used to adjust the routing parameter K further to match the peak time in the hydrologic model with the modelled peak time in the hydraulic model.

7.6.4 Design flow estimation

7.6.4.1 Hydrologic model peak flow assessment

A critical duration assessment was undertaken to determine which storm duration(s) produced peak flood flows at the stream flow gauges and at major waterways that are intersected by the Project alignment. To assess the critical storm duration the following methodology was adopted:

- The models were modelled for a range of AEP events: 20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 and PMF:
  - Each AEP was modelled for a range of durations from 30 minutes to 12 hours design storm events

  - Each duration was modelled for each of the ten associated temporal patterns
- Peak water levels were mapped for each storm duration
- A critical duration assessment was undertaken at the key locations to determine which duration produced the highest median flow of the ten temporal patterns for each event

7.6.4.2 Design events

The design flow for the 1% AEP and extreme events were estimated using the calibrated hydrologic model. Comparison was undertaken between peak flows modelled by the hydrologic model and the FFA results and previous studies to ensure the adopted loss values were within accepted ranges.

Table 7.21 compares the 1% AEP peak flow modelled by the hydrologic model with FFA results at the Oakey and Cranley gauges. The results show that the difference between FFA and the hydrologic model is less than 10% for both gauges.

Table 7.21 Comparison between peak flows from flood frequency analysis with hydrologic model flows

Stream gauge	FFA flow for 1% AEP (m^3/s)	Hydrologic model 1% AEP flow (m^3/s)	Difference %
Oakey	705[1] - 780[2]	708[3]	+0.4% to -9.3%
Cranley	369	404[3]	+9.3%

Table notes:

1 LP3 probability model – using original data
2 LP3 probability model – using revised data
3 Median value of all 10 temporal patterns (bursts) for the critical duration

A comparison between peak flows modelled in this current investigation and the TRC (2014b) study at two locations, Cotswold Hill East catchment (C17) and Cotswold West Catchment (C31) is shown in Table 7.22. The differences in results between the two studies may be because of the following factors:

- The previous study used ARR 1987 while this assessment uses ARR 2016 data and methodologies
- The critical duration adopted in the previous study was 60 minutes while in this assessment it is 90 minutes. Assumed velocities for channels were higher in the previous model in comparison to this assessment.
- The previous study did not use areal reduction factors and as a result the applied rainfall intensity was higher than in this assessment
- This assessment uses a routing method for channel flow routing in the hydrologic model while the previous model used the lagging method
- The previous model had focused on the Cotswold catchment and was not calibrated to any stream gauges while this assessment focuses on the Gowrie Creek catchment and was calibrated against two historical events.

Table 7.22 Comparison between modelled peak flow in the hydrologic model with TRC (2014b) study

Event	Cotswold Hill East (C17)		Cotswold Hill West (C31)	
	TRC (2014b) (m^3/s)	B2G hydrologic model (m^3/s)	TRC (2014b) (m^3/s)	B2G hydrologic model (m^3/s)
January 2011	120	114	71	80
1% AEP event	141	103[1]	108	89[1]

Table note:

1 Median of all 10 temporal patterns (bursts) for critical duration of 90 minutes

7.7 Hydraulic modelling – Gowrie Creek

7.7.1 Design event modelling

Two hydraulic models were developed, representing the current state of development (i.e. Existing Case) and a scenario where the Project alignment is implemented including proposed drainage structures and associated infrastructure works (i.e. Developed Case). The design event flows were modelled in the hydraulic model for both cases for the suite of AEP events (20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 and PMF).

The modelled hydrographs from the hydrologic model for the design events were exported and used as flow boundaries in the hydraulic model. Total flow of the Gow1.05 sub-catchment was applied as the upstream flow boundary to the hydraulic model. The exported hydrographs for the rest of the sub-catchments within the hydraulic model domain were applied as internal flow boundaries.

A critical duration assessment was undertaken to determine which storm duration produced peak water levels across the hydraulic model and more specifically the study area. The hydrologic modelling results for the 1% AEP and 1 in 10,000 AEP events at key locations are presented in Table 7.23 and

Table 7.24, respectively. 1% AEP peak water levels and velocities are presented in Figure A-3e and Figure A-4a in Volume II – Appendix A, respectively.

Table 7.23 Peak flow for 1% AEP event at key locations

Sub-catchment name	Peak flow (m^3/s)	Critical storm duration
U/S model boundary	433	120 minutes

Table 7.24 Peak flow for 1 in 10,000 AEP event at key locations

Sub-catchment name	Peak flow (m^3/s)	Critical storm duration
U/S model boundary	1,343	120 minutes

The critical durations for each AEP at Old Homebush Rd are outlined in Table 7.25.

Table 7.25 Critical duration assessment for Gowrie Creek hydraulic model

Location	Event	Duration (minutes)	Peak flow (m^3/s)
Old Homebush Road	20% AEP	720	191
Old Homebush Road	10% AEP	360	257
Old Homebush Road	5% AEP	180	336
Old Homebush Road	2% AEP	120	446
Old Homebush Road	1% AEP	120	534
Old Homebush Road	1 in 2,000 AEP	90	980
Old Homebush Road	1 in 10,000 AEP	120	1,724

7.8 Existing Case modelling results – Gowrie Creek

7.8.1 Existing Case flood maps

Flood maps illustrating indicative flood extents and peak water levels were prepared and are presented in Volume II – Appendix A:

- 20% AEP: Figure A-3a
- 10% AEP: Figure A-3b

- 5% AEP: Figure A-3c
- 2% AEP: Figure A-3d
- 1% AEP: Figure A-3e
- 1 in 2,000 AEP: Figure A-3f
- 1 in 10,000 AEP: Figure A-3g
- PMF: Figure A-3h.

Figure A-4a presents peak flood velocities for the 1% AEP event.

7.8.2 Flood inundation extent and flood levels

Figure A-3e in Volume II – Appendix A shows the 1% AEP peak water levels and flood inundation extent for the Gowrie Creek floodplain for the Existing Case. The peak depth in the main channel of Gowrie Creek is estimated at greater than 6 m and the 1% AEP flood appears to be contained in-channel upstream of Gowrie Junction Road.

The existing QR line runs parallel to the proposed rail on the northern side between Draper Road and Ganzer Morris Road. The top of rail earthen embankment is defined as the rail formation level. The top of the rail is approximately 0.7 m above the formation level, which includes a 0.5 m depth of ballast and a 0.2 m high rail. The 1% AEP event overtops the existing QR rail formation in several sections.

7.8.3 Flood immunity of existing infrastructure

Table 7.26 presents a summary of overtopping depths for the existing QR rail line and key roads near the Project alignment under a range of design events. The overtopping depths for the QR rail line are estimated levels above the rail formation level.

Table 7.26 Gowrie Creek – Existing Case – overtopping depths of key infrastructure

Infrastructure	Location	Maximum overtopping depth (m)							
		PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Chamberlain Road	At the end of road near Gowrie Creek	3.99	2.15	1.18	0.70	0.63	0.50	0.33	0.27
Kingsthorpe-Tilgonda Road	Near Gowrie Creek (about 650 m west of Lessons Road)	4.62	3.06	2.08	1.09	0.90	0.75	0.45	0.17
Leeson Road	Gowrie Creek Crossing	3.62	2.11	1.45	0.94	0.87	0.75	0.65	0.53
Existing QR Rail Line	About 950 m west of Gowrie Creek/QR Rail Line crossing	1.14	0.54	0.45	0.25	0.23	0.17	-	-

7.8.4 Existing Case velocities

Peak Existing Case velocities for the 1% AEP event in the Gowrie Creek channel are high, in the order of 2 to 5 m/s. On the floodplain velocities are generally in the order of 1 to 2 m/s as shown in Figure A4-a in Volume II – Appendix A.

7.9 Developed Case modelling results – Gowrie Creek

7.9.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW model (1D and 2D approach).

On the Gowrie Creek floodplain, the Project includes the following floodplain (or regional structures):

- Nine reinforced concrete pipe (RCP) locations (a total of 51 cells)
- One reinforced concrete box culvert (RCBC) location (a total of 16 cells)
- Two rail-over-road bridges that also convey flows within minor drainage lines.

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and depending on their interaction with the Gowrie Creek floodplain were incorporated in the hydraulic model.

A blockage factor of 25% was applied to all proposed culverts based on guidelines set out in ARR 2016. The adopted blockage factor for the proposed bridges was between 5% and 10% based on the waterway area blockage due to bridge piers.

The proposed drainage structures are summarised in Table 7.27 and Table 7.28 and shown in Figure A-1f in Volume II – Appendix A.

Table 7.27 Gowrie Creek – proposed rail bridge locations and details

Chainage (km)	Structure ID	Type	Diameter/ bridge length (m)	Bridge soffit level (m AHD)
203.06	310-BR34	Warrego Highway Rail Bridge	132	465.3
204.46	310-BR35	Chamberlain Road Rail Bridge	299	481.3

Table 7.28 Gowrie Creek – proposed floodplain culvert locations and details

Chainage (km)	Structure ID	Type	U/S invert level (m AHD)	D/S invert level (m AHD)	Diameter/ width (m)	Height (m)	Number of cells
203.17	C203.17	RCP	471.40	469.50	1.05	-	2
204.92	C204.92	RCP	446.90	446.35	1.05	-	2
205.09	C205.09	RCP	446.40	446.20	1.05	-	12
205.14	C205.14	RCP	446.80	445.86	1.05	-	2
205.30	C205.30	RCP	447.06	446.10	1.05	-	4
205.37	C205.37	RCP	447.18	446.98	1.05	-	15
205.47	C205.47	RCP	448.09	447.40	1.05	-	5
205.60	C205.60	RCP	448.45	447.75	1.05	-	2
205.87	C205.87	RCP	449.20	448.80	1.05	-	7
206.95	C206.95	RCBC	456.80	456.40	2.40	1.20	16

7.9.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

7.9.2.1 Rail immunity and structures results

Appendix C summarises the peak 1% AEP water levels on the upstream side of the proposed alignment. Local drainage structures (i.e. those not included in the flood model) are not reported.

The results of flood modelling indicate that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the Gowrie Creek floodplain. There is over 0.5 m freeboard above the culvert obvert levels to the rail formation in a 1% AEP event, except for culvert C205.87 which has a freeboard of 0.2 m.

7.9.2.2 Velocities and scour

Appendix C summarises the peak 1% AEP outlet flows and velocities at structures. The 1% AEP peak velocity through the proposed drainage structures is generally less than 2.0 m/s.

Scour protection requirements for culverts were calculated based on the velocities predicted from the hydraulic modelling. The scour protection was 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.

Table 7.29 lists the soil types encountered along the B2G alignment and the allowable soil velocity based on AGRD.

Table 7.29 Allowable soil velocities along the Project alignment

Soil type	Soil description	Maximum allowable soil velocity as per AGRD (m/s)
Sodosols	Firm loam or stiff clay	2 m/s
Vertosols, Dermosols, Rudosols and Kandosols	Sandy or silty clay	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.

Scour protection requirements for culverts have been calculated based on the velocities predicted from the hydraulic modelling. Appendix D presents the proposed scour protection at each structure.

A conservative scour estimation has been undertaken at the bridge site based on available information and will be refined during detailed design.

7.9.2.3 Flood immunity for extreme events

The risk of overtopping of the rail alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF) with Table 7.30 presenting the depth of water above the formation level and over the top of rail at each structure. It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the alignment. As such, overtopping of the rail is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario.

Table 7.30 Gowrie Creek – extreme events – depth of water above formation level and/or over top of rail level

Chainage (km)	Structure ID	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
203.06	310-BR35	-	-	-	-	-	-
203.17	C203.17	-	-	-	-	-	-
204.46	310-BR34	-	-	-	-	-	-
204.92	C204.92	-	-	-	-	-	-
205.09	C205.09	-	-	-	-	-	-
205.14	C205.14	-	-	-	-	-	-
205.30	C205.30	-	-	-	-	-	-
205.37	C205.37	-	-	-	-	-	-
205.47	C205.47	-	-	-	-	-	-
205.60	C205.60	-	-	-	-	-	-
205.87	C205.87	-	-	1.5	-	-	0.8
206.95	C206.95	-	-	0.6	-	-	-

7.9.3 Flood impact objectives outcomes – Gowrie Creek

The impact of the Developed Case was assessed through inclusion of the proposed rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, duration of inundation, velocities and peak flows within the floodplain:

- Changes in peak water levels for the AEP's assessed are presented in Figures A-5a to A-5h in Volume II – Appendix A
- Changes in 1% AEP duration of inundation are presented in Figure A-5i in Volume II – Appendix A
- Changes in 1% AEP velocities are presented in Figure A-5j in Volume II – Appendix A.

All impacts are required to be agreed with relevant stakeholders and affected landowners as part of the EIS process. One-one-one consultation with landowners who are expected to experience changes in flooding behaviour on the property was conducted by ARTC supported by FFJV.

The Project design outcomes relative to the flood impact objectives (refer Table 4.2) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Volume II – Appendix N.

7.9.3.1 Flood impacts at proposed hydraulic structures

The estimated potential impacts on peak water levels at each proposed structure is presented in Table 7.31 for the 1% AEP event. Peak water levels were extracted immediately upstream of each culvert and at the control line of each bridge.

The design achieved a freeboard of at least 300 mm between the 1% AEP peak water and formation levels.

Table 7.31 Gowrie Creek – 1% AEP event – estimated impacts to peak water levels at proposed hydraulic structures

Chainage (km)	Structure ID	Structure type	Rail formation level or bridge deck height (m AHD)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Change in peak water level (mm)
203.06	310-BR35	Rail Bridge	483.15	473.58	473.87	+290
203.17	C203.17	RCP	482.86	471.92	471.99	+70
204.46	310-BR34	Rail Bridge	468.24	449.80	449.80	-
204.92	C204.92	RCP	462.46	447.26	447.40	+140
205.09	C205.09	RCP	460.14	446.87	447.19	+320
205.14	C205.14	RCP	459.38	447.04	447.27	+230
205.30	C205.30	RCP	457.53	447.39	447.72	+330
205.37	C205.37	RCP	456.67	447.47	447.93	+460
205.47	C205.47	RCP	455.56	-	448.55[1]	-
205.60	C205.60	RCP	453.98	449.10	449.33	+230
205.87	C205.87	RCP	450.97	449.88	450.36	+480
206.95	C206.95	RCBC	459.72	458.13	458.36	+230

Table note:

1 Due to overbank flow, moreover this culvert is assessed separately as local flow is dominant at this culvert

7.9.3.2 Flood impacts on flood sensitive receptors

Based on the available aerial imagery, no buildings or critical infrastructure are located within the area predicted to be impacted by changes in peak water levels on the Gowrie Creek floodplain for events up to the 1% AEP.

7.9.3.3 Flood impacts on Queensland Rail

No impacts on the existing Queensland Rail line to the north of the Project alignment are expected in a 1% AEP event.

7.9.3.4 Flood impacts on state-controlled roads

The following sections describe the impacts to state-controlled roads in both the Existing Case and the Developed Case and summarises the differences between the two. The reporting locations used in the following result summaries are shown in Figure 10.

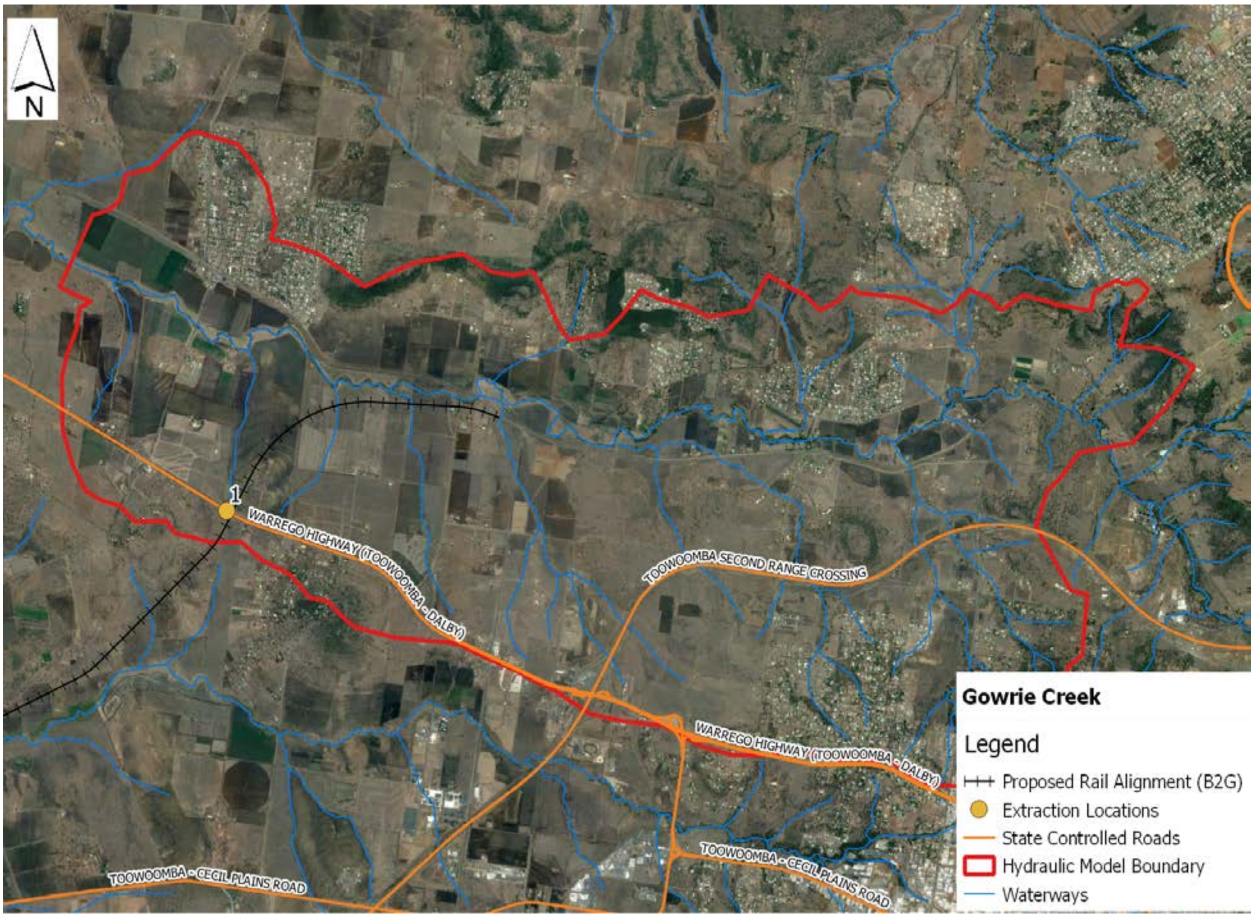

Figure 10 Gowrie Creek – hydraulic model extent and associated state-controlled roads

Based on Existing Case model results, the Warrego Highway exhibits 5% AEP immunity in regard to regional Gowrie Creek flooding. Overtopping depths in the 2% and 1% AEP events are typically less than 100 mm and in segments of 10 to 35 m in width.

Existing Case flooding conditions

Table 7.32 Gowrie Creek – Existing Case flood depths

Reporting location	Road	Estimated depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
1	Warrego Highway	0.00	0.00	0.00	0.08	0.09	0.12	0.14	0.22

Table 7.33 Gowrie Creek – Existing Case flood inundation length

Reporting location	Road	Approximate length of inundation (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
1	Warrego Highway	0	0	0	8	35	60	67	87

Table 7.34 Gowrie Creek – Existing Case time of submergence

Reporting location	Road	Estimated time of submergence (hrs)								AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
1	Warrego Highway	0	0	0	3.8	3.3	5.4	3.5	3.9	0.14

Developed Case flooding conditions

Table 7.35 Gowrie Creek – Developed Case flood depths

Reporting location	Road	Estimated depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
1	Warrego Highway	0.00	0.00	0.00	0.08	0.09	0.12	0.14	0.22

Table 7.36 Gowrie Creek – Developed Case flood inundation length

Reporting location	Road	Approximate length of inundation (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
1	Warrego Highway	0	0	0	8	34	60	64	82

Table 7.37 Gowrie Creek – Developed Case time of submergence

Reporting location	Road	Estimated time of submergence (hrs)								AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
1	Warrego Highway	0	0	0	1.7	1.9	5.4	2.6	2.8	0.08

Impacts of Project alignment

Table 7.38 Gowrie Creek – change in flood depths

Reporting location	Road	Estimated change in depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
1	Warrego Highway	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table 7.39 Gowrie Creek – change in time of submergence

Reporting Location	Road	Estimated change in time of submergence (hrs)								Estimated change in AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
1	Warrego Highway	0	0	0	-2.1	-1.4	0.0	-0.9	-1.1	-0.1

Change in flood hydrographs

Figure 11 shows the Existing Case and Developed Case 1% AEP water level time series result at the sag point of the Warrego Highway. As shown by the figure, there is negligible difference between the time series results.

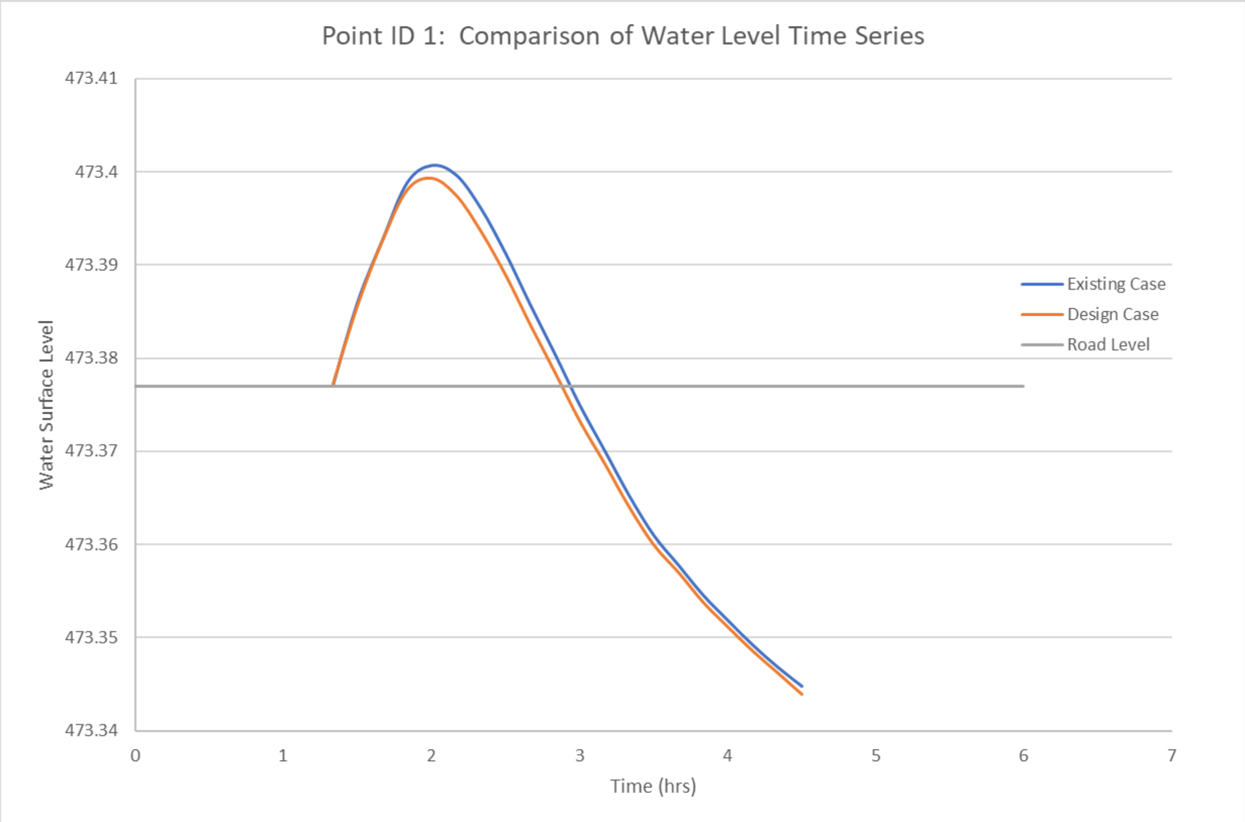

Figure 11 Extraction Point 1 – comparison of water level time series, 1% AEP

7.9.3.5 Flood impacts on local public roads

The change in peak water levels and flood hazard (velocity-depth) for the 1% AEP event were evaluated on local public roads within the hydraulic model domain. Local public roads that are expected to experience an increase in flood hazard and/or increases in peak flood levels are reported in Table 7.40.

Table 7.40 Gowrie Creek – changes in peak water levels and velocity depth and flood hazard for local public roads, 1% AEP

Location	Existing flood hazard (m^2/s)	Design flood hazard (m^2/s)	Existing maximum flood depth (m)	Design maximum flood depth (m)	Maximum change in peak water levels (mm)[1]
Draper Road	1.66	2.88	0.84	1.07	+230
Kingsthorpe Haden Road	5.57	5.58	1.89	1.89	+3
Kingsthorpe Tilgonda Road	4.19	4.20	1.27	1.27	+5
Tilgonda Kingsthorpe Road	5.27	5.27	1.89	1.89	+2

Table note:

1 The maximum change in peak water level does not necessarily occur at the same location as where the existing and/or design maximum flood depth occur

Duration of inundation

Assessment of the time of submergence (ToS) and average annual time of submergence (AAToS) was undertaken for local public roads within the hydraulic model domain. Local public roads that are expected to experience an increase in ToS and/or AAToS are presented in Table 7.41.

Table 7.41 Gowrie Creek – ToS and AATOS for local public roads

Location	Existing 1% AEP ToS (hrs)	1% AEP ToS diff. (hrs)	2% AEP ToS diff. (hrs)	5% AEP ToS diff. (hrs)	10% AEP ToS diff. (hrs)	AAToS Existing Case (hrs)	AAToS Developed Case (hrs)	AAToS diff. (hrs)
Draper Road	5.31	-	-	-	-	5.13	5.69	0.56
Ganzer Morris Road	4.56	0.16	0.23	2.19	1.52	0.73	1.39	0.66
Leesons Road	4.67	-	-	-	-0.03	5.80	5.80	-0.01
Paulsens Road	5.37	-0.01	0.10	-0.02	0.07	7.78	7.79	0.01

7.9.3.6 Flood impacts on private land outside the rail disturbance footprint

Most of the area where afflux is predicted is agricultural land or open land on which nominal afflux is unlikely to cause any adverse impact. Table 7.42 presents the modelled changes in flood conditions during the 1% AEP event on a lot basis according to the following thresholds:

- Peak water levels increased by greater than +10 mm
- Peak velocities increased by greater than 0.25 m/s
- Duration of inundation changed by more than 25% of its original duration of inundation across the lot.

Table 7.42 Gowrie Creek – summary of flood impacts on private land outside the rail disturbance footprint for 1% AEP

Approximate chainage (km)	Changes in peak water levels[1]		Changes in peak velocities		Changes in duration of inundation	
	Maximum change (mm)	Total area affected by change > 10 mm (ha)	Maximum change (m/s)	Total area affected by change (ha)	Maximum change (%)	Total area affected by change (ha)
206.65 to 206.95	+410[2]	1.43	+0.79	0.05	-	-
206.70 to 206.90	+70	0.02	+0.79	0.05	-	-
205.30 to 205.85	+260	2.87	+0.68	0.12	-	-
205.85 to 206.05	+200	0.17	-	-	-	-
204.50 to 205.30	+684[2]	6.76	+0.85	1.13	-	-

Table notes:

1 Afflux on lots that exceed the flood impact objectives are summarised in the EIS Surface Water Chapter
2 Area affected by afflux exceeding 200 mm is contained in a small area within existing creek channels

7.9.3.7 Flow distribution

A key landowner concern is changes to flow distributions. To understand the magnitude of these flowpaths, flows were extracted from the hydraulic model at key locations. The difference between the Existing Case and Developed Case was considered and reported in Table 7.43. The results indicate negligible changes in a 10% AEP event, and minimal changes in a 1% AEP event.

Figure 12 presents the selected flowpath comparison locations. The flow is calculated across the length of the line. Therefore, the lines presented are either calculating the flow across the width of the floodplain (for the longer flow lines) or the main flowpath of the waterways (generally for smaller flow lines).

Table 7.43 Gowrie Creek - flow comparison

Flow location ID	10% AEP			1% AEP		
	Existing Case flow (m^3/s)	Developed Case flow (m^3/s)	% Change	Existing Case flow (m^3/s)	Developed Case flow (m^3/s)	% Change
L1	11.45	11.44	-0.08	35.94	35.94	-
L2	19.34	19.45	0.54	65.08	65.11	0.04
L3	227.22	227.14	-0.04	485.37	485.35	-
L4	256.88	255.50	-0.54	532.16	532.82	0.12
L5	20.59	20.60	0.06	68.90	68.84	-0.08
L6	288.04	287.83	-0.07	600.12	603.37	0.54
L7	12.60	12.46	-1.17	28.06	28.03	-0.10
L8	300.05	299.84	-0.07	629.06	608.94	-3.20
L9	15.43	15.47	0.28	57.25	57.22	-0.06
L10	5.05	5.04	-0.16	20.07	20.02	-0.27
L11	322.65	322.46	-0.06	694.52	695.44	0.13
L12	325.84	325.99	0.05	709.39	715.97	0.93
L13	3.44	3.44	0.03	8.03	8.02	-0.14
L14	3.15	3.14	-0.20	10.70	10.26	-4.12
L15	328.77	328.91	0.04	747.48	749.27	0.24

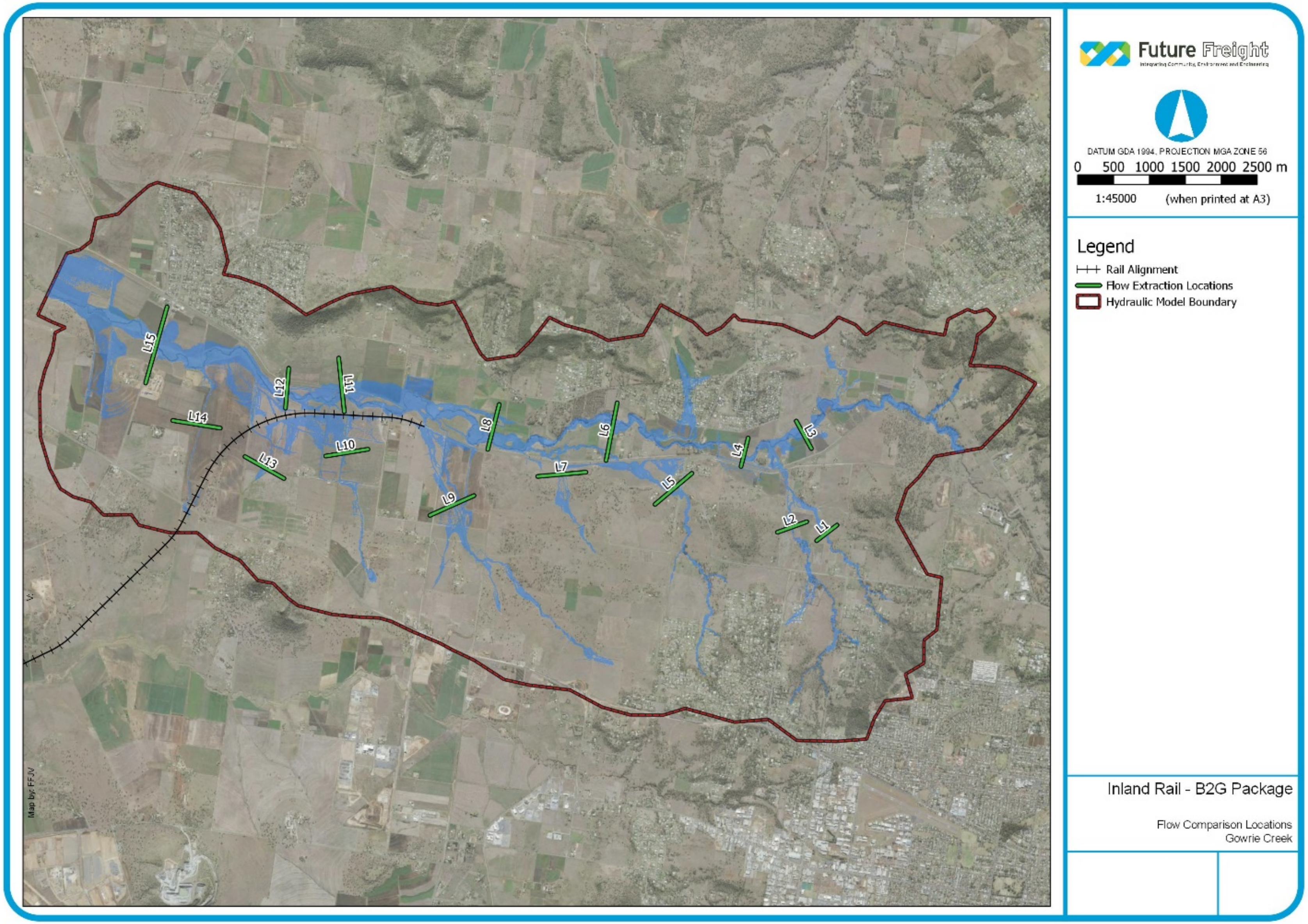

Figure 12 Gowrie Creek – flow comparison locations

7.9.4 Sensitivity analysis – Gowrie Creek

The sensitivity of the model to various parameters was assessed using the following three scenarios:

- An increase in rainfall intensity i.e. to reflect climate change scenario
- Increase in blockage of culverts from 25% to 50%
- Decrease in blockage of culverts from 25% to 0%.

7.9.4.1 Blockage

Blockage was assessed in accordance with ARR 2016. The blockage assessment undertaken resulted in a blockage factor of 25% being adopted for culverts. A minimum culvert size of 900 mm diameter was adopted to reduce potential for blockage and maintenance. A significant 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% and 50% blockage.

There is little change to the predicted impacts on the downstream and upstream afflux as a result of reducing the applied culvert blockage allowance to 0%. As a result of increasing the blockage factor to 50%, minor increases are predicted in localised areas upstream of the alignment in particular at Culvert C205.09 where predicted increases in flood levels directly upstream exceed 200 mm.

The adopted blockage factor for the proposed bridges was between 5% and 10% based on the waterway area blockage due to bridge piers.

Two blockage sensitivity scenarios were tested with both 0% and 50% blockage of all culverts. Table 7.44 provides a summary of 1% AEP peak flood levels at cross drainage structures for the blockage scenarios.

Table 7.44 Gowrie Creek – 1% AEP event – culvert blockage assessment

Structure ID	Structure type	1 % AEP Peak water levels (m AHD)			Increase from Developed Case to 50 % blockage scenario (mm)
		0 % blockage	Developed Case (25 % blockage)	50 % blockage	
C203.17	RCP	471.96	471.99	472.05	+60
C204.92	RCP	447.39	447.40	447.40	-
C205.09	RCP	447.13	447.19	447.41	+220
C205.14	RCP	447.23	447.27	447.41	+140
C205.30	RCP	447.67	447.72	447.78	+60
C205.37	RCP	447.88	447.93	448.01	+80
C205.47	RCP	448.54	448.55	448.57	+20
C205.60	RCP	449.32	449.33	449.35	+20
C205.87	RCP	450.18	450.24	450.32	+80
C206.95	RCBC	450.30	458.36	458.53	+170

Table 7.45 outlines the changes in peak water levels at flood sensitive receptors for the 50% blockage scenario where the increase exceeds 10 mm.

Table 7.45 Gowrie Creek – Summary of 50% blockage impacts at flood sensitive receptors

Flood sensitive receptor ID	Existing case flood depth (m)	Change in peak water level (mm)
Draper Rd	1.07	+260
Kingsthorpe Haden Rd	4.95	+10
Chamberlain Rd	0.40	+280

Maps demonstrating the impacts of blockage are shown in Volume II – Appendix A, Figures A-6a (0%) and A-6b (50%).

During detail 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.

7.9.4.2 Impacts during extreme events

Table 7.46 outlines the changes in peak water levels at flood sensitive receptors for the extreme events where the increase exceeds 10 mm under one of the events. The existing depth of flooding is also detailed and as can be seen the larger impacts that occur under the PMF event occur generally when there are already high flood depths as would be expected under such a rare event.

Flood immunity of the Project alignment is discussed in Section 7.9.2.3, and maps demonstrating the impacts during extreme events are shown in Volume II – Appendix A, Figures A-5f to A-5h.

Table 7.46 Gowrie Creek – Summary of extreme event impacts at flood sensitive receptors

Flood sensitive receptor ID	1 in 2,000 AEP event		1 in 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)
GOW_ID_2	-	-	+39	0.14	+77	0.75
GOW_ID_4	-	-	-	-	+264	0.74
GOW_ID_5	-	-	-	-	+240	0.15
GOW_ID_6	-	-	-	-	+96	0.21
GOW_ID_7	-	-	-	-	+51	0.07
Kingsthorpe Tilgonda Road	+52	2.47	+161	3.42	+449	4.98
Existing QR Rail Line	+40	0.25	+100	1.20	+430	2.76
Leesons Road	+17	2.80	+202	4.11	+308	6.10
Tilgonda Kingsthorpe Road	-	2.17	+21	2.63	+432	3.90
Draper Road	+549	1.31	+699	1.62	+517	3.09
Kingsthorpe Haden Road	+15	5.45	+1	6.25	-30	8.02
Chamberlain Road	+281	0.42	+295	1.16	+1,898	3.00

7.9.4.3 Climate change

The potential impacts of climate change in the Gowrie Creek floodplain were assessed for the 1% AEP event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2016.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario has been adopted for the Project with an associated increase in rainfall intensity of 21% across the catchment area.

For the 1% AEP event, the change in peak water levels for the Representative Concentration Pathways 8.5 climate change scenario is presented in Figure A-6c in Volume II – Appendix A. The change in peak water levels is calculated from the difference between the Developed Case and the Existing Case with 21% increase to rainfall intensity applied to both cases.

The hydraulic model predicts that, with an increase in rainfall intensity of 21% across the catchment, peak water levels upstream of the Project alignment are likely to increase by up to 0.5 m between Chainages 204.70 km and 205.15 km. The Project alignment is not predicted to overtop under the Representative Concentration Pathways 8.5 climate change scenario.

Table 7.47 presents the structure performance under Representative Concentration Pathways 8.5 climate change conditions.

Table 7.47 Gowrie Creek – 1% AEP event with Representative Concentration Pathways 8.5 conditions – structure performance

Structure ID	Structure type	U/S peak water level (m AHD)	Freeboard to formation level (m)	Outlet velocity (m/s)	Peak flow (m^3/s)
310-BR35	Rail Bridge	473.90	9.3	0.7	3.4
C203.17	RCP	472.13	10.7	1.6	1.5
310-BR34	Rail Bridge	449.81	18.4	1.4	13.9
C204.92	RCP	447.47	15.0	1.4	1.0
C205.09	RCP	444.48	15.7	1.8	12.8
C205.14	RCP	447.49	11.9	1.6	1.4
C205.30	RCP	447.82	9.7	1.6	3.1
C205.37	RCP	448.06	8.6	1.7	12.2
C205.47	RCP	448.65	6.9	1.2	1.8
C205.60	RCP	449.40	4.6	1.9	2.2
C205.87	RCP	450.38	0.6	2.3	10.6
C206.95	RCBC	458.66	1.1	1.9	64.4

Table 7.48 outlines the changes in peak water levels at flood sensitive receptors for the climate change scenario where the increase exceeds 10 mm.

Table 7.48 Gowrie Creek – Summary of climate change impacts at flood sensitive receptors

Flood sensitive receptor ID	1% AEP climate change event	
	Change in peak water level (mm)	Existing case flood depth (m)
Draper Road[1]	+332	1.07
Kingsthorpe Haden Road[1]	+37	4.95
Chamberlain Road[1]	+285	0.40

Table note:

1 These roads are affected by climate change regardless of the Project and so the amenity of the roads is not compromised by the Project.

The downstream extents of these impacts are similar to those under the 1% AEP event.

8 Westbrook and Dry Creeks

The Project alignment crosses Westbrook Creek and Dry Creek to the west of Toowoomba Wellcamp Airport. The western section of the Toowoomba Wellcamp Airport is protected by a flood levee. The proposed Inland Rail crossing location is approximately 800 m upstream of the confluence of Westbrook Creek and Dry Creek. The combined Existing Case 1% AEP event inundated floodplain width at the proposed crossing point is approximately 1.7 km. At the Brimblecombe Road crossing of Dry Creek the inundated floodplain width is approximately 500 m. At the Toowoomba-Cecil Plains Road crossing of Westbrook Creek the inundated floodplain width is approximately 800 m.

The Westbrook Creek and Dry Creek floodplains are well defined with a few minor localised breakouts and small tributary drainage lines. Under the 1% AEP event, around the Project alignment crossing of Westbrook Creek, the peak depth of water is approximately 4.5 m in the channel and up to 3 m on the floodplain. At Dry Creek the 1% AEP event water depth in the channel is around 3.5 m deep and up to 1 m on the floodplain.

The location of the Project rail alignment in relation to Westbrook Creek and Dry Creek is shown in Figure B-1a in Volume II – Appendix B.

8.1 Data collection and review – Westbrook and Dry Creeks

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:

- TRC – existing flood studies and stream gauging data
- The BoM – rainfall and stream gauging data
- DNRME – stream gauging data
- QR – existing infrastructure details
- DTMR – existing infrastructure details.

8.1.1 Previous studies

A number of previous hydrology and hydraulic studies were sourced as part of this assessment. A review of each study was undertaken to determine suitability for use on the Project as documented in the following sections.

Work Package 8, 2D Flood Study for Westbrook Final Report, DHI/TRC 2014

This study focused on Westbrook town and included the development of a 2D rain-on-grid MIKE21 hydraulic model. A hydrologic model was not developed for this study.

Spring Creek Flood Study - Rev 2, TRC 2017

This study focused on the upstream catchment area of Spring Creek and reported on estimated flow at a location 10 km upstream of the Inland Rail study corridor. The study focuses on a small localised area with catchment characteristics (e.g. urbanisation and catchment slope) different to the overall Westbrook Creek catchment.

2D Flood Study for Dry Creek, TRC 2015

This study focused on the upstream catchment area of Dry Creek and reported on estimated flow at a location 7 km upstream of the Inland Rail study corridor. The study focuses on a small localised area with catchment characteristics (e.g. urbanisation and catchment slope) different to the overall Westbrook Creek catchment.

Engineering Report Hydraulic Assessment – Westbrook Creek Wellcamp Project No 10525, RMA Engineers Pty Ltd 2015

A hydraulic assessment was undertaken in support of the Wellcamp Airport development at 1511 Toowoomba-Cecil Plains Road. The study explored the impacts of Westbrook Creek on the Wellcamp Airport site. To support the study an XP-STORM model was developed for Westbrook Creek and calibrated against the ARR Regional Flood Frequency Estimation (RFFE) model. A 2D hydraulic model was developed in TUFLOW covering an area from upstream of the Westbrook and Dry Creek confluence to upstream of the Wellcamp Airport site.

8.1.2 Survey

ARTC provided LiDAR data from 2015 as 1 m grid DEM tiles. Using GIS software, a DEM was generated with a 1 m grid resolution for use in the Project based on the 2015 dataset. This was used for modelling within the disturbance footprint and up to the full extent of the 2015 LiDAR where relevant.

Additional LiDAR data extents were required to appropriately model downstream boundary conditions and facilitate calibration against streamflow gauges. In areas that were not covered by the LiDAR provided by ARTC, LiDAR tiles were sourced from Geoscience Australia. The DEM datasets utilised for modelling were based on surveys flown between 2009 and 2015, with preference given to the most recent data available.

SRTM data was used for catchment delineation where no LiDAR data could be sourced, to inform the hydrologic modelling.

The survey data sources and DEM developed for Gowrie Creek is shown in Figure B-1b in Volume II – Appendix B.

8.1.3 Aerial imagery

Aerial imagery of the study area was provided by ARTC and was used to identify and confirm topographic and vegetative characteristics of the study area. Aerial imagery captured in 2015 was made available. Additional imagery outside the study area was sourced from QGIS imagery in an open source format.

8.1.4 Existing drainage structure data

At the time of documenting this report, there was no information available on existing drainage structures within the Westbrook Creek and Dry Creek catchments. Critical locations were identified based on existing flowpaths and aerial imagery. The DEM was modified at several locations to represent natural drains at road or embankment crossing locations where the existing cross-drainage structure data was not available (refer Section 8.3.3).

8.1.5 Streamflow gauges

There are no streamflow gauges located in the Westbrook Creek or Dry Creek catchments.

8.1.6 Rainfall data

A number of daily and sub-daily rainfall stations are located in and around the Westbrook Creek and Dry Creek catchments. However, no historical rainfall data was sought for calibration purposes because other required data for calibration was not available.

8.1.7 Anecdotal and observed flood data

No anecdotal or observed flood data was available for this area of Westbrook and Dry Creeks.

8.1.8 Site inspection

A site inspection was undertaken during February 2018. During the site inspection, all major waterway crossings were visited and inspected with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

8.2 Hydrologic model development – Westbrook and Dry Creeks

8.2.1 Model setup

A hydrologic model for the combined Westbrook Creek and Dry Creek catchments was developed using RAFTS software Version 2018.1.2. The RAFTS software is a rainfall runoff routing model that estimates catchment flows generated by rainfall depths applied to a network of parameterised sub-catchments and waterways.

As part of the Inland Rail project hydrology and hydraulic assessment, a RAFTS hydrologic model was developed and calibrated for the Gowrie Creek catchment located to the north of the Westbrook and Dry Creeks catchment. For the development of the Westbrook and Dry Creeks RAFTS model, the calibrated parameters of the Gowrie Creek hydrologic model were used as guidance along with the parameters presented in the previous studies and relevant data downloaded from the ARR 2016 Data Hub. Other adopted parameters are discussed in the following sections.

The Westbrook and Dry Creeks RAFTS hydrologic model was validated against the RFFE model flow predictions. The estimated flows were also compared with reported flows in the previous studies where the information was available.

8.2.2 Sub-catchments

Catchment-based information used in the hydrologic model depends on the geographic, physical and topographic characteristics. The catchment data includes catchment area, catchment slope, fraction impervious, catchment roughness and routing parameters.

The delineation of sub-catchments for the Westbrook and Dry Creeks catchment was based on topographic data (SRTM data) using CatchmentSIM 3.5 software.

The hydrologic model extent and sub-catchment map is presented in Figure B-1c in Volume II – Appendix B.

8.2.3 Fraction impervious and roughness

The catchment roughness (PERN) and percentage impervious were based on the land use layer from the Queensland Globe database and aerial imagery. Fraction imperviousness of all sub-catchments was initially estimated based on land use data downloaded from Queensland Globe in April 2018. Then, for urbanised sub-catchments, the impervious area was estimated manually using aerial imagery. Accordingly, the impervious area was calculated as 80% of the estimated urbanised area considering the area comprises low-medium density residential, roads, district and local centres (based on fraction impervious values presented in TRC, 2017 study).

8.2.4 Routing parameters

Routing between sub-catchments was modelled using the channel routing link approach (K and X Parameters). The parameter K is a storage constant expressing the ratio between storage and flow and is usually expressed in hours. It may also be viewed as the lag or travel time through the reach. The dimensionless parameter X is indicative of the relative importance of inflow and outflow to storage.

Initial estimates of K parameters were based on approximate flow distance between sub-catchment nodes and average flow velocities based on catchment slope. The adopted value for X was 0.25, which is a typical value for natural streams. The K values were lag time in hours for each routing link based on assumed flow velocity. The routing parameter K was slightly adjusted to account for large flow for a 1% AEP event. The TUFLOW hydraulic model was used to adjust the routing parameter K further to match the peak time in the RAFTS hydrologic model with the modelled peak time in the hydraulic model (refer Section 8.3).

8.2.5 Loss model

In this assessment, an initial and continuing loss model was applied, where the lost rainfall depth does not contribute to the catchment runoff. In the loss model, it was assumed that there is an initial loss of rainfall depth followed by a constant continuing loss (mm/hour) throughout the rainfall event.

As discussed previously a hydrologic model was developed for the Gowrie Creek catchment, located in the north of the Westbrook and Dry Creeks catchment. The total catchment was divided into two sub-areas where the Area 2 (downstream of the Gowrie catchment) is similar to the Westbrook and Dry Creeks catchment. Therefore, the rainfall loss ranges used for the Gowrie Creek catchment (Area 2) along with data downloaded from the ARR 2016 Data Hub were used as initial loss estimates for the Westbrook and Dry Creeks hydrologic model and were adjusted through an iterative process.

Table 8.1 summarise the range of losses presented in the Gowrie Creek (2018) study and downloaded from ARR 2016 Data Hub. The loss model is shown in Figure B-1c in Volume II – Appendix B.

Table 8.1 Range of rainfall losses applicable for the Westbrook and Dry Creeks catchment

Source	Impervious area		Pervious area	
	Initial loss (mm)	Continuous loss (mm/hr)	Initial loss (mm)	Continuous loss (mm/hr)
Range reported in the Gowrie Creek (2018) study for 5% AEP to 1% AEP design storms	0.0 - 1.5	0.0	15 - 40.0	1.0 - 2.5
Adopted value for 1% AEP design storm for Gowrie Creek in the Gowrie Creek (2018) study	1.0	0.0	15.0	2.5
Data downloaded from ARR 2016 Data Hub[1]	N/A	N/A	40.0	1.0

Table note:

1 Accessed on November 2018

8.3 Hydraulic model development – Westbrook and Dry Creeks

8.3.1 Model setup

The Gowrie Creek hydraulic model was developed in the TUFLOW HPC software package using a 5 m grid size. The spatial extent of the hydraulic model was defined so that the proposed section of Project rail structures within the Westbrook Creek and Dry Creek catchments was included and flooding mechanisms of the Westbrook Creek system were represented in the model. The same model extent was used for both the Existing and the Developed Cases.

An overview of the model setup and key parameters is provided in Table 8.2. The hydraulic model extent and adopted land use is presented in Figure B-1d in Volume II – Appendix B.

Table 8.2 Westbrook and Dry Creeks - hydraulic model setup overview

Parameter	Information
Completion Date	November 2018
AEP's Assessed	20% AEP, 10%, 5%, 2%, 1%, 1 in 2,000 and 1 in 10,000 AEP and PMF
Hydrologic Modelling Approach	Exported hydrographs from developed hydrologic model in RAFTS were used as external and internal boundaries
Hydraulic Modelling Approach	TUFLOW software version Build 2018-03-AB – GPU
Grid Size	5 m
DEM (year flown)	1m DEM – 2015
Roughness	Based on the land use. Refer to Table 7.7
Eddy Viscosity	SMAGORINSKY (default)
Model Calibration	Model was not validated against any historical event
U/S Model Boundary	Flow/Time (QT) boundary obtained from RAFTS model
D/S Model Boundary	Height/Flow boundary based on the Westbrook Creek steam slope D/S of the model
Internal Boundaries	SA polygons and internal Flow/Time (QT) boundary lines based on the RAFTS model
Time step	2 seconds (2D) and 1 second (1D)

8.3.2 Model topography and grid size

The topography within the TUFLOW hydraulic model was based on 2m DEM data collected during 2015, which was supplied by ARTC. A 5 m x 5 m grid size was selected to allow the features of the channels and floodplain to be represented with sufficient accuracy while maintaining efficient model run times. The 5m x 5m grid size is considered adequate to represent floodplain topography and is consistent with the approach typically adopted in similar flood studies.

The 2D model topography was modified adjacent to the culverts so that the existing drainage flows, existing road/rail crowns and levee banks were represented accurately in the model.

For the Developed Case, the topography was modified to include the Project rail embankment, drainage structures and associated infrastructure.

8.3.3 Hydraulic structures

There was no information available about the existing drainage structures within the Westbrook Creek and Dry Creek catchments. Initial 2D modelling results show ponding at road and embankment crossings and in some other areas. These locations were further inspected using aerial imagery and reviewing the topographic representation of open channels and hydraulic structures. This clearly showed that the model topography required adjustment to provide a more realistic reorientation of flow paths. Therefore, the DEM was lowered at five locations to create the identified flow paths.

Also, it was reported that there is an informal levee bank around Wellcamp Airport. Since there was no information available about this levee, a breakline was created using 1m DEM and aerial imagery to make sure that the levee bank was represented in the hydraulic model.

8.3.4 Roughness

The hydraulic roughness generally reflects the types of development and ground cover that exists within the hydraulic model extent. The distribution of roughness categories adopted for this assessment was based on the information supplied in the land use layer from Queensland Globe database, aerial imagery and confirmed during the site inspection. Specific roughness values applied to the model are detailed in Table 8.3.

Table 8.3 Adopted roughness values

Category	Manning's "n" value
Floodplain	0.050
Roads	0.025
Developed area	0.083
Vegetated waterways	0.050
Waterways	0.033
Dense vegetation	0.100
Rail Embankment	0.040

Figure B-1e in Volume II – Appendix B shows the spatial breakdown of land use within the hydraulic model.

8.3.5 Boundary conditions

8.3.5.1 External and internal boundary conditions

The inflow boundary and internal source boundaries were based on results of the hydrologic modelling. A normal depth boundary was applied for the downstream outflow model boundary. It was confirmed that it was a sufficient distance away from the area of interest so as not to impact on modelling results.

Dry Creek is a tributary of Westbrook Creek running east to west and joining Westbrook Creek from the north-east on the downstream side of the Project alignment. Several Westbrook Creek and Dry Creek sub-catchments including S1.04, S11.02, S3.04, S12.02, S9.01, S10.01 and S14.01 are located upstream of the hydraulic model domain (refer Figure B-1c and B-1d in Volume II – Appendix B). Accordingly, the upstream boundary was applied as a Flow/Time (QT) boundary with total flow of these sub-catchments obtained from the hydrologic model.

Delineated sub-catchments in the hydrologic model were used as internal inflow boundaries within the hydraulic model extent. The modelled flows in the hydrologic model were applied to the lowest point of each SA polygon. Since the lowest point of the SA polygons for sub-catchments S2.02, S6.02, S7.01, S8.01 and S1.11 lie downstream of the proposed alignment, extra internal inflow boundaries were added as a Flow/Time (QT) boundary and the total flow was divided accordingly based on catchment area upstream and downstream of the Project alignment.

Figure B-1c in Volume II – Appendix B shows the hydrologic model boundaries.

The same boundary conditions were used for both the Existing Case and the Developed Case.

8.4 Existing Case hydrologic modelling – Westbrook and Dry Creeks

The RAFTS hydrological model for the Westbrook and Dry Creeks catchment was used to estimate the design hydrographs for the 20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF events. A climate change scenario was also considered for the 1% AEP event based on Representative Concentration Pathways 8.5 scenario for 2090.

Details of the adopted hydrologic model parameters are presented in the following sections.

8.4.1 Design rainfall for up to 1 in 2,000 AEP event

8.4.1.1 Spatial distribution

The ARR 2016 design event methodology was adopted for this assessment. ARR 2016 recommends adopting the spatially varying method for catchments larger than 20 km^2. The Westbrook and Dry Creeks catchment is approximately 317 km^2. Therefore, the catchment was divided into four IFD areas as shown in Figure B-1c in Volume II – Appendix B. Table 8.4 presents latitude and longitude of the centroid of IFD areas.

Table 8.4 IFD areas – total area and latitude and longitude of the centroid of each area

IFD area number	Area (km^2)	Longitude (°E)	Latitude (°S)
1	42.9	151.841673	-27.534918
2	81.7	151.844230	-27.572356
3	100.1	151.828434	-27.617558
4	92.7	151.737083	-27.535577

8.4.1.2 Rainfall depth estimation

For 20% AEP to 1% AEP events, the design rainfalls were directly downloaded by the RAFTS software from the ARR 2016 Data Hub at the centroid of each IFD area and assumed to be uniform across each IFD area.

At the time this model was developed, the RAFTS software could not download design rainfalls for rare events with short durations, e.g. the 1 in 2,000 AEP event. Therefore, for the 1 in 2,000 AEP event, the design rainfalls directly downloaded from ARR 2016 Data Hub (in November 2018) at the centroid of each IFD area and manually added to the RAFTS hydrologic model.

Table 8.5 to Table 8.8 show the adopted rainfall intensities for each IFD area.

Table 8.5 Rainfall intensity (mm/hr) used for the IFD1 area

Duration (hr)	Annual Exceedance Probability (% AEP)						
	50	20	10	5	2	1	0.05
0.25	71.6	98.0	116.4	134.0	157.6	175.6	272.4
0.50	49.0	67.6	80.4	93.0	109.6	122.6	190.0
1.00	31.0	42.8	51.0	59.2	70.2	78.9	122.0
2.00	18.7	25.6	30.4	35.4	42.1	47.5	73.5
3.00	13.7	18.6	22.1	25.7	30.6	34.7	53.7
4.50	10.0	13.6	16.1	18.6	22.2	25.1	38.9

Duration (hr)	Annual Exceedance Probability (% AEP)						
	50	**20**	**10**	**5**	**2**	**1**	**0.05**
6.00	8.1	10.8	12.8	14.8	17.7	19.8	31.0
12.00	4.8	6.4	7.6	8.8	10.3	11.7	18.1
24.00	2.9	3.9	4.6	5.3	6.3	7.1	11.0
48.00	1.8	2.4	2.9	3.4	4.0	4.5	6.9
72.00	1.3	1.8	2.2	2.6	3.1	3.5	5.4

Table 8.6 Rainfall intensity (mm/hr) used for the IFD2 area

Duration (hr)	Annual Exceedance Probability (% AEP)						
	50	**20**	**10**	**5**	**2**	**1**	**0.05**
0.25	71.6	98.0	116.4	134.0	157.6	175.6	272.4
0.50	49.0	67.6	80.4	93.0	109.6	122.6	190.0
1.00	31.0	42.8	51.0	59.2	70.2	78.9	122.0
2.00	18.7	25.6	30.4	35.4	42.1	47.5	73.5
3.00	13.7	18.6	22.1	25.7	30.6	34.7	53.7
4.50	10.0	13.6	16.1	18.6	22.2	25.1	38.9
6.00	8.1	10.8	12.8	14.8	17.7	19.8	31.0
12.00	4.8	6.4	7.6	8.8	10.3	11.7	18.1
24.00	2.9	3.9	4.6	5.3	6.3	7.1	11.0
48.00	1.8	2.4	2.9	3.4	4.0	4.5	6.9
72.00	1.3	1.8	2.2	2.6	3.1	3.5	5.4

Table 8.7 Rainfall intensity (mm/hr) used for the IFD3 area

Duration (hr)	Annual Exceedance Probability (% AEP)						
	50	**20**	**10**	**5**	**2**	**1**	**0.05**
0.25	71.2	98.0	116.4	134.0	158.0	176.0	274.0
0.50	49.0	67.6	80.4	93.0	110.0	123.0	190.8
1.00	30.9	42.7	50.9	59.2	70.3	79.1	122.0
2.00	18.6	25.5	30.4	35.3	42.1	47.5	73.5
3.00	13.7	18.6	22.1	25.6	30.6	34.7	53.7
4.50	10.0	13.5	16.0	18.6	22.1	24.9	38.9
6.00	8.1	10.8	12.8	14.8	17.5	19.8	30.8
12.00	4.8	6.4	7.5	8.7	10.3	11.6	18.0
24.00	2.9	3.9	4.6	5.3	6.3	7.0	11.0
48.00	1.8	2.4	2.9	3.4	4.0	4.5	6.9
72.00	1.3	1.8	2.2	2.6	3.1	3.4	5.3

Table 8.8 Rainfall intensity (mm/hr) used for the IFD4 area

Duration (hr)	Annual Exceedance Probability (% AEP)						
	50	**20**	**10**	**5**	**2**	**1**	**0.05**
0.25	71.6	98.4	116.4	134.0	157.6	175.2	268.4
0.50	49.2	67.6	80.2	92.8	109.2	122.0	186.6
1.00	31.0	42.6	50.7	58.8	69.7	78.3	120.0

Duration (hr)	Annual Exceedance Probability (% AEP)						
	50	20	10	5	2	1	0.05
2.00	18.6	25.4	30.2	35.1	41.8	47.1	72.0
3.00	13.6	18.5	21.9	25.5	30.4	34.3	52.7
4.50	9.9	13.4	15.9	18.4	22.0	24.9	38.0
6.00	8.0	10.7	12.7	14.7	17.5	19.8	30.3
12.00	4.7	6.3	7.4	8.6	10.3	11.5	17.7
24.00	2.8	3.8	4.5	5.2	6.2	6.9	10.6
48.00	1.7	2.3	2.8	3.2	3.8	4.3	6.5
72.00	1.2	1.7	2.0	2.4	2.8	3.2	4.9

8.4.1.3 Areal reduction factors

The catchment area of Westbrook and Dry Creeks upstream of the Project alignment is 216 km^2. This area was used to estimate the Areal Reduction Factor (ARF).

The ARF was estimated using the coefficients presented in Table 8.9 and was applied to all IFD areas. The RAFTS software directly downloaded these coefficients from ARR 2016 Data Hub to estimate the ARF using Equation 2.4.1 of ARR 2016 Book 2.

Table 8.9 Aerial reduction factor coefficients from ARR 2016 Data Hub

a	b	c	d	e	f	g	h	i
0.159	0.283	0.25	0.308	7.3e-07	1	0.039	0	0

8.4.1.4 Temporal patterns

Temporal patterns for 20% AEP to 1 in 2,000 AEP events were adopted based on 10 bursts ensemble presented in the ARR 2016. The temporal patterns and pre-burst data were directly downloaded by the RAFTS software from the ARR 2016 Data Hub at the centroid of the Westbrook and Dry Creeks catchment.

Median Pre-Burst depths for durations from 1 hour to 4 hours for a 1% AEP design event are between 2 mm and 8.4 mm (ARR 2016 Data Hub accessed November 2018). Due to the small depth of Pre-Burst in this area, the impact of pre-burst condition is considered to be minimal in this catchment. Three Pre-burst time steps were selected as Pre-Burst rainfall option.

8.4.2 Design rainfall for 1 in 10,000 AEP and PMF events

8.4.2.1 Rainfall depth estimation

PMP depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the methodology presented in the Estimation of Probable Maximum Precipitation in Australia: Generalised Short Duration Method (BOM, 2003). Once the PMP was estimated, rainfall for events between 1 in 2,000 AEP and the PMF was then interpolated. The parameters adopted for PMP rainfall estimation are presented in Table 8.10.

Table 8.10 Location data for PMP estimation

Factor	Value
Topographic Adjustment Factor (TAF)	1.02
Decay Amplitude Factor (DAF)	0.94
Extreme Precipitable Water (EPW	84.49
Moisture Adjustment Factor (MAF)	0.81

For the 1 in 10,000 AEP event, areal reduction factors were applied based on interpolation methodology presented in ARR 2016 Section 3.5 in Book 8 and the rainfall depths were estimated accordingly as shown in Table 8.11.

Table 8.11 Rainfall intensity for the 1 in 10,000 AEP and PMP events (note: no areal reduction factor was applied to the PMP rainfall)

Duration (hr)	Areal reduction factor	1 in 10,000 AEP rainfall intensity (mm/hr)	PMP rainfall intensity (mm/hr)
0.25	0.56	156.9	440.0
0.50	0.63	123.6	320.0
0.75	0.70	113.9	280.0
1.00	0.77	102.9	250.0
1.50	0.81	88.0	206.7
2.00	0.84	75.8	180.0
2.50	0.86	67.7	160.0
3.00	0.88	61.8	146.7
4.00	0.89	51.4	122.5
5.00	0.91	45.3	106.0
6.00	0.91	40.2	95.0

8.4.2.2 Temporal patterns

Temporal patterns for the 1 in 10,000 AEP and PMF design events for duration up to 6 hours were obtained from The Estimation of Probable Maximum Precipitation in Australia: Generalised Short Duration Method (BOM, 2003).

8.4.3 Design rainfall losses

A range of design rainfall losses and routing parameters were trialled for the design events in accordance with the ranges detailed in Section 8.2.5. Loss rates shown in Table 8.12 were selected as a result of:

- Improved correlation between the estimated flow in the RAFTS hydrologic model and the RFFE results
- Comparison between the hydrologic and hydraulic modelled hydrographs and peak flows at few critical locations along the Project alignment.

For impervious area, the same loss parameters were applied to the entire catchment. There is variability in characteristics of the pervious area with the upstream area of the Westbrook and Dry Creeks catchment more urbanised and steeper than the rest of the catchment. Two different loss models were applied in the upstream and the rest of catchment as follow:

- Pervious U/S: applied to the upstream catchment where the loss values are smaller
- Pervious: applied to the rest of catchment where loss values are higher.

Figure B-2 in shows the two loss areas and applied loss values to different sub-catchments.

Table 8.12 Adopted rainfall losses

Design event	Area type	Initial loss (mm)	Continuous loss (mm/hr)
20% AEP to 5% AEP	Impervious	1.0	0.0
	Pervious U/S	20.0	1.0
	Pervious	35.0	1.5
1% AEP and 2% AEP	Impervious	1.0	0.0
	Pervious U/S	15.0	1.0
	Pervious	25.0	1.5
1 in 2,000 AEP	Impervious	0.0	0.0
	Pervious U/S	10.0	1.0
	Pervious	15.0	1.0
1 in 10,000 AEP and PMF	Impervious	0.0	0.0
	Pervious U/S	0.0	1.0
	Pervious	0.0	1.0

8.4.4 Routing parameters

The RAFTS software has three types of link that provide connectivity between the hydrologic model catchment nodes. In this assessment, the routing link method that was used requires definition of two routing parameters, K and X. The parameter K is a storage constant expressing the ratio between storage and flow and is usually expressed in hours. It may also be viewed as the lag or travel time through the reach. The dimensionless parameter X is indicative of the relative importance of inflow and outflow to storage.

The adopted value for X was 0.25, which is a typical value for natural streams. The K values (lag time in hours) for each routing link was based on the assumed flow velocity using catchment slope.

Figure 13 to Figure 16 present the comparison of flow estimates for critical sub-catchments.

8.4.5 Hydrologic model validation

The RAFTS hydrologic model for the Westbrook and Dry Creeks catchment was not calibrated due to unavailability of observed streamflow and anecdotal flood level data in the catchment. However, the loss and routing parameters were adjusted to match the RFFE results and through comparison of the modelled hydrographs in the RAFTS hydrologic and TUFLOW hydraulic models. The estimated flows were also compared with reported flows in the previous studies where the information was available.

8.4.5.1 Regional flood frequency estimation

Since there are no stream gauge data available within the Westbrook and Dry Creeks catchment, FFA was not able to be undertaken. However, a Regional Flood Frequency Estimation Model (RFFE) from ARR 2016 was used for the validation of the hydrologic model of the Westbrook and Dry Creeks catchment.

The RFFE online tool was used to obtain an estimation of catchment peak flow at the outlet of three sub-catchments including outlets of sub-catchment S1.09 and S11.05 where the proposed alignment crosses Westbrook Creek and Dry Creek respectively and at the outlet of sub-catchment S1.11 where the two creeks merge. The RFFE online site was accessed on 28 and 29 November 2018 and the RFFE inputs and outputs are summarised in Table 8.13 and Table 8.14 respectively.

Table 8.13 **Model inputs for the regional flood frequency estimation for Westbrook and Dry Creek sub-catchments**

Model parameter	Input value
Sub-catchment S1.11	
Region name	East Coast
Region code	1
Latitude at S1.11 (degree)	-27.53
Longitude at S1.11 (degree)	151.77
Latitude at catchment centroid (degree)	-27.57
Longitude at catchment centroid (degree)	151.83
Distance of the nearest gauged catchment in the database (km)	16.8
Catchment area (km^2)	247
Design rainfall intensity, 50% AEP and 6 hr duration (mm/h)	8.07
Design rainfall intensity, 2% AEP and 6 hr duration (mm/h)	17.61
Sub-catchment S1.09	
Region name	East Coast
Region code	1
Latitude at S1.11 (degree)	-27.539
Longitude at S1.11 (degree)	151.77
Latitude at catchment centroid (degree)	-27.594
Longitude at catchment centroid (degree)	151.824
Distance of the nearest gauged catchment in the database (km)	16.9
Catchment area (km^2)	205
Design rainfall intensity, 50% AEP and 6 hr duration (mm/h)	7.99
Design rainfall intensity, 2% AEP and 6 hr duration (mm/h)	17.46
Sub-catchment S11.05	
Region name	East Coast
Region code	1
Latitude at S1.11 (degree)	-27.53
Longitude at S1.11 (degree)	151.78
Latitude at catchment centroid (degree)	-27.53
Longitude at catchment centroid (degree)	151.85
Distance of the nearest gauged catchment in the database (km)	15.8
Catchment area (km^2)	41
Design rainfall intensity, 50% AEP and 6 hr duration (mm/h)	8.06
Design rainfall intensity, 2% AEP and 6 hr duration (mm/h)	17.62

Table 8.14 Regional flood frequency estimation results obtained from ARR 2016 website for Westbrook and Dry Creek sub-catchments

AEP (%)	Flow (m^3/s)	95% probability limit	
Sub-catchment S1.11			
50	82.0	31.5	213
20	189.0	76.2	470
10	296.0	113.0	781
5	431.0	151.0	1,210
2	665.0	206.0	2,140
1	891.0	250.0	3,150
Sub-catchment S1.09			
50	68.2	26.2	177
20	157.0	63.4	392
10	246.0	94.2	652
5	359.0	126.0	1,010
2	554.0	172.0	1,790
1	743.0	208.0	2,630
Sub-catchment S11.05			
50	19.9	7.6	51.8
20	45.4	18.3	114.0
10	70.8	27.0	188.0
5	103.0	35.9	290.0
2	158.0	48.8	509.0
1	211.0	59.2	744.0

8.4.5.2 Hydrologic model validation against hydraulic model

The RAFTS and TUFLOW hydrographs were compared at critical locations at the Project alignment to adjust rainfall losses and routing parameters accordingly. The final values were adopted as a result of improved correlation between RAFTS and TUFLOW modelling results.

Figure 13 to Figure 16 illustrate the comparison of modelled hydrographs and peak flow estimates from the RAFTS hydrologic and TUFLOW hydraulic models. The graphs demonstrate that the models correlate well.

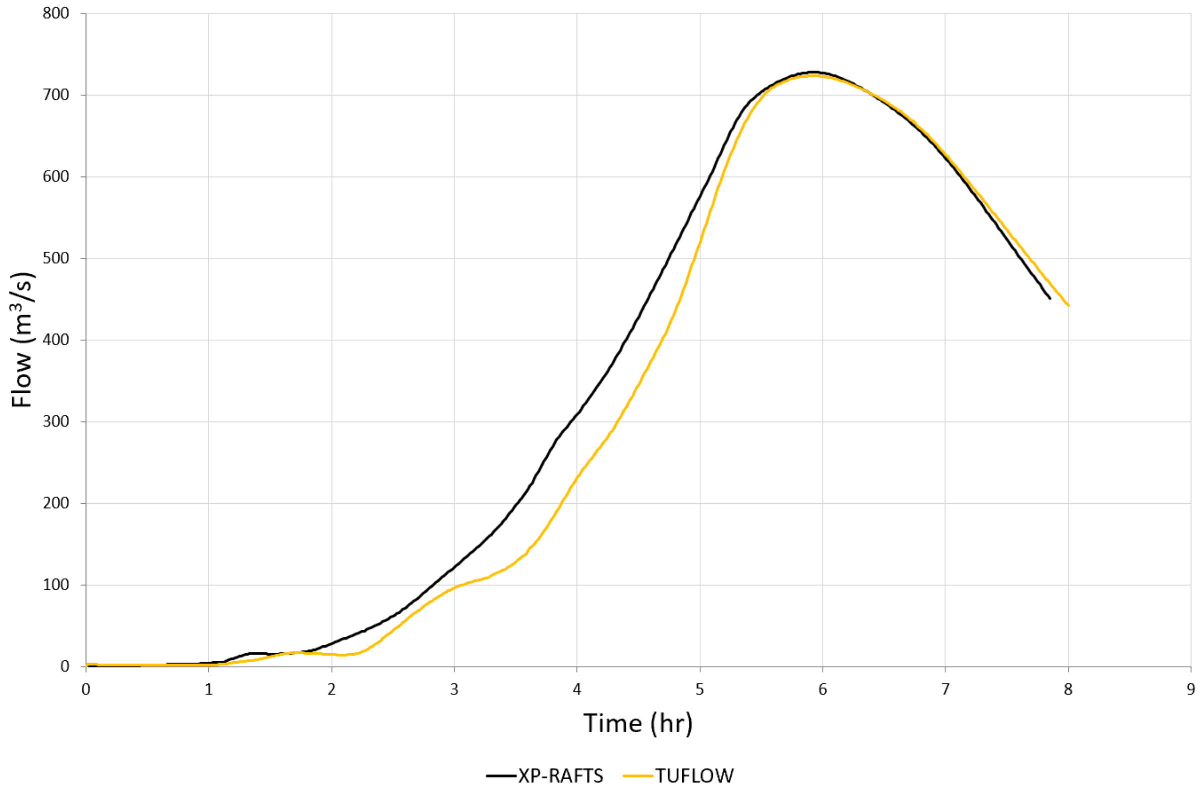

Figure 13 **Comparison of sub-catchment S1.08 modelled hydrographs in TUFLOW and RAFTS models – storm 4.5hrs burst 9**

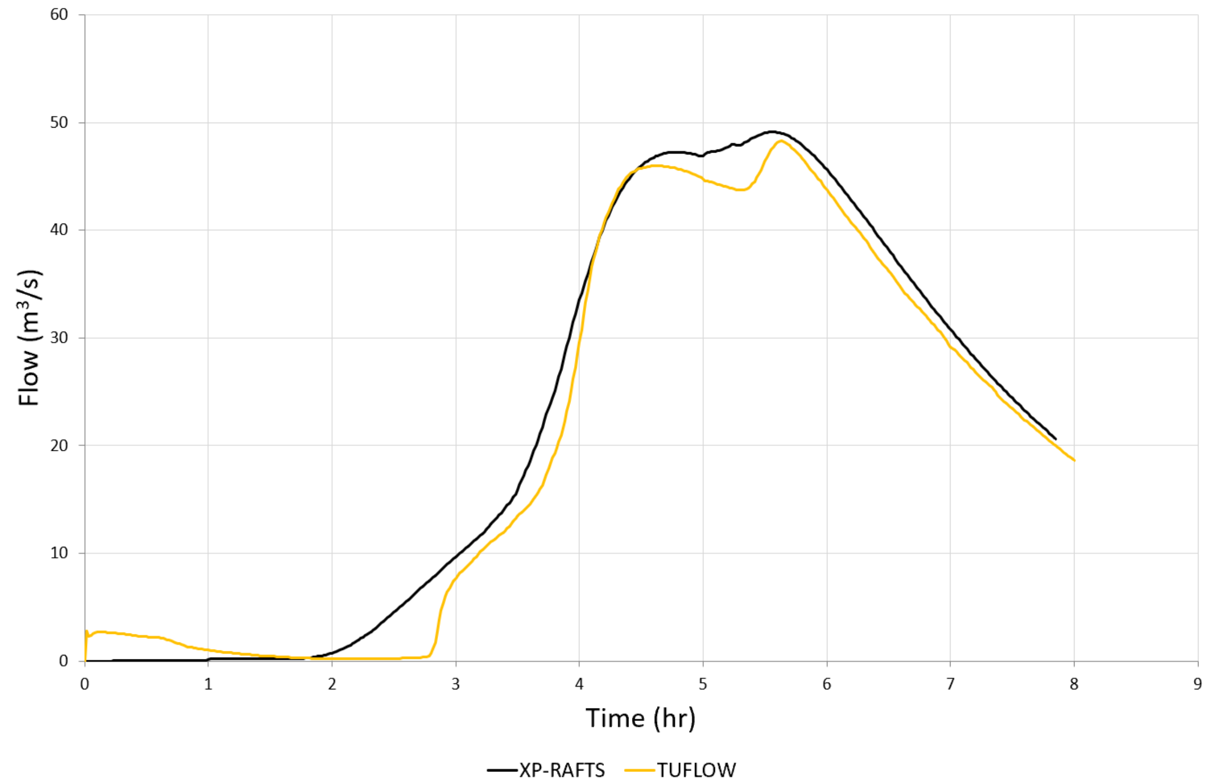

Figure 14 **Comparison of sub-catchment S1.10 modelled hydrographs in TUFLOW and RAFTS models – storm 4.5hrs burst 9**

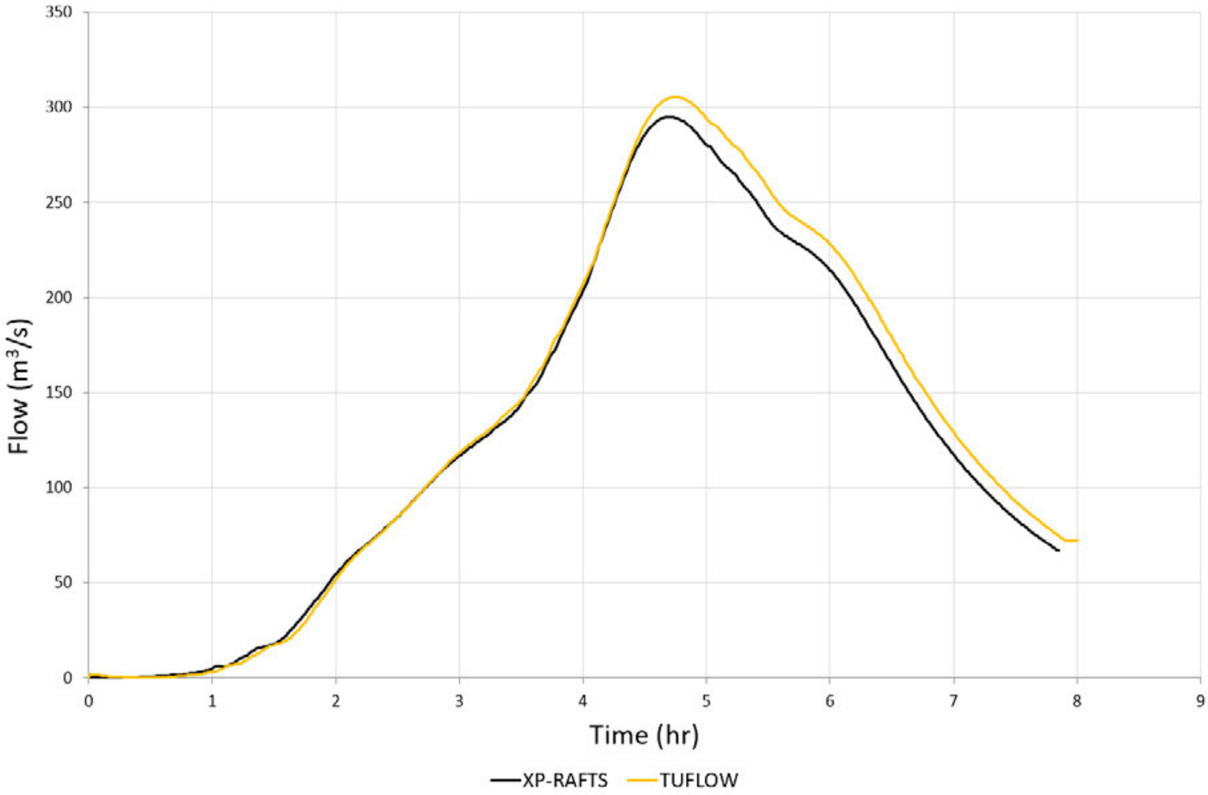

Figure 15 **Comparison of sub-catchment S3.04 modelled hydrographs in TUFLOW and RAFTS models – storm 4.5hrs burst 9**

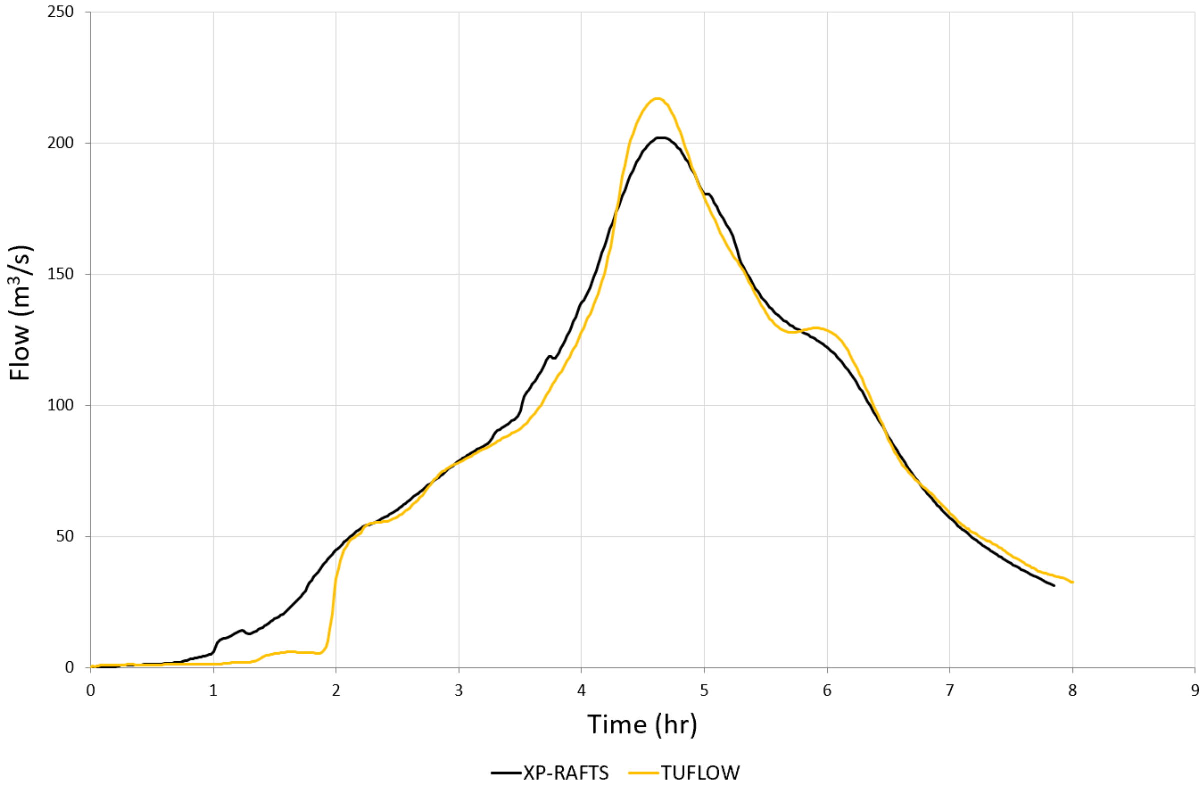

Figure 16 **Comparison of sub-catchment S1.05 modelled hydrographs in TUFLOW and RAFTS models – storm 4.5hrs burst 9**

8.4.6 Estimated design flows

The design flow for the 1% AEP event was estimated using the developed RAFTS hydrologic model for the Westbrook Creek catchment. The model peak flow estimates were compared against the RFFE estimates and the results from the previous studies to ensure the estimated flows are within the accepted ranges.

Table 8.15 provides comparison of the peak flow modelled by RAFTS model and RFFE estimates at three sub-catchments. The results show that the difference between the RFFE and RAFTS model estimations at the area of interest is less than 6% for 1% AEP and less than 15% for 2% AEP event. The difference between the RFFE and RAFTS model estimations for 10% is up to 45%.

Table 8.15 Comparison between estimated peak flows by RFFE with the RAFTS model

Sub-catchment	Peak flow by RFFE (m^3/s)			Peak flow by RAFTS model (m^3/s)[1]			Difference (%)		
	1% AEP	2% AEP	10% AEP	1% AEP	2% AEP	10% AEP	1% AEP	2% AEP	10% AEP
S1.11	891	665	296	921	756	429	+ 3.4	+13.7	+44.9
S1.09	743	554	246	770	633	356	+ 3.6	+14.3	+44.7
S11.05	211	158	71	200	167	85	- 5.2	+5.7	+19.7

Table note:

1 Median value of all 10 temporal patterns (bursts) for the critical duration

Table 8.16 shows a comparison between the reported 1% AEP peak flows in this assessment with estimates from the previous studies. The differences between the previous study and the current study results can be related to the following factors:

- RMA (2015) study used ARR 1987 methodology while the current study used ARR 2016 methodology. The rainfall depths used in RMA (2015) are smaller than the ones used in this current study, and the ensemble storm approach was not used in the ARR 1987 methodology.
- The location of reported peak flows in the previous studies is not the same as those used in this current investigation. Therefore, the total catchment areas for the reported locations are slightly different from the area used in this assessment.

Table 8.16 Comparison between flows for 1% AEP event in previous studies with the RAFTS model developed in this assessment

B2G hydrologic model sub-catchment	1% AEP peak flow (m^3/s)[1]	Previous investigation and the associated sub-catchment in that study	B2G hydrologic model (m^3/s)	Difference (%)
S1.09	770	RMA (2015) at catchment outlet	691	- 10%
S3.02	211	TRC (2017) at R3A7 sub-catchment	213	+ 0.1%

Table note:

1 Median value of all 10 temporal patterns (bursts) for the critical duration

The estimated peak flows in this assessment at the Project alignment have a reasonable correlation with the RFFE model results and results from TRC (2017) and RMA (2015) studies. This comparison provides confidence in the current 1% AEP event flow estimates.

8.5 Existing Case modelling results – Westbrook and Dry Creeks

8.5.1 Hydrologic model peak flow assessment

A critical duration assessment was undertaken to determine which storm duration(s) produced peak flood flows at the stream flow gauges locations and where the major waterways are intersected by the Project alignment. To assess the critical storm duration the following methodology was adopted:

- The models were modelled for a range of AEP events: 20% AEP, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF:
  - Each AEP was modelled for a range of durations, and
  - Each duration was modelled for each of the ten associated temporal patterns (TPs)
- Peak water levels were mapped for each storm duration
- A critical duration assessment was undertaken at the locations mentioned above to determine which duration produced the highest median flow of the ten temporal patterns for each event.

Table 8.17 presents the estimated peak flow applied to the hydraulic model at outlets of several sub-catchments (Figure B-1c in Volume II – Appendix B).

Table 8.17 1% AEP event peak flow at key locations as applied in the hydraulic model

Sub-catchment name	1% AEP event peak flow (m^3/s)	Critical storm (hrs) duration and temporal pattern	1 in 10,000 AEP event peak flow (m^3/s)	Critical storm duration hours)
1.08	719	3 hour – TP 7	2,570	3
1.09	770	4.5 hour – TP 9	2,752	4.5
1.10	50	4.5 hour – TP 8	204	3
1.11	921	4.5 hour – TP 9	3,294	4.5
1.15	1,050	12 hour – TP 9	3,734	4.5
2.02	50	4.5 hour – TP 1	200	3
6.02	56	4.5 hour – TP 8	227	3
7.01	16	4.5 hour – TP 8	65	3
8.01	16	4.5 hour – TP 9	63	3
11.05	200	3 hour – TP 6	644	3

8.5.2 Existing Case flood maps

Maps illustrating indicative flood extents and peak water levels were prepared and are presented in Volume II – Appendix B:

- 20% AEP: Figure B-2a
- 10% AEP: Figure B-2b
- 5% AEP: Figure B-2c
- 2% AEP: Figure B-2d
- 1% AEP: Figure B-2e
- 1 in 2,000 AEP: Figure B-2f
- 1 in 10,000 AEP: Figure B-2g
- PMF: Figure B-2h.

Figure B-3a presents peak flood velocities expected in a 1% AEP event.

8.5.3 Flood inundation extent and flood levels

Figure B-2e in Volume II – Appendix B shows the 1% AEP indicative flood extent and peak water levels within the Westbrook and Dry Creeks floodplain for the Existing Case. The peak flood depth is within the channel downstream of the Project alignment and is approximately 6.5 m.

8.5.4 Flood immunity of existing infrastructure

Toowoomba-Cecil Plains Road is located upstream of the Project alignment. Table 8.18 presents a summary of overtopping depths for key roads near the Project alignment under a range of design events. Modelling results indicates that Toowoomba Wellcamp Airport is not flooded under a 1% AEP event.

Table 8.18 Westbrook and Dry Creeks – Existing Case – overtopping depths of key infrastructure

Infrastructure	Location	Maximum overtopping depth (m)							
		PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Anderson Road	D/S Westbrook Bridge	6.73	5.16	4.38	3.64	3.36	2.94	2.53	2.05
Brimblecombe Road	Dry Creek Crossing	2.03	1.51	1.30	1.05	0.98	0.85	0.68	0.52
Toowoomba-Cecil Plains Road	Westbrook Creek Crossing	2.57	1.36	0.90	0.57	0.50	0.41	0.31	0.18

8.5.5 Existing Case velocities

Peak Existing Case velocities for the 1% AEP event in the channels of Westbrook and Dry Creeks at the Project alignment are relatively high, in the order of 2 to 3 m/s, and on the floodplain, velocities are generally in the order of 1 to 1.5 m/s as shown in Figure B-3a in Volume II – Appendix B.

8.6 Developed Case modelling results – Westbrook and Dry Creeks

8.6.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW hydraulic model (1d and 2d approach).

On the Westbrook and Dry Creek floodplain, the Project includes the following floodplain (or regional structures):

- Two waterway bridges
- 10 RCP locations (a total of 94 cells)
- Two RCBC location (a total of 13 cells)
- Two rail-over-road bridges that also convey flood flows in large events.

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and depending on their interaction with the Westbrook and Dry Creek floodplain were incorporated in the hydraulic model.

A blockage factor of 25% was applied to all proposed culverts based on guidelines set out in ARR 2016. The adopted blockage factor for the proposed bridges was between 5% and 10% based on the waterway area blockage due to bridge piers.

The proposed drainage structures are summarised in Table 8.19 and Table 8.20 and shown in Figure B-1f in Volume II – Appendix B.

Table 8.19 Westbrook and Dry Creeks – proposed floodplain culvert locations and details

Chainage (km)	Structure ID	Type	U/S invert (m AHD)	D/S invert (m AHD)	Diameter/ width(m)	Height (m)	Number of cells
188.72	C188.72	RCBC	510.46	510.08	1.8	1.2	11
191.83	C191.83	RCP	462.96	462.39	2.7	-	5
193.38	C193.38	RCBC	469.50	469.22	1.5	0.9	2
193.41	C193.41	RCP	469.00	468.70	1.05	-	3
195.64	C195.64	RCP	432.67	432.20	1.05	-	14
195.93	C195.93	RCP	432.91	432.21	1.05	-	2
196.03	C196.03	RCP	432.80	432.49	1.05	-	2
197.42	C197.42	RCP	423.88	423.67	2.4	-	15
197.49	C197.49	RCP	425.09	424.94	1.5	-	11
197.53	C197.53	RCP	425.48	425.34	1.2	-	10
197.71	C197.71	RCP	425.44	425.34	1.05	-	17
198.26	C198.26	RCP	426.29	425.80	1.05	-	15

Table 8.20 Westbrook and Dry Creeks – proposed bridge locations and details

Chainage (km)	Structure ID	Bridge name	Soffit level (m AHD)	Deck depth (m)	Bridge length (m)
196.12	310-BR20	Toowoomba-Cecil Plains Road Rail Bridge	442.3	2.0	92
197.26	310-BR31	Westbrook Creek Waterway Bridge	430.3	2.0	230
197.96	310-BR32	Dry Creek Waterway Bridge	428.7	2.0	184
198.73	310-BR33	Brimblecombe Road Rail Bridge	436.7	2.0	75

8.6.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

8.6.2.1 Rail immunity and structures results

Appendix C summarises the peak 1% AEP water levels on the upstream side of the proposed alignment. Local drainage structures (i.e. those not included in the flood model) and road culverts are not reported.

The results of flood modelling indicate that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the Westbrook and Dry Creeks floodplain. There is over 2.5 m freeboard above the culvert obvert levels to the rail formation level in a 1% AEP event. At the bridge locations, the 1% AEP peak water levels are below the bridge soffit levels.

8.6.2.2 Velocities and scour

Appendix C summarises the peak 1% AEP outlet flows and velocities at structures. The 1% AEP event velocities through the proposed culverts are generally less than 2.5 m/s except at four culverts where velocities are higher than 2.5 m/s.

Scour protection requirements for culverts were calculated based on the velocities predicted from the hydraulic modelling. The scour protection was 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.

Table 8.21 lists the soil types encountered along the Project alignment and the allowable soil velocity based on AGRD.

Table 8.21 Allowable soil velocities along the Project alignment

Soil type	Soil description	Maximum allowable soil velocity as per AGRD (m/s)
Sodosols	Firm loam or stiff clay	2 m/s
Vertosols, Dermosols, Rudosols and Kandosols	Sandy or silty clay	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.

Scour protection requirements for culverts have been calculated based on the velocities predicted from the hydraulic modelling. Appendix D presents the proposed scour protection at each structure.

A conservative scour estimation has been undertaken at the bridge site based on available information and will be refined during detailed design.

8.6.2.3 Flood immunity for extreme events

The risk of overtopping of the rail alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with Table 8.22 presenting the depth of water above the rail formation level and over the top of rail at each structure location. It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the alignment. As such, overtopping of the rail is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario.

Table 8.22 Westbrook and Dry Creeks – extreme events – depth of water above formation and top of rail levels

Chainage (km)	Structure ID	Depth of water above formation level (m)			Depth of water over 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
196.12	310-BR20	-	-	-	-	-	-
197.26	310-BR31	-	-	-	-	-	-
197.96	310-BR32	-	-	0.01	-	-	-
198.73	310-BR33	-	-	-	-	-	-

Chainage (km)	Structure ID	Depth of water above formation level (m)			Depth of water over 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
188.72	C188.72	-	0.15	0.49	-	-	-
191.83	C191.83	-	-	-	-	-	-
193.38	C193.38	-	-	-	-	-	-
193.41	C193.41	-	-	0.11	-	-	-
195.64	C195.64	-	-	-	-	-	-
195.93	C195.93	-	-	-	-	-	-
196.03	C196.03	-	-	-	-	-	-
197.42	C197.42	-	-	-	-	-	-
197.49	C197.49	-	-	0.53	-	-	-
197.53	C197.53	-	-	0.72	-	-	0.02
197.71	C197.71	-	-	1.78	-	-	1.08
198.26	C198.26	-	-	-	-	-	-

8.6.3 Flood impact objectives outcomes – Westbrook and Dry Creeks

The impact of the Developed Case was assessed through inclusion of the proposed rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, duration of inundation, velocities and peak flows within the floodplain.

- Changes in peak water levels for the AEP's assessed are presented in Figures B-4a to B-4h in Volume II – Appendix B
- Changes in 1% AEP duration of inundation are presented in Figure B-4i in Volume II – Appendix B
- Changes in 1% AEP velocities are presented in Figure B-4j in Volume II – Appendix B.

All impacts are required to be agreed with relevant stakeholders and affected landowners as part of the EIS process.

The Project design outcomes relative to the flood impact objectives (refer Table 4.2) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Volume II – Appendix N.

8.6.3.1 Flood impacts at proposed hydraulic structures

The estimated potential impacts on peak water levels at each proposed structure is presented in Table 8.23 for the 1% AEP event. Peak water levels were extracted immediately upstream of each culvert and at the control line of each bridge.

The design achieved a freeboard of at least 300 mm between the 1% AEP peak water and formation levels.

Table 8.23 Westbrook and Dry Creeks – 1% AEP event – estimated impacts to peak water levels at proposed hydraulic structures

Chainage (km)	Structure ID	Structure type	Rail formation level or bridge deck height (m AHD)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Change in peak water level (mm)
196.12	310-BR20	Rail bridge	446.64/444.64	432.93	433.32	+390
197.26	310-BR31	Waterway bridge	436.09/434.09	426.63	426.75	+120
197.96	310-BR32	Waterway bridge	432.89/430.89	425.72	425.80	+70
198.73	310-BR33	Rail bridge	440.86/438.86	430.00	430.01	+10
188.72	C188.72	RCBC	518.49	511.27	512.02	+750
191.83	C191.83	RCP	487.15	465.39	466.14	+750
193.38	C193.38	RCBC	471.61	473.06	470.25	-2,810
193.41	C193.41	RCP	471.27	473.06	470.25	-2,810
195.64	C195.64	RCP	448.96	432.61	433.58	+970
195.93	C195.93	RCP	446.34	-	433.52	+530
196.03	C196.03	RCP	445.03	432.93	433.41	+480
197.42	C197.42	RCP	431.29	426.23	426.59	+360
197.49	C197.49	RCP	430.50	426.02	426.47	+450
197.53	C197.53	RCP	430.08	426.02	426.47	+450
197.71	C197.71	RCP	428.71	425.87	426.41	+540
198.26	C198.26	RCP	433.84	426.51	427.08	+570

8.6.3.2 Flood impacts on flood sensitive receptors

Based on the available aerial imagery, no buildings or critical infrastructure are located within the area predicted to be impacted by changes in peak water levels on the Westbrook and Dry Creeks floodplain for events up to the 1% AEP.

8.6.3.3 Flood impacts on state-controlled roads

Within the extent of the hydraulic model, the only state-controlled road which is influenced by flooding and the Project alignment is the Toowoomba-Cecil Plains. The extent of the hydraulic model developed for Westbrook and Dry Creeks and the extent of the state-controlled road is shown in Figure 17.

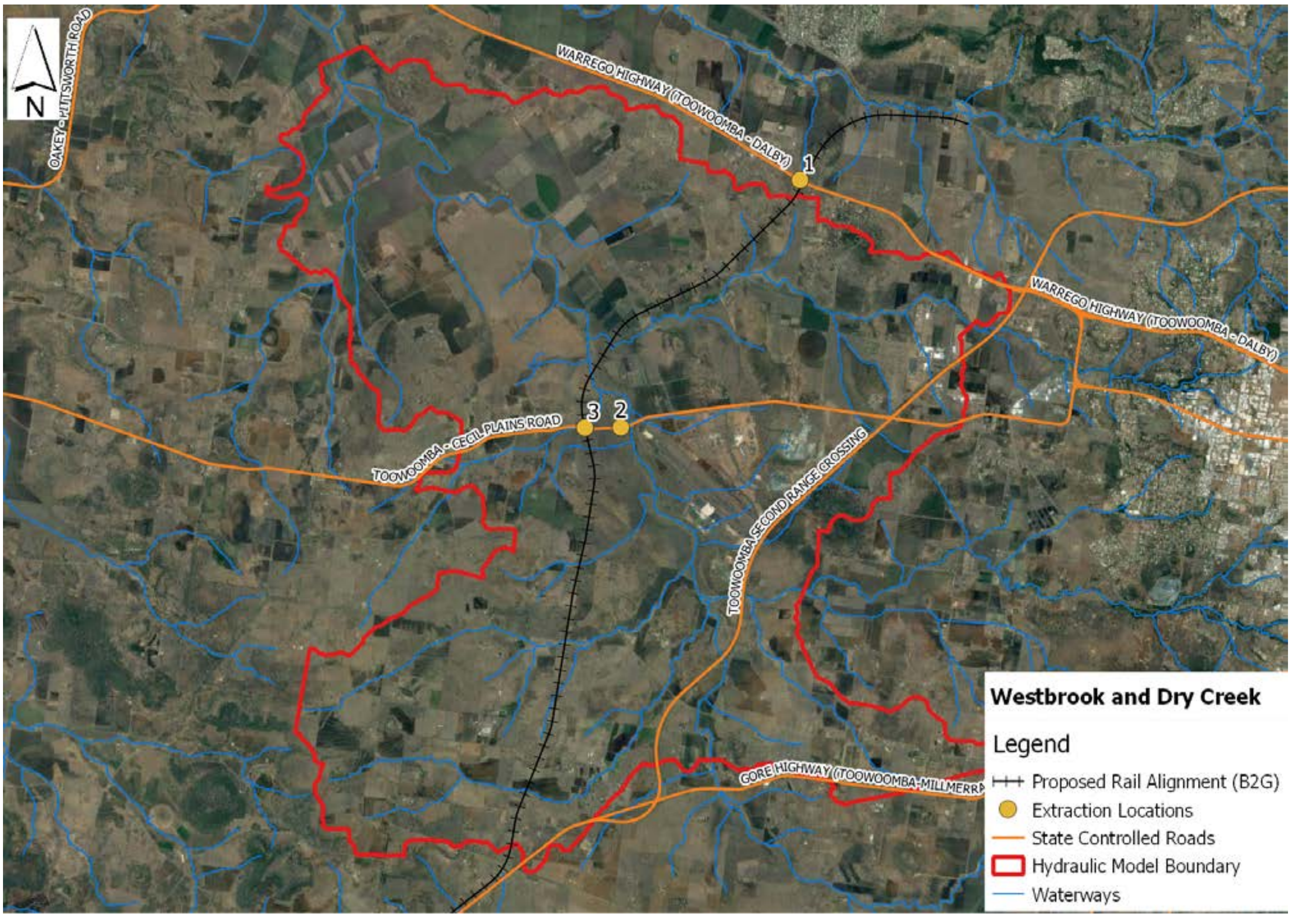

Figure 17 Westbrook and Dry Creek Hydraulic Model Extent and Associated State-controlled Roads

The Existing Case model results indicate that Toowoomba-Cecil Plains roads has very low flood immunity, with sections overtopping with small flood depths in the 20% AEP event. The impact of the Project alignment has minimal impact on the overall flood immunity. It is noted that in a 1% AEP event, the Project alignment does result in another small road segment being overtopped; however, as the road is already submerged in other parts of the link, it does not influence immunity.

Existing Case flooding conditions

Table 8.24 Westbrook and Dry Creeks – Existing Case flood depths

Reporting location	Road	Estimated depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
2	Toowoomba-Cecil Plains Roads	0.04	0.11	0.18	0.26	0.33	0.58	0.96	2.09
3	Toowoomba-Cecil Plains Roads	0.00	0.00	0.00	0.00	0.00	0.08	0.19	0.53

Table 8.25 Westbrook and Dry Creeks – Existing Case flood inundation length

Reporting location	Road	Approximate length of inundation (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
2	Toowoomba-Cecil Plains Roads	167	251	263	284	533	754	885	1,318
3	Toowoomba-Cecil Plains Roads	0	0	0	0	0	123	175	

Table 8.26 Westbrook and Dry Creeks – Existing Case time of submergence

Reporting Location	Road	Estimated time of submergence (hrs)								AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
2	Toowoomba-Cecil Plains Roads	0.1	3.3	4.4	3.0	3.7	4.5	5.2	5.7	0.60
3	Toowoomba-Cecil Plains Roads	0.0	0.0	0.0	0.0	0.0	1.7	3.9	4.4	0.0

Developed Case flooding conditions

Table 8.27 Westbrook and Dry Creeks – Developed Case flood depths

Reporting location	Road	Estimated depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
2	Toowoomba-Cecil Plains Roads	0.04	0.11	0.18	0.26	0.33	0.56	0.91	2.18
3	Toowoomba-Cecil Plains Roads	0.00	0.00	0.00	0.00	0.07	0.25	0.80	2.01

Table 8.28 Westbrook and Dry Creeks – Developed Case flood inundation length

Reporting location	Road	Approximate length of inundation (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
2	Toowoomba-Cecil Plains Roads	167	251	263	284	533	747	861	1,318
3	Toowoomba-Cecil Plains Roads	0	0	0	0	12	137	212	

Table 8.29 Westbrook and Dry Creeks – Developed Case time of submergence

Reporting location	Road	Estimated time of submergence (hrs)								AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
2	Toowoomba-Cecil Plains Roads	0.5	3.4	4.4	3.0	3.7	5.2	5.2	5.6	0.8
3	Toowoomba-Cecil Plains Roads	0.0	0.0	0.0	0.0	1.1	5.0	5.0	5.1	0.0

Impacts of Project alignment

Table 8.30 Westbrook and Dry Creeks – change in flood depths

Reporting location	Road	Estimated change in depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
2	Toowoomba-Cecil Plains Roads	0.00	0.00	0.00	0.00	0.00	-0.02	-0.05	0.09
3	Toowoomba-Cecil Plains Roads	0.00	0.00	0.00	0.00	0.07	0.17	0.61	1.48

Table 8.31 Westbrook and Dry Creeks – change in time of submergence

Reporting Location	Road	Estimated change in time of submergence (hrs)								Estimated change in AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
2	Toowoomba-Cecil Plains Roads	0.4	0.0	0.0	0.0	0.0	0.7	0.0	-0.1	0.2
3	Toowoomba-Cecil Plains Roads	0.0	0.0	0.0	0.0	1.1	3.3	1.1	0.7	0.0

Change in flood hydrographs

Figure 18 shows the height time series for the Existing Case and Developed Case for the 1% AEP event. The time series have been extracted from extraction point 2. The differences between the Developed and Existing Case hydrographs are minimal in terms of shape and peak level, with a marginal decrease in flood height being present in the design scenario.

However, as shown in Figure 18, the length of model simulation does not allow for the full hydrograph to route through the model. As such, with the results from the current model iteration, the impact of the Project alignment on the receding limb is unable to be confirmed.

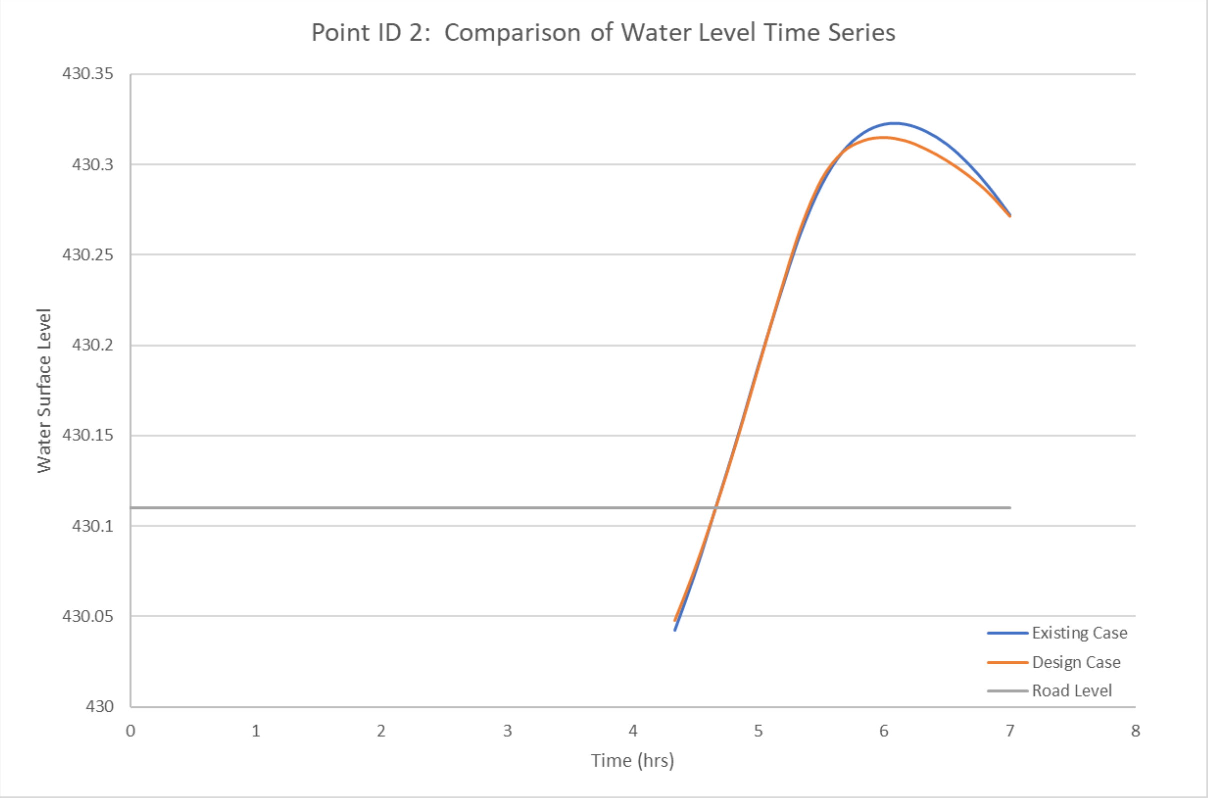

Figure 18 Extraction Point 2 - comparison of water level time series, 1% AEP

8.6.3.4 Flood impacts on local public roads

The change in peak water levels and flood hazard (velocity-depth) for the 1% AEP event were evaluated on local public roads within the hydraulic model domain. Local public roads that are expected to experience an increase in flood hazard and/or increases in peak flood levels are reported in Table 8.32.

Table 8.32 Westbrook and Dry Creeks – change in peak water levels and flood hazard for local public roads, 1% AEP

Location	Existing flood hazard (m^2/s)	Design flood hazard (m^2/s)	Existing maximum flood depth (m)	Design maximum flood depth (m)	Maximum change in peak water levels (mm)[1]
Brimblecombe Road	2.56	2.57	1.13	1.14	+33
Omara Road	2.26	2.26	0.71	0.71	-
Wegener Road	1.95	1.95	0.66	0.67	-

Table note:

1 The maximum change in peak water level does not necessarily occur at the same location as where the existing and/or design maximum flood depth occur

Duration of inundation

Assessment of the time of submergence (ToS) and average annual time of submergence (AAToS) was undertaken for local public roads within the hydraulic model domain. No local public roads are expected to experience an increase in ToS or AAToS.

8.6.3.5 Flood impacts on private land outside the rail disturbance footprint

Most of the area where afflux is predicted to occur is agricultural land or open land on which nominal afflux is unlikely to cause any adverse impact. Table 8.33 presents the modelled changes in flood conditions during the 1% AEP event on a lot basis according to the following thresholds:

- Peak water levels increased by greater than +10 mm
- Peak velocities increased by greater than 0.25 m/s
- Duration of inundation changed by more than 25% of its original duration of inundation across the lot.

Table 8.33 Westbrook and Dry Creeks – summary of flood impacts on private land outside the rail disturbance footprint for 1% AEP

Approximate chainage (km)	Changes in peak water levels[1]		Changes in peak velocities		Changes in Duration of inundation	
	Maximum change (mm)	Total area affected by change > 10 mm (ha)	Maximum change (m/s)	Total area affected by change (ha)	Maximum change (%)	Total area affected by change (ha)
193.50	-	-	-	-	+38%	0.17
197.90 to 198.00	+40	0.33	+0.84	0.04	-	-
195.50 to 195.90	+110	2.77	+0.46	0.09	-	-
197.20 to 197.50	+380	11.40	+0.43	0.30	-	-
188.70	+250[2]	0.40	+0.28	0.00	-	-
191.80	+160[2]	0.16	-	-	-	-
193.40	+440[2]	0.18	+1.12	0.16	+40%	0.38
198.90 to 199.00	+110	2.37	-	-	-	-
197.50	+380	4.29	-	-	-	-
197.85 to 198.70	+300	6.10	+0.78	0.04	-	-
195.50 to 195.90	+760[3]	2.90	+1.71	0.06	-	-
197.90	+40	0.27	-	-	-	-

Approximate chainage (km)	Changes in peak water levels[1]		Changes in peak velocities		Changes in Duration of inundation	
	Maximum change (mm)	Total area affected by change > 10 mm (ha)	Maximum change (m/s)	Total area affected by change (ha)	Maximum change (%)	Total area affected by change (ha)
197.50 to 197.80	+440	6.50	+0.86	0.26	-	-
197.15	+140	2.82	+0.49	0.02	-	-

Table notes:

1 Afflux on lots that exceed the flood impact objectives are summarised in the EIS Surface Water Chapter
2 Change in peak water levels at these locations are confined to existing creek channels
3 Change in peak water levels at this location is localised and directly adjacent to the Project alignment

8.6.3.6 Flow distribution

A key landowner concern is potential change to flow distribution. To understand the magnitude of these flows, flows were extracted from the hydraulic model at key locations. The difference between the Existing Case and Developed Case was considered and reported in Table 8.34. The results indicate negligible changes in flow distribution.

Figure 19 presents the selected flow path comparison locations. The flow is calculated across the length of the line. Therefore, the lines presented are either calculating the flow across the width of the floodplain (for the longer flow lines) or the main flow path of the waterways (generally for smaller flow lines).

Table 8.34 Westbrook and Dry Creeks – flow comparison

Flow location ID	10% AEP			1% AEP		
	Existing Case flow (m^3/s)	Developed Case flow (m^3/s)	% Change	Existing Case flow (m^3/s)	Developed Case flow (m^3/s)	% Change
L1	67.6	67.6	-0.14	144.2	146.7	1.74
L2	90.4	93.1	2.95	193.0	193.0	-0.01
L3	100.9	101.0	0.01	217.6	217.5	-0.01
L4	105.9	105.9	-0.01	228.4	228.4	-
L5	341.0	341.1	0.01	691.6	691.6	-
L6	361.4	364.4	0.83	732.7	729.3	-0.47
L7	474.4	479.7	1.12	967.8	964.9	-0.30
L8	491.9	496.4	0.91	931.3	929.6	-0.19

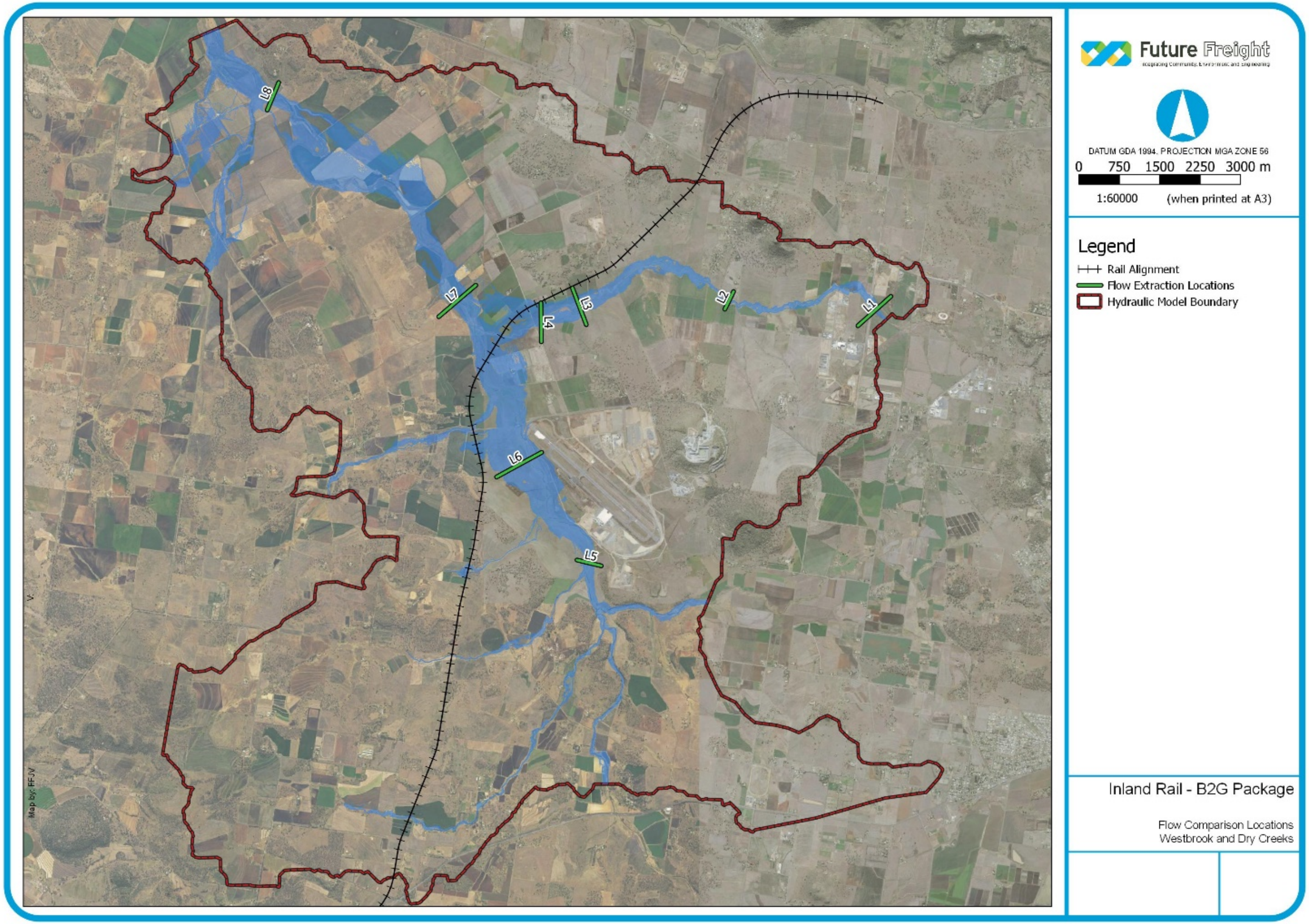

Figure 19 Westbrook and Dry Creeks – flow comparison locations

8.6.4 Sensitivity analysis – Westbrook and Dry Creeks

The sensitivity of the model to various parameters was assessed using the following three scenarios:

- An increase in rainfall intensity, i.e. to reflect climate change scenario
- Increase in blockage of culverts from 25% to 50%
- Decrease in blockage of culverts from 25% to 0%.

8.6.4.1 Blockage

Blockage was assessed in accordance with ARR 2016. The blockage assessment undertaken resulted in a blockage factor of 25% being adopted for culverts. A minimum culvert size of 900 mm diameter was adopted to reduce potential for blockage and maintenance. A significant 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% and 50% blockage.

There is little change to the predicted afflux as a result of reducing the applied culvert blockage allowance to 0%. As a result of increasing the blockage factor to 50%, minor increases are predicted in localised areas upstream of the alignment in particular between the two proposed bridges over Westbrook and Dry Creeks, and upstream of culverts C188.72 and C191.83, where predicted increases in upstream 1% AEP peak flood levels exceed 500 mm.

The adopted blockage factor for the proposed bridges was between 5% and 10% based on the waterway area blockage due to bridge piers.

Table 8.35 provides a summary of 1% AEP peak flood levels at cross drainage structures for the blockage scenarios.

Table 8.35 Westbrook Creek and Dry Creek – 1 % AEP event – culvert blockage assessment

Structure ID	Structure type	1 % AEP Peak water levels (m AHD)			Increase from Developed Case to 50 % blockage scenario (mm)
		0 % blockage	Developed Case (25 % blockage)	50 % blockage	
C188.72	RCBC	511.83	512.02	512.91	+890
C191.83	RCP	465.92	466.14	466.73	+590
C193.38	RCBC	470.20	470.25	470.34	+90
C193.41	RCP	470.20	470.25	470.34	+90
C195.64	RCP	433.51	433.58	433.67	+90
C195.93	RCP	433.57	433.52	433.60	+80
C196.03	RCP	433.37	433.41	433.46	+50
C197.42	RCP	426.54	426.59	426.66	+70
C197.49	RCP	426.42	426.47	426.57	+100
C197.53	RCP	426.42	426.47	426.56	+90
C197.71	RCP	426.37	426.41	426.49	+80
C198.26	RCP	427.07	427.08	427.12	+40

Table 8.36 outlines the changes in peak water levels at flood sensitive receptors for the 50% blockage scenario where the increase exceeds 10 mm.

Table 8.36 Westbrook and Dry Creeks – Summary of 50% blockage impacts at flood sensitive receptors

Flood sensitive receptor ID	Existing case flood depth (m)	Change in peak water level (mm)
Brimblecombe Road	1.12	+34

Maps demonstrating the effects of blockage are shown in Figures B-5a (0%) and B-5b (50%) in Volume II – Appendix B.

During detail 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.

8.6.4.2 Impacts during extreme events

Table 8.37 outlines the changes in peak water levels at flood sensitive receptors for the extreme events where the increase exceeds 10 mm under one of the events. The existing depth of flooding is also detailed and as can be seen the larger impacts that occur under the PMF event occur generally when there are already high flood depths as would be expected under such a rare event.

Flood immunity of the Project alignment is discussed in Section 8.6.2.3, and maps demonstrating the impacts during extreme events are shown in Volume II – Appendix B, Figures B-4f to B-4h.

Table 8.37 Westbrook and Dry Creeks – Summary of extreme event impacts at flood sensitive receptors

Flood sensitive receptor ID	1 in 2,000 AEP event		1 in 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)
WES_ID_1	+4	0.52	+128	1.17	+846	2.93
WES_ID_2	+19	0.28	+235	0.98	+964	2.79
WES_ID_3	+11	0.38	+214	1.04	+947	2.82
Omara Road	-	3.01	+1	3.11	+19	3.88
Brimblecombe Road	+68	1.26	+110	1.48	+479	2.11
Athol School Road	-	0.57	+2,912	0.76	+3,737	1.13
F G G Couper Road	+1	3.70	+15	5.06	+4	7.38

8.6.4.3 Climate change

The potential impacts of climate change in the Westbrook and Dry Creeks floodplain were assessed for the 1% AEP design event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2016.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario has been adopted for the Project with an associated increase in rainfall intensity of 21% across the catchment area.

For the 1% AEP event, the change in peak water levels for the Representative Concentration Pathways 8.5 climate change scenario is presented in Volume II – Appendix B, Figure B-5c. The change in peak water levels is calculated from the difference between the Developed Case and the Existing Case with 21% increase to rainfall intensity applied to both cases.

The hydraulic model predicts that, with an increase in rainfall intensity of 21% across the catchment, peak water levels increase, and an afflux of more than 500 mm is expected between Dry Creek and Westbrook Creek bridges upstream of the alignment and upstream of culverts C01A, C04 and C05. The Project alignment is predicted still has 1% AEP flood immunity to formation level under the climate change scenario.

Table 8.38 presents the structure performance with Representative Concentration Pathways 8.5 climate change conditions.

Table 8.38 Westbrook and Dry Creeks – 1% AEP event with Representative Concentration Pathways 8.5 conditions – structure performance

Structure ID	Structure type	U/S peak water level (m AHD)	Freeboard to formation level (m)	Outlet velocity (m/s)	Peak flow (m^3/s)
310-BR20	Rail Bridge	433.48	9.2	2.9	75.2
310-BR31	Waterway Bridge	427.07	5.0	2.7	731.7
310-BR32	Waterway Bridge	425.89	3.0	3.3	292.8
310-BR33	Rail Bridge	430.07	6.8	1.5	20.1
C188.72	RCBC	512.72	5.8	5.4	57.5
C191.83	RCP	466.73	20.4	3.3	71.2
C193.38	RCBC	470.37	1.2	2.4	3.0
C193.41	RCP	470.37	0.9	2.8	5.3
C195.64	RCP	433.72	15.2	2.4	15.4
C195.93	RCP	433.67	12.7	2.6	1.5
C196.03	RCP	433.58	11.5	2.0	1.4
C197.42	RCP	426.95	4.3	2.4	120.8
C197.49	RCP	426.87	3.6	2.4	34.8
C197.53	RCP	426.87	3.2	2.3	17.5
C197.71	RCP	426.77	1.9	2.3	23.9
C198.26	RCP	427.29	6.5	1.9	16.8

Table 8.39 outlines the changes in peak water levels at flood sensitive receptors for the climate change scenario where the increase exceeds 10 mm.

Table 8.39 Westbrook and Dry Creeks – Summary of climate change impacts at flood sensitive receptors

Flood sensitive receptor ID	1% AEP climate change event	
	Change in peak water level (mm)	Existing case flood depth (m)
Brimblecombe Road[1]	+45	1.12

Table note:

1 Brimblecombe Road is affected by climate change regardless of the Project and so the amenity of this road is not compromised by the Project

9 Condamine River

The Condamine River drains the northern portion of the Darling Downs and forms part of the Balonne River catchment that in turn drains into the Murray-Darling River basin. The river rises on the western slopes of the Great Dividing Range and drains a total catchment area of approximately 14,000 km^2.

The upper Condamine catchments rise in elevated country around Killarney with elevations up to 1,400 m above sea level, but about two-thirds of the catchment is flat floodplain country where elevations are around 100 to 200 m above sea level. The lower part of the catchment consists of a complex system of rivers and creeks. The eastern part of the catchment has an annual average rainfall of 600 to 800 mm and the floodplains of the south-west have an annual average rainfall of 300 to 500 mm.

Upstream of the proposed Inland Rail alignment between Millmerran and Pittsworth, where the proposed rail crosses the Condamine River floodplain, the river breaks out around the area of Tummaville into a braided and intricate system covering an area around 13 km wide. The floodplain is formed by three main river branches; the Condamine River North Branch, main Condamine River and a southern branch known as Grasstree Creek.

On the Condamine River floodplain, the terrain is flat and the sinuous creek channels begin to break their banks in a 50% AEP event, and then flow between branches in 20% AEP and larger events. Due to the minimal slope throughout the majority of the floodplain, flooding in this area is typically characterised by slow moving flood waters. The Existing Case 1% AEP inundated floodplain width at the Project alignment is approximately 12.5 km.

Under existing conditions, there are multiple pieces of infrastructure impacted by flooding, including the existing QR Rail Line, multiple State-controlled roads, and various structures including houses and sheds. The existing State-controlled roads within the floodplain, which includes the Gore Highway, Millmerran Leyburn Road and Pampas-Horrane Road, all have low flood immunity. The Gore Highway is estimated to have an existing flood immunity of approximately 10% AEP, and Pampas-Horrane Road and Millmerran-Leyburn Road have around 50% AEP flood immunity.

Under the 1% AEP event, the Existing Case peak depth of water is approximately 4 m in the Condamine River channel, 1.5 m in the Condamine North Branch, and approximately 1 m deep in the Grasstree Creek channel.

The location of the Project rail alignment in relation to the Condamine River is shown in Figure C-1a in Volume II – Appendix C.

9.1 Data collection and review – Condamine River

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:

- TRC – existing flood studies
- The BoM – rainfall data
- DTMR – existing infrastructure details.

9.1.1 Previous studies

A number of previous hydrology and hydraulic studies were sourced as part of this assessment. A review of each study was undertaken to determine suitability for use on the Project as documented in the following sections.

Upper Condamine River Flood Study, SKM, 2013

In 2013, SKM undertook a 2D flood study for the Upper Condamine River catchment on behalf of TRC. This flood study included historical and design event modelling and was based on an URBS hydrological model. The hydraulic model utilised a 60 m grid due to lack of LiDAR information at the time. The BoM developed a number of URBS models for the use in its flood forecasting and flood warning system in 2003. The model was calibrated to the 1976 flood event. SKM undertook a review and revision of the BoM URBS model and developed a new Upper Condamine URBS model in 2012. This model was calibrated to a number of flood events including the 2010/11 event.

In 2013 an URBS model for TRC was developed by extending the 2012 Upper Condamine URBS model to Cecil Plains using catchment data from the BoM 2003 URBS model. The extended URBS model was validated against the 1976 and 2010/11 flood events. In 2013 SKM used the extended URBS model to derive the design flows for a number of design events (10%, 5%, 1% and 0.2% AEP) for input into the hydraulic model. The critical duration of the design events was identified as 72 hours. Information regarding observed flood extents during the December 2010/January 2011 flood event was collected around the township of Ellangowan, located approximately 5 km upstream of the SKM model extent, as part of a community consultation process.

Toowoomba Regional Council SP051 Flood Studies - Work Package 7 - 2D Flood Study for areas within the Upper Condamine River Floodplain, SKM, 2014

The Upper Condamine River Flood Study (2013) was updated and published as the Toowoomba Regional Council SP051 Flood Studies - Work Package 7 - 2D Flood Study for areas within the Upper Condamine River Floodplain in June 2014.

Historical Study for Brookstead, WRM Water & Environment, 2014

This study focused on the hydraulic analysis of flooding in the town of Brookstead. The hydraulic modelling was undertaken using a coupled MIKE FLOOD 1D/2D hydrodynamic model and utilised a steady state peak flow of 700 m^3/s with no hydrological modelling as basis. As part of the study a sensitivity analysis was carried out which determined that peak water levels are not very sensitive to changes in flow, roughness or blockage of hydraulic structures.

This study recommended that calibration for at least two historical flood events should be performed to improve the model accuracy. The model developed for this study was validated by adjusting steady-state flows to achieve a match with historic flood marks within ± 0.5 m. A total of eight flood marks for the December 2010 flood event were used for validation, and the modelled peak water levels achieved a match for seven out of eight flood mark heights within the targeted tolerance of ± 0.5 m.

This study also provides a good explanation with regards to the uncertainty associated with observed flood marks, and states: "*Available recorded historical flood information was supplied by TRC. The flood information was collected following the January 2011 event. However, it should be noted that significant flooding occurred in the study area in December 2010 as well as January 2011. It is therefore assumed that the historical flood data collected following the January 2011 event could have originated from either of the two events. Please note that TRC has collected flood data for this study from a variety of sources including debris marks, flood marks visible and accessible at the time of survey after the January 2011 flood, eyewitness accounts, community consultation, etc. It is possible that some the flood data available to TRC may not be accurate or complete. Information used is the best information available at this time for the purposes of this study. Marks observed and other anecdotal information obtained after flood events have been obtained from a range of sources and have varying degrees of certainty*".

Condamine River Flood Study, TRC, 2015

Water Modelling Solutions (WMS) built a new hydraulic model of the Upper Condamine River catchment using the MIKEFLOOD FlexiMesh software on behalf of TRC in 2015. This model used LiDAR data and extended the SKM model domain to include the township of Ellangowan. The model domain extends from approximately 10 km south of Ellangowan to approximately 14 km north of Cecil Plains, and includes the study area for the B2G Hydrology and Flooding assessment.

The model includes cross-drainage structures under the Gore Highway downstream of the Project alignment. Dimensional data for these structures were used in the development of the TUFLOW model for the B2G Hydrology and Flooding assessment.

The MIKEFLOOD model was validated against observed records for the Upper Condamine River catchment by WMS. The model results had reasonable (-0.25 m) fits against observed records at the DNRM gauge (422347 – North Condamine River at Pampas) within the study area.

Inland Rail, Border to Gowrie Phase 1 Report, AECOM, 2017

During 2017 AECOM was commissioned by ARTC to undertake a hydraulic assessment of the Condamine River and floodplain at its intersection with the proposed Inland Rail corridor at the time. The hydraulic assessment was undertaken to establish existing flood conditions, determine potential flood impacts, and inform the design of cross drainage infrastructure to establish comparative cost estimates and enable a Multi-Criteria Analysis (MCA) of the four corridor options under consideration at the time.

The hydrology inflows adopted for this study for the Condamine River and its tributaries were based on flow hydrographs extracted from this WMS MIKEFLOOD model. No additional hydrologic assessment was undertaken. For the hydraulic assessment AECOM developed a hydraulic model using the TUFLOW CPU software package. The model was used to provide an estimate of existing flood levels, extents and velocities; inform the design of the cross-drainage solutions for the Inland Rail Phase 1 concept, and to estimate any potential impacts on flooding as a result of the Project. Tributaries of the Condamine River such as Rocky Creek, Back Creek, Punches Creek and Hermitage Creek were not modelled as part of this work package.

9.1.2 Survey

ARTC provided LiDAR data from 2015 as 1 m grid DEM tiles. Using GIS software, a DEM was generated with a 1 m grid resolution for use in the Project based on the 2015 dataset. This was used for modelling within the disturbance footprint and up to the full extent of the 2015 LiDAR where relevant.

The DEM datasets utilised for modelling were based on surveys flown between 2009 and 2015. SRTM data was used for catchment delineation where no LiDAR data could be sourced, to inform the hydrologic modelling.

The LiDAR data that was used for the Condamine River model development is summarised in Table 9.1.

Table 9.1 LiDAR datasets

LiDAR project	LiDAR location	Date flown	Vertical accuracy	Output grid resolution
Melbourne to Brisbane Inland Rail LiDAR	Millmerran to Toowoomba	26/3/2015	0.15m	1 m
Inland_Towns_Stage_6_2014	CondamineRiv_Twmba_2014_Prj	30/8/2014	0.15 m @ 68% CL	1 m
Inland_Towns_Stage_4_2012	Brookstead_2012_Twn	25/7/2012	0.15 m @ 68% CL	1 m
Toowoomba_2010	Millmerran_2010_Twn	16/7/2010	0.15 m @ 68% CL	1 m
Inland_Towns_Stage_2_2011	Tummaville_2011_Loc	5/6/2011	0.13 m @ 67% CL	1 m
Inland_Towns_Stage_2_2011	Leyburn_2011_Twn	5/6/2011	0.13 m @ 67% CL	1 m

LiDAR project	LiDAR location	Date flown	Vertical accuracy	Output grid resolution
Southern_Downs_2010	Southern_Downs_2010_Rgn	23/10/2010	0.15 m @ 68% CL	1 m
Toowoomba_2010	Tummaville_2010_Loc	16/7/2010	0.15 m @ 68% CL	1 m
Inland_Towns_Stage_6_2014	Clifton_2014_Twn	30/8/2014	0.15 m @ 68% CL	1 m
Toowoomba_2010	Clifton_2010_Twn	16/7/2010	0.15 m @ 68% CL	1 m
Inland_Towns_Stage_4_2012	Maryvale_to_Goomburra_2012_Rgn	31/7/2012	0.15 m @ 68% CL	1 m

Detailed ground and structure survey was incorporated at the Gore Highway crossing at the main Condamine River branch.

The survey data sources and DEM developed for the Condamine River are shown in Figure C-1b in Volume II – Appendix C.

Discrepancies between differently dated LiDAR datasets have been observed, which may be attributed to landuse practises and seasonal potentially vegetation cover. Updated LiDAR data for the whole model domain will be acquired to facilities model updates during detailed design.

9.1.3 Aerial imagery

Aerial imagery of the study area was provided by ARTC and was used to identify and confirm topographic and vegetative characteristics of the study area. Aerial imagery captured in 2015 was made available. Additional imagery outside the study area was sourced from QGIS imagery in an open source format.

9.1.4 Existing drainage structure data

The DTMR GIMS database contained structure information for the following bridges:

- Back Creek Bridge on the Gore Highway – Bridge ID 24899. Dwg. 114676
- Condamine River Bridge on the Gore Highway – Bridge ID 361. Dwg. 149608B
- Bridges on Millmerran-Leyburn Road – Bridge ID 243. Dwg. 166904, Bridge ID 244. Dwg. 42359, Bridge ID 245. Dwg. 42359
- Bridge on Toowoomba-Karara Road – Bridge ID 582. Dwg. 276940, Bridge ID 581. Dwg. 150712, Bridge ID 580. Dwg. 51744.

This structure data was built into the hydraulic model.

9.1.5 Stream gauge data

There are 21 major stream gauges within the study area with 16 operated by Queensland Government's Department of Natural Resources Mines and Energy (DNRME), 4 by SunWater, and one by TRC. Although a large number of stream gauges exist within the study catchment area, most are of limited value to the development of the flood model. Many of the stream gauges either have a short or incomplete data records or are positioned on a small tributary of the Condamine River. The gauges used for model calibration were selected by their data quality and quantity, proximity to the rail alignment and the proportion of catchment area that drains to them. Key gauges used, and their purpose are summarised in Table 9.2. Data at these gauges was sourced from DNRME Water Monitoring Information Portal (WMIP) and requested from SunWater directly.

Due to floodwaters in larger events breaking out and recombining within the Condamine floodplain, particularly downstream of Tummaville, flood flow estimates at some gauges are reliable only for small events. Despite this, the water levels recorded at the gauges are reliable and are useful for hydraulic model calibration. Gauges at Tummaville, Yarramalong and Pampas were used for hydraulic model development. Flood height data from the Centenary Bridge BoM logbook was used for the 1991 event.

The accuracy of the stream gauges varies considerably in the Upper Condamine catchment. The stream gauge at Warwick is rated as reliable up to a depth of 10.61 m, which corresponds to a flood flow of 3,160 m^3/s. The highest recorded gauge height was 3.79 m on 23 December 1975.

Moving downstream to Talgai Tailwater (TW), the gauge is situated upstream of a stream confluence as shown in Figure 20. Talgai TW is rated as reliable up to a depth of 4.77 m which corresponds to a flood flow of 910 m^3/s. Above this height, flows are approximated using log-log extrapolation up to 3,000 m^3/s at 13.8 m deep. The highest record gauge height was 6.58 m on 6 May 1996.

Adding uncertainty to the reliability of data at Talgai TW gauge is the Dalrymple Creek catchment to the east (approximately 500 km^2 in size). Due to upstream breakouts, flood flows from Dalrymple Creek may get recorded at the Talgai TW gauge during larger events.

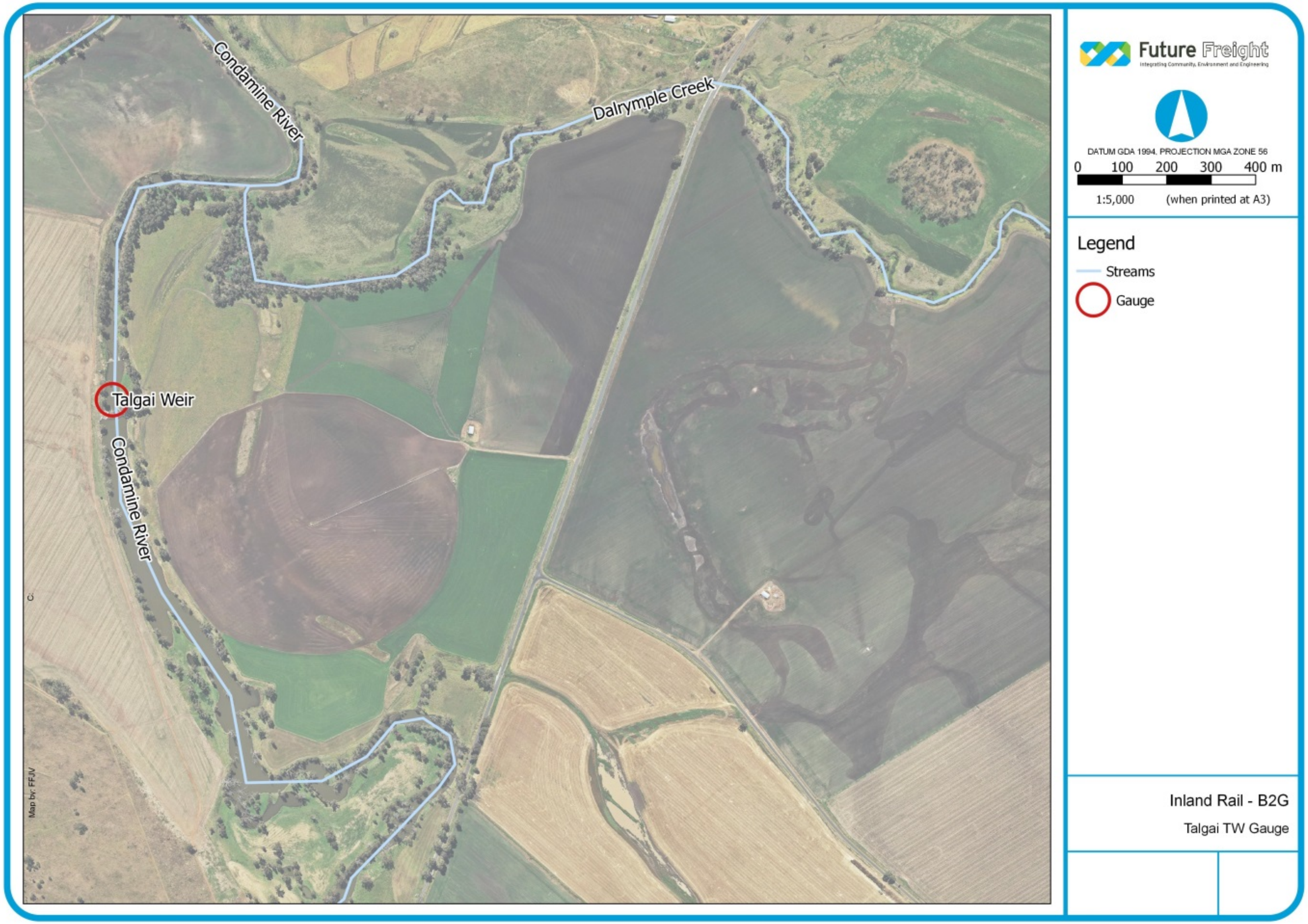

Figure 20 **Talgai TW gauge location**

Table 9.2 Stream gauges adopted

Gauge	Name	Catchment area (km^2)	Owner	Latitude (ºS)	Longitude (ºE)	Start	End	Purpose
422310C	Condamine River at Warwick	1,360	DNRME	28.214	152.049	1/10/1960	Not closed	Calibration of hydrological model using flow data
422355A	Condamine River at Talgai TW	3,105	DNRME	27.99	151.758	26/10/1989	Not closed	Calibration of hydrological model using flow data
422323A	Condamine River at Tummaville	6,475	DNRME	27.87	151.511	29/08/1961	Not closed	Calibration of hydrological using flow data and hydraulic model using height data
422347B	North Condamine River at Pampas	378[1]	DNRME	27.783	151.424	25/03/1988	Not closed	Calibration of hydraulic model using height data
422353A	Yarramalong Weir TW	-	SunWater	27.836	151.449	28/10/1989	Not closed	Calibration of hydraulic model using height data
422936	Centenary Bridge on Gore Highway	-	Toowoomba Regional Council/ Lyndon Pfeffer	27.809	151.363	1/2/1982	-	Validation of hydraulic model using height data. Note: Automatic data logging begun December 2017. Prior events from February 1982 were recorded by logbook.
422316A	Condamine River at Cecil Weir	7,795[1]	DNRME	27.534	151.203	24/10/1947	Not closed	Calibration of hydrological model using flow data
422345	North Condamine River at Lone Pine	710[1]	DNRME	27.669	151.347	13/10/1978	Not closed	Calibration of hydrological model using flow data

Table note:

1 Catchment areas stated in the table above are approximate. In larger events, floodwaters from the Condamine River breakout near Tummaville and flow into the North Branch where the Pampas and Lone Pine gauges are located

The Tummaville gauge is rated as reliable up to a depth of 4.23 m, which corresponds to a flood flow of 160 m^3/s. The highest recorded gauge height was 8.75 m on 30 January 2013. A flood flow of 700 m^3/s at 8.93 m deep is rated a fair estimate, but around this level flood waters have already broken out significantly across the floodplain to the north and all estimates of flood flows are unreliable.

The Tummaville gauge was re-rated several times during its record, which has resulted in considerable variation in flow estimates, particularly at large depths. The most recent rating curve was adopted some time in 2008 and subsequent flood events have smaller recorded flows for a given depth than equivalent floods during the 1970s, 1980s and 1990s. For example, the depths recorded at Tummaville during the 1976 and 2010 events were very similar, 11.19 m and 11.14 m respectively, yet the estimated flows were 1,788.4 m^3/s and 957.9 m^3/s respectively.

Adding further uncertainty to the flows estimated at the Tummaville gauge, there are substantial breakouts upstream of the Tummaville gauge which allow floodwaters to completely bypass the gauge (refer Figure 21) and not be recorded. Therefore, using this gauge for hydrological model development was only considered for small events and with great caution.

Downstream of the Project alignment are the Cecil Weir and Lone Pine gauges. Cecil Weir is rated as reliable up to a depth of 5.55 m, which corresponds to a flood flow of 2,025 m^3/s. The highest recorded gauge height was 4.84 m on 6 April 1988.

Lone Pine is a very small gauge used for monitoring low flows and is rated as accurate up to a depth of 1.5 m with a flow of 1.0 m^3/s. Above 1.5 m, the flood flows are estimated to be 1,000 m^3/s at 3.7 m deep. The highest recorded gauge height was 1.66 m on 16 February 1988.

Flows at these gauges were combined to represent all floodwaters exiting the study area. These combined flows were used to aid in the calibration of the hydrological model. The limited accuracy of the Lone Pine gauge was outweighed significantly by the higher accuracy of the Cecil Weir gauge.

9.1.6 Historical rainfall data

Historical rainfall information used for model development came in two forms: gauging station records and gridded daily rainfall data.

There are over 200 daily rainfall gauging stations in and around the Upper Condamine catchment. Data at these gauges was sourced from the BoM website. Sub-daily data (pluviograph data) is available at six-minute intervals and this resolution facilitates a better understanding of the temporal patterns of storms for calibration.

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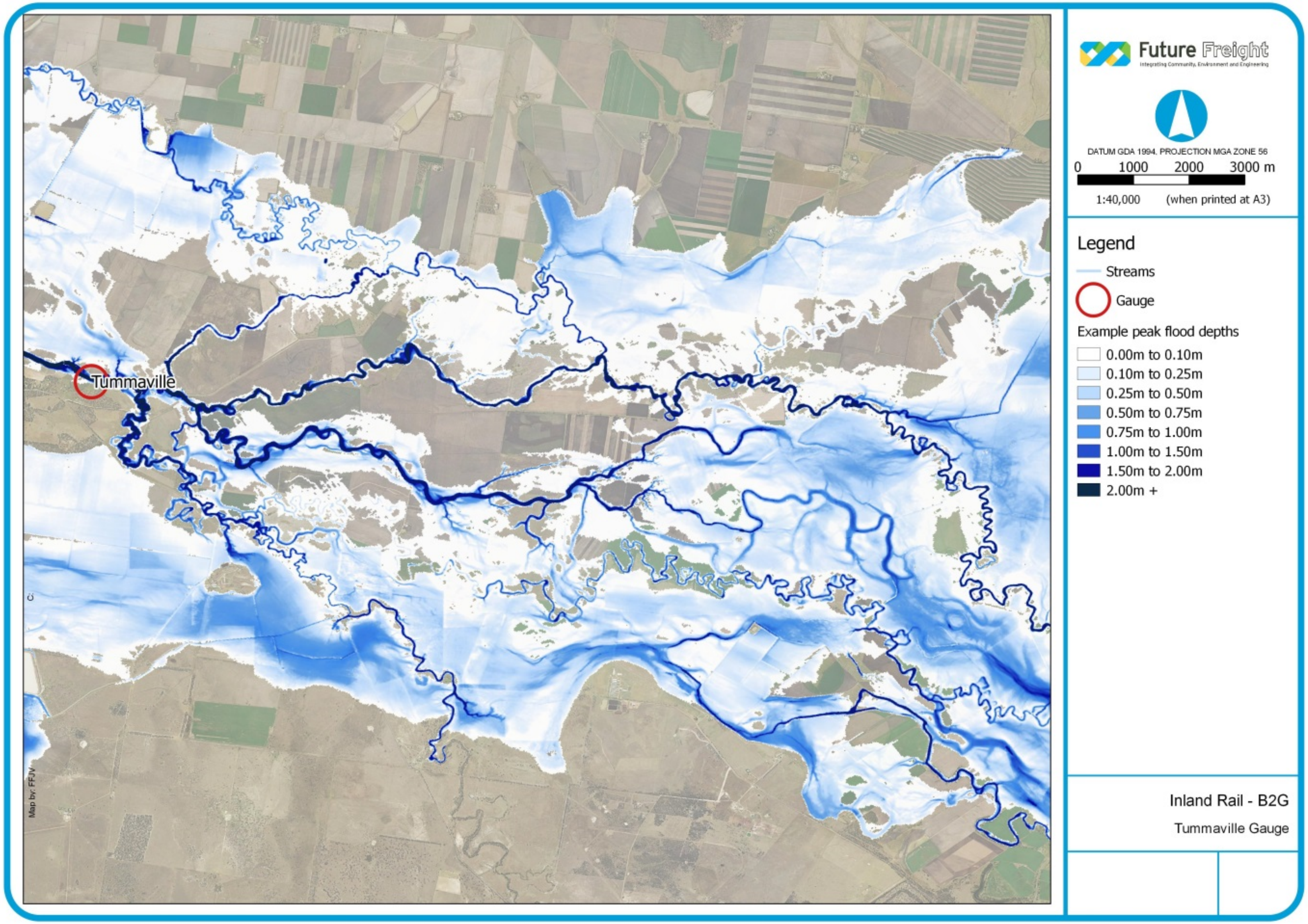


Figure 21 **Tummaville gauge location**

Data for the gauges as listed in Table 9.3 was sourced from the BoM.

Table 9.3 Pluviograph data

Gauge No.	Name	Longitude	Latitude	Record start	Record end
041018	Clifton Post Office	-27.9317	151.9058	19/09/1972	31/07/2010
041044	Hermitage	-28.2061	152.1003	18/02/1952	28/02/2001
041056	Killarney Post Office	-28.3344	152.2953	24/09/1972	31/01/2010
041445	Leslie Dam	-28.2144	151.9194	30/04/1986	30/06/2009
041060	Leyburn	-28.0092	151.5861	1/01/1959	21/11/1996
041063	Leyburn Post Office	-28.0106	151.5856	1/03/1997	31/05/2010
041361	Pittsworth DPI	-27.7167	151.6292	9/09/1970	14/12/1984
041082	Pittsworth	-27.7156	151.6333	25/03/1959	24/02/1996
041525	Warwick	-28.2061	152.1003	1/04/1999	30/04/2010
041467	Toowoomba City Council	-27.5667	151.8850	1/01/1957	1/01/1984

The Queensland Government also provides gridded climatic data through the Scientific Information for Land Owners (SILO) online portal. The data uses advanced splining or kriging techniques which spatially interpolate between data points – in this case, rainfall gauge information, in order to provide an estimate of daily rainfall depths across Australia.

9.1.7 Anecdotal and observed flood data

Local landowners provided the following anecdotal flood information which was invaluable for flood model calibration and validation:

Flood marks on local landowners' properties. With landowner permission, some of these flood marks were surveyed and their heights assisted in validating the hydraulic model. See Figure 51 and Figure 53 for location of marks and indication of their reliability. Validation of hydraulic modelling results against floodmark data is discussed further in Section 9.3.6.

- Timestamped photographs of historical floods:
  - Some photographs were taken from aircraft which provided information on broader flood extents
  - Some photographs show flood heights at buildings and fences
  - The results of the hydraulic model were compared against the information shown in these photos
- Anecdotes of flood extents and behaviour. For instance, landowners have provided insight into:
  - Where and how floods breakout from the main Condamine flow paths
  - The velocity of flows, which tend to vary considerably across the floodplain
  - How flood direction can be significantly affected by accumulated flood debris and crop types
  - The impact that the Gore Highway upgrades have had on flood behaviour
  - Their concern about scour at culverts and other locations of concentrated flow.

9.1.8 Site inspection

A site inspection was undertaken during February 2018. During the site inspection, all major waterway crossings were visited and inspected with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

9.2 Hydrologic model development – Condamine River

9.2.1 Model setup

It is understood that Toowoomba Regional Council, SunWater and BoM possess calibrated hydrological models of the Upper Condamine catchment. However, as none of these models were made available for use on this assessment, a new, calibrated model was built to establish historical event flows and design flood event flows for input into the hydraulic model.

A runoff routing network model (hydrologic model) was established using the latest procedures detailed in ARR 2016 and using the latest rainfall data from BoM. The Unified River Basin Simulator (URBS) software package was chosen for estimating flood flows in the Condamine River catchment. URBS is a sophisticated rainfall runoff program that was widely used in the hydrology industry for over 25 years.

The hydrologic model covers approximately 8,542 km^2 of the Condamine River catchment, spanning the upper reaches at Killarney and extending 146 km downriver to Cecil Plains. The catchment was delineated into 147 sub-catchments to capture the variability of hydrological parameters such as losses and rainfall, and to better represent the network of creeks and streams within the catchment.

These sub-catchments were grouped into four subzones according to the stream gauges being used for calibration (refer Section 9.2.3). The hydrologic model setup including extent and sub-catchment map is presented in Figure C-1c in Volume II – Appendix C.

Table 9.4 Stream gauges adopted for calibration

URBS sub-model	Stream gauges used for calibration
A	422310C – Condamine River at Warwick
B	422355A – Condamine River at Talgai Tailwater
C	422323A – Condamine River at Tummaville
D	422345A – North Condamine River at Lone Pine, 422316A – Condamine River at Cecil Weir

URBS can accommodate up to seven routing variables which have varying degrees of influence on the size and shape of flood hydrographs produced by the model. This model uses only two routing variables (catchment areas and reach lengths) to estimate flood flows. During the model calibration process, two additional routing variables were tested (channel slope and catchment slope) with negligible improvement to model outputs. Therefore, these additional parameters were omitted from the model.

It must be noted that Leslie and Connolly Dams are situated in the upper reaches of the Condamine River catchment. Given their catchment sizes compared to the size of the overall catchment is small and their storage Volume is also small, any contributing attenuation from these dams would be negligible. Therefore, both dams have not been included in the URBS model.

A summary of the URBS model inputs is provided in Table 9.5.

Table 9.5 Summary of URBS model calibration inputs

Input parameter	Remarks
URBS model type	Split
Routing variables	Catchment area, stream lengths
Rainfall data used	Daily data at over 200 rainfall gauges downloaded from the BoM's database.
Pluviograph data used	BoM pluviographs - 041018, 041044, 041056, 041445, 041060, 041063, 041361, 041082, 041525, 041467 ,041457, 040677, 041359, 041352, 041536, 422310C, 422313B, 422355A, 422394A, 541041, 541062
Gridded rainfall data used	Daily rainfall data was sourced from SILO's online portal
Stream gauge data used	DNRME's gauges at 422310C, 422355A, 422323A, 422347B, 422316A

9.2.2 Event selection for calibration

Stream gauges within the catchment were prioritised by quality of gauge record and their position within the catchment. Five gauges were identified as high priority for model calibration (refer Table 9.4). Flood frequency analyses (FFA) using Log Pearson Type III to fit the data were conducted at each of these gauges to determine key flood events which would be suitable for model calibration purposes. The aim was to select one large and one small flood event for calibration to provide confidence that the final, calibrated model can replicate events of these magnitudes and be used to interpolate/extrapolate floods of other magnitudes.

The two largest events on record appear to be the 1956 and 1976 events, of which several community members recall their magnitudes and impacts. However, available quality data for these events is scarce and the floodplain has changed significantly in the last 30 to 50 years. Therefore, calibrating to these events would not produce a reliable model. Instead the focus shifted to the December 2010 and January 2011 events. Both events were large and there exists a large amount of stream gauge and rainfall information for these events.

Furthermore, local landowners were able to provide valuable anecdotal information in the form of photographs and flood marks on their properties. The December 2010 event was finally chosen since the January 2011 event began before the 2010 event had fully finished.

Selecting a smaller event was somewhat more difficult than choosing a large event. Farmers are legally permitted to pump floodwater from the Condamine River and its various branches into dams (or ring tanks) for irrigation. Given the scale of agriculture in the Condamine floodplain, the amount of water harvested can be significant compared to the size of the flood. For instance, in the 2004 event, flood Volume recorded at the Cecil Weir gauge was approximately half of what was recorded at the Talgai TW gauge 75 km upstream. Calibrating a hydrological model to event like 2004 is risky and may produce an unreliable model.

Using historical aerial photography, it can be seen that the number of ring tanks in the area grew substantially in the mid-to-late 1990s, which correlates to larger volumes of floodwater being pumped from the floodplain during flood events.

Within the extent of the regional TUFLOW model, as described in Section 8.3, a total of 63 significant ring tanks were identified. The construction dates were estimated using historical aerial photographs from Google Earth ®. The number of ring tanks built since 1984, as well as the cumulative increase, is shown in Figure 22.

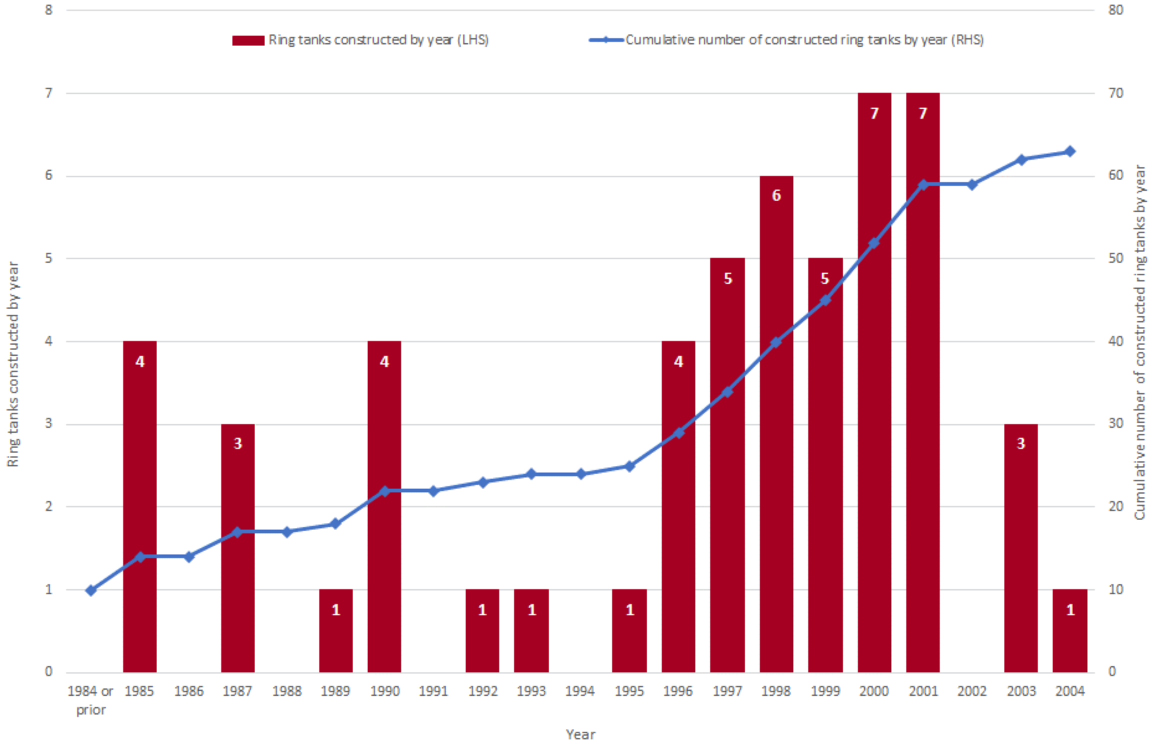


Figure 22 **Estimated proliferation of ring tanks within the regional flood model extent**

The February 1991 event and December 2010 event were selected for calibration as they represented a relatively small and relatively large flood event respectively. A third event was selected for validation purposes. The January 2013 was judged to be a suitable event due its magnitude and recent occurrence. A summary of peak depths and flows recorded at gauges for each flood event and their estimated AEP is summarised in Table 9.6. For comparison purposes, the 1956 and 1976 events were included.

Table 9.6 Historic gauged results

Event	Warwick (422310C)	Talgai TW (422355A)	Tummaville (422323A)	Cecil Weir (422316A)
1956	No data	No data	No data	8.8 m – 1,717.8 m^3/s 3.7% AEP
1976	8.3 m – 1,519.4 m^3/s 0.6% AEP	No data	11.2 m – 1,788.4 m^3/s 1.0% AEP	9.2 m – 2,023.9 m^3/s 2.2% AEP
1991	3.9 m – 236.4 m^3/s 33.2% AEP	14.2 m – 290.6 m^3/s 26.0% AEP	8.3 m – 602.2 m^3/s 25.1% AEP	7.5 m – 872.4 m^3/s 23.3% AEP
2010	7.1 m – 1,029.9 m^3/s 2.6% AEP	15.7 m – 2,264.2 m^3/s 8.9% AEP	11.1 m – 957.9 m^3/s 13.0% AEP	9.2 m – 2,046.8 m^3/s 0.8% AEP
2013	6.9 m – 956.1 m^3/s 3.7% AEP	15.7 m – 2,375 m^3/s 2.1% AEP	10.8 m – 866.5 m^3/s 19.9% AEP	8.3 m - 1,342.9 m^3/s 12.1% AEP

9.2.2.1 February 1991 event

The 1991 event appeared to be of suitably small magnitude, early enough to have negligible pumping intervention, and recent enough to have good rainfall and stream gauge data coverage.

Historical aerial photography indicates that very few private storages had been constructed prior to the 1991 event. It is estimated that as few as three had been constructed along the Condamine River between Tummaville and the Gore Highway; one on Grasstree Creek; and one on the North Branch near Pampas. The potential influence of irrigation pumping would be minimal on this event. FFAs at the Warwick and Cecil Weir gauges estimate the February 1991 event was a 50 to 20% AEP event. The spatial distribution of rainfall during the 1991 event is shown in Figure 23.

9.2.2.2 December 2010 event

In the months leading up to the 2010 flood event there were a series of large convective storms over South East Queensland. It is highly likely the catchment was saturated, and all private storages were at full capacity, despite the large number of private storages constructed by that time. Additionally, the rainfall event that contributed to the flood lasted at least three weeks, which suggests that if any storages were not already full, any pumping to top them up would have ceased prior to the flood peak. The potential influence of irrigation pumping would be minimal on this event. FFAs at the Warwick and Cecil Weir gauges estimate the December 2010 was approximately a 5% to 1% event. The spatial distribution of rainfall during the 2010 event is shown in Figure 24.

9.2.2.3 January 2013 event

The 2013 flood event in the Upper Condamine catchment was one of the largest on record. The event reached a peak depth of 10.76 m at Cecil Weir at approximately 1pm on 29th January 2013. The rainfall burst which contributed to the peak flood event lasted approximately six days. On average, 160 mm fell across the entire Upper Condamine catchment during this time. Some areas upstream of Warwick received in excess of 540 mm during this period. The event is estimated to be a 16% AEP flood event at Cecil Weir. The spatial distribution of rainfall during the 2010 event is shown in Figure 25.

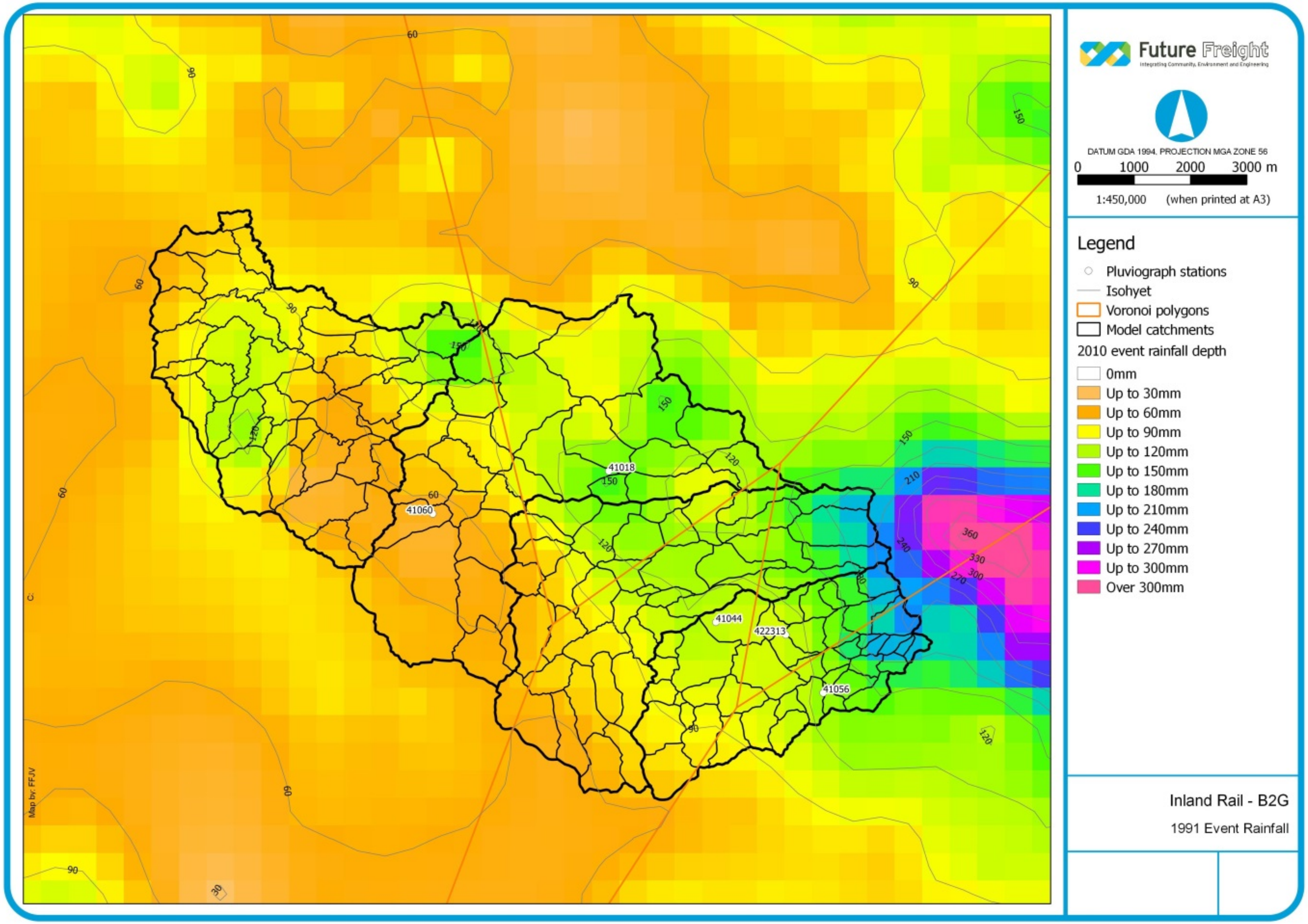


Figure 23 Gridded rainfall depths during 1991 event

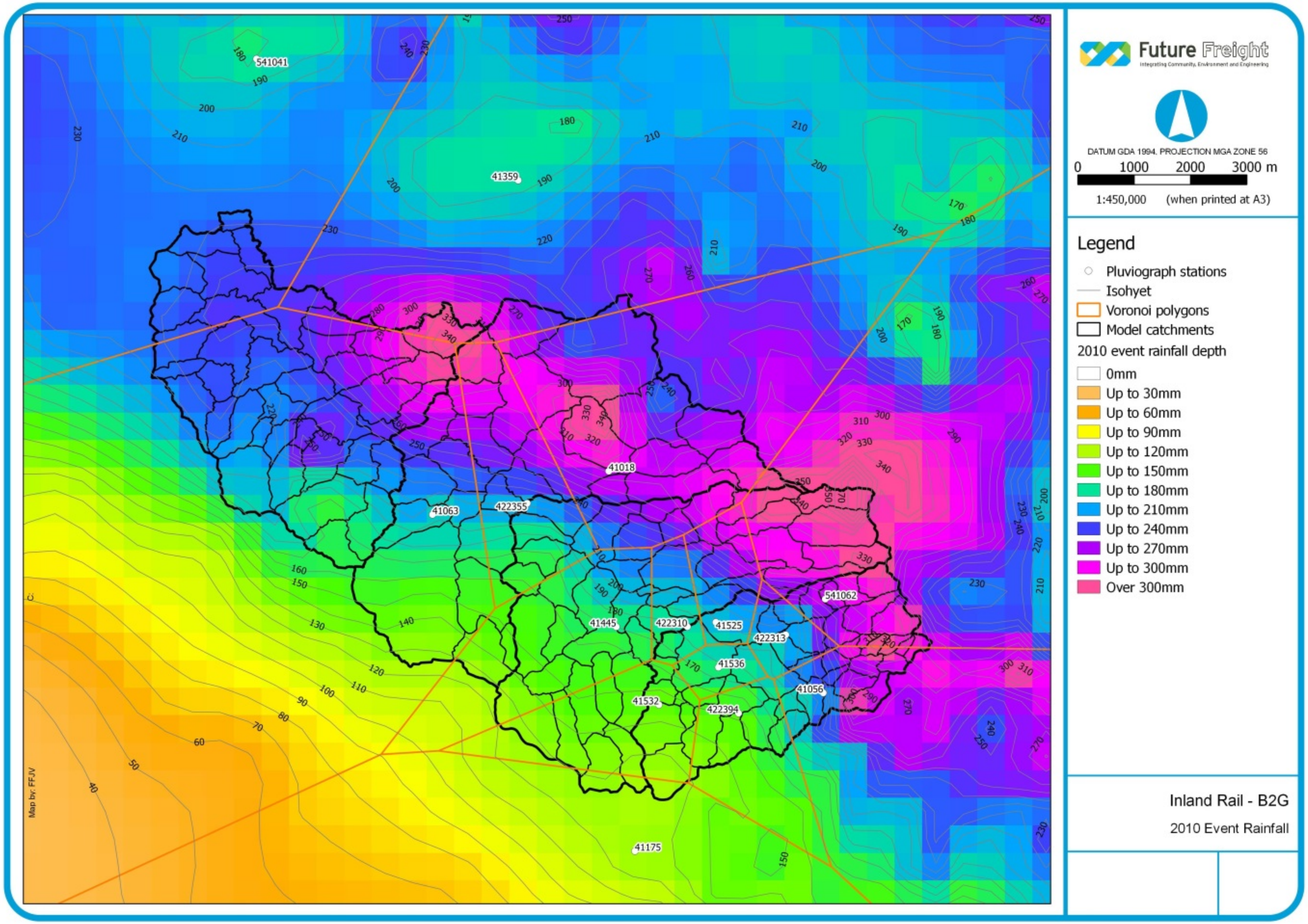


Figure 24 Gridded rainfall depths during 2010 event

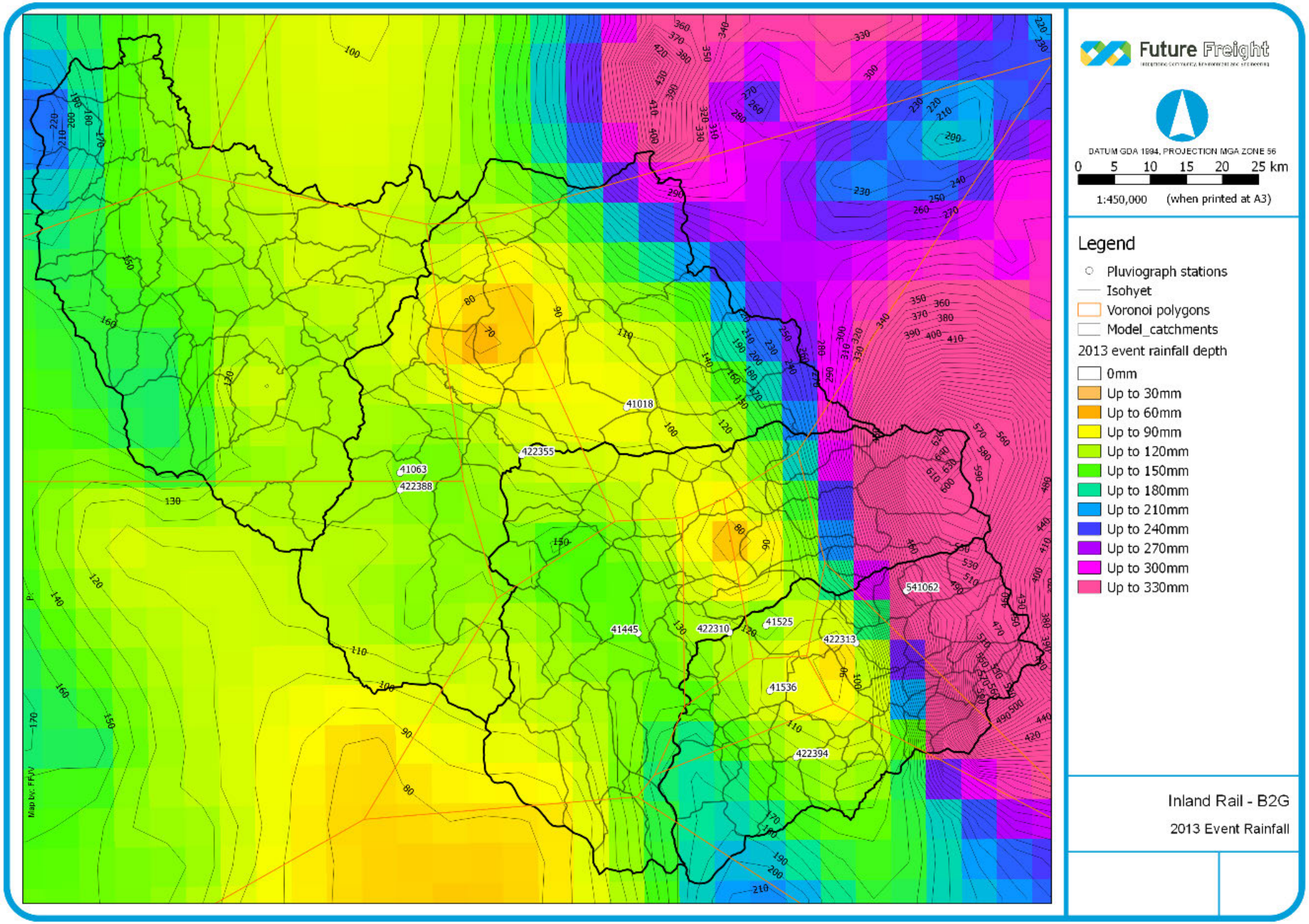


Figure 25 Gridded rainfall depths during 2013 event

9.2.3 Model calibration

Gridded rainfall data provided an estimate of rainfall variation across the entire model catchment for each calibration event (refer Figure 23, Figure 24 and Figure 25). Area-weighted averages of gridded rainfall depths were applied to each model sub-catchment. Pluviograph data was used to capture the temporality of each calibration event. The sub-daily rainfall patterns from the pluviographs were applied to each model sub-catchment based on their proximity to the gauges using Voronoi analysis.

The rainfall depths and temporal patterns were applied to the URBS model to produce estimates of flood hydrographs at the stream gauges identified for calibration. URBS model routing parameters alpha, beta, m, rf, and the initial and continuing rainfall losses (IL and CL) were adjusted until good fits to historical stream gauge hydrographs was obtained.

Measures of good fit to historical data were:

- Peak flow
- Hydrograph volume
- Time to peak
- Matching shape of flood event

As an overall check of flood flow estimation within the model, the URBS hydrographs at Cecil Weir and Lone Pine were combined and compared against a combined stream gauge hydrograph. This combination is referred to as the model 'Outlet'.

Calibration hydrographs at each gauge and calibration event are displayed in Figure 26 to Figure 29 (1991 event) and Figure 30 to Figure 33 (2010 event).

A summary of the parameters used to achieve the calibration hydrographs is contained in Table 9.7 (1991 event) and Table 9.9 (2010 event).

The performance metrics of the calibration are summarised in Table 9.8 (1991 event) and Table 9.10 (2010 event). A discussion on results follows the figures and tables.

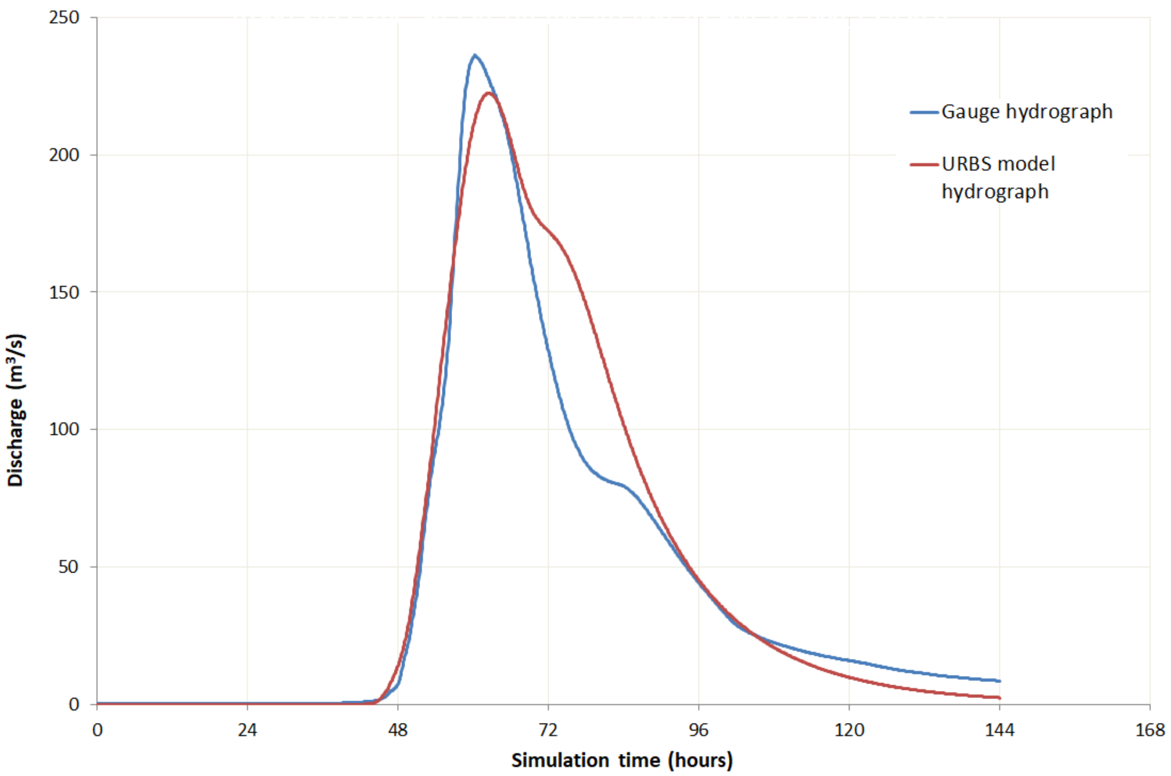


Figure 26 Calibration hydrographs at Warwick 422310C – 1991 event

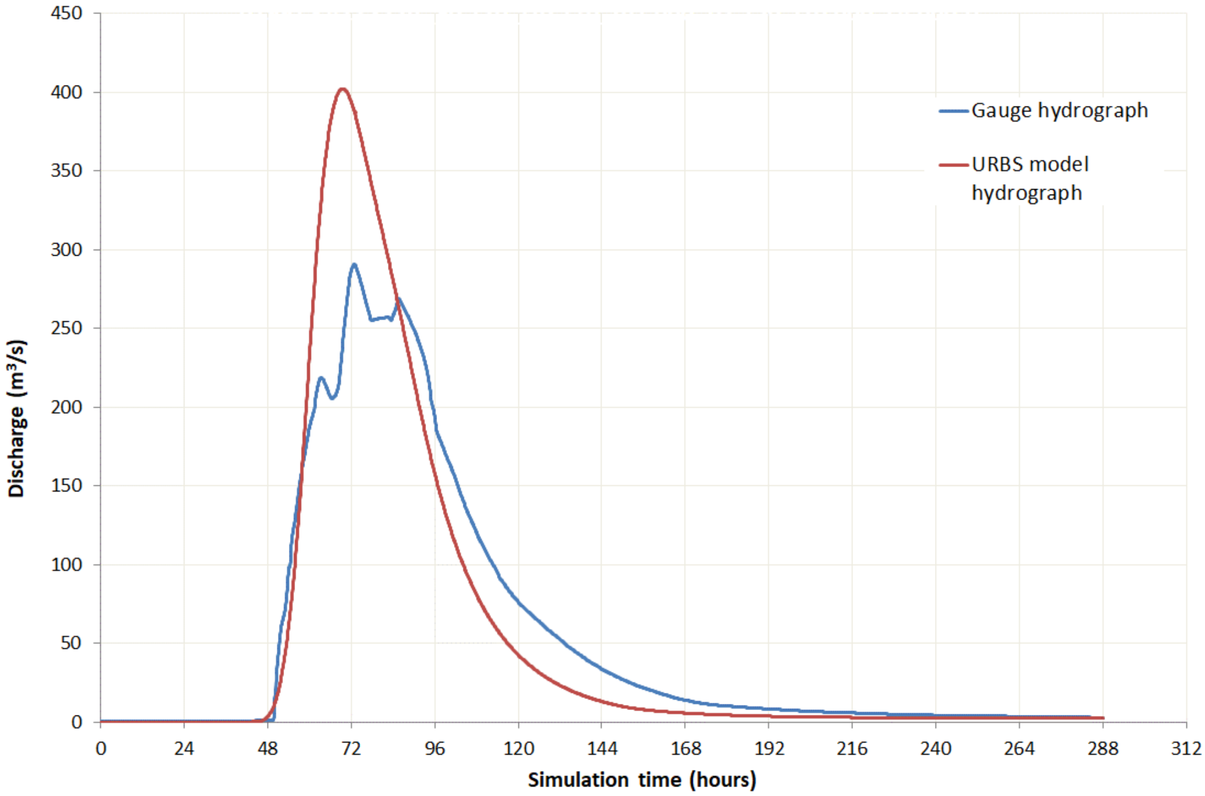


Figure 27 **Calibration hydrographs at Talgai TW 422355A – 1991 event**

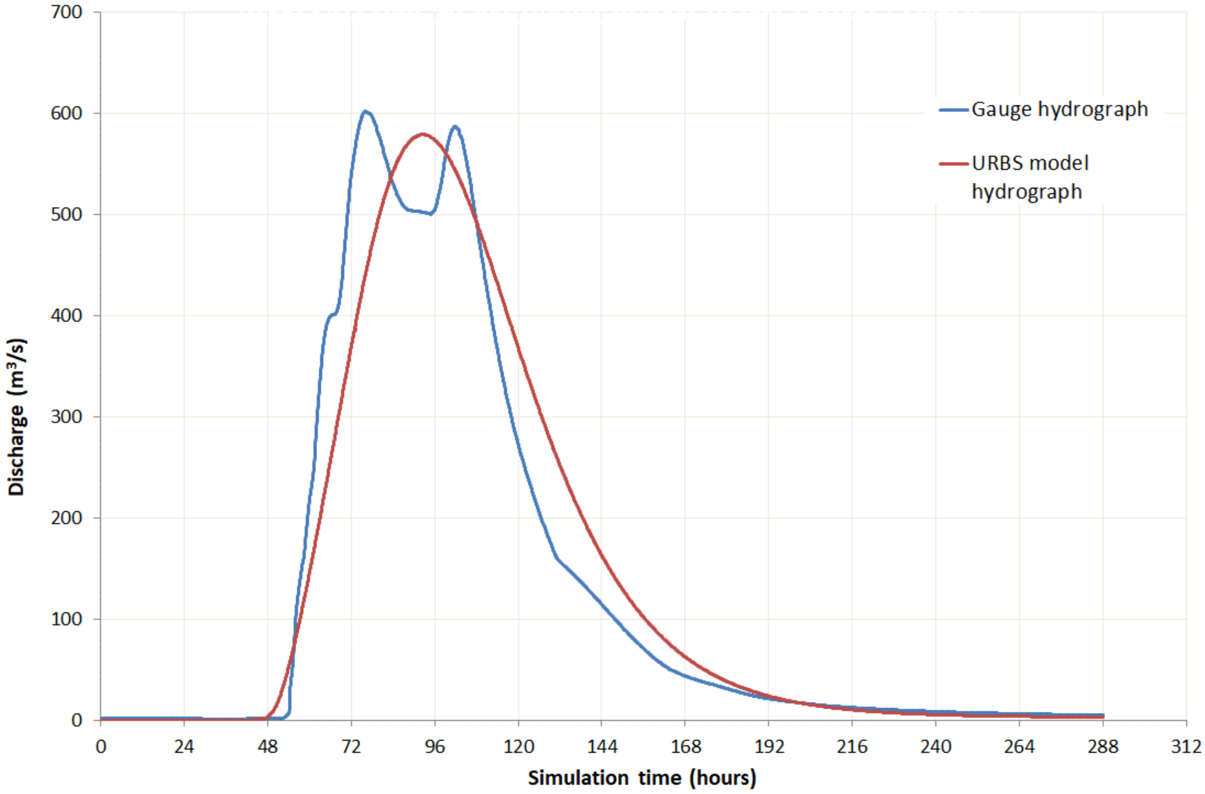


Figure 28 **Calibration hydrographs at Condamine River at Tummaville 422323A – 1991 event**

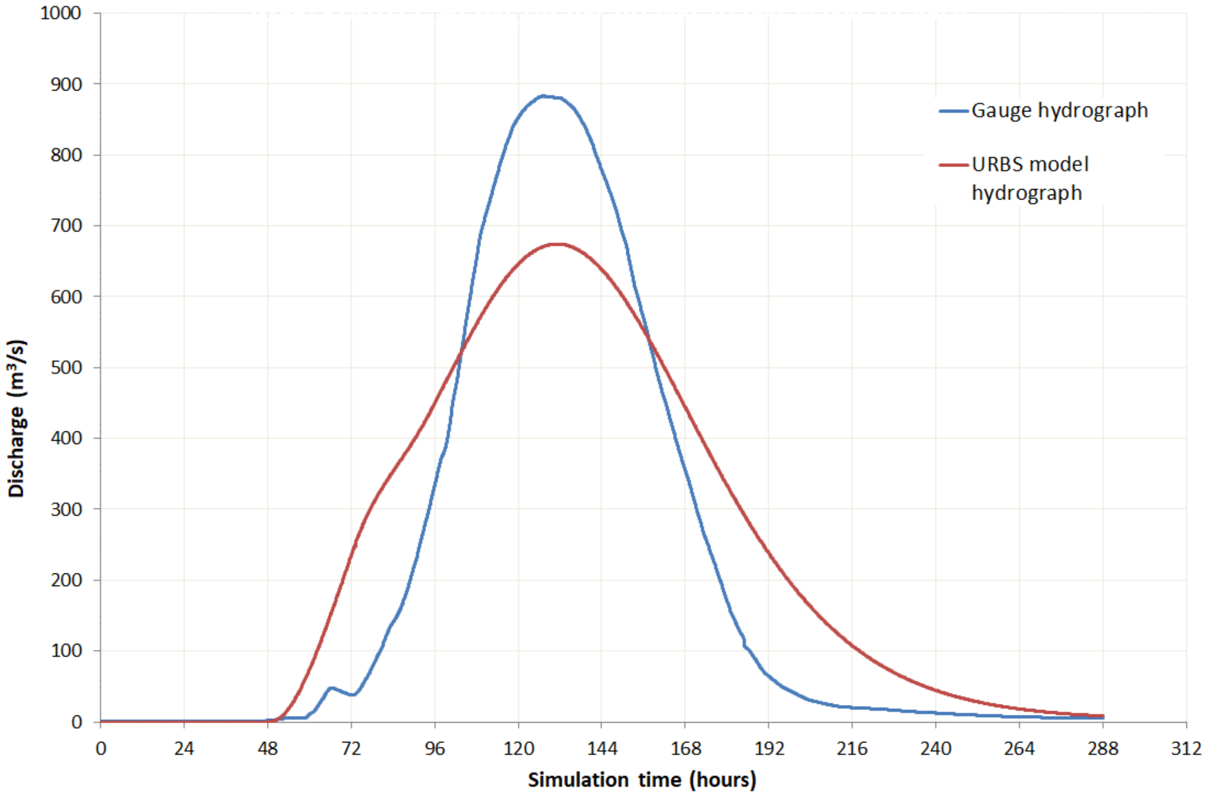


Figure 29 **Calibration hydrographs at combined 'outlet' – 1991 event**

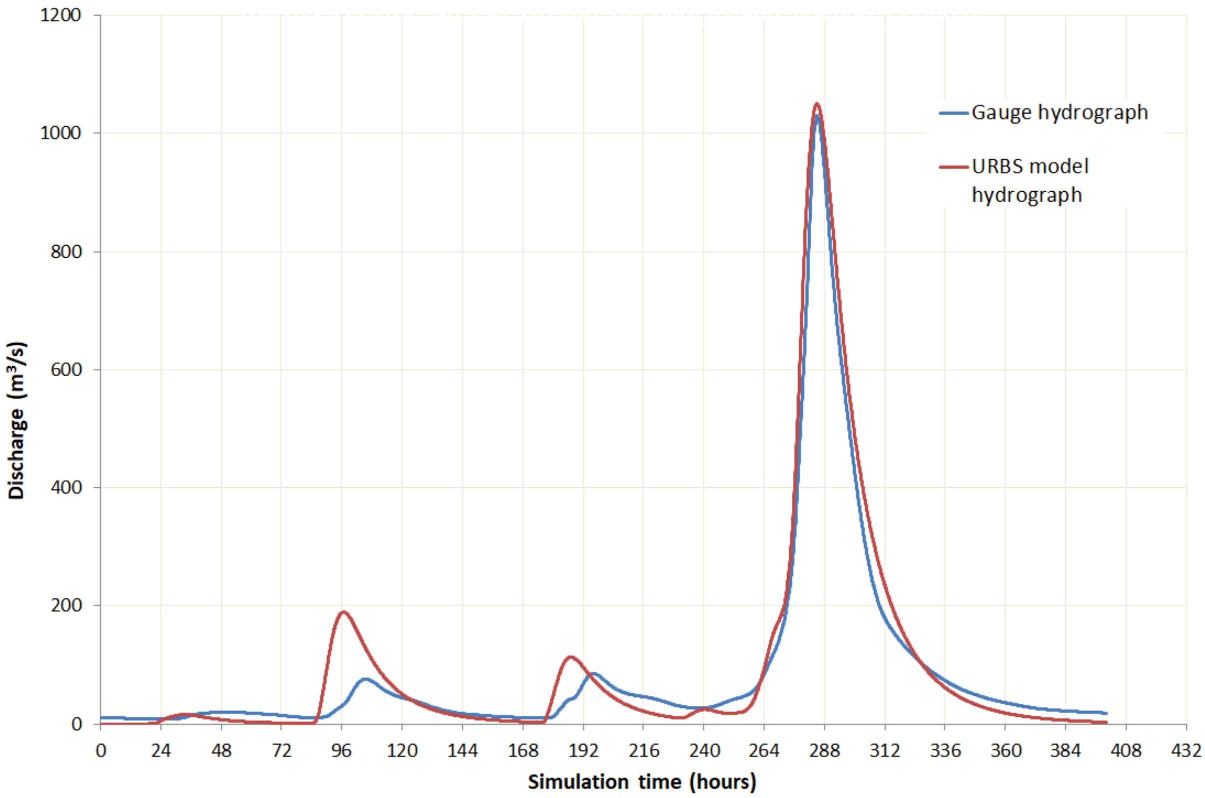


Figure 30 **Calibration hydrographs at Warwick 422310C – 2010 event**

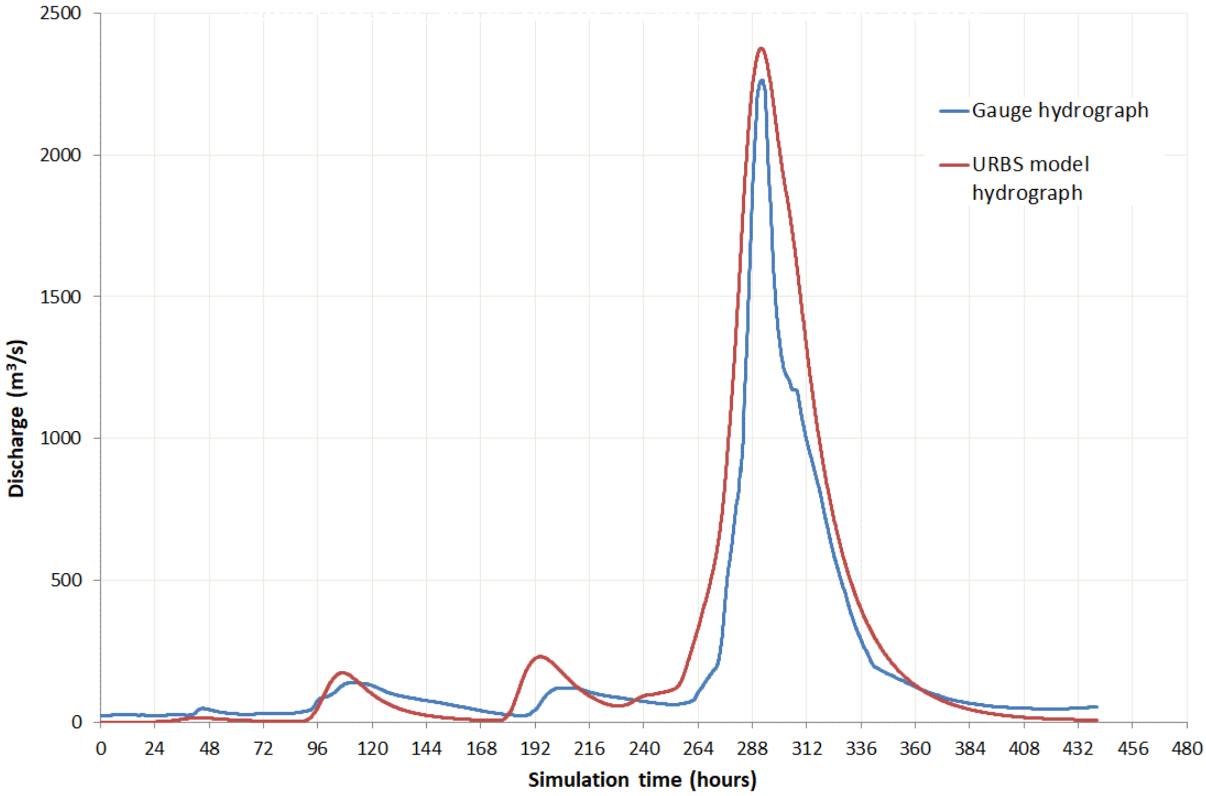


Figure 31 Calibration hydrographs at Talgai TW 422355A – 2010 event

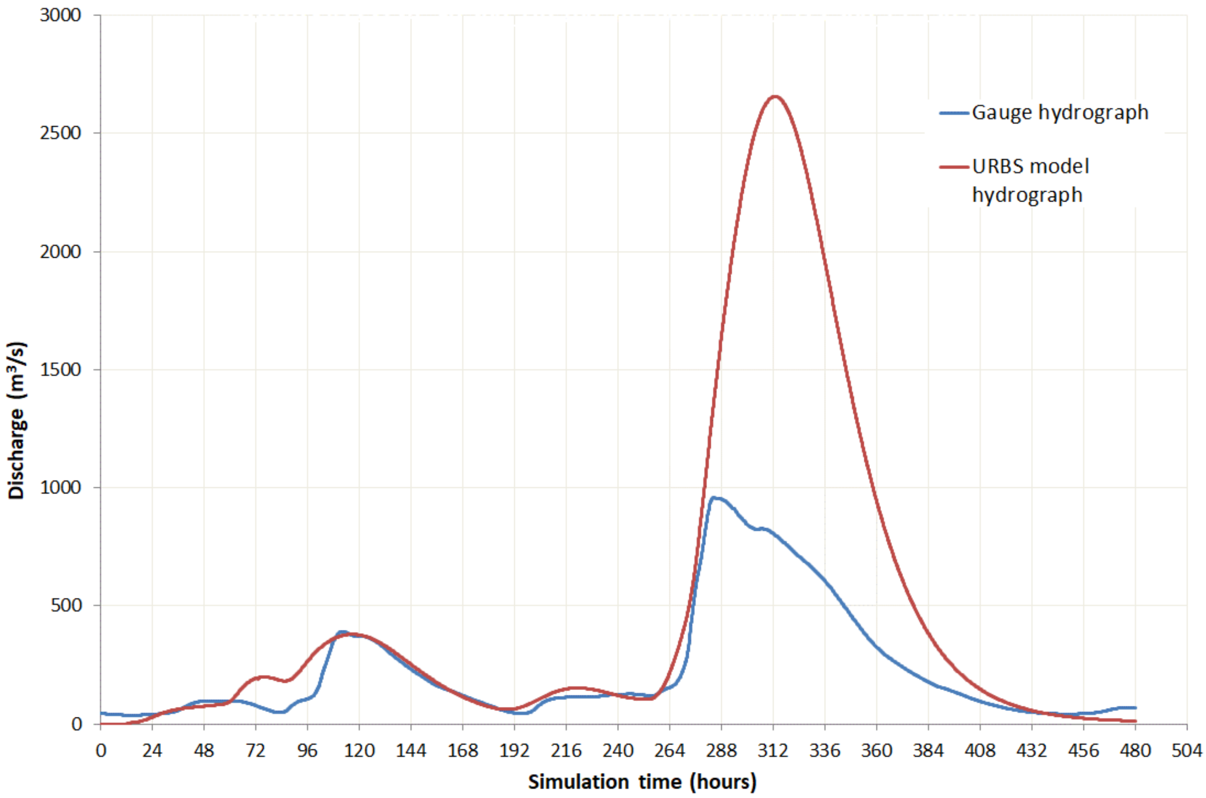


Figure 32 Calibration hydrographs at Condamine River at Tummaville 423323A – 2010 event

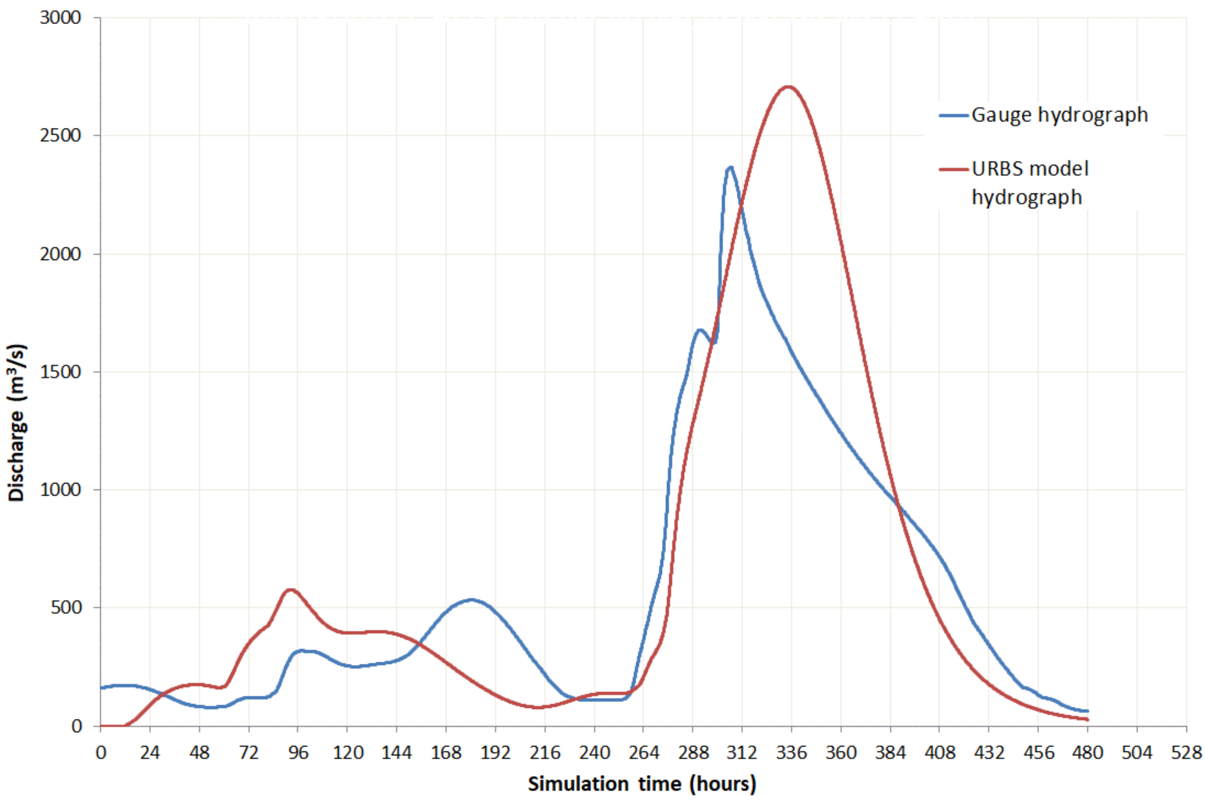


Figure 33 **Calibration hydrographs at combined 'outlet' – 2010 event**

Table 9.7 **Summary of URBS model calibration parameters – 1991 event**

Parameter description		Sub-model A	Sub-model B	Sub-model C	Sub-model D
Channel lag parameter	α	0.15	0.12	0.30	0.26
Catchment lag parameter	β	2.80	2.75	2.00	2.00
Initial loss recovery factor	rf	0.10	0.11	0.10	0.10
Catchment non-linearity parameter	m	1.00	1.00	1.00	1.00
Initial loss	IL	91.0	70.0	50	4
Continuing loss	CL	3.45	4.00	3.0	0.5

Table 9.8 **URBS model calibration performance – 1991 event**

Gauge	Difference in peak flow	Difference in hydrograph volume	Difference in time to peak (hours)
Warwick 422310C	-5.8%	9.0%	2.20
Talgai TW 422355A	38.4%	-1.4%	-3.40
Tummaville 422323A	-3.8%	3.2%	16.50
OUTLET	-23.6%	14.0%	3.70

Table 9.9 Summary of URBS model calibration parameters – 2010 event

Parameter description		Sub-model A	Sub-model B	Sub-model C	Sub-model D
Channel lag parameter	α	0.15	0.12	0.30	0.13
Catchment lag parameter	β	2.80	2.75	2.00	2.00
Initial loss recovery factor	rf	0.10	0.11	0.1	0.10
Catchment non-linearity parameter	m	1.00	1.00	1.00	1.00
Initial loss	IL	18.0	78.5	25.5	20.5
Continuing loss	CL	2.00	0.23	1.15	1.80

Table 9.10 URBS model calibration performance – 2010 event

Gauge	Difference in peak flow	Difference in hydrograph volume	Difference in time to peak (hours)
Warwick 422310C	2.0%	9.3%	0.00
Talgai TW 422355A	5.0%	26.9%	-0.50
Tummaville 422323A	177.3%	109.8%	28.90
OUTLET	14.4%	9.8%	28.30

9.2.3.1 Discussion of calibration results

The primary aim of calibration in the upper catchment was to match URBS model peak flows, hydrograph volumes and times to peak to those recorded at the gauge. A reasonably good calibration was achieved at Warwick and Talgai TW in the upper catchment as shown in Figure 26, Figure 27, Figure 30 and Figure 31.

Aside from the inherent approximation of physical processes in any hydrological model, the level of accuracy a calibration can achieve is largely governed by the quality and coverage of data available. As stated in Section 9.1.5 (stream gauges), the reliability of the gauge at Talgai TW is limited by its location upstream of the confluence with Dalrymple Creek and the potential for breakouts in that area. During both calibration events there is uncertainty surrounding how much flow Dalrymple Creek is contributing to the flood event.

The accuracy of results declines as the model moves past the Tummaville gauge into the Condamine floodplain and down to Cecil Weir and Lone Pine gauges. Due to the complex braided nature of the main Condamine floodplain, calibration of the hydrology model in this area tends to be unproductive. URBS, nor any hydrological modelling package, cannot sufficiently capture flows breaking out and recombining during different flood events.

As an example of the complex flood behaviour around Tummaville (refer Figure 21) illustrates how a substantial proportion of any significant flood event can break out and completely bypass the Tummaville gauge in multiple locations. During the 2010 event the peak flow at the Talgai TW gauge is higher than that recorded at Tummaville, which strongly suggests that either water is bypassing the gauge; flows derived at Tummaville are inaccurate; or some combination of both. Therefore, the validity in adopting flow data from the Tummaville gauge (especially in large events like 2010) is very low.

The complex flood behaviour continues for approximately 60 km until the Condamine River and the North Branch recombine 5 km downstream of Cecil Weir. Calibrating peak flow and timing of peak at Tummaville, Cecil Weir and Lone Pine was not pursued, and instead the focus shifted to the calibration of flood volumes, particularly at Cecil Weir and Lone Pine.

Calibrating flood volumes was feasible, and it was important to achieve a match in flood volumes since the flood water levels in the broad, flat floodplain are driven predominantly by volume. The URBS model calibrated well to flood volumes at the 'Outlet'. A flood hydrograph Volume match within 10% at the 'Outlet' was considered satisfactory.

The URBS model produces estimates of local runoff in the Condamine floodplain, but all complex routing is determined by the hydraulic model. Therefore, calibration to gauges in the floodplain was made using gauge heights in the two-dimensional hydraulic model (refer Section 9.3).

Since there was negligible variability in routing parameters across calibration events for each sub-model, it was simple to adopt a single set of URBS parameters for the validation and design event hydrology. Only the channel lag parameter in Sub-model D differed between 1991 and 2010, but a test performed in the TUFLOW hydraulic model showed that the model results are not sensitive to that parameter in that part of the model. Table 9.11 summarises the URBS routing parameters that were adopted for the validation and design events.

Table 9.11 URBS routing parameters adopted for validation and design events

Parameter description		Sub-model A	Sub-model B	Sub-model C	Sub-model D
Channel lag parameter	α	0.15	0.12	0.30	0.13
Catchment lag parameter	β	2.80	2.75	2.00	2.00
Initial loss recovery factor	rf	0.10	0.11	0.10	0.10
Catchment non-linearity parameter	m	1.00	1.00	1.00	1.00

9.2.4 Model validation

The results of the 2013 event validation are shown in Figure 34 to Figure 37 (hydrographs) and Table 9.12 (model validation performance metrics). The validation results indicate that the hydrological model outputs using parameters adopted in Table 9.11 achieve a good match with the historical event.

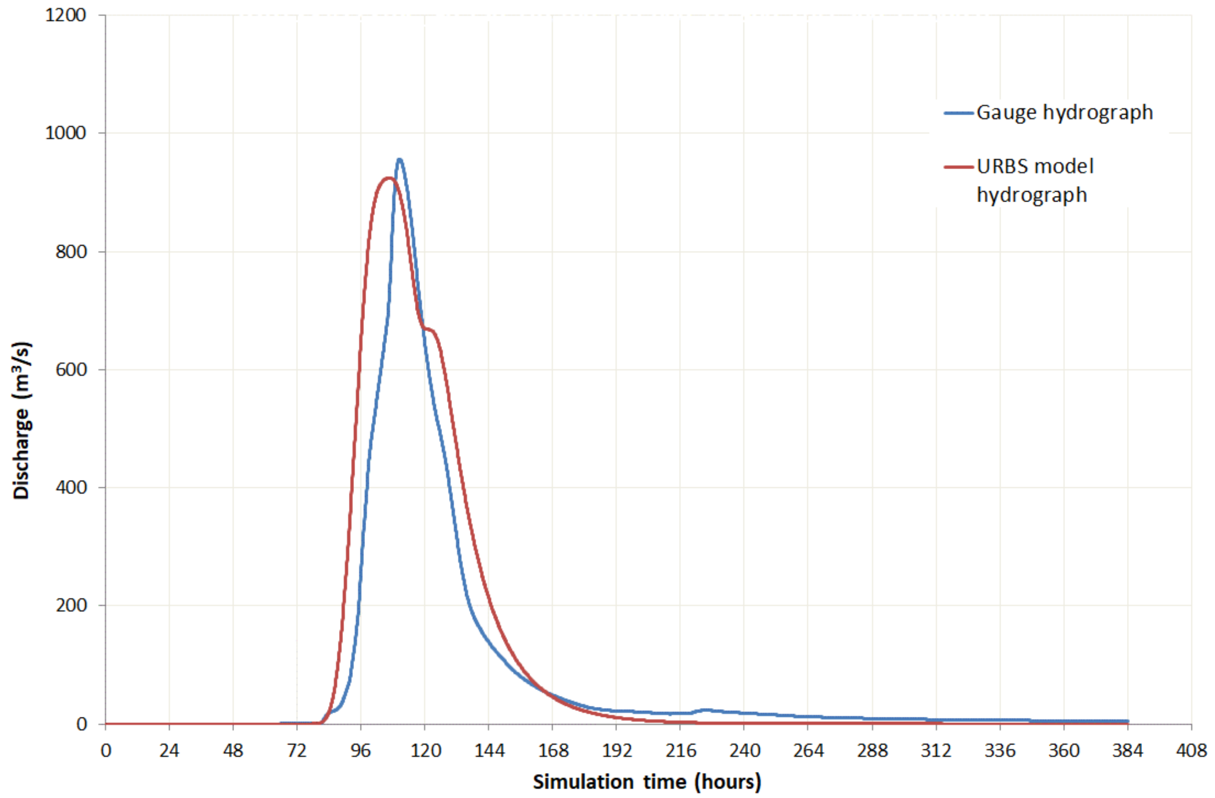


Figure 34 Validation hydrographs at Warwick 422310C – 2013 event

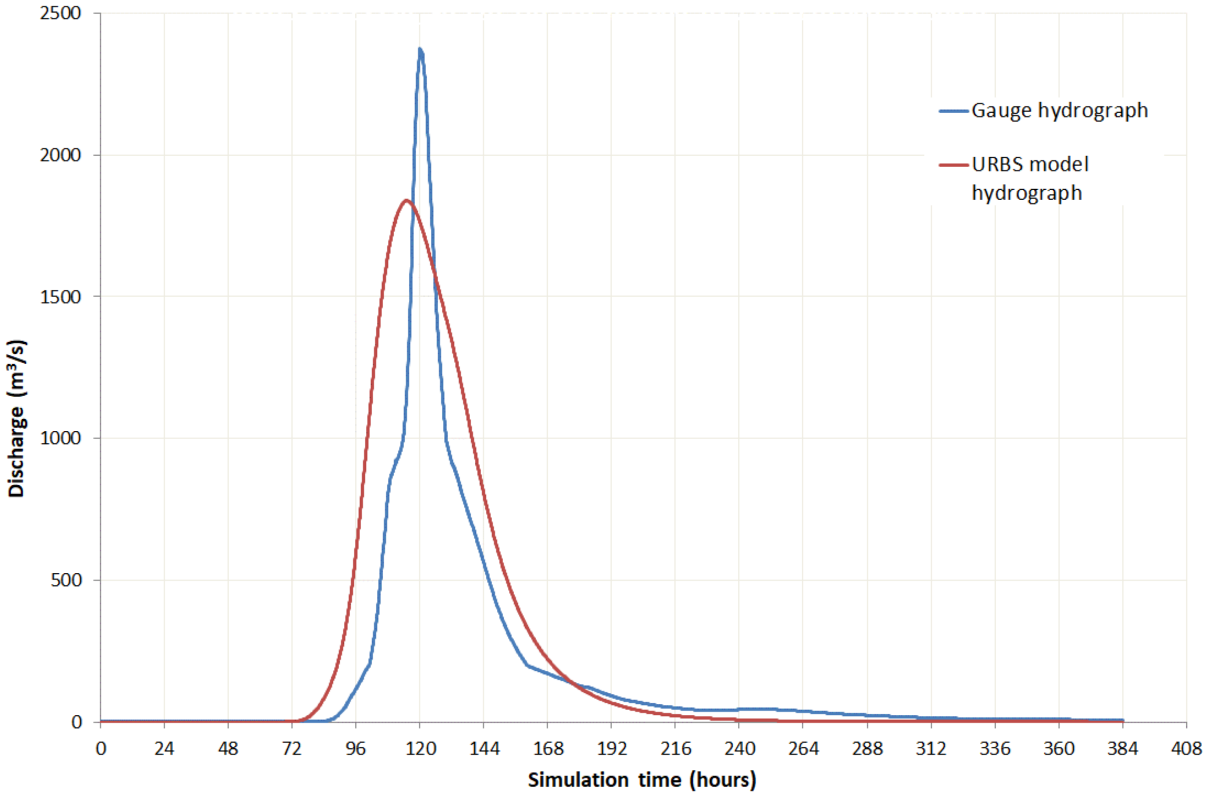


Figure 35 Validation hydrographs at Talgai TW 422355A – 2013 event

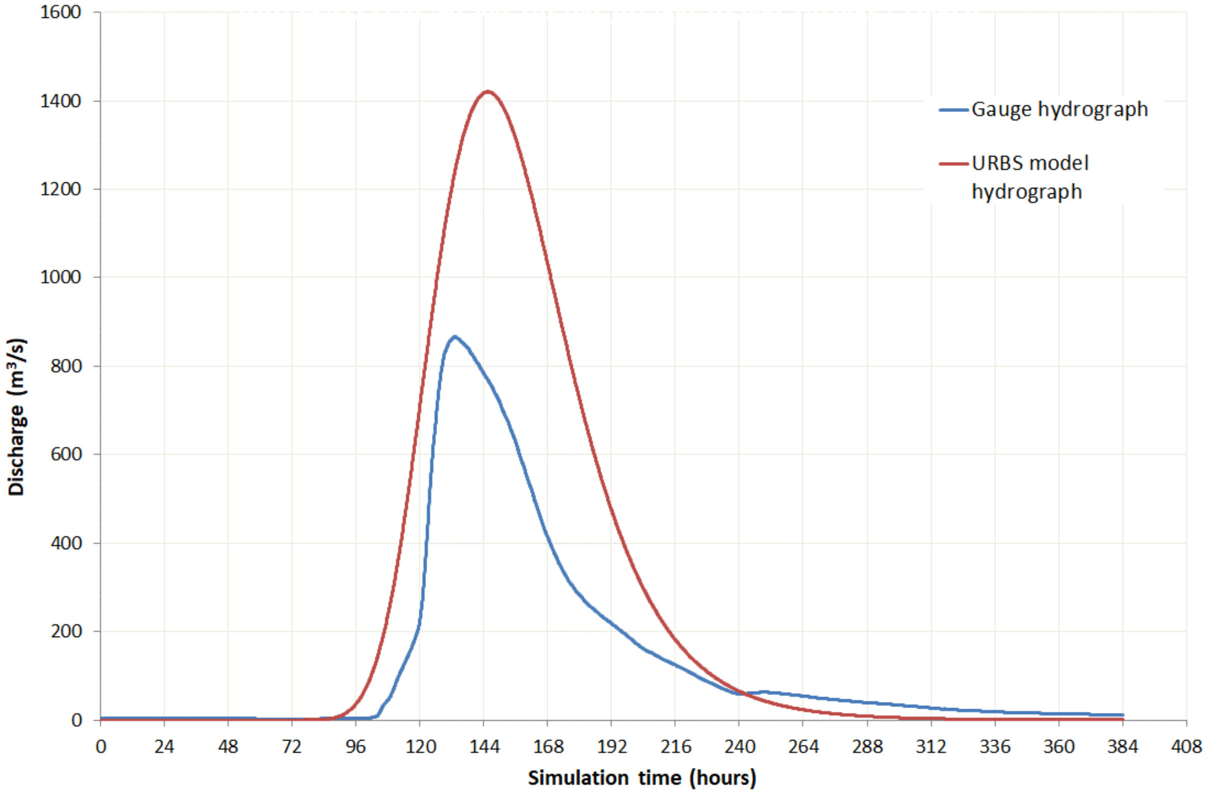


Figure 36 Validation hydrographs at Tummaville 423323A – 2013 event

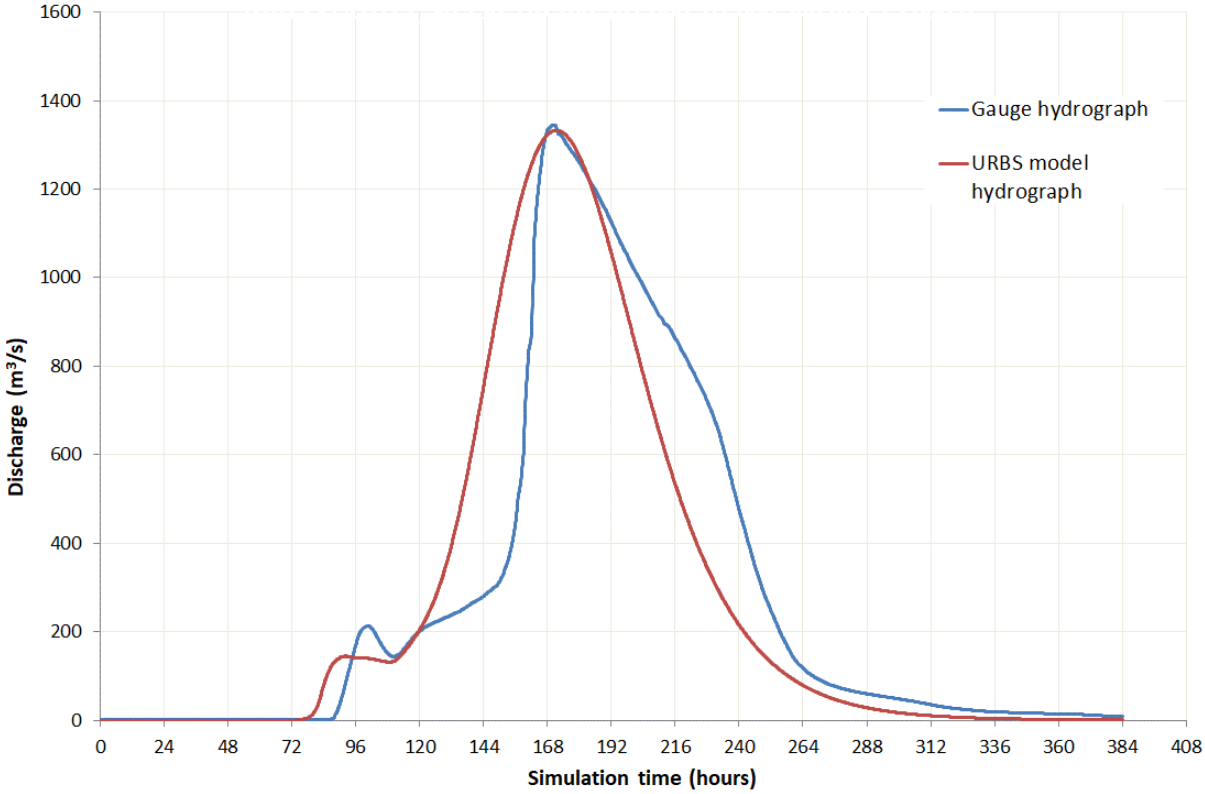


Figure 37 **Validation hydrographs at combined 'outlet' – 2013 event**

Table 9.12 **URBS model validation performance metrics – 2013 event**

Gauge	Difference in peak flow	Difference in hydrograph volume	Difference in time to peak (hours)
Warwick 422310C	-3.3%	22.4%	-4.40
Talgai TW 422355A	-22.6%	34.5%	-4.90
Tummaville 422323A	63.9%	82.7%	12.50
OUTLET	-0.9%	-2.4%	0.30

9.2.5 Design event parameters

Hydrologic information to assist estimation of design event flows was sourced from the ARR Data Hub as summarised in Table 9.13 below.

Table 9.13 **Summary of URBS model design event inputs**

Input parameter	Remarks
Design rainfall	IFDs for each sub-catchment were downloaded from the BoM's website to simulate the relatively high variation in rainfall across the catchment.
Extreme event rainfall	PMP depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the method presented in the Bulletin 53 (BOM, 2003). The rainfall depths for the 1 in 10,000 AEP event were estimated using the interpolation method presented in ARR 2016 Book 8 Section 3.5.
Losses	Rainfall loss parameters were downloaded from the ARR Data Hub for each sub-model. Sub-model A – IL: 31mm, CL: 3.0mm/hr Sub-model B – IL: 25mm, CL: 1.7mm/hr Sub-model C – IL: 37mm, CL: 1.0mm/hr Sub-model D – IL: 46mm, CL: 0.4mm/hr

Input parameter	Remarks
Areal reduction factor	Parameters were adopted for the Semi-Arid Inland Queensland region. The catchment area U/S of the proposed rail crossing on the Condamine River is approximately 7,064 km^2, which yields an ARF between 78.5% and 86.3% depending on design storm event AEP and duration.
Ensemble temporal patterns	As the study catchment area exceeds 75 km^2, the standard ensemble rainfall patterns from ARR 2016 do not apply to this catchment. These were replaced with the areal temporal patterns for the Central Slopes region.
Preburst depths	Median preburst depths were downloaded from the ARR Data Hub for each sub-catchment. Preburst depths vary by design storm event AEP and duration. Preburst depths were applied to the model by reducing the initial losses for each storm event.

The calibrated URBS model was set up to run the ten ensemble temporal patterns for the nine AEPs (50%, 20%, 10%, 5%, 2%, 1%, 1 in 2000, 1 in 10,000, PMF) and 7 storm durations (720, 1440, 2880, 4320, 5760, 7200, 8640 minutes), which amounted to 560 individual design storms per sub-model area. Each storm was passed from one sub-model to the next in series.

Initially the URBS model was run with the loss values obtained from the ARR Data Hub. The losses derived during calibration were not adopted since it was difficult to confidently judge antecedent conditions and the true magnitude of the losses in each calibration event.

The results of the 560 design storms were statistically analysed to determine which temporal pattern generated the fifth-highest (Rank-5) flow for each duration and AEP. Then the adopted design flow for a given AEP was the maximum value across the corresponding Rank-5 flows. The results informed how much these losses needed to be scaled (up or down) to reconcile the differences between FFAs and the flows estimated by the URBS model at the gauges.

Adjustments to rainfall losses were made only in Sub-model A to reconcile modelled flows to the FFA at Warwick. Several iterations were necessary to reconcile differences to less than 5% for events smaller than the 1% AEP, and to less than 10% for events larger than the 1% AEP. The broader tolerance for the rarer, larger events was due to the larger uncertainty in the FFAs at those AEPs. No adjustments to rainfall losses were made downstream of Warwick because of the theoretical nature of its FFA.

Figure 38 and Table 9.14 summarise the required adjustments to the loss values in order to achieve a good match to the FFAs.

Figure 39 and Figure 40 display how the design event flows plot against each FFA and Table 9.15 and Table 9.16 summarise the differences. Note that due to the much higher reliability of the FFAs at Warwick and Cecil Weir, compared to the FFAs at Talgai TW and Tummaville, reconciliation was performed at Warwick and Cecil Weir only.

As an additional check, DTMR's Quantile Regression Technique (QRT) was used. Although QRT is applicable only to catchments up 1,000 km^2, it serves as a reasonable, arbitrary point of comparison.

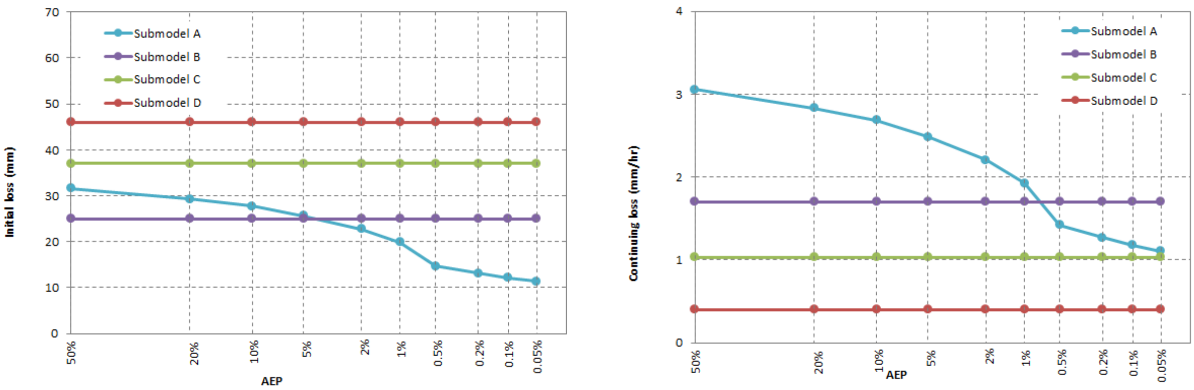


Figure 38 Adjusted design event losses

Table 9.14 Adjusted design event losses

AEP		ARR Data Hub value	50%	20%	10%	5%	2%	1%	1 in 2,000
Sub-model A	IL	31.0	31.57	29.28	27.74	25.68	22.77	19.88	11.40
	CL	3.0	3.05	2.83	2.68	2.49	2.20	1.92	1.10
Sub-model B	IL	25.0	25.00	25.00	25.00	25.00	25.00	25.00	25.00
	CL	1.7	1.70	1.70	1.70	1.70	1.70	1.70	1.70
Sub-model C	IL	37.0	37.00	37.00	37.00	37.00	37.00	37.00	37.00
	CL	1.0	1.03	1.03	1.03	1.03	1.03	1.03	1.03
Sub-model D	IL	46.0	46.00	46.00	46.00	46.00	46.00	46.00	46.00
	CL	0.4	0.40	0.40	0.40	0.40	0.40	0.40	0.40

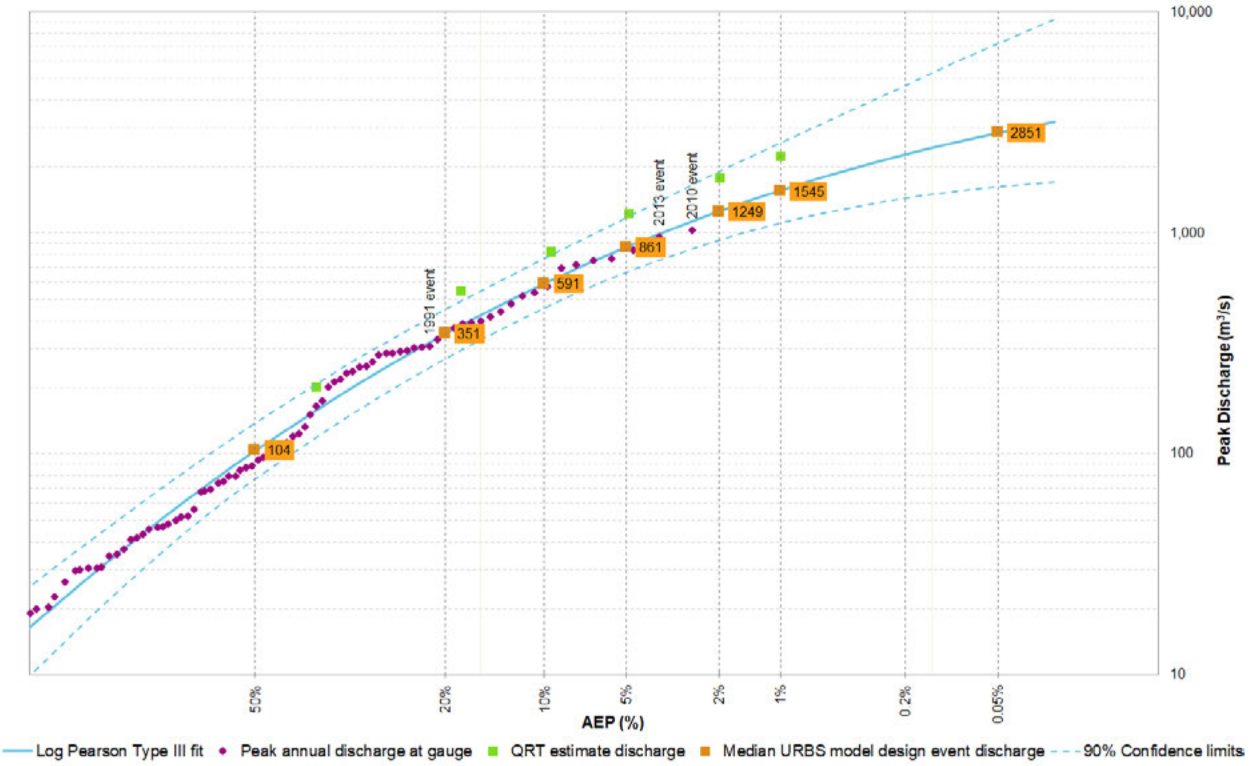


Figure 39 Flood frequency analysis at Warwick (422310C)

Table 9.15 Data output from flood frequency analysis at Warwick (422310C)

AEP	FFA estimate - lower bound 90% confidence interval (m^3/s)	FFA – estimate of flow (m^3/s)	FFA estimate - upper bound 90% confidence interval (m^3/s)	DTMR quantile regression technique (m^3/s)	URBS model flows (m^3/s)	Difference between FFA estimate and URBS model
50%	77	103	138	-	104	0.5%
20%	269	349	452	-	351	0.7%
10%	457	589	767	-	591	0.3%
5%	664	861	1,176	1,224	861	0.0%
2%	926	1,250	1,892	1,767	1,249	-0.1%
1%	1,106	1,554	2,559	2,213	1,545	-0.6%
1 in 2,000	1,616	2,826	7,213	-	2,851	-1.2%

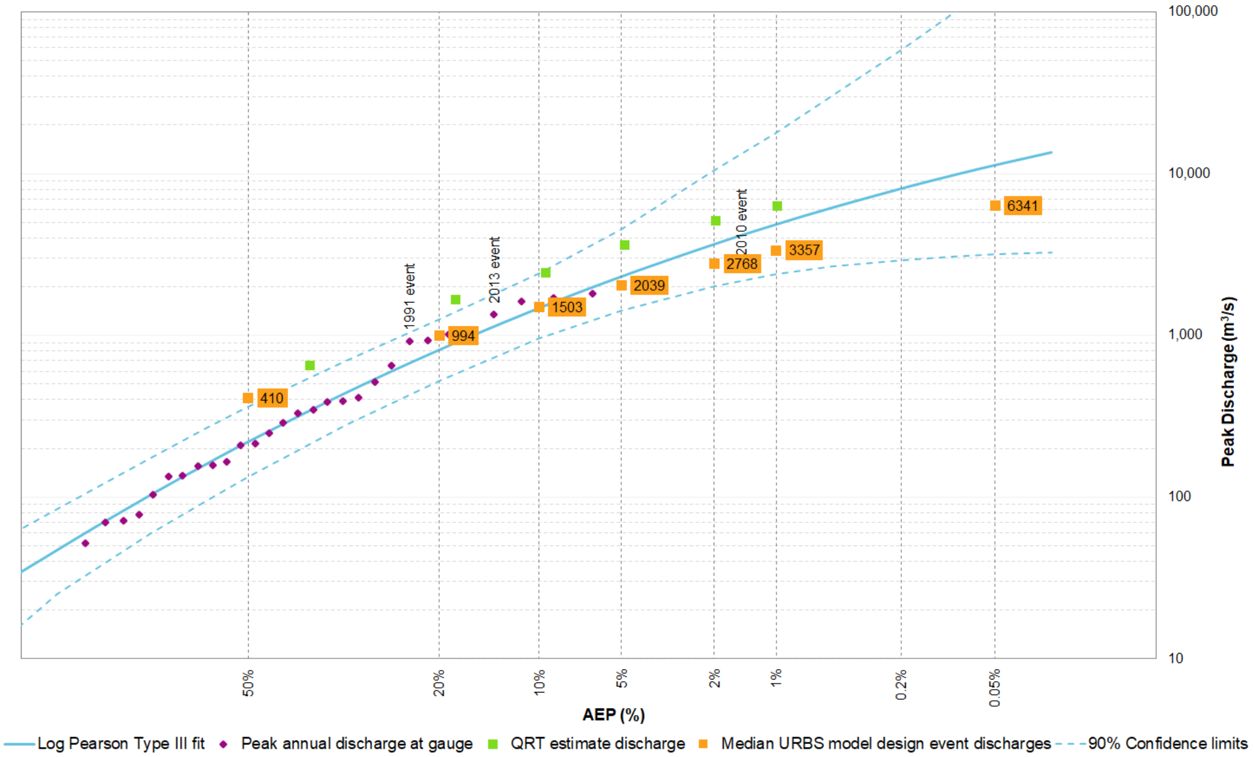


Figure 40 Combined flood frequency analysis at URBS model outlet (422316A & 422345A)

Table 9.16 Data output from combined flood frequency analysis at URBS model outlet

AEP	FFA estimate - lower bound 90% confidence interval (m^3/s)	FFA – estimate of flow (m^3/s)	FFA estimate - upper bound 90% confidence interval (m^3/s)	DTMR quantile regression technique (m^3/s)	URBS model flows (m^3/s)	Difference between FFA estimate and URBS model
50%	133	221	362	-	410	+85.7%
20%	524	817	1,256	-	994	+21.7%
10%	959	1,483	2,437	-	1,503	+1.4%
5%	1,432	2,323	4,565	3,624	2,039	-12.2%
2%	2,020	3,681	10,472	5,139	2,768	-24.8%
1%	2,380	4,878	17,968	6,354	3,357	-31.2%
1 in 2,000	3,173	11,348	152,272	-	6,341	-44.1%

9.2.6 Design hydrograph selection

Three storm durations were selected for each AEP for testing in the TUFLOW model to account for potential subtle differences between design event hydrographs causing localised differences to the flood hydraulics.

The critical durations which produced the highest peak flow tended to be between 2,880 and 7,200 minutes, while the critical durations which produced the highest flood Volume tended to be between 2,880 and 8,640 minutes. Storm event durations of 2,880, 4,320 and 5,760 minutes were chosen as their flood peaks and volumes were consistently among the highest for all AEP at locations within the hydraulic model domain.

The URBS results used for design event selection were taken at the outlet of Sub-model C (Tummaville) as this is the last point of reference upstream of the proposed rail and upstream of the braided floodplain. The peaks and the temporal pattern chosen for design event hydrographs are summarised in Table 9.17.

Table 9.17 Summary of selected design event hydrographs

AEP	Peak flow (m^3/s)			Ensemble pattern (0 to 9)		
	Storm duration (mins)					
	2,880	4,320	5,760	2,880	4,320	5,760
50%	284	284	296	6	6	3
20%	761	752	744	0	6	3
10%	1,147	1,111	1,152	0	4	9
5%	1,553	1,530	1,590	0	0	9
2%	2,223	2,148	2,182	4	0	9
1%	2,765	2,655	2,707	4	0	7
1 in 2,000	4,927	5,186	5,421	5	7	7

9.3 Hydraulic model development – Condamine River

Due to the expansive floodplain and braided flowpaths associated with the lower reaches of the Condamine River catchment, particularly in the agricultural land adjacent Millmerran and Brookstead, a two-dimensional modelling approach was adopted to appropriately simulate flood mechanisms within the catchment. The platform which was utilised for hydraulic modelling for the Condamine system is the TUFLOW HPC software package. The processes and assumptions adopted throughout the development of the hydraulic model are described in the following sections.

9.3.1 Model setup

Integrated 1d/2d numerical TUFLOW HPC hydraulic models were developed to simulate flood behaviour within the study area. Two TUFLOW models were developed, one being a regional model and the second being the local model, which covered a smaller area within the extents of the regional model to enable faster design optioneering.

The local model was used to test a large number of design configurations, however following the selection of the preferred design, the selected option was run using the full regional model.

The regional model was developed to simulate flows throughout the broader Condamine floodplain, extending approximately 50 km along the floodplain from just downstream of the Talgai Weir to the Lone Pine stream gauge station. At its widest point, the model spans 34 km. The model was used for calibration and to understand flood depths and velocities during design events in existing and design conditions.

The hydraulic model extent set up is shown in Figure C-1d in Volume II – Appendix C.

An overview of the model setup and key parameters for the regional TUFLOW model is provided in Table 9.18.

Table 9.18 Regional Condamine River hydraulic model summary

Parameter	Information
Completion date	June 2019
AEPs assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF
Hydrologic modelling approach	Inflows determined through aforementioned URBS model. Inflows applied to model as SA polygons within 2d domain utilising the SA ALL command
IFD input parameters	Hydrologic approach based upon ARR 2016
Hydraulic modelling approach	TUFLOW HPC GPU – version 2017-09-AC-w64-iSP
Model extent	Refer to Figure 41
Grid size	20m

Parameter	Information
DEM (year flown)	Multiple Topographic datasets, varying from 2010 to 2015.
Roughness	Spatially varying roughness values compliant with industry norms.
Eddy viscosity	Smagorinsky (default)
Model calibration	1991 Event, 2010 Event and 2013 Event.
D/S model boundary	Height-Discharge (HQ) Boundary with normal slope approximated based upon topography dataset.
Hydraulic model timestep	Adaptive Timestep
Hydraulic model wetting and drying depths	Cell centre set at 0.0002m Cell side set at 0.0001 m
Mapping cut-off depth	100 mm post processed cut-off depth applied.
Modelled scenarios	Existing conditions
Sensitivity analysis	4x inflow application techniques 14x roughness profiles 1x climate change

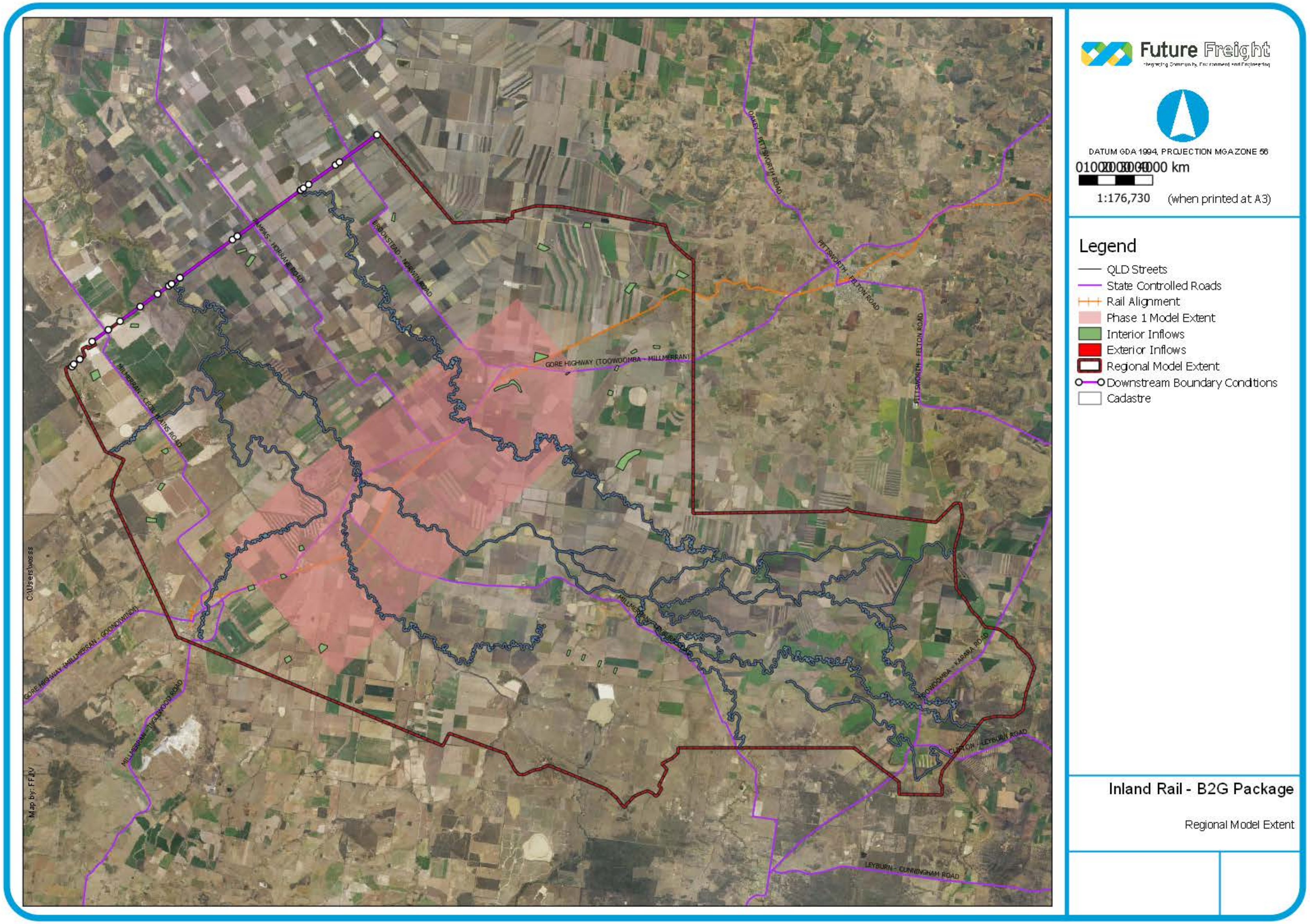


Figure 41 New regional TUFLOW model extent (with Study Corridor and 2017 Phase 1 AECOM model extent superimposed)

9.3.2 Hydraulic structures

Numerous data sources were used in defining existing structures within the hydraulic model. Survey information was utilised where possible along with DTMR GIMS data. In addition to this information, records from site visits undertaken in February and structure geometry information contained within the previous hydraulic models was used. Where no data was available and culverts were visible within topography and aerial imagery, assumptions of structure sizes were made based upon measured structure widths and topography levels. In total, 15 culverts and eight bridges identified within the hydraulic model domain as summarised in Table 9.19 and Table 9.20.

Table 9.19 Identified existing structures within the hydraulic model extent

Structure modelling ID	Type	U/S invert (m AHD)	D/S invert (m AHD)	Diameter/width (m)	Height (m)	Number of cells
G02	RCBC	379.67	379.65	2.4	1.2	8
G03	RCBC	376.20	376.20	2.4	1.2	8
G04	RCBC	376.29	376.29	2.4	1.2	2
G05	RCBC	376.39	376.39	2.4	1.2	8
G06	RCBC	376.80	376.80	2.4	1.2	3
P01	RCBC	381.75	381.72	1.2	0.3	6
P02	RCBC	381.73	381.67	1.2	0.3	6
P03	RCBC	381.74	381.67	1.2	0.3	6
P04	RCBC	381.67	381.66	1.2	0.3	6
P05	RCBC	381.71	381.68	1.2	0.3	6
P06	RCBC	381.72	381.68	1.2	0.3	6
P07	RCBC	381.72	381.69	1.2	0.3	6
P08	RCBC	381.69	381.65	1.2	0.3	6
P09	RCBC	381.69	381.69	1.2	0.3	6
P10	RCBC	381.76	381.64	1.2	0.3	6

Table 9.20 Identified existing bridges within the hydraulic model extent

Bridge name	Obvert level (m AHD)	Deck depth (m)
Grass Tree Creek (Millmerran-Leyburn Road) - BIS ID 243	377.80	1.00
Dogtrap Creek (Millmerran-Leyburn Road) - BIS ID 244	392.20	0.61
Canal Creek (Millmerran-Leyburn Road) - BIS ID 245	394.59	0.61
Back Creek (Gore Highway) - BIS ID 24899	396.10	1.00
Condamine River (Gore Highway) - BIS ID 361	376.91	1.00
Condamine River (Toowoomba-Karara Road) - BIS ID 580	406.09	0.55
Middle Creek (Toowoomba-Karara Road) - BIS ID 581	406.10	1.00
Thanes Creek (Toowoomba-Karara Road) - BIS ID 582	405.51	1.00

9.3.3 Roughness

The roughness values adopted in both the regional and local model were spatially varied based upon both aerial imagery and land-use classifications sourced from the QSpatial and published by the Department of Infrastructure, Local Government and Planning. Information gained from landowners during the site inspection such as typical crop types and sowing/harvesting timeframes, was also used to inform the material roughness values. The adopted roughness values are listed in Table 9.21.

Table 9.21 Condamine River hydraulic model – Manning's n values

Material	Manning's n value
River	0.035
Riparian Vegetation	0.04
Sealed Road Corridor	0.015
Unsealed Road Corridor	0.025
Rural Residential	0.090
Urban Residential	0.100
Commercial	0.060
Open Space – Minimum Vegetation	0.045
Open Space – Moderate Vegetation	0.055
Open Space – Heavy Vegetation	0.065
Open Water	0.025
Rail	0.025
Crops – Uniform[1]	0.093 to 0.1085

Table note:

1 A uniform value was adopted for the crop areas, due to the seasonality of the vegetation and the uncertainty this brings to the design event approach

The values above were tested through the calibration and validation runs, particularly in regard to the crop roughness value, and the values adopted were observed to result in relatively good fits for timings of flow peaks and floodplain attenuation.

As shown in Table 9.21 the Manning's n value for crop-generic is a range with a lower value of 0.093 and an upper value of 0.1085. These values were adopted as a result of multiple testing iterations during model calibration that found that one single roughness value for crops did not adequately account for the uncertainty and variation in crop type and seasonality. This range of roughness values was adopted as two separate scenarios in all design events to provide the lower and upper bounds to expected peak flood water surface elevations, irrespective of event magnitude and time of flood occurrence. Additional tests on the sensitivity of results to spatial variation in roughness parameters is discussed in Section 9.5.4.1.

The hydraulic model extent and the spatial distribution of land use in the 2D model domain is presented in Figure C-1e in Volume II – Appendix C.

9.3.4 Boundary conditions

The Condamine River URBS hydrologic model outputs were applied as inflows into the TUFLOW hydraulic model. Total inflows from catchments upstream of the hydraulic model extent were applied at the upstream model boundary and local inflows from areas within the TUFLOW hydraulic model were applied throughout the model.

Internal inflow boundaries were applied as SA polygons along the streams and rivers within the hydraulic model extent. The ALL command was adopted in the application of the internal SA boundaries, to ensure that flow is equally distributed to all cells within the SA polygon. This approach was utilised to better represent the application of flows along the river reaches.

Downstream boundaries were modelled under normal depth conditions. Downstream boundary lines were separate to represent the independent flow paths which would each incur their own normal depth condition.

Boundary locations are shown in Figure C-1d in Volume II - Appendix C.

9.3.5 Calibration

Model calibration and validation was undertaken on the December 2010 event, January 2013 and the February 1991 event. Calibration for the hydraulic model was based upon comparison made between hydrographs at key gauge locations, as well as level and depth comparisons at both anecdotal floodmarks and surveyed floodmarks.

The comparison of modelled and gauged hydrographs for the 1991, 2010 and 2013 event are shown in Figure 42 to Figure 44 (1991 event), Figure 45 to Figure 47 (2010 event) and Figure 48 to Figure 50 (2013 event).

A summary of modelled and observed flood heights are shown in Table 9.22, Table 9.23 and Table 9.24.

Inundation extent maps for the calibration results are shown in Volume II – Appendix C, Figures C-2a to C-2c.

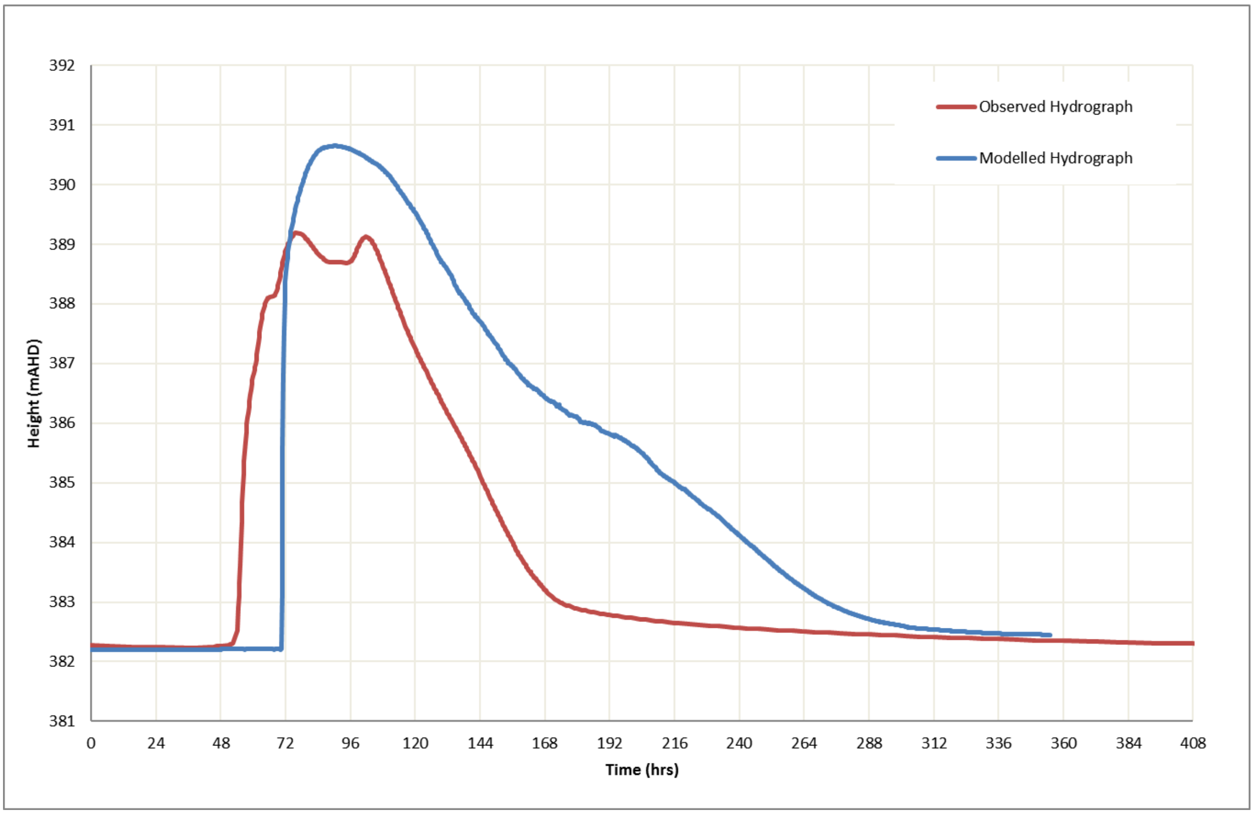


Figure 42 **Hydraulic model calibration hydrographs at Tummaville 422323A – 1991 event**

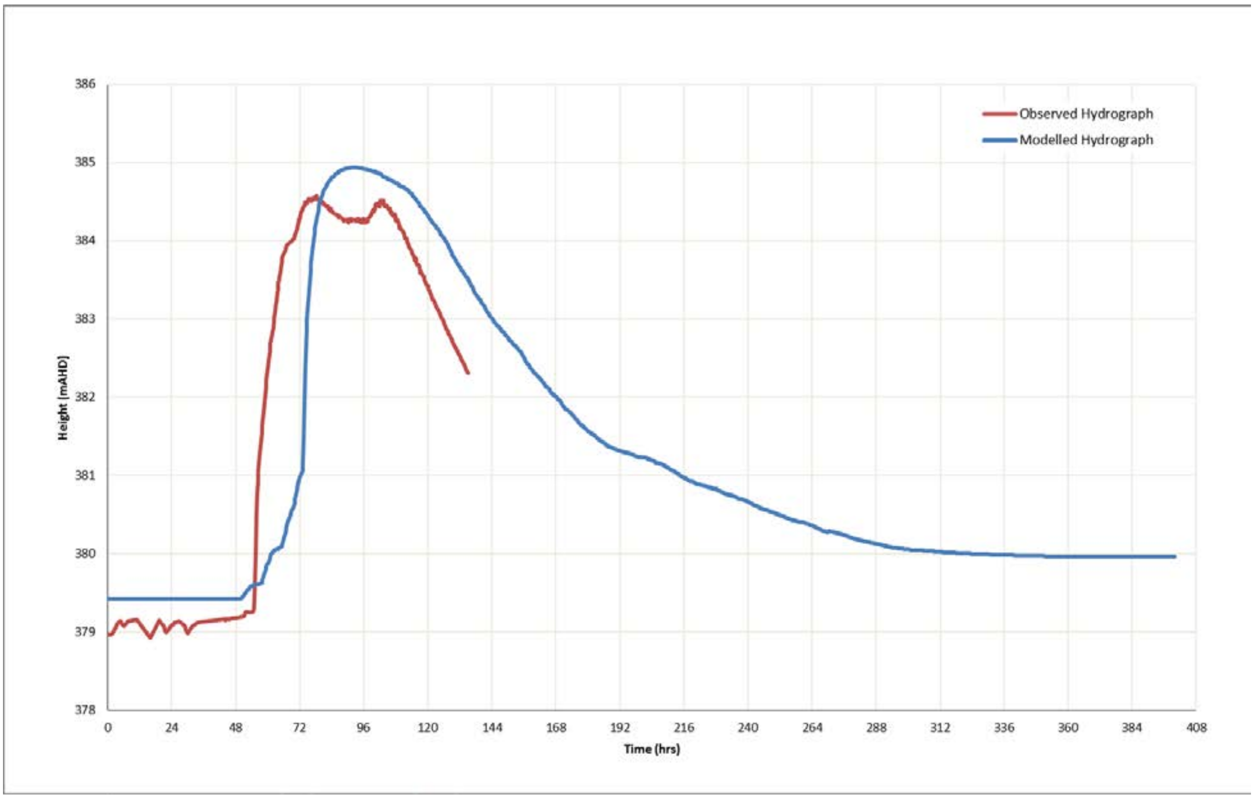


Figure 43 Hydraulic model calibration hydrographs at Yarramalong Weir 422353A – 1991 event

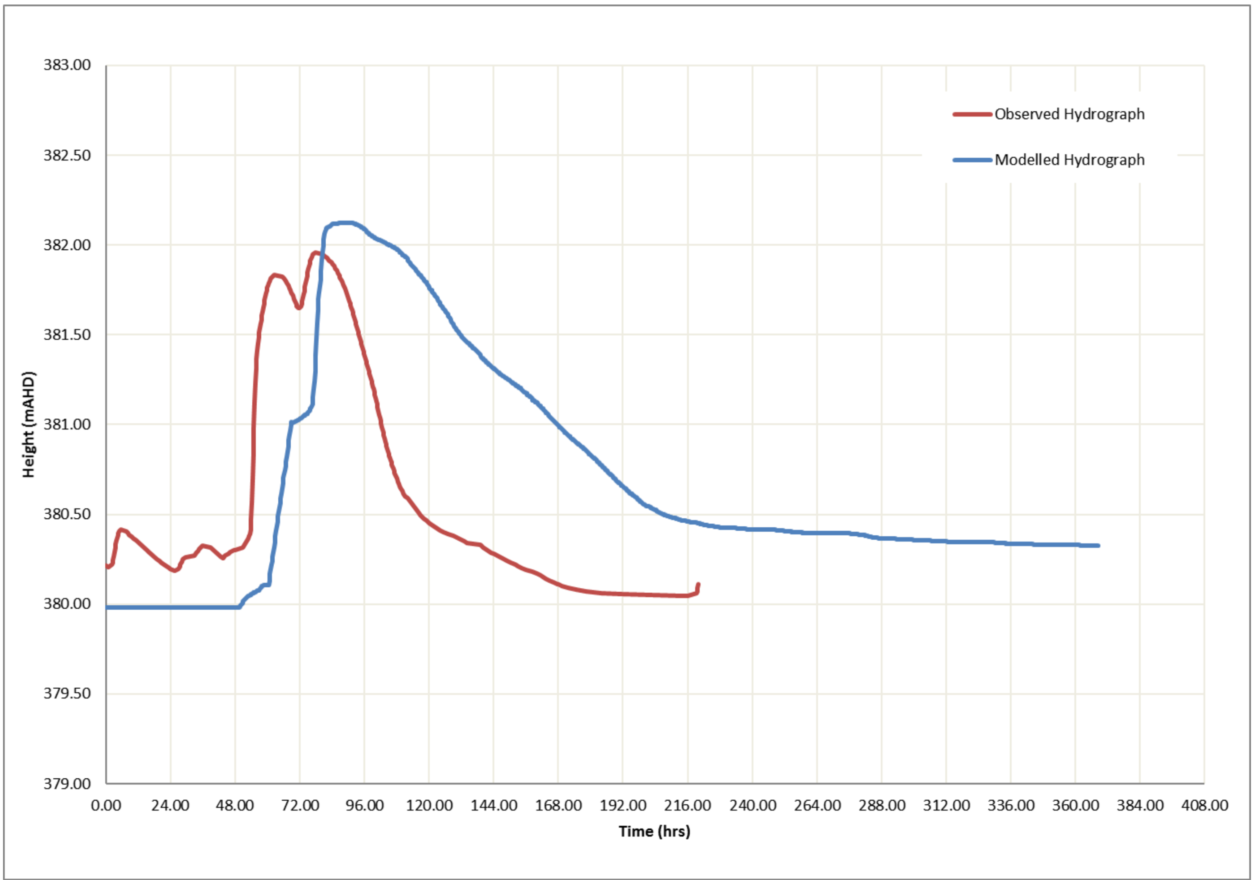


Figure 44 Hydraulic model calibration hydrographs at Pampas 422347B – 1991 event

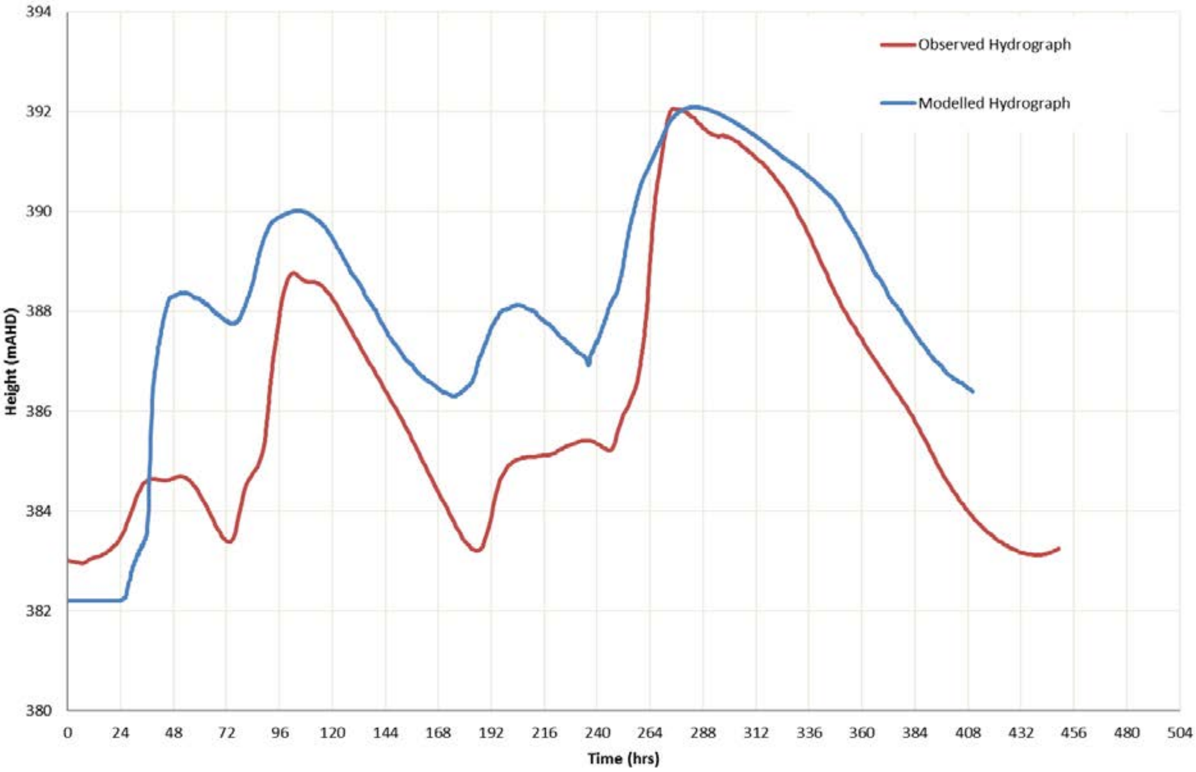


Figure 45 **Hydraulic model calibration hydrographs at Tummaville 422323A – 2010 event**

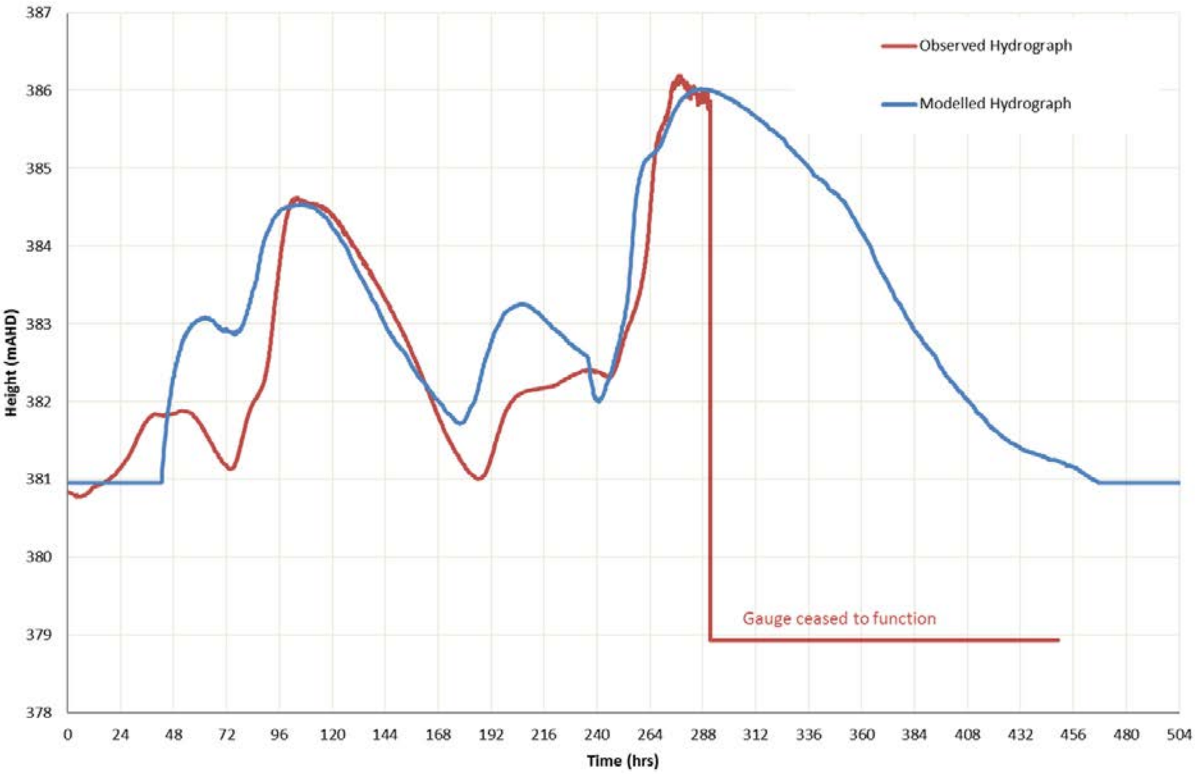


Figure 46 **Hydraulic model calibration hydrographs at Yarramalong Weir 422353A – 2010 event**

Figure notes: Yarramalong gauge ceased to operate during the event as can be seen on the 'observed hydrograph' plot

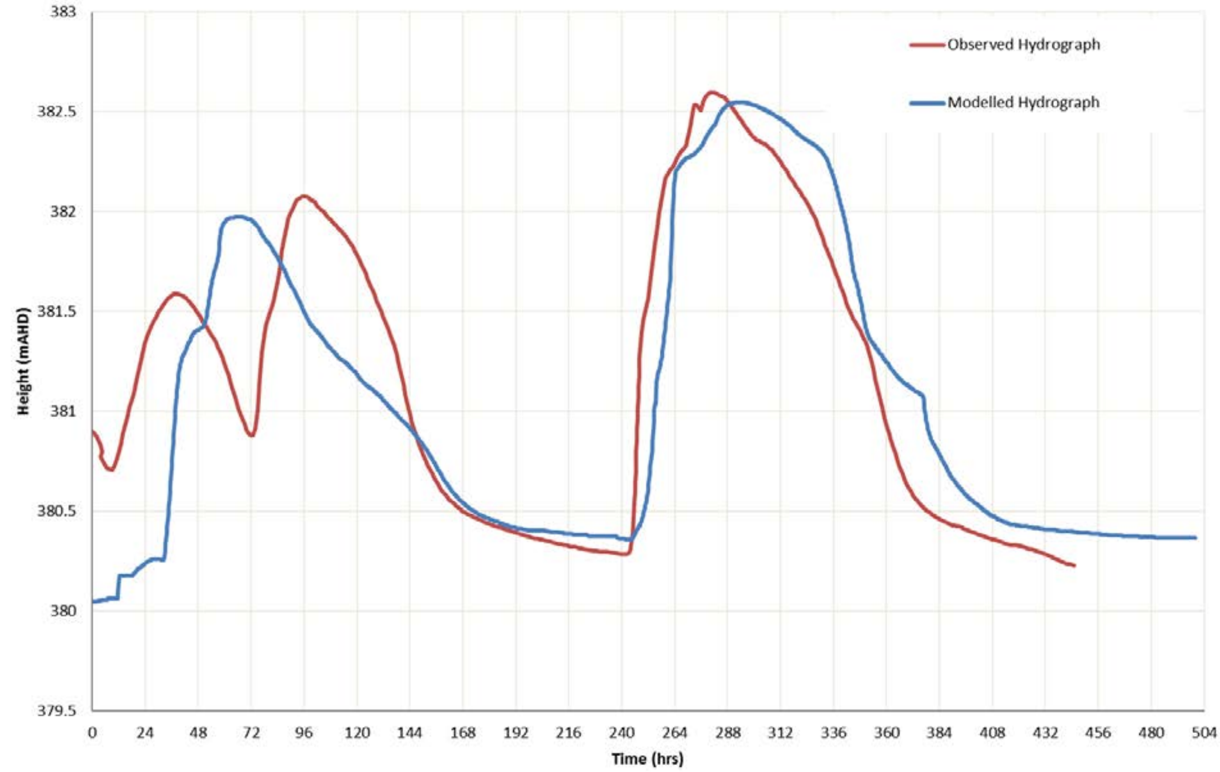


Figure 47 **Hydraulic model calibration hydrographs at Pampas 422347B – 2010 event**

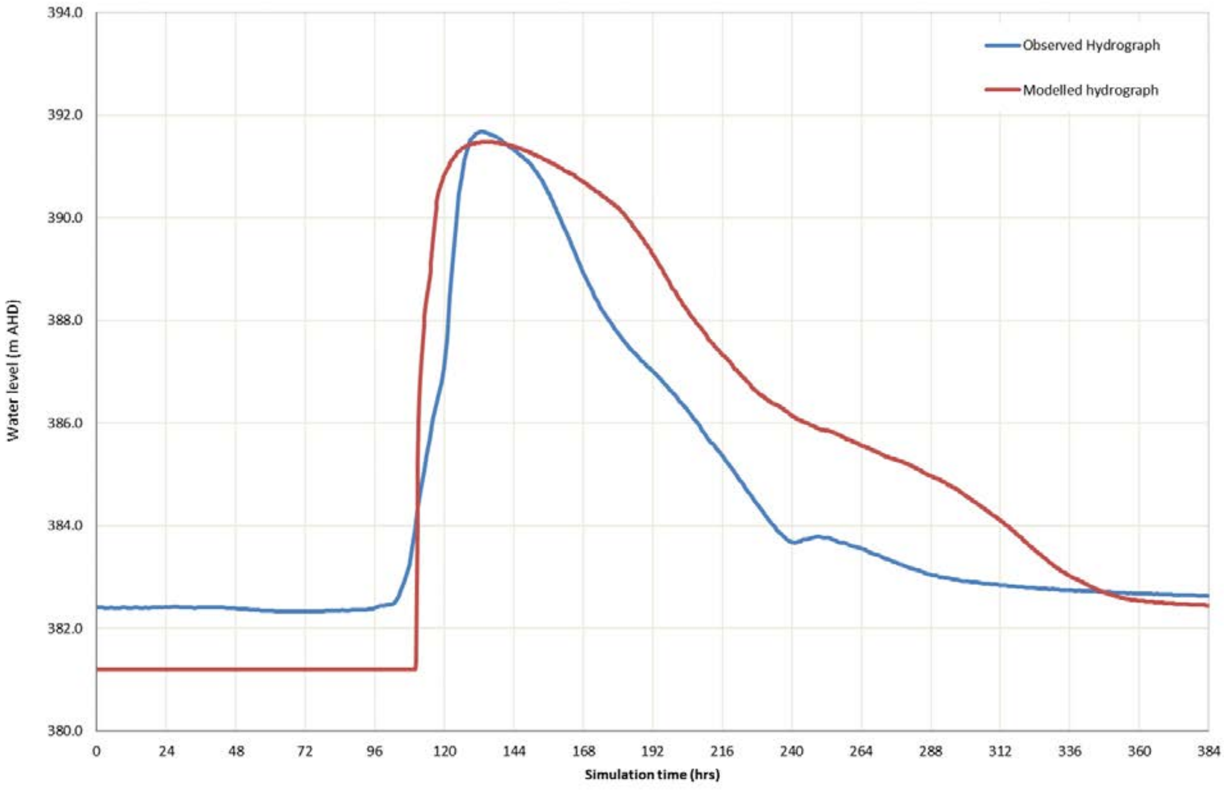


Figure 48 **Hydraulic model calibration hydrographs at Tummaville 422323A – 2013 event**

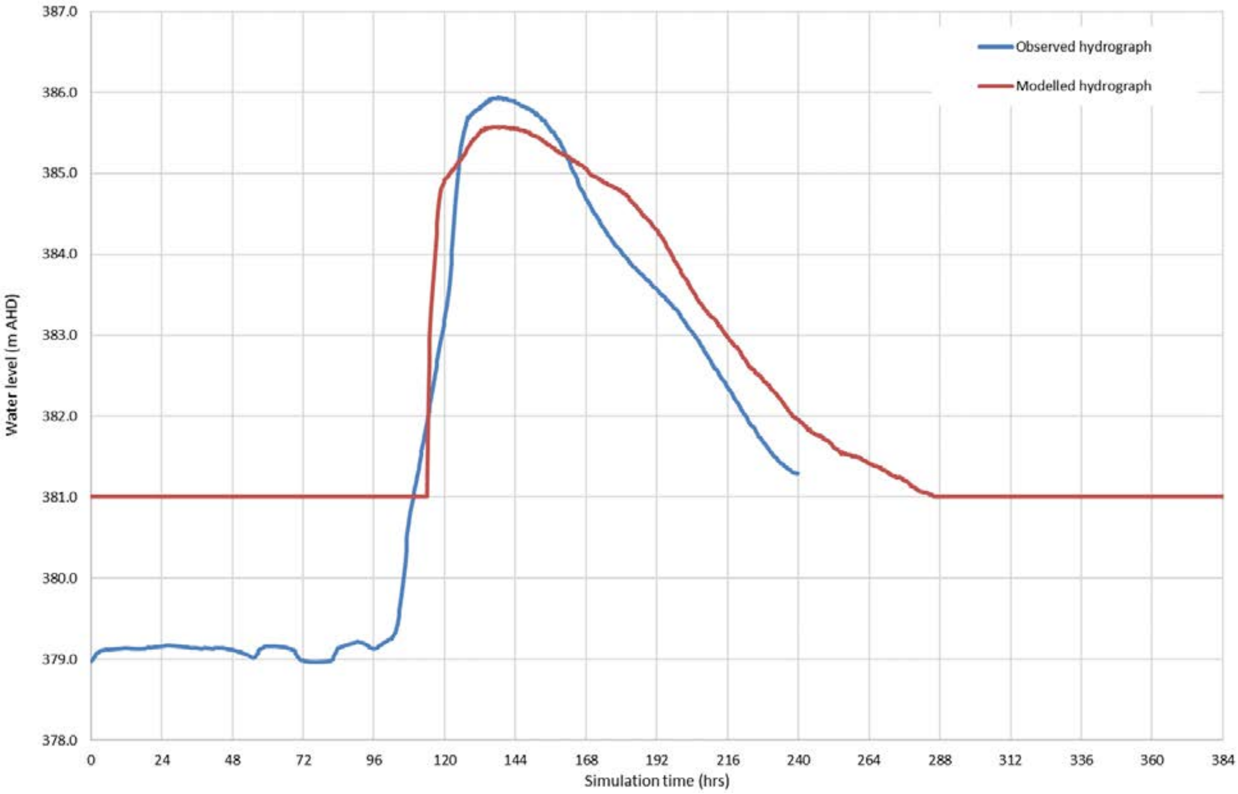


Figure 49 Hydraulic model calibration hydrographs at Yarramalong Weir 422353A – 2013 event

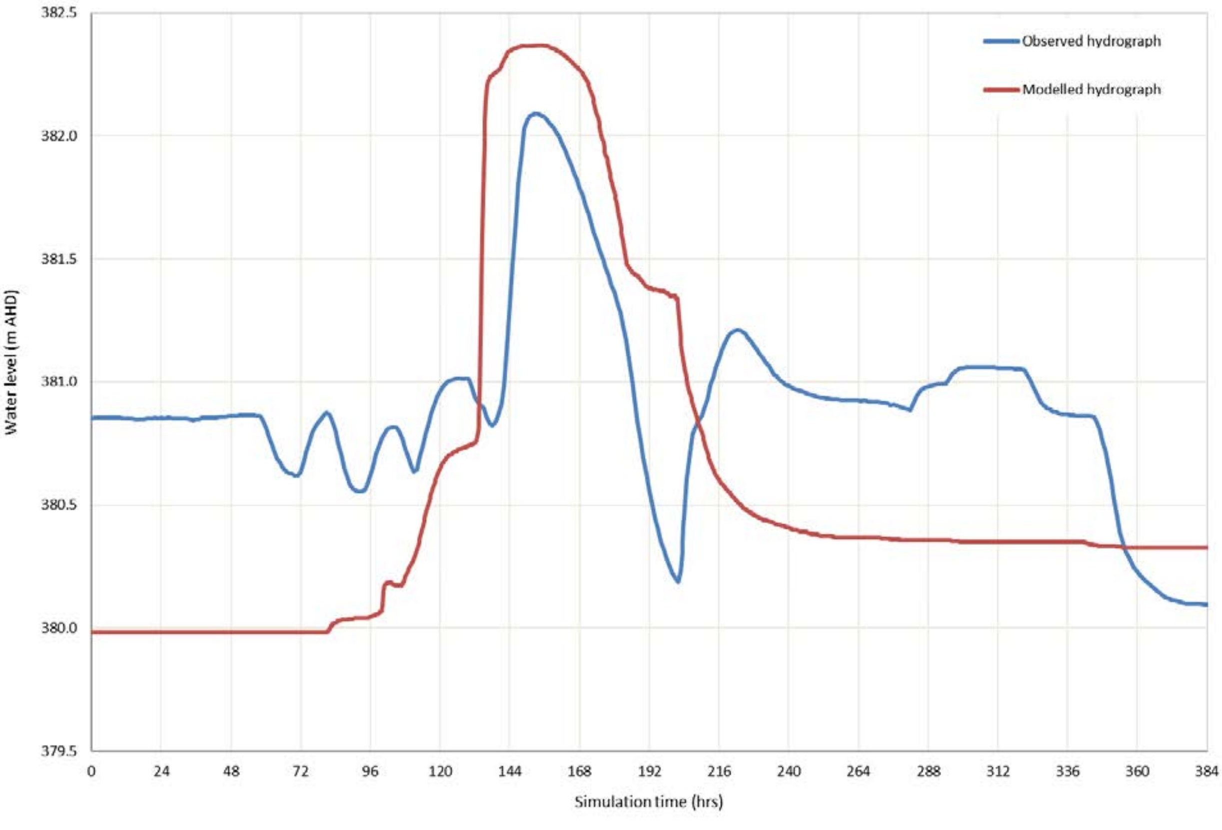


Figure 50 Hydraulic model calibration hydrographs at Pampas 422347B – 2013 event

Table 9.22 Summary of hydraulic model calibration – 1991 event[1]

Gauge	Ground surface level (m AHD)	Observed water level (m AHD)	Modelled water level (m AHD)	Observed water depth (m)	Modelled water depth (m)	Difference in water level (m)	Difference in water depth (%)
Tummaville	380.926	389.195	390.649	8.27	9.72	+1.454	+17.6%
Yarramalong Weir	378.930	384.577	384.888	5.65	5.96	+0.311	-5.5%
Pampas	379.000	381.956	382.123	2.96	3.12	+0.167	-5.6%

Table 9.23 Summary of hydraulic model calibration – 2010 event[1]

Gauge	Ground surface level (m AHD)	Observed water level (m AHD)	Modelled water level (m AHD)	Observed water depth (m)	Modelled water depth (m)	Difference in water level (m)	Difference in water depth (%)
Tummaville	380.926	392.061	392.086	11.14	11.16	+0.025	+0.2%
Yarramalong Weir	378.930	386.192	386.016	7.26	7.09	-0.176	-2.4%
Pampas	379.000	382.594	382.546	3.59	3.55	-0.048	-1.3%

Table 9.24 Summary of hydraulic model calibration – 2013 event[1]

Gauge	Ground surface level (m AHD)	Observed water level (m AHD)	Modelled water level (m AHD)	Observed water depth (m)	Modelled water depth (m)	Difference in water level (m)	Difference in water depth (%)
Tummaville	380.926	391.690	391.465	10.76	10.54	-0.225	-2.09%
Yarramalong Weir	378.930	385.939	385.521	7.01	6.59	-0.418	-5.96%
Pampas	379.000	382.090	382.303	3.09	3.30	+0.213	+6.90%

Table note:

1 Values in the tables above are shown to three decimal places for comparative purposes only, and not to imply absolute accuracy of results

Calibration may require further refinement and it is recommended that the calibration should be improved in following phases using additional topographic survey that is focused on specific areas. However, throughout the development of the model, it was observed that changes made throughout calibration iterations have not resulted in significant changes to corresponding design event runs because the broad and complex floodplain is not overly sensitive to tested parameters such as inflow application methods or hydraulic roughness parameters.

Consequently, it is suggested that the results produced by this calibrated hydraulic model are fit-for-purpose in determining the impacts of various feasibility design scenarios on existing floodplain conditions.

9.3.6 Validation

Through consultation a total of 47 floodmarks of the December 2010 flood event were supplied by landowners on the Condamine River floodplain. These floodmarks were surveyed by a professional surveying company. Further confidence in model calibration is gained when validating the anecdotal and surveyed floodmarks against modelling results.

Figure 51 illustrates the geographical location of the floodmarks, including the relative quality of each. In addition, data at three streamflow gauges, namely Tummaville, Yarramalong Weir and Pampas were also available for validation.

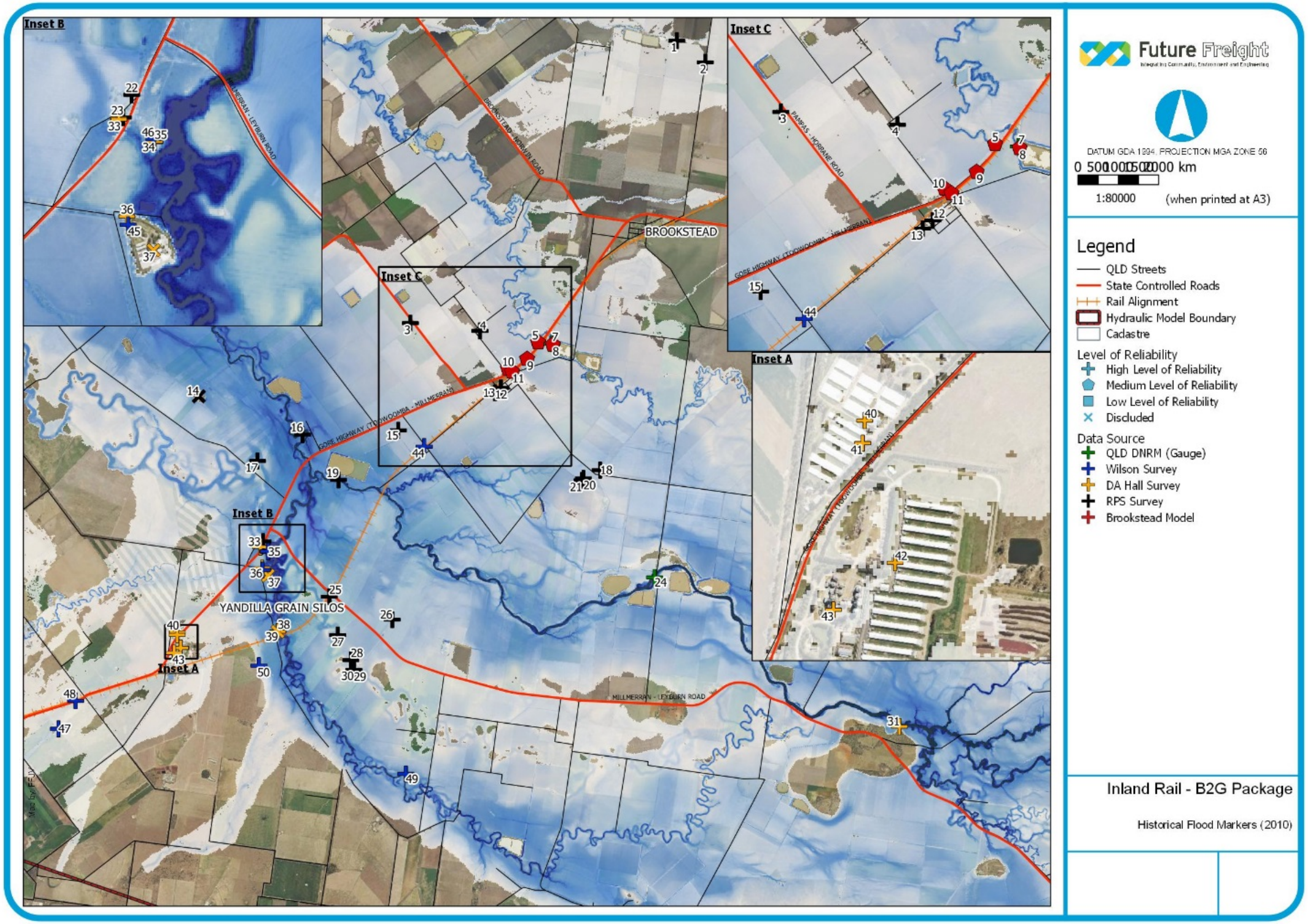


Figure 51 Floodmark locations and relative quality

Table 9.25 summarises the validation grouping of floodmarks.

Table 9.25 Floodmark grouping

Description	Grouping
Good match	+/-200 mm
Acceptable match	+/-200 – 300 mm
Poor match	+/-300 – 500 mm

For the purpose of this study floodmarks matched to within +/- 200 mm in model validation were considered to be a *good* match, with matches within +/- 300 mm considered to be an *acceptable* match. These targeted tolerances are in line with industry best practice and had previously been adopted for other large infrastructure projects such as the Yeppen floodplain crossing, and industry benchmark flood studies such as the Brisbane River Flood Study.

Table 9.28 summarises the floodmark validation results. It should be noted that modelled peak water levels are rounded to three decimal places not to imply accuracy, but to enable a thorough assessment against surveyed levels.

The 50 entries include the three streamflow gauges used for calibration. Excluding the three gauges and the four outliers a total of 32 out of 43 remaining floodmarks (or 74%) achieved a good match, and 38 out of 43 (or 88%) achieve a good or acceptable match.

Commentary on the five *poor* matches are provide in Table 9.28. Figure 53 provides a graphical illustration of floodmark validation results.

9.3.6.1 Manual gauge at Centenary Bridge

A manual gauge at Centenary Bridge was in operation during the December 2010 food event. Table 9.26 compares the modelled peak water levels for the December 2010 calibration event with the observed peak water level (as per SKM, 2014) at Centenary Bridge. It also includes a comparison with the modelled peak water levels obtained by SKM in their flood study for TRC in 2014.

Table 9.26 Recorded and modelled peak flood levels for December 2010 event at Centenary Bridge (SKM observed water level)

Gauge	Observed water level as per SKM (m AHD)	SKM/TRC Flood Study (2014)		This Study (FFJV)	
		Modelled water level (m AHD)	Difference in water level (mm)	Modelled water level (m AHD)	Difference in water level (mm)
Centenary Bridge	379.59[1]	378.85	-740	379.18	-410

Table note:

1 The observed water level quoted by SKM in their 2014 report seems to be incorrect. In their report they quote a difference of 440 mm but 379.59 m AHD minus 378.85 m AHD is 740 mm. However, if one adds 440 mm to 378.85 m AHD, one gets 379.29 m AHD.

Given the uncertainty associated with the observed water level quoted by SKM, the formal record for the Centenary Bridge manual gauge was requested from the BoM. Figure 52 presents the manual gauged readings during the December 2010 flood event. It is unable to tell whether the flood peak was captured. The last data point before the record peak depth of 8.3 m was 11 hours prior.

Based on the gauge zero of 370.99 m AHD a peak flood depth of 8.3 m yields a peak flood level of 379.29 m AHD. This level corresponds with the ‘corrected’ SKM level (see table note 1 above), providing further confidence.

Table 9.27 compares the BoM December 2010 flood peak level with the FFJV modelled water level at Centenary Bridge.

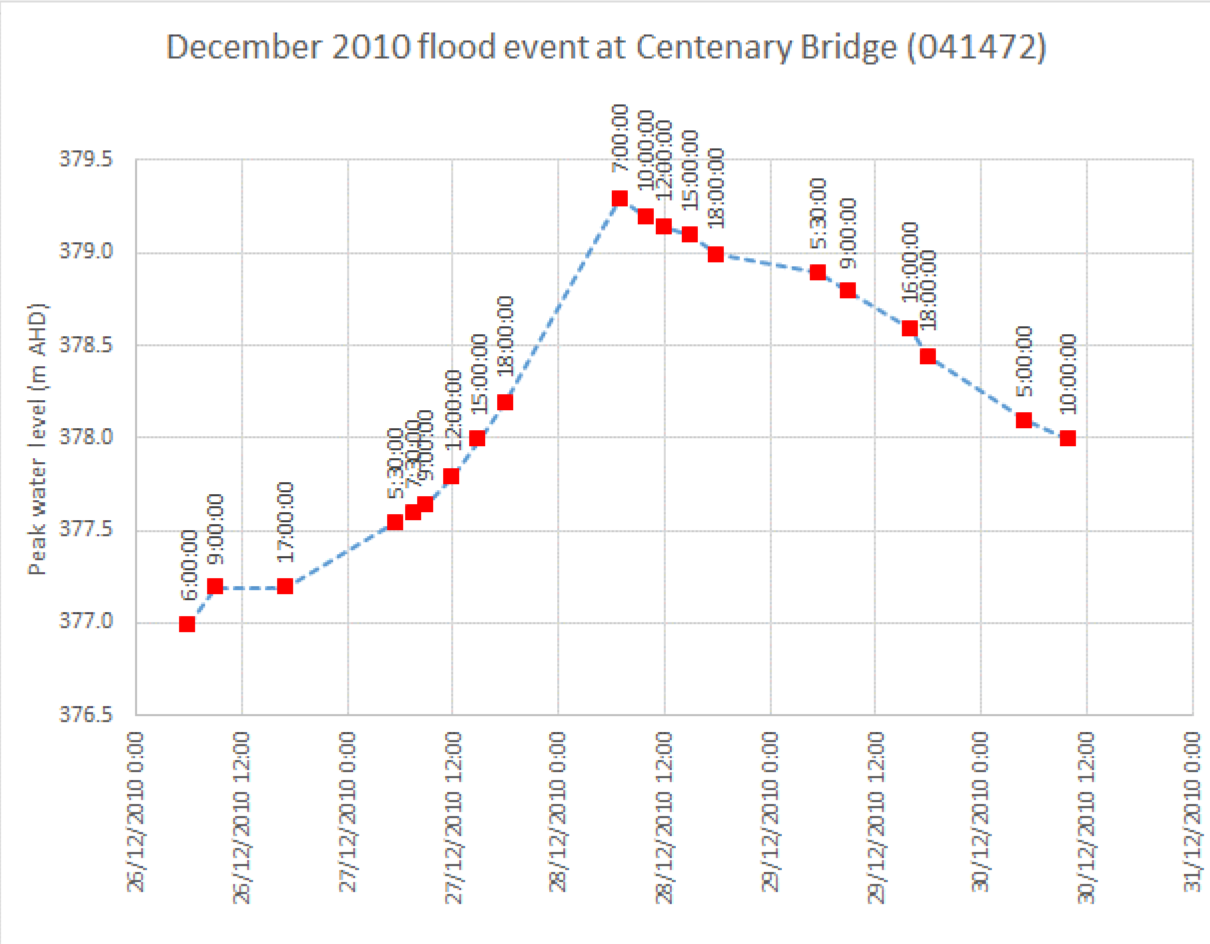


Figure 52 **Manual gauged readings at Centenary Bridge (041472) during the Dec 2010 flood event**

Table 9.27 **Recorded and modelled peak flood levels for December 2010 event at Centenary Bridge (BoM observed water level)**

Gauge	Observed water level as per BoM (m AHD)	This Study (FFJV)	
		Modelled water level (m AHD)	Difference in water level (mm)
Centenary Bridge	379.29	379.18	-110

The BoM data suggests that the FFJV flood model validates well against observed data at Centenary Bridge for the December 2010 event.

SKM/TRC (2014) concluded that the lower simulated level at Centenary Bridge is "*likely to be due to the model not implicitly representing the bridge structures and embankments in these locations due to the 60 m grid size".* The FFJV hydraulic model has a finer grid size of 20 m and therefore achieved a better match with the observed historic flood level; however, uncertainties associated with the terrain data and model grid size remain.

Refer to Section 9.3.7 which discusses uncertainty in topography further.

9.3.6.2 Verification against flood photos

In addition to floodmark validation, an assessment was undertaken to verify modelling results against available photographs of the 2010 flood event. Photographs were supplied directly by some landowners, whilst other photographs were acquired from a recent exhibition in the Millmerran Library.

Out of >400 available photographs, >40 photographs were obtained that relate directly to available floodmarks. Except for the three gauges all the floodmarks are surveyed levels based on information provided by landowners. A degree of uncertainty therefore exists in terms of the accuracy of the floodmarks, given the variables involved such as timing of the observation, wave action, localised hydraulic effects, infrastructure failure etc.

As previously stated WRM (2014), in their study of Condamine River flooding at Brookstead discusses the uncertainty involved in using historic flood marks for validation and verification (refer Section 9.1.1).

9.3.7 Topography representation across calibration events

The floodplain topography has changed considerably over the past 50 years, and it is likely to continue to change. This shortens the horizon of the model's capability to recreate past events and forecast future events. Nevertheless, the base case topography adopted for the Existing Case and Developed Case model scenarios is a composite of the latest LiDAR tiles available, which reflects the most robust representation of the floodplain topography. The topography includes the existing QR railway line embankment

Table 9.28 Surveyed floodmark comparison – 2010 event

Floodmarker ID	Easting	Northing	2010 Peak Water Level (m AHD)	Modelled Peak Water Level (m AHD)	Difference in Peak Water Level (mm)	Grouping	Comments
1	347851	6933298	389.610	389.4421	-168	+/-200 mm	
2	348557	6932762	392.130	392.1436	14	+/-200 mm	
3	341255	6926305	378.720	378.2509	-469	+/-300 – 500 mm	Poor match potentially due to terrain inaccuracies, landuse/cropping density assumption, or local hydraulic controls.
4	342977	6926114	382.010	381.724	-286	+/-200 – 300 mm	
5	344418	6925831	381.780	382.167	387	Outlier[1]	Same location as ID 6 (which validates well). ID 5 has previously been discounted by WRM (2014) as outlier.
6	344418	6925831	382.440	382.167	-273	+/-200 – 300 mm	
7	344762	6925794	382.594	382.545	-49	+/-200 mm	Pampas Gauge
8	344779	6925771	382.590	382.553	-37	+/-200 mm	
9	344145	6925418	382.210	382.407	197	+/-200 mm	
10	343673	6925174	382.300	382.191	-109	+/-200 mm	
11	343774	6925102	382.630	382.469	-161	+/-200 mm	
12	343498	6924703	382.330	381.926	-404	+/-300 – 500 mm	Poor match potentially due to terrain inaccuracies, landuse/cropping density assumption, or local hydraulic controls.
13	343349	6924581	382.090	381.814	-276	+/-200 – 300 mm	
14	336017	6924495	376.270	-	0	Outlier[2]	
15	340957	6923652	379.980	379.872	-108	+/-200 mm	
16	338590	6923546	379.000	378.939	-61	+/-200 mm	
17	337470	6922892	378.420	378.922	502	+/-300 – 500 mm	Based on aerial imagery this property is protected by a levee. The FFJV model assumes no levee, hence it overpredicts 2010 flood depths at that location. There is a valid argument to also discount this floodmark as an outlier.
18	345953	6922666	384.090	384.090	0	+/-200 mm	
19	339476	6922425	379.430	379.727	297	+/-200 – 300 mm	
20	345539	6922459	383.480	383.475	-5	+/-200 mm	
21	345501	6922414	383.540	383.433	-107	+/-200 mm	

Floodmarker ID	Easting	Northing	2010 Peak Water Level (m AHD)	Modelled Peak Water Level (m AHD)	Difference in Peak Water Level (mm)	Grouping	Comments
22	337612	6920898	380.010	379.958	-52	+/-200 mm	
23	337567	6920775	380.080	379.993	-87	+/-200 mm	
24	347300	6920011	386.190	385.991	-199	+/-200 mm	Yarramalong Gauge
25	339247	6919533	380.410	380.428	18	+/-200 mm	
26	340806	6918963	381.160	380.993	-167	+/-200 mm	
27	339456	6918583	380.780	380.692	-88	+/-200 mm	
28	339779	6917966	381.150	380.858	-292	+/-200 – 300 mm	
29	339858	6917719	381.680	381.220	-460	+/-300 – 500 m	ID 29 and ID 30 are located <50m apart at the same location and could therefore be treated as one floodmark. The poor match is potentially due to terrain inaccuracies, land use/cropping density assumption, or local hydraulic controls.
30	339845	6917698	381.630	381.220	-410	+/-300 – 500 m	
31	353364	6916318	392.061	392.114	53	+/-200 mm	Tummaville Gauge
32	337541	6920771	380.092	379.996	-96	+/-200 mm	
33	337543	6920764	380.078	379.998	-80	+/-200 mm	
34	337736	6920641	380.221	380.036	-184	+/-200 mm	
35	337737	6920644	380.220	380.036	-183	+/-200 mm	
36	337586	6920239	380.187	380.116	-70	+/-200 mm	
37	337731	6920056	380.496	-	0	Outlier[2]	
38	337972	6918669	380.684	380.426	-257	+/-200 – 300 mm	
39	337972	6918669	-	-	-	Outlier[3]	2013 flood level
40	335486	6918651	384.088	383.906	-182	+/-200 mm	Taken from adjacent cell where topography is closer match to surveyed level.
41[4]	335481	6918589	384.238	-	-129	+/-200 mm	Surveyed ground level is 400 mm above LiDAR and DEM Z
42[4]	335569	6918265	384.812	-	69	+/-200 mm	Surveyed ground level is 200 mm above LiDAR and 300 mm above DEM Z
43	335402	6918138	385.425	385.225	-200	+/-200 mm	
44[4]	341597	6923248	380.988	-	-192	+/-200 mm	Topography, almost 300 mm below survey level within cell

Floodmarker ID	Easting	Northing	2010 Peak Water Level (m AHD)	Modelled Peak Water Level (m AHD)	Difference in Peak Water Level (mm)	Grouping	Comments
45	337593	6920194	380.308	380.253	-55	+/-200 mm	Mark is taken at top of bund and measured to FL (600 mm below). Water levels and topography match well.
46	337733	6920658	380.213	380.029	-184	+/-200 mm	
47	332543	6916262	393.927	393.945	18	+/-200 mm	
48	332968	6916955	392.070	391.945	-125	+/-200 mm	
49[4]	341143	6915145	382.752	-	-85	+/-200 mm	Surveyed ground level is approximately 1.5 m higher than DEM Z in this spot
50[4]	337506	6917826	380.989	-	-129	+/-200 mm	Topography ranges from -50 mm to 150 mm higher, as such the water level is marginally out

Table notes:

1 Same location as ID 6 (which validates well). ID 5 has previously been discounted by WRM (2014) as outlier.

2 Floodmarks surveyed within flood levee which had failed during 2010 flood event. The flood model assumes flood levee to be structurally sound.

3 2013 flood level at abandoned hut; therefore discounted.

4 Depth was adopted in assessment instead of water surface elevation; justification for doing so highlighted in table.

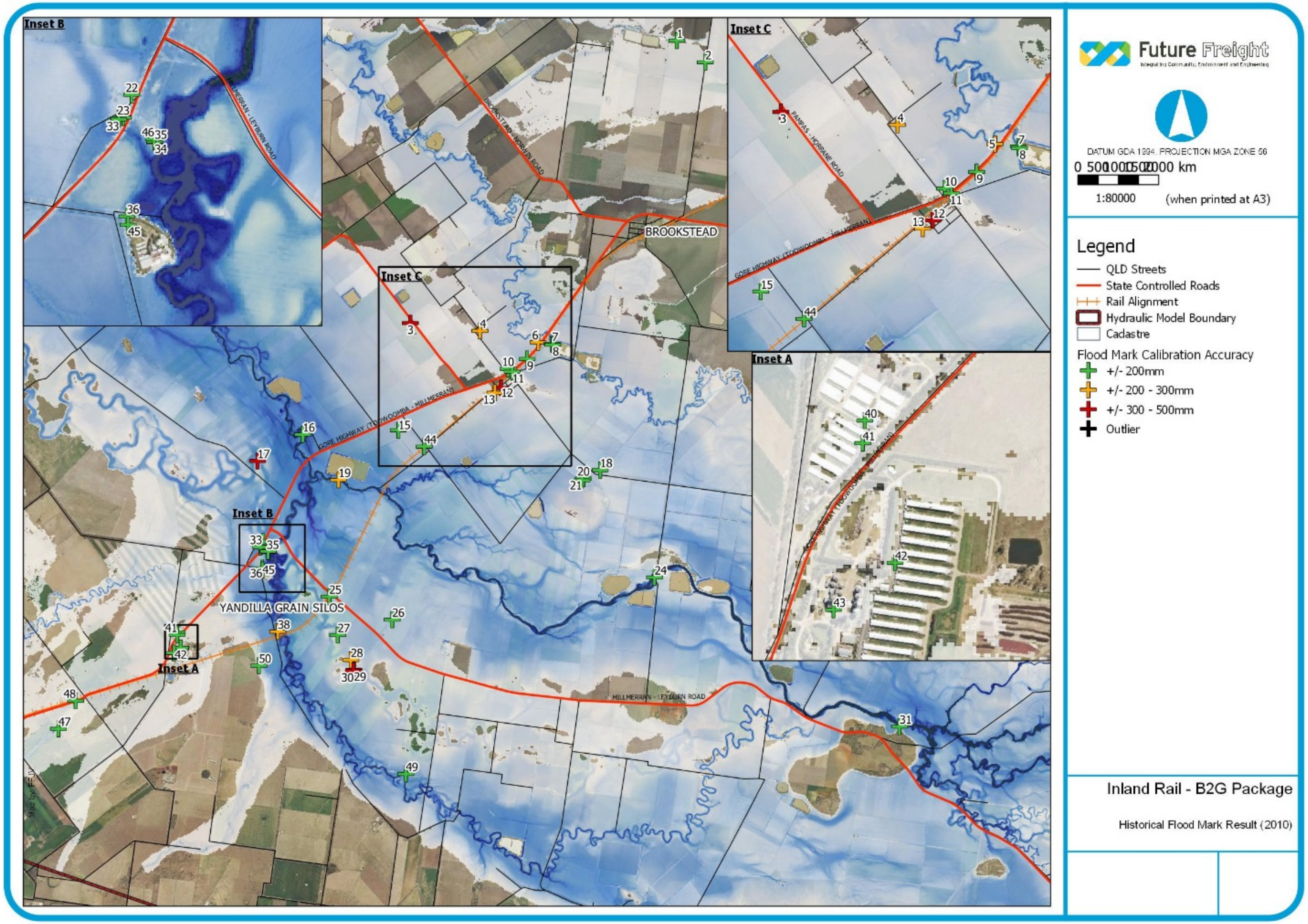
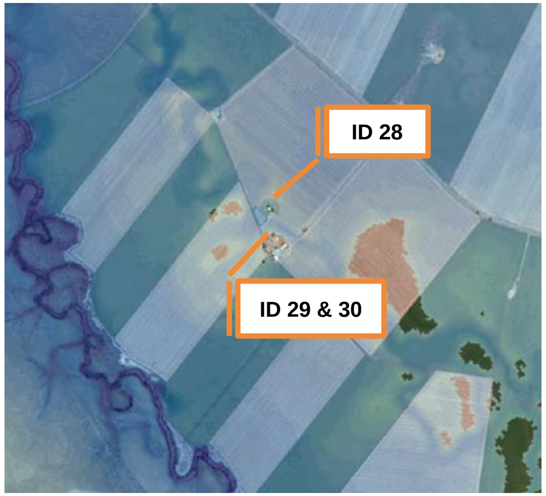
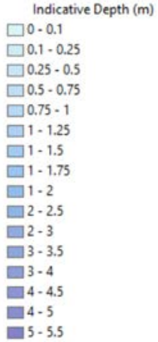
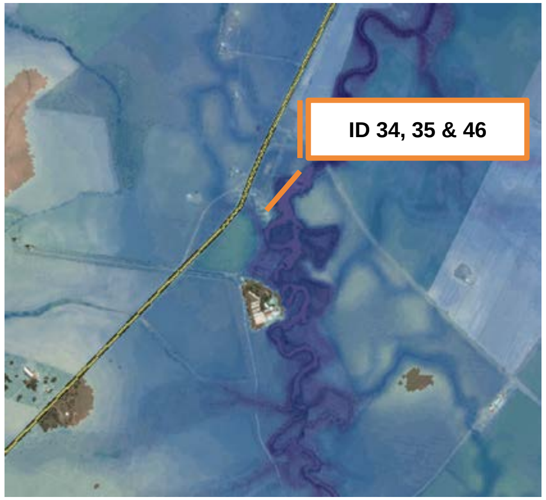
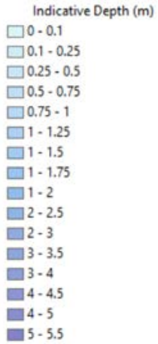
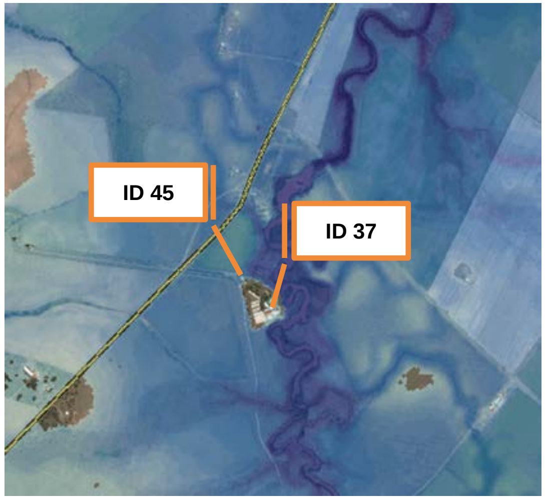
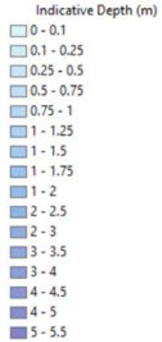
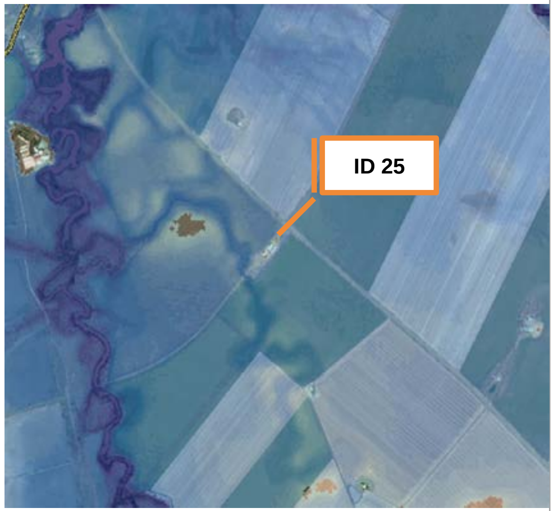
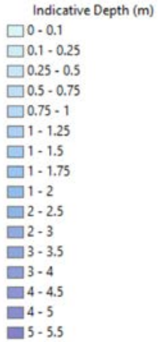


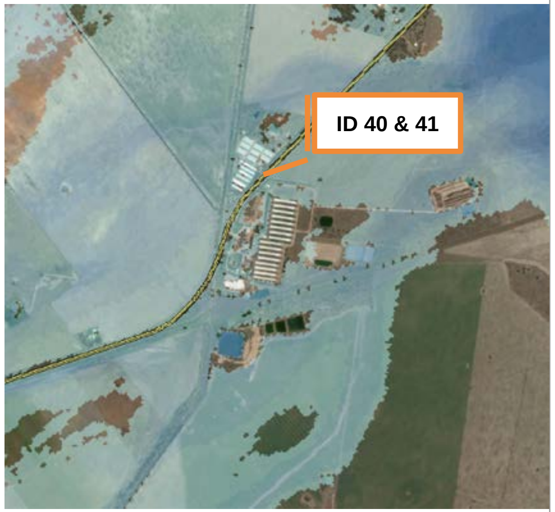
Figure 53 Graphical representation of floodmark validation results

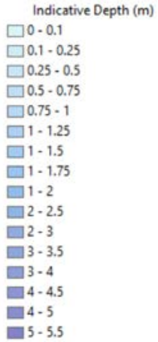
Table 9.29 Modelling result verification against photographs

Floodmark ID	Photograph	Flood model extract (estimated depth, m)	Legend	Commentary
ID 28, 29 & 30	 House and sheds on Lotplan 3RP35147, Millmerran-Leyburn Road			Photographed in 2010 during the flooding event at an unknown time. The flood model underestimates the peak flood level, but the general flood extent and behaviour is a match.
ID 34, 35 & 46	 House on Lotplan 1DY939, Yandilla			Photo taken after peak. Peak modelled levels are within +/- 200 mm of surveyed peak flood levels.

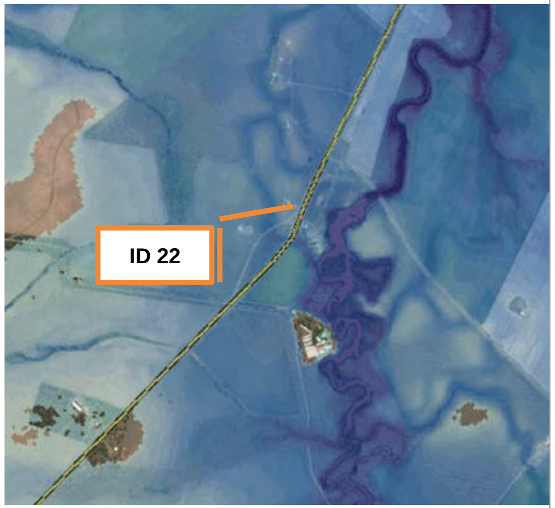
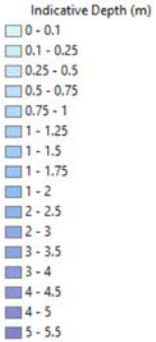
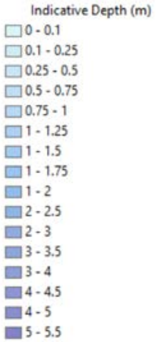
Floodmark ID	Photograph	Flood model extract (estimated depth, m)	Legend	Commentary
ID 37 and 45	 Hall's Poultry offices and sheds surrounded by levee bank. 2010 level came right up to levee crest but did not overtop 			Peak modelled level is within +/- 200 mm of surveyed floodmark. Note that marker 37 is a projected level by Hall's to determine where the flood level would have come up to had there been a breach in the levee bank. The area inside the levee bank did not actually flood.

Floodmark ID	Photograph	Flood model extract (estimated depth, m)	Legend	Commentary
ID 25	 Aerial photograph after peak (time unknown)  Photograph of debris line at Yandilla Grain Silos denoting peak flood level			Peak modelled level is within +/- 200 mm of surveyed floodmark. Silos pad is dry but surrounded by flood water.

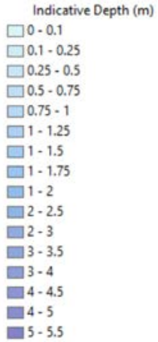
Floodmark ID	Photograph	Flood model extract (estimated depth, m)	Legend	Commentary
ID 40 & 41	  Halls Poultry Sheds – north of Gore Highway		Indicative Depth (m) 0 - 0.1 0.1 - 0.25 0.25 - 0.5 0.5 - 0.75 0.75 - 1 1 - 1.25 1 - 1.5 1 - 1.75 1 - 2 2 - 2.5 2 - 3 3 - 3.5 3 - 4 4 - 4.5 4 - 5 5 - 5.5	Peak modelled level is within +/- 200 mm of surveyed floodmarks.

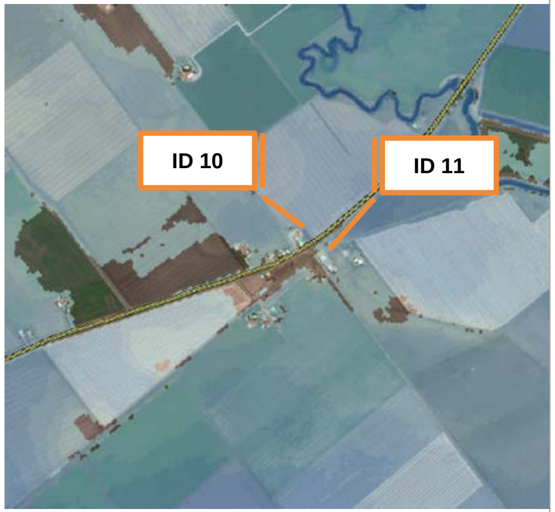
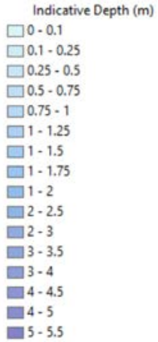
Floodmark ID	Photograph	Flood model extract (estimated depth, m)	Legend	Commentary
ID 43	   Halls Poultry feedmill – south of Gore Hwy			Peak modelled level is within +/- 200 mm of surveyed floodmark.

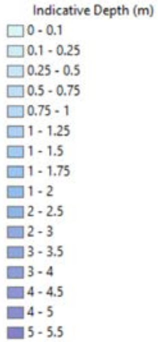
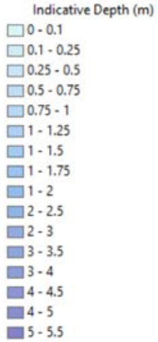
Floodmark ID	Photograph	Flood model extract (estimated depth, m)	Legend	Commentary
ID 23, 32 and 33	   Yandilla Church		Indicative Depth (m) 0 - 0.1 0.1 - 0.25 0.25 - 0.5 0.5 - 0.75 0.75 - 1 1 - 1.25 1 - 1.5 1 - 1.75 1 - 2 2 - 2.5 2 - 3 3 - 3.5 3 - 4 4 - 4.5 4 - 5 5 - 5.5	Good match +/- 200 mm (one marker in building and two on fence).

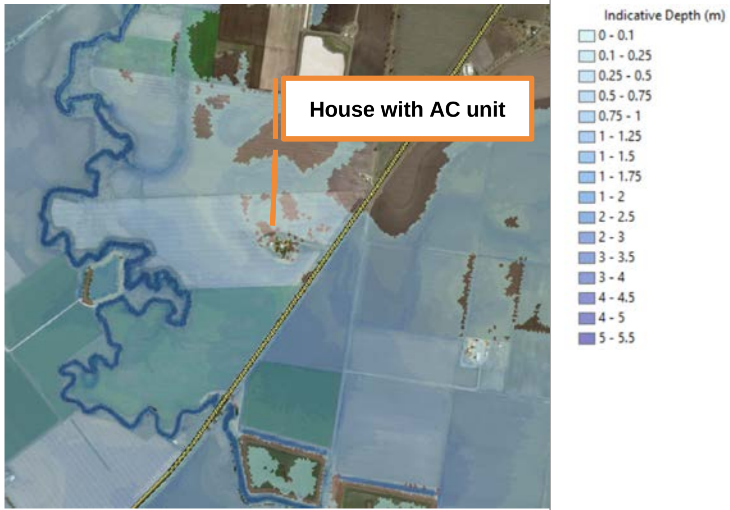
Floodmark ID	Photograph	Flood model extract (estimated depth, m)	Legend	Commentary
ID 22	 Letterbox in driverway of Lotplan 2RP71762, Gore Hwy			Peak modelled level is within +/- 200 mm of surveyed floodmark.
ID 12	 316 Fysh Road, Pampas			Not a good match in terms of level; however, flooding around the house is evident. Difference could be due to cropping patterns at the time of the flood or topography (i.e. LiDAR) inaccuracies.

Floodmark ID	Photograph	Flood model extract (estimated depth, m)	Legend	Commentary
ID 18	 Car travelling on Fysh Road  House on Lotplan 1RP42427, Fysh Road		Indicative Depth (m) 0 - 0.1 0.1 - 0.25 0.25 - 0.5 0.5 - 0.75 0.75 - 1 1 - 1.25 1 - 1.5 1 - 1.75 1 - 2 2 - 2.5 2 - 3 3 - 3.5 3 - 4 4 - 4.5 4 - 5 5 - 5.5	Peak modelled level is within +/- 200 mm of surveyed floodmark.

Floodmark ID	Photograph	Flood model extract (estimated depth, m)	Legend	Commentary
ID 20 & 21	 House on Lotplan 1RP48984, Fysh Road after peak  Shed on Lotplan 1RP48984, Fysh Road			Peak modelled level is within +/- 200 mm of surveyed floodmark.

Floodmark ID	Photograph	Flood model extract (estimated depth, m)	Legend	Commentary
ID 10 & 11	 Pampas			This image was taken on the 28th December 2010, around 3hrs after the peak. Modelled peak levels are within +/- 200 mm of the surveyed flood levels at ID locations 10 and 11.
ID 3	 House on Lotplan 4RP55115, Pampas-Horrane Road			Photo taken at 11am Tuesday 28th December ("few hours after the peak"). Flood extent matches observed flood extent; however, the modelled levels do not match the recorded levels.

Floodmark ID	Photograph	Flood model extract (estimated depth, m)	Legend	Commentary
ID 5, 6 & 9	 Gore Hwy between Pampas and North Branch			During peak of flood. Peak modelled level of ID 9 is within +/- 200mm of surveyed floodmark, and ID 6 is within +/- 300mm. At IDs 5 the flood extents match; however, the modelled level do not match the recorded level within the targeted range.
No floodmark	 Bunded property near Lemontree Road/Bosto Creek Road intersection			Good match despite floodmark not being available.

Floodmark ID	Photograph	Flood model extract (estimated depth, m)	Legend	Commentary
No floodmark	 Driveway on Lotplan 2SP119342 north of Gore Hwy, between Pampas and Brookstead  Lotplan 2SP119342 north of Gore Hwy, between Pampas and Brookstead (water reaching AC unit)			Timing unknown but debris line on AC unit suggests the photo was taken after the peak. Despite floodmarks not being available there is a good match between modelling information and photographs.

Floodmark ID	Photograph	Flood model extract (estimated depth, m)	Legend	Commentary
ID 26	 House on Lotplan 1RP53346 at Millmerran-Leyburn Road		Indicative Depth (m) 0 - 0.1 0.1 - 0.25 0.25 - 0.5 0.5 - 0.75 0.75 - 1 1 - 1.25 1 - 1.5 1 - 1.75 1 - 2 2 - 2.5 2 - 3 3 - 3.5 3 - 4 4 - 4.5 4 - 5 5 - 5.5	Peak modelled level is within +/- 200 mm of surveyed floodmark.

9.4 Existing Case modelling results – Condamine River

9.4.1 Existing Case flood maps

Maps illustrating indicative flood extents and peak water levels were prepared and are presented in Volume II – Appendix C:

- 20% AEP: Figure C-3a
- 10% AEP: Figure C-3b
- 5% AEP: Figure C-3c
- 2% AEP: Figure C-3d
- 1% AEP: Figure C-3e
- 1 in 2,000 AEP: Figure C-3f
- 1 in 10,000 AEP: Figure C-3g
- PMF: Figure C-3h.

Figure C-4a presents peak flood velocities predicted under a 1% AEP event.

9.4.2 Flood inundation extent and flood levels

The Condamine hydraulic model predicts 1% AEP depths of between 6 m and 11 m throughout the main Condamine River channel. Corresponding velocities in main channel typically range between 1.5 m/s and 2 m/s, however there are localised areas where velocities reach magnitudes as high as 4 m/s.

In the North Condamine River, the channel has a lower flow capacity than the main branch, and 1% AEP depths within the channel are typically between 2 to 3 m and corresponding velocities are typically 1.5 m/s to 2 m/s. In the 1% AEP event, depths range up to approximately 1.5 m on the floodplain with flood velocities remaining below 1 m/s.

Model results indicate that the channels within the Condamine River floodplain have low flow capacity, as even in the 50% AEP event, flooding is seen to spill out from channel banks. Major breakout flows can be seen from the 20% AEP event and events of larger magnitude, particularly from upstream of Tummaville into Grasstree Creek, and from North Condamine River into the Main Condamine River Branch.

Figure C-3e in Volume II – Appendix C shows the 1% AEP indicative flood extent and peak water levels within the Condamine River floodplain for the Existing Case.

9.4.3 Flood immunity of existing infrastructure

Table 9.30 presents a summary of overtopping depths for the existing QR Rail Line and key roads near the Project alignment under a range of design events.

Table 9.30 Condamine River – Existing Case – overtopping depths of key infrastructure

Infrastructure	Location	Maximum overtopping depth (m)							
		PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Gore Highway	D/S of the proposed main branch bridge	4.14	2.84	1.96	1.23	1.03	0.77	0.52	0.19
Millmerran-Leyburn Road	Immediately U/S of proposed alignment	4.02	2.61	1.64	0.80	0.57	0.34	0.23	0.18

Infrastructure	Location	Maximum overtopping depth (m)							
		PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Pampas-Horrane Road	D/S of Project alignment	2.77	1.45	0.64	0.23	0.19	0.12	0.06	-
Existing QR Rail Line	Adjacent to Main Condamine Crossing	4.99	3.66	2.77	2.09	1.93	1.72	1.51	1.24
Existing QR Rail Line	Adjacent to North Branch Condamine Crossing	2.01	1.27	0.95	0.76	0.69	0.55	0.48	0.45
Yandilla Grain Silos	Yandilla, Immediately U/S of Project alignment	3.43	2.02	1.06	0.25	0.05	-	-	-

9.4.4 Existing Case velocities

Peak Existing Case velocities for the 1% AEP event in the Condamine River are typically in the order of 1 to 2 m/s, and on the floodplain, velocities are generally in the order of <0.5 m/s as shown in Volume II – Appendix C Figure C-4a.

9.5 Developed Case modelling results – Condamine River

9.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW hydraulic model (1d and 2d approach).

On the Condamine River floodplain, the Project includes the following floodplain (or regional structures):

- Six waterway bridges (at four main locations)
- Seventy-one RCP locations (a total of 452 cells)
- Fourteen RCBC locations (a total of 76 cells)

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and depending on their interaction with the Condamine River floodplain were incorporated in the hydraulic model.

A blockage factor of 25% was applied to all proposed culverts based on guidelines set out in ARR 2016. The adopted blockage factor for the proposed bridges was between 5% and 10% based on the waterway area blockage due to bridge piers.

The proposed drainage structures are summarised in Table 9.31 and Table 9.32, and shown in Figure C-1f to C-1k in Volume II – Appendix C.

Table 9.31 Condamine River – proposed bridge locations and details

Chainage (km)	Bridge design ID	Bridge name	Soffit level (m AHD)	Deck depth (m)	Bridge length (m)
138.01-138.35	310-BR21	Grasstree Creek #1 Rail Bridge	382.05	2	336
138.78-139.33	310-BR22	Grasstree Creek #2 Rail Bridge	382.05	2	952
141.34-142.00	310-BR24	Condamine River #1 Rail Bridge	382.06	2	658
142.60-144.51	310-BR25	Condamine River #2 Rail Bridge	382.06	2	1,918
144.54-145.14	310-BR26	Condamine River #3 Rail Bridge	382.06	2	602
147.78-149.33	310-BR27	Condamine River North Branch Rail Bridge	383.79	2	1,568

Table 9.32 Condamine River – proposed floodplain culvert locations and details

Chainage (km)	Structure ID	Structure type	U/S invert (m AHD)	D/S invert (m AHD)	Diameter/width (m)	Height (m)	Number of cells
131.39	C131.39	RCP	401.00	400.50	2.1	-	1
131.49	C131.49	RCP	400.50	400.00	2.1	-	1
137.83	C137.83	RCP	379.96	379.83	1.35	-	8
137.88	C137.88	RCP	379.69	379.57	1.65	-	11
137.92	C137.92	RCP	379.46	379.35	1.8	-	8
139.37	C139.37	RCP	379.43	379.33	1.8	-	11
139.44	C139.44	RCP	379.12	379.10	2.1	-	8
139.50	C139.5	RCP	379.22	379.18	2.1	-	8
139.56	C139.56	RCP	379.40	379.36	1.8	-	11
139.71	C139.71	RCP	379.27	379.23	1.65	-	9
139.73	C139.73	RCBC	378.10	378.08	2.4	1.8	4
139.78	C139.78	RCP	379.81	378.73	2.1	-	10
140.09	C140.09	RCP	379.69	379.34	1.8	-	7
140.11	C140.11	RCP	379.68	379.35	1.8	-	7
140.17	C140.17	RCP	379.38	379.29	2.1	-	6
140.21	C140.21	RCP	379.45	379.35	2.1	-	6
140.23	C140.23	RCP	379.43	379.33	2.1	-	6
140.25	C140.25	RCP	379.43	379.33	2.1	-	6
140.27	C140.27	RCP	379.54	379.29	2.1	-	6
140.32	C140.32	RCP	379.44	379.31	2.1	-	6
140.38	C140.38	RCP	379.37	379.27	2.1	-	6
140.40	C140.4	RCP	379.20	379.10	2.1	-	6
140.43	C140.43	RCP	379.18	379.08	1.8	-	7
140.46	C140.46	RCP	379.11	379.03	2.1	-	5
140.49	C140.49	RCP	379.12	379.04	2.1	-	6
140.51	C140.51	RCP	379.09	379.00	2.1	-	6
140.55	C140.55	RCP	379.11	379.01	2.1	-	5
140.59	C140.59	RCP	379.08	378.99	2.1	-	5
140.64	C140.64	RCP	379.11	378.96	2.1	-	6
140.67	C140.67	RCP	379.09	378.97	2.1	-	5
140.78	C140.78	RCP	379.15	378.68	2.1	-	6

Chainage (km)	Structure ID	Structure type	U/S invert (m AHD)	D/S invert (m AHD)	Diameter/width (m)	Height (m)	Number of cells
140.83	C140.83	RCP	379.20	378.83	2.1	-	6
140.87	C140.87	RCP	379.18	378.85	2.1	-	4
140.91	C140.91	RCP	379.14	378.81	2.1	-	6
140.98	C140.98	RCP	379.08	378.86	2.1	-	6
141.03	C141.03	RCP	379.02	378.87	2.1	-	4
141.07	C141.07	RCP	379.01	378.77	2.1	-	6
141.11	C141.11	RCP	379.08	378.81	2.1	-	6
141.20	C141.2	RCP	379.08	378.95	2.1	-	6
141.24	C141.24	RCP	379.11	378.83	2.1	-	6
141.29	C141.29	RCP	379.10	379.01	2.1	-	6
141.32	C141.32	RCP	378.86	378.60	2.1	-	4
142.02	C142.02	RCP	379.04	378.84	2.1	-	6
142.04	C142.04	RCP	379.07	378.88	2.1	-	6
142.08	C142.08	RCP	379.08	378.87	2.1	-	6
142.13	C142.13	RCP	379.16	378.99	2.1	-	6
142.15	C142.15	RCP	379.14	379.02	2.1	-	6
142.19	C142.19	RCP	379.21	378.93	2.1	-	6
142.22	C142.22	RCP	379.13	378.93	2.1	-	6
142.25	C142.25	RCP	379.29	379.02	2.1	-	6
142.28	C142.28	RCP	379.28	379.02	2.1	-	5
142.36	C142.36	RCP	379.31	379.12	2.1	-	6
142.41	C142.41	RCP	379.33	379.20	2.1	-	6
142.44	C142.44	RCP	379.29	379.11	2.1	-	6
142.48	C142.48	RCP	379.26	379.08	2.1	-	6
142.50	C142.5	RCP	379.25	379.04	2.1	-	5
142.54	C142.54	RCP	379.20	379.01	2.1	-	4
142.58	C142.58	RCP	379.26	378.96	2.1	-	5
145.16	C145.16	RCBC	380.27	380.19	1.2	0.9	4
145.21	C145.21	RCBC	380.36	380.35	1.2	0.9	4
145.25	C145.25	RCBC	380.41	380.40	1.2	0.9	4
145.32	C145.32	RCBC	380.60	380.55	1.2	0.9	2
145.40	C145.4	RCBC	380.80	380.75	1.2	0.9	6
145.72	C145.72	RCBC	380.92	380.74	1.5	0.9	10
145.83	C145.83	RCBC	381.14	380.65	1.2	0.9	4
145.89	C145.89	RCBC	380.86	380.66	1.5	0.9	10
145.92	C145.92	RCBC	381.00	380.70	1.2	0.9	4
145.98	C145.98	RCBC	381.06	380.78	1.2	0.9	4
146.03	C146.03	RCBC	380.94	380.89	1.5	0.9	10
146.56	C146.56	RCBC	381.65	381.40	1.2	0.6	6
146.62	C146.62	RCBC	381.66	381.48	1.2	0.6	4
147.58	C147.58	RCP	381.48	381.42	1.05	-	6
147.63	C147.63	RCP	381.50	381.44	1.05	-	6

Chainage (km)	Structure ID	Structure type	U/S invert (m AHD)	D/S invert (m AHD)	Diameter/width (m)	Height (m)	Number of cells
147.66	C147.66	RCP	381.45	381.38	1.05	-	6
147.73	C147.73	RCP	381.42	381.34	1.05	-	7
149.39	C149.39	RCP	381.81	381.73	1.35	-	10
149.42	C149.42	RCP	381.83	381.76	1.2	-	12
149.45	C149.45	RCP	381.83	381.77	1.35	-	3
149.76	C149.76	RCP	381.84	381.81	1.2	-	8
149.80	C149.8	RCP	381.86	381.86	1.2	-	8
149.83	C149.83	RCP	381.85	381.81	1.2	-	8
149.87	C149.87	RCP	381.82	381.78	1.35	-	6
149.91	C149.91	RCP	381.81	381.78	1.35	-	6
149.96	C149.96	RCP	382.10	382.01	1.2	-	8
150.01	C150.01	RCP	382.23	382.17	1.05	-	8

Table 9.33 notes that over half of the proposed culvert cells are large 2.1 m diameter barrels.

Table 9.33 Condamine River – summary of proposed number of floodplain culvert cells

Structure type	Diameter (m)	Width (m)	Number of cells
RCBC	2.40	1.80	4
RCBC	1.50	0.90	30
RCBC	1.20	0.90	32
RCBC	1.20	0.60	10
Subtotal		**76**	
RCP	2.10	-	271
RCP	1.80	-	51
RCP	1.65	-	20
RCP	1.35	-	33
RCP	1.20	-	44
RCP	1.05	-	33
Subtotal		**452**	
Total		**528**	

9.5.1.1 Cross drainage structure blockage

Cross drainage structure blockage was considered and applied in accordance with the latest ARR 2016 guidelines. In accordance with ARR 2016, blockage factors can vary with respect to event magnitude and the catchment conditions, such as slopes and ambient vegetation. However, as the catchment characteristics were determined to be fairly consistent throughout the catchment, a uniform blockage factor at each structure was expected. After each culvert was independently assessed, a blockage factor of 25% was confirmed as suitably conservative to be adopted across all culverts.

Bridges and viaduct structures were represented within the TUFLOW model through use of layered flow constrictions. Each bridge/viaduct within the model has had a flow constriction coefficient and percentage blocked value applied to represent obstruction of waterway area due to the piers. Using standard ARTC bridge pier sizes and span spacing, the waterway blockage due to piers typically comprised 3 to 5% of the total waterway area. And despite the catchment being vegetated, the likelihood of significant amounts of debris accumulating against the piers of the bridge is low. Therefore, no additional debris blockage factor was applied at the proposed Condamine bridges. Consequently, a fixed blockage factor of 5% was adopted.

9.5.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

9.5.2.1 Rail immunity and structures results

Appendix C summarises the peak 1% AEP water levels on the upstream side of the proposed alignment. Local drainage structures (i.e. those not included in the flood model) and road culverts are not reported. The results of flood modelling indicate that a 1% AEP event flood immunity to the proposed rail formation level is achieved for the Project alignment across the Condamine River floodplain. There is over 0.69 m freeboard above the culvert obvert levels to the rail formation in a 1% AEP event.

9.5.2.2 Velocities and scour

Appendix C summarises the peak 1% AEP outlet flows and velocities at structures. The 1% AEP peak velocity through the proposed drainage structures is generally less than 2.2 m/s.

Scour protection requirements for culverts were calculated based on the velocities predicted from the hydraulic modelling. The scour protection was 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.

Table 9.34 lists the soil types encountered along the Project alignment and the allowable soil velocity based on AGRD.

Table 9.34 Allowable soil velocities along the Project alignment

Soil type	Soil description	Maximum allowable soil velocity as per AGRD (m/s)
Sodosols	Firm loam or stiff clay	2 m/s
Vertosols, Dermosols, Rudosols and Kandosols	Sandy or silty clay	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.

Scour protection requirements for culverts have been calculated based on the velocities predicted from the hydraulic modelling. Appendix D presents the proposed scour protection at each structure.

A conservative scour estimation has been undertaken at the bridge site based on available information and will be refined during detailed design.

9.5.2.3 Flood immunity for extreme events

The risk of overtopping of the rail alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF) with Table 9.35 presenting the depth of water above the formation level and over the top of rail at each structure. It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the alignment. As such, overtopping of the rail is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario.

Table 9.35 Condamine River – extreme events – depth of water above formation and top of rail levels

Chainage (km)	Depth of water above formation level (m)			Depth of water over top of rail (m)[1]		
	1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
132.00-132.94	-	0.18	0.27	-	-	-
137.93-138.00	-	0.17	1.47	-	-	0.77
138.18	-	-	1.55	-	-	0.85
139.35-141.27	-	0.28	1.56	-	-	0.86
141.67	-	-	0.50	-	-	-
142.00-142.58	-	-	1.12	-	-	0.42
144.88	-	-	0.34	-	-	-
145.24-147.50	-	0.42	1.12	-	-	0.42

Table note:

1 Assuming top of rail is 700 mm above formation level

9.5.3 Flood impact objectives outcomes – Condamine River

The impact of the Developed Case was assessed through inclusion of the proposed rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, duration of inundation, velocities and peak flows within the floodplain:

- Changes in peak water levels for the AEP's assessed are presented in Figures C-5a to C-5h in Volume II – Appendix C
- Changes in 1% AEP duration of inundation are presented in Figure C-5i in Volume II – Appendix C
- Changes in 1% AEP velocities are presented in Figure C-5j in Volume II – Appendix C.

All impacts are required to be agreed with relevant stakeholders and affected landowners as part of the EIS process. One-on-one consultation with landowners who are expected to experience changes in flooding behaviour on the property was conducted by ARTC supported by FFJV.

The Project design outcomes relative to the flood impact objectives (refer Table 9.36) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Volume II – Appendix N.

9.5.3.1 Flood impacts at proposed hydraulic structures

The estimated potential impacts to peak water levels at each proposed structure is presented in Table 9.36. Peak water levels were extracted immediately upstream of each culvert and at the control line of each bridge.

The design achieved a freeboard of at least 300 mm between the 1% AEP peak water and formation levels.

Table 9.36 Condamine River – 1% AEP event – estimated impacts to peak water levels at proposed hydraulic structures

Structure ID	Rail formation level or bridge deck height (m AHD)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Change in peak water level (mm)
C131.39	406.04	401.14	401.79	+650
C131.49	406.04	400.80	401.79	+980
C137.83	382.57	380.57	380.65	+80

Structure ID	Rail formation level or bridge deck height (m AHD)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Change in peak water level (mm)
C137.88	382.38	380.56	380.62	+60
C137.92	382.24	380.56	380.64	+70
310-BR21	383.05	380.56	380.57	+10
310-BR22	383.05	380.57	380.60	+40
C139.37	382.24	380.56	380.63	+70
C139.44	382.24	380.56	380.64	+80
C139.50	382.24	380.56	380.64	+80
C139.56	382.24	380.56	380.63	+80
C139.71	382.24	380.56	380.65	+90
C139.73	382.24	380.56	380.65	+90
C139.78	382.24	380.55	380.63	+80
C140.09	382.24	380.51	380.62	+110
C140.11	382.24	380.51	380.62	+100
C140.17	382.24	380.50	380.57	+70
C140.21	382.24	380.50	380.57	+70
C140.23	382.24	380.49	380.56	+70
C140.25	382.24	380.49	380.56	+70
C140.27	382.24	380.49	380.56	+70
C140.32	382.24	380.49	380.56	+70
C140.38	382.24	380.49	380.56	+70
C140.40	382.24	380.49	380.56	+70
C140.43	382.24	380.48	380.56	+80
C140.46	382.24	380.48	380.55	+70
C140.49	382.24	380.48	380.54	+70
C140.51	382.24	380.47	380.54	+70
C140.55	382.24	380.47	380.55	+80
C140.59	382.34	380.46	380.55	+90
C140.64	382.33	380.45	380.54	+90
C140.67	382.33	380.45	380.55	+100
C140.78	382.33	380.42	380.52	+100
C140.83	382.33	380.42	380.51	++90
C140.87	382.33	380.41	380.51	+100
C140.91	382.33	380.41	380.51	+100
C140.98	382.34	380.39	380.49	+100
C141.03	382.33	380.39	380.50	+110
C141.07	382.24	380.38	380.49	+100
C141.11	382.24	380.38	380.48	+100
C141.20	382.24	380.37	380.47	+100
C141.24	382.24	380.37	380.46	+90
C141.29	382.24	380.36	380.45	+80
C141.32	382.24	380.37	380.43	+60

Structure ID	Rail formation level or bridge deck height (m AHD)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Change in peak water level (mm)
310-BR24	383.06	380.35	380.37	+20
C142.02	382.24	380.36	380.39	+30
C142.04	382.24	380.36	380.40	+40
C142.08	382.24	380.35	380.39	+40
C142.13	382.24	380.34	380.39	+60
C142.15	382.24	380.34	380.39	+50
C142.19	382.24	380.34	380.39	+50
C142.22	382.24	380.33	380.39	+50
C142.25	382.24	380.33	380.40	+70
C142.28	382.24	380.33	380.41	+80
C142.36	382.24	380.33	380.40	+70
C142.41	382.24	380.35	380.41	+70
C142.44	382.24	380.35	380.41	+70
C142.48	382.24	380.36	380.41	+50
C142.50	382.24	380.36	380.42	+60
C142.54	382.24	380.37	380.43	+60
C142.58	382.24	380.37	380.41	+40
310-BR25	383.06	380.62	380.66	+40
310-BR26	383.06	380.76	380.74	-20
C145.16	382.24	380.81	380.87	+60
C145.21	382.24	380.89	380.98	+90
C145.25	382.24	380.95	381.07	+120
C145.32	382.24	381.02	381.10	+80
C145.40	382.26	381.08	381.14	+70
C145.72	382.26	381.14	381.32	+170
C145.83	382.35	381.22	381.39	+170
C145.89	382.36	381.18	381.38	+200
C145.92	382.37	381.23	381.39	+160
C145.98	382.38	381.35	381.41	+60
C146.03	382.39	381.31	381.45	+130
C146.56	382.50	381.77	381.80	+20
C146.62	382.51	381.80	381.82	+20
C147.58	383.37	382.47	382.47	-
C147.63	383.55	382.47	382.51	+40
C147.66	383.68	382.47	382.45	-20
C147.73	383.92	382.45	382.40	-60
310-BR27	384.79	382.64	382.64	-
C149.39	383.97	382.66	382.65	-10
C149.42	383.97	382.67	382.66	-
C149.45	383.97	382.67	382.67	-
C149.76	383.97	382.71	382.71	-

Structure ID	Rail formation level or bridge deck height (m AHD)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Change in peak water level (mm)
C149.80	383.97	382.71	382.71	-
C149.83	383.97	382.71	382.71	-
C149.87	383.97	382.71	382.71	-
C149.91	383.97	382.71	382.71	-
C149.96	383.98	382.71	382.70	-10
C150.01	384.19	382.71	382.70	-10

9.5.3.2 Flood impacts on flood sensitive receptors

Flood sensitive receptors were identified from aerial imagery. Details of where afflux is greater than 10 mm, for events up to the 1% AEP are summarised in Table 9.37. Impacted flood sensitive receptors are labelled in the impact figures in Volume II - Appendix C, Figure C-4a to C-5j.

Impacts to flood sensitive receptors that exceed the flood impact objectives are reported in the EIS Surface Water Chapter.

Table 9.37 Condamine River – estimated impacts to peak water levels at flood sensitive receptors

Flood sensitive receptor ID	Description	Afflux > +/- 10 mm				
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
CON_ID_4	Silos	-	-	-	-	+97
CON_ID_5	Silos	-	-	-	-	+147
CON_ID_6	Silos	-	-	-	-	+117
CON_ID_7	Silos	-	-	-	-	+160
CON_ID_8	Silos	-	-	-	+66	+99
CON_ID_9	Silos	-	-	-	+63	+99
CON_ID_10	Silos	-	-	-	+59	+94
CON_ID_68	Shed	-	-	-	+30	+37
CON_ID_78	House	-	-	-	+33	+42
CON_ID_81	Shed	-	-9	-7	-8	-9
CON_ID_82	House	+12	-	-	-	-
CON_ID_99	House	-	-	-	-	+15
CON_ID_101	Shed	-	-	-	+13	+25
CON_ID_102	Shed	-	-	-	+15	+26
CON_ID_103	Shed	-	-	-	-	+23
CON_ID_118	Shed	-	-	-	-	+136
CON_ID_119	Shed	-	-	-	+21	+36
CON_ID_120	Shed	-	-	-	+25	+38
CON_ID_146	Shed	-	-	-	-	+25
CON_ID_147	Shed	-	-	-	-	+24
CON_ID_148	House	-	-	-	-	+26
CON_ID_150	House	-	-	-	-	-22
CON_ID_151	Shed	-	-	-	-13	-22
CON_ID_152	Shed	-	-	-	-	-22
CON_ID_154	Shed	-	-	-	-	-30

Flood sensitive receptor ID	Description	Afflux > +/- 10 mm				
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
CON_ID_155	House	-	-	-	-10	-31
CON_ID_203	Shed	-	-	+2	+8	+16
CON_ID_204	Shed	-	-	-	+8	+16
CON_ID_205	House	-	-	-	+8	+16
CON_ID_206	Shed	-	-	-	+8	+16
CON_ID_207	Shed	-	-	-	+7	+16
CON_ID_235	Shed	-	-	-	-	-10
CON_ID_247	Shed	-	-	-	+7	+25
CON_ID_274	Shed	-	-	-	-	-15
CON_ID_275	House	-	-	-	-	+32
CON_ID_277	House	-	-	-	-	+36
CON_ID_278	Shed	-	-	-	+1	-5
CON_ID_283	Shed	-	-	-	-8	-10
CON_ID_284	House	-	-	-	-8	-19

9.5.3.3 Flood impacts on state-controlled roads

The extent of the hydraulic model developed for the Condamine River is shown in Figure 54. Within the extent of the hydraulic model, the state-controlled roads which are influenced by flooding and the Project alignment are:

- Gore Highway
- Millmerran-Leyburn Road
- Pampas-Horrane Road.

The location of the state-controlled roads are shown in Figure 54.

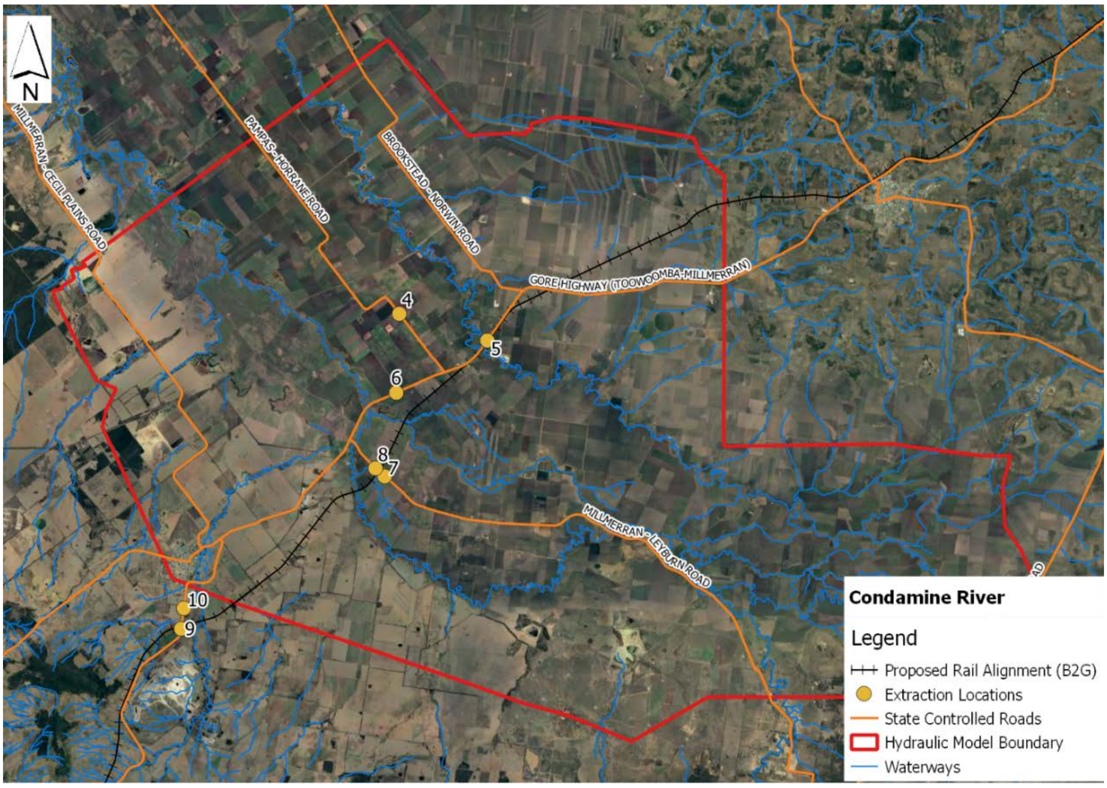


Figure 54 **Condamine River – hydraulic model extent and associated state-controlled roads**

The following sections describe the impacts to state-controlled roads in both the Existing Case and the Developed Case and summarises the differences between the two.

Due to the expanse of the Condamine floodplain, multiple points have been used to depict changes in flood behaviours throughout the model extent. Extraction points 5 and 6 both show results for the Gore Highway; however, point 5 relates primarily to the north branch of the Condamine, whereas point 6 relates primarily to the main branch of the river. Similarly, points 7 and 8 both represent Millmerran Leyburn road; however, point 7 represents the change in conditions upstream of the Project alignment and point 8 reflects the changes in flood behaviours downstream of the Project alignment.

All state-controlled roads within the Condamine floodplain typically exhibit a low flood immunity, being less than 20% AEP immune. The majority of the Gore Highway has 20% immunity and has the highest immunity of roads within the Condamine model.

Existing Case flooding conditions

Table 9.38 **Condamine River – Existing Case flood depths**

Reporting location	Road	Estimated depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
4	Pampas-Horrane Road	0.0	0.14	0.17	0.20	0.31	0.98	2.04	3.29
5	Gore Highway	0.0	0.0	0.0	0.07	0.11	0.30	0.67	1.44
6	Gore Highway	0.01	0.10	0.15	0.24	0.39	1.04	1.93	3.26
7	Millmerran Leyburn Road	0.04	0.21	0.35	0.56	0.78	1.60	2.57	3.98
8	Millmerran Leyburn Road	0.01	0.04	0.25	0.54	0.78	1.61	2.58	3.98

Table 9.39 Condamine River – Existing Case flood inundation length

Reporting location	Road	Approximate length of inundation (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
4	Pampas-Horrane Road	0	98	743	921	2,140	4,620	13,800	17,100
5	Gore Highway	0	0	0	430	647	1476	2880	13,900
6	Gore Highway	3,320	5,400	7,030	7,390	7,600	9,605	9,910	
7	Millmerran Leyburn Road[1]	677	2,200	2,318	2,440	3,560	7,490	8,270	8,990
8	Millmerran Leyburn Road[1]	60	347	499	2,205	2,205	2,205	2,205	2,205

Table note:

1 Segment measured up until proposed rail, to allow for comparison with Existing Case

Table 9.40 Condamine River – Existing Case time of submergence

Reporting location	Road	Estimated time of submergence (hrs)								AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
4	Pampas-Horrane Road	0	89	102	102	107	115	135	138	14.45
5	Gore Highway	0	0	0	49	64	94	122	151	2.1
6	Gore Highway	0	54	59	75	84	107	132	143	9.3
7	Millmerran Leyburn Road	50	60	73	86	93	112	138	146	33.1
8	Millmerran Leyburn Road	76	90	98	105	110	121	144	149	48.7

Developed Case flooding conditions

Table 9.41 Condamine River – Developed Case flood depths

Reporting location	Road	Estimated depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
4	Pampas-Horrane Road	0.0	0.14	0.17	0.20	0.30	0.95	2.03	3.29
5	Gore Highway	0.0	0.0	0.0	0.07	0.11	0.32	0.74	1.53
6	Gore Highway	0.01	0.10	0.15	0.24	0.39	1.06	1.95	3.27
7	Millmerran Leyburn Road	0.04	0.23	0.39	0.62	0.86	1.78	2.81	4.09
8	Millmerran Leyburn Road	0.03	0.04	0.06	0.10	0.17	0.67	1.62	3.09

Table 9.42 Condamine River – Developed Case flood inundation length

Reporting location	Road	Approximate length of inundation (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
4	Pampas-Horrane Road	0	98	745	918	2,240	4,170	13,800	17,100
5	Gore Highway	0	0	0	422	647	1,476	3,020	13,900
6	Gore Highway	3,320	5,400	7,030	7,390	7,600	9,605	9,910	
7	Millmerran Leyburn Road[1]	677	2,200	2,318	2,440	3,560	7,490	8,270	8,990
8	Millmerran Leyburn Road[1]	10	76	286	1,990	1,990	2,005	2,085	2,205

Table note:

1 Segment measured up until proposed rail, to allow for comparison with Existing Case

Table 9.43 Condamine River – Developed Case time of submergence

Reporting location	Road	Estimated time of submergence (hrs)								AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
4	Pampas-Horrane Road	0	82	100	102	107	114	135	138	13.8
5	Gore Highway	0	0	0	62	76	100	126	152	2.5
6	Gore Highway	0	54	60	75.	84	107	132	143	9.3
7	Millmerran Leyburn Road	61	64	77	89	96	112	138	146	38.6
8	Millmerran Leyburn Road	116	123	127	101	108	100	125	143	70.1

Impacts of Project alignment

Table 9.44 Condamine River – change in flood depths

Reporting location	Road	Estimated change in depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
4	Pampas-Horrane Road	0.00	0.00	0.00	0.00	-0.01	-0.03	-0.01	0.00
5	Gore Highway	0.00	0.00	0.00	0.00	0.00	0.02	0.07	0.09
6	Gore Highway	0.00	0.00	0.00	0.00	0.00	0.02	0.02	0.01
7	Millmerran Leyburn Road	0.00	0.02	0.04	0.06	0.08	0.18	0.24	0.11
8	Millmerran Leyburn Road	0.02	0.00	-0.19	-0.44	-0.61	-0.94	-0.96	-0.89

Table 9.45 Condamine River – change in time of submergence

Reporting location	Road	Estimated change in time of submergence (hrs)								Estimated change in AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
4	Pampas-Horrane Road	0	-7	-2	0	0	-1	0	0	-0.6
5	Gore Highway	0	0	0	13	12	6	4	1	0.4
6	Gore Highway	0	0	1	0	0	0	0	0	0.0
7	Millmerran Leyburn Road	11	4	4	3	3	0	0	0	5.5
8	Millmerran Leyburn Road[1]	40	33	29	-4	-2	-21	-19	-6	21.4

Table note:

1 There is a significant increase in AATOS for Millmerran Leyburn road downstream of the Project alignment. This increase is due to the increased conveyance area under the Project alignment in comparison to the existing rail alignment. The effects are noticeable in the smaller events as flow is able to pass through the Project alignment far easier than the existing rail alignment which acts as a weir in smaller events. Difference in larger events are negligible as the existing rail alignment is overtopped

Change in flood hydrographs

Figure 55 presents the Developed Case and Existing Case water level time series for the 1% AEP event at extraction point 5, located along Pampas-Horrane Road. While the difference in peak levels and initial hydrograph shape are negligible, it can be seen that the through the tail end of the event, there is a flatter gradient in the receding limb, and as such it is likely that if the event was simulated for a longer period, a greater difference in time of submergence may be present.

Figure 56 presents Developed Case and Existing Case levels for the 1% AEP at extraction point 6, located along Gore Highway. As shown by the result presented above, as well as the similar shape and magnitude of the hydrographs presented in Figure 56, the Project alignment has negligible impact on flood behaviours at this location.

Figure 57 presents Developed Case and Existing Case levels for the 1% AEP at extraction point 7, located along Millmerran-Leyburn Road. Results indicate marginal increases in inundation length and peak flood height, but in the scheme of the overall flood hydrograph these results are negligible.

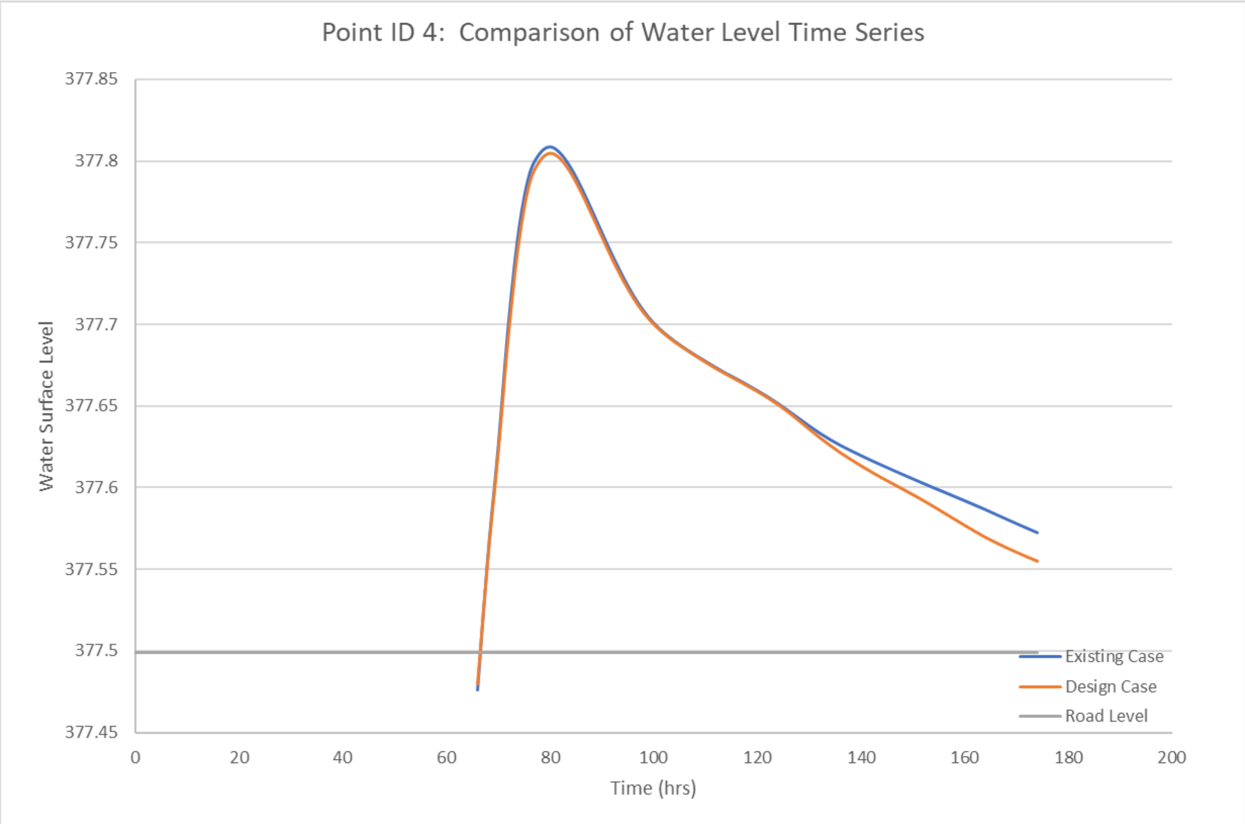


Figure 55 **Extraction Point 4 – comparison of water level time series, 1% AEP**

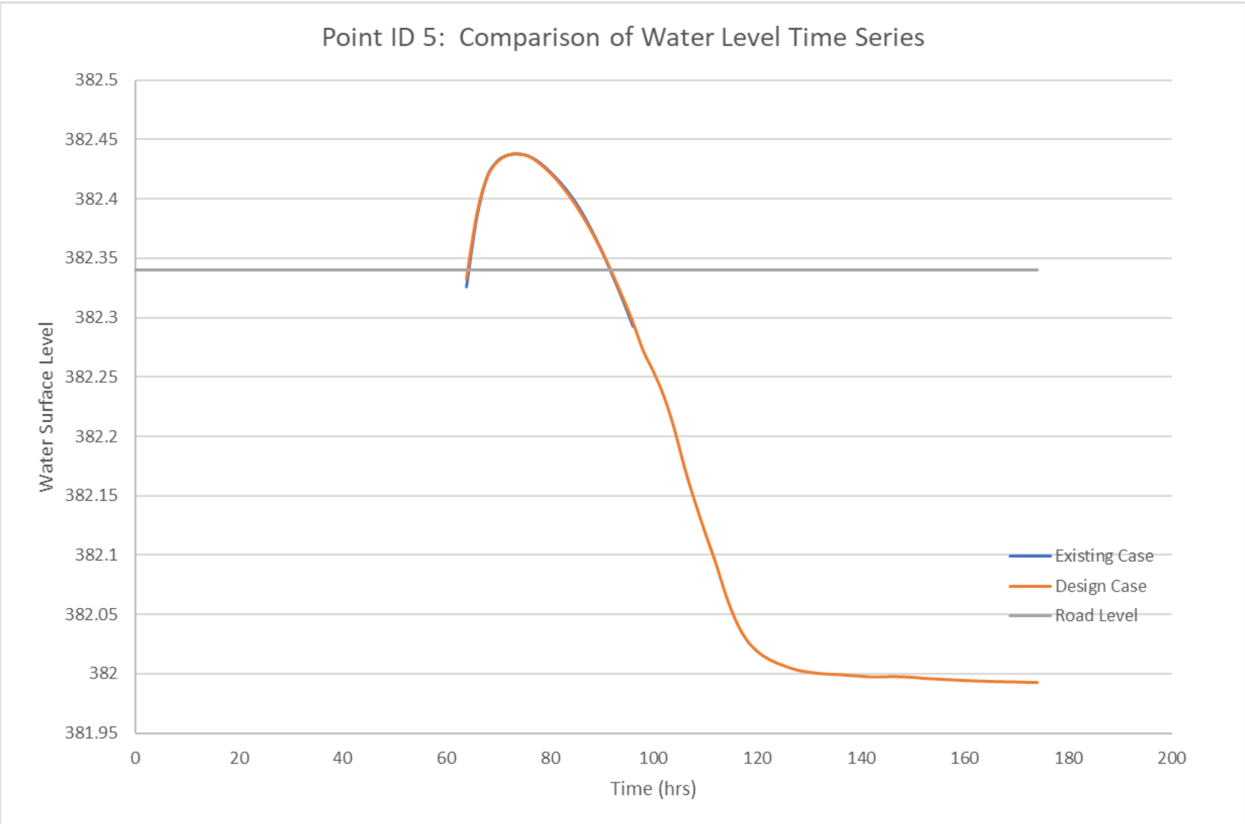


Figure 56 **Extraction Point 5 – comparison of water level time series, 1% AEP**

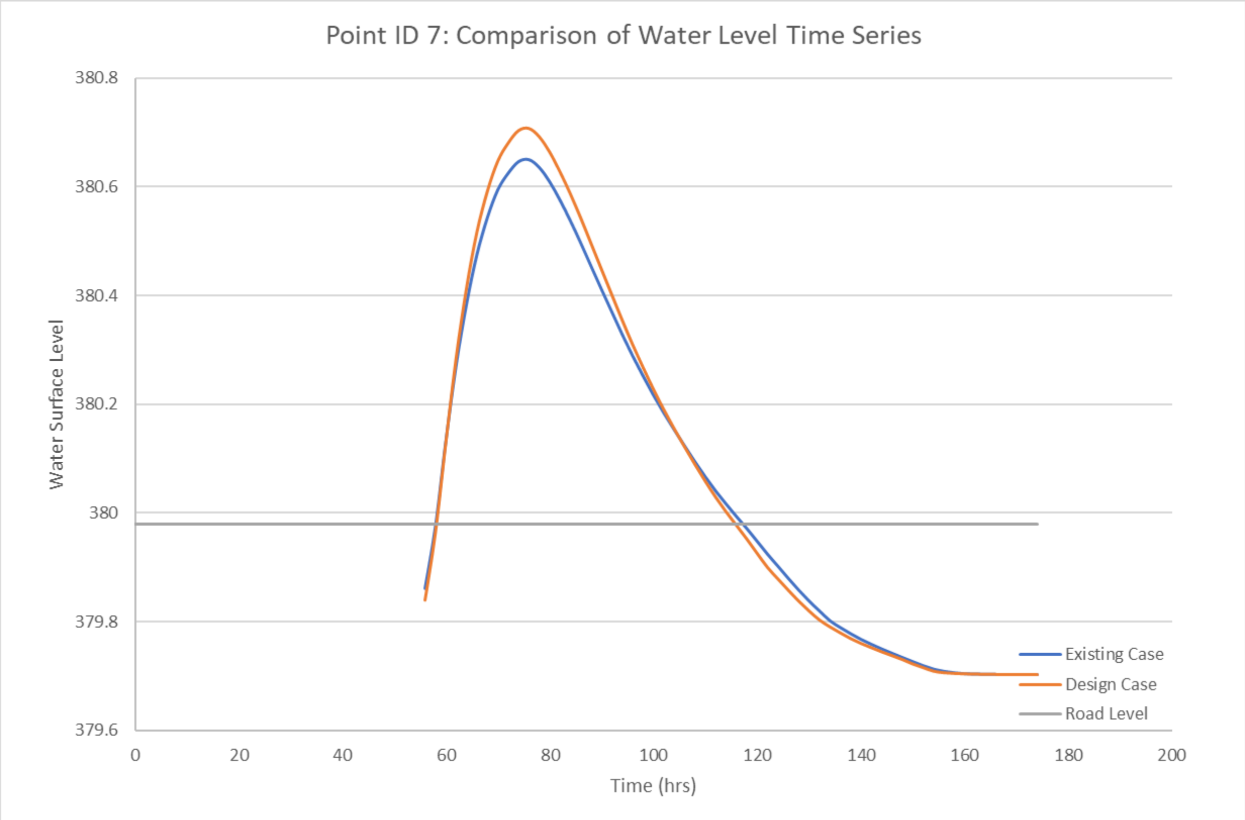


Figure 57 Extraction Point 7 – comparison of water level time series, 1% AEP

9.5.3.4 Flood impacts on local public roads

The change in peak water levels and flood hazard (velocity-depth) for the 1% AEP event were evaluated on local public roads within the hydraulic model domain. Local public roads that are expected to experience an increase in flood hazard and/or increases in peak flood levels are reported in Table 9.46.

Table 9.46 Condamine River- changes in peak water levels and flood hazard for local public roads, 1% AEP

Location	Existing flood hazard (m^2/s)	Design flood hazard (m^2/s)	Maximum existing flood depth (m)	Maximum design flood depth (m)	Maximum change in peak water levels (mm)[1]
Evanslea Road	4.13	4.13	0.84	0.84	-
Fysh Road	1.17	1.17	1.69	1.69	+188
Gilgai Lane	2.67	2.66	3.34	3.35	+119
Hall Road	2.98	2.73	3.49	3.50	+212
Holmes Road	2.04	2.04	2.60	2.60	-
Lemontree Road	1.45	1.45	2.04	2.05	+2
Pampas Pit Road	2.53	2.53	2.35	2.35	+37
Pump Road	0.98	0.98	2.19	2.19	+2

Table note:

1 The maximum change in peak water level does not necessarily occur at the same location as where the existing and/or design maximum flood depth occur

Duration of inundation

Assessment of the time of submergence (ToS) and average annual time of submergence (AAToS) was undertaken for local public roads within the hydraulic model domain. Local public roads that are expected to experience an increase in ToS and/or AAToS are presented in Table 9.47.

Table 9.47 Condamine River – ToS and AAToS for local public roads

Location	Existing 1% AEP ToS (hrs)	1% AEP ToS diff. (hrs)	2% AEP ToS diff. (hrs)	5% AEP ToS diff. (hrs)	10% AEP ToS diff. (hrs)	AAToS Existing Case (hrs)	AAToS Developed Case (hrs)	AAToS diff. (hrs)
Backhouse Road	149.45	0.01	-	-	-	86.19	86.19	-
Bailey Road	159.82	0.01	-	-	-	93.03	93.04	-
Bellevue Road	107.03	-	-	0.01	0.01	8.03	8.04	-
Blackgully Road	150.22	0.01	-	0.01	-	84.97	84.97	-
Bligh Road	113.74	-0.02	-0.01	-0.01	-	34.08	34.13	0.06
Bostock Road	163.86	-	0.01	-	-0.01	95.51	95.51	-
Brookstead – Norwin Road	163.33	-	0.01	-	-	93.64	93.64	-
Charles Street	114.31	0.01	-	-	-0.03	63.51	63.51	-
Clifton Leyburn Road	157.84	-	0.01	-	-	91.72	91.72	-
Clifton Road	160.41	-	0.01	-0.01	-	87.13	87.13	-
Dooley Road	165.97	-	0.01	-	-	96.92	96.92	-
Elsden Road	140.55	0.01	-	-	-0.01	82.46	82.46	-
Gibbs Road	124.07	-	-	-	0.08	31.08	31.07	-
Gilgai Lane	136.36	0.09	0.09	0.09	0.08	73.28	73.34	0.05
Grieves Road	127.15	-	-	0.01	-	54.64	54.64	-
Gurney Road	154.63	-	0.02	-0.01	-	86.18	86.18	-
Hanlon Road	158.8	-0.12	-0.09	-0.01	0.75	90.66	91.1	0.44
Hogarth Road	156.51	0.01	-	-	-	90.52	90.53	-
Hogarths Road	144.76	0.01	-	-	-	86.54	86.55	-
Keeley Road	141.06	0.01	-	-	-	79.61	79.61	-
Kelly Road	145.14	0.01	-	-	-	84.77	84.77	-
King Road	135.21	-	-	0.01	-	67.39	67.39	-
Kyle Road	166.32	-	0.01	-0.01	-	97.05	97.05	-
Ladner Road	123.18	0.01	-0.01	-	-	4.19	4.19	-
Lemontree Road	132.98	0.02	0.01	0.01	-	78.91	78.91	-
Lindenmayer Road1	163.84	-	0.01	-	-	95.41	95.42	-
Lindenmayer Road	143.67	-	-	-0.01	-0.2	89.65	89.63	-0.01
Macwilliam Road	153.42	0.01	0.01	-0.02	-0.01	80.01	80.01	-
Maddern Lane	153.36	0.01	-	-	-	86.88	86.88	-
Mann Silo Road	134.41	0.01	0.01	-	-	72.77	72.77	-
Margaret Street	143.12	-	0.01	-	-	82.67	82.67	-
Millmerran – Cecil Plains Road	124.13	0.01	0.01	-	0.01	65.22	65.22	-
Molloy Road	135.92	0.01	-	-	-	70.69	70.69	-
Murphy Road	134.48	0.01	-0.01	0.01	0.01	65.78	65.78	-

Location	Existing 1% AEP ToS (hrs)	1% AEP ToS diff. (hrs)	2% AEP ToS diff. (hrs)	5% AEP ToS diff. (hrs)	10% AEP ToS diff. (hrs)	AAToS Existing Case (hrs)	AAToS Developed Case (hrs)	AAToS diff. (hrs)
Owens Scrub Road	163.28	-	0.01	-	-	94.95	94.95	-
Pump Road	134.51	0.03	0.04	0.03	0.03	52.98	52.99	0.02
Saal Road	158.12	0.01	-	-	-	92.15	92.15	-
Tummaville Road	156.84	-	-	0.01	0.01	88.47	88.46	-0.01
Waco Lane	151.65	0.01	-	-	-	90.2	90.2	-
Wallace Road	93.11	-	-	-	0.01	37.6	37.61	-
Yandilla Pit Road	144.43	0.01	-	-	-	81.57	81.57	-
Yarramalong Road	160.47	-0.01	0.02	-	-	93.84	93.85	0.01

Table note:

1 Roads duplicated in the table above represent roads that are cut by floodwaters at multiple locations

9.5.3.5 Flood impacts on private land outside the rail disturbance footprint

Most of the area where afflux is expected is agricultural land or open land on which nominal afflux is unlikely to cause any adverse impact. Table 9.48 presents the modelled changes in flood conditions during the 1% AEP event on a lot basis according to the following thresholds:

- Peak water levels increased by greater than +10 mm
- Peak velocities increased by greater than 0.25 m/s
- Duration of inundation changed by more than 25% of its original duration of inundation across the lot.

Table 9.48 Condamine River– summary of flood impacts on private land outside the rail disturbance footprint for 1% AEP

Approximate chainage (km)	Changes in peak water levels[1]		Changes in peak velocities		Changes in duration of inundation	
	Maximum change (mm)	Total area affected by change > 10 mm (ha)	Maximum change (m/s)	Total area affected by change (ha)	Maximum change (%)	Total area affected by change (ha)
146.80 - 147.20	+37	0.1	-	-	-	-
146.80 - 147.20	+37	0.1	-	-	+21	<0.1
146.80 - 147.20	+34	0.1	-	-	-59	<0.1
146.80 - 147.20	+33	0.1	-	-	-55	<0.1
146.80 - 147.20	+32	0.1	-	-	-49	<0.1
146.80 - 147.20	+32	0.1	-	-	-36	<0.1
146.80 - 147.20	+31	0.1	-	-	-	-
146.80 - 147.20	+28	0.1	-	-	-	-
146.80 - 147.20	+28	0.1	-	-	-	-
146.80 - 147.20	+22	<0.1	-	-	-	-
146.80 - 147.20	+18	0.1	-	-	-	-
146.80 - 147.20	+13	0.1	-	-	-	-
146.80 - 147.20	+12	0.1	-	-	-	-
146.80 - 147.20	+11	0.1	-	-	-	-
146.80 - 147.20	+11	0.1	-	-	-	-
146.80 - 147.20	+11	0.1	-	-	+22	0.1

Approximate chainage (km)	Changes in peak water levels[1]		Changes in peak velocities		Changes in duration of inundation	
	Maximum change (mm)	Total area affected by change > 10 mm (ha)	Maximum change (m/s)	Total area affected by change (ha)	Maximum change (%)	Total area affected by change (ha)
146.80 - 147.20	+11	0.1	-	-	-	-
146.80 - 147.20	+17	0.1	-	-	-	-
146.80 - 147.20	+29	0.1	-	-	-28	<0.1
146.80 - 147.20	+31	0.1	-	-	-	-
146.80 - 147.20	+33	0.1	-	-	-	-
146.80 - 147.20	+19	0.1	-	-	-	-
146.80 - 147.20	+20	0.1	-	-	-	-
146.80 - 147.20	+21	0.1	-	-	-	-
146.80 - 147.20	+22	0.1	-	-	-	-
146.80 - 147.20	+23	0.1	-	-	-	-
146.80 - 147.20	+24	0.1	-	-	-	-
146.80 - 147.20	+24	0.1	-	-	-	-
146.80 - 147.20	+23	0.1	-	-	-	-
146.80 - 147.20	+21	0.1	-	-	-	-
146.80 - 147.20	+21	0.1	-	-	-	-
146.80 - 147.20	+20	0.1	-	-	-	-
146.80 - 147.20	+20	0.1	-	-	-	-
146.80 - 147.20	+20	0.1	-	-	-	-
146.80 - 147.20	+19	0.1	-	-	-	-
146.80 - 147.20	+19	0.1	-	-	-	-
146.80 - 147.20	+20	<0.1	-	-	-	-
146.80 - 147.20	+24	0.1	-	-	-	-
146.80 - 147.20	+29	0.1	-	-	-	-
146.80 - 147.20	+28	0.1	-	-	-	-
146.80 - 147.20	+28	0.1	-	-	-	-
146.80 - 147.20	+28	0.1	-	-	-	-
146.80 - 147.20	+28	0.1	-	-	-	-
146.80 - 147.20	+24	<0.1	-	-	-	-
146.80 - 147.20	+37	<0.1	-	-	-	-
147.20	+37	0.1	-	-	-	-
146.80 - 147.20	+18	0.1	-	-	-	-
146.80 - 147.20	+19	0.1	-	-	-	-
146.80 - 147.20	+19	0.1	-	-	-	-
146.80 - 147.20	+20	0.1	-	-	-	-
146.80 - 147.20	+20	0.1	-	-	-	-
146.80 - 147.20	+21	0.1	-	-	-	-
146.80 - 147.20	+21	0.1	-	-	-	-
146.80 - 147.20	+21	0.1	-	-	-	-
146.80 - 147.20	+22	0.1	-	-	-	-

Approximate chainage (km)	Changes in peak water levels[1]		Changes in peak velocities		Changes in duration of inundation	
	Maximum change (mm)	Total area affected by change > 10 mm (ha)	Maximum change (m/s)	Total area affected by change (ha)	Maximum change (%)	Total area affected by change (ha)
146.80 - 147.20	+23	0.1	-	-	-	-
146.80 - 147.20	+23	0.1	-	-	-	-
146.80 - 147.20	+32	0.1	-	-	-	-
146.80 - 147.20	+28	0.1	-	-	-	-
146.80 - 147.20	+27	0.1	-	-	-	-
146.80 - 147.20	+26	0.1	-	-	-	-
131.10 - 133.40	+900	1.5	-	-	-	-
146.80 - 147.20	+26	0.1	-	-	-	-
146.80 - 147.20	+31	0.1	-	-	-	-
146.80 - 147.20	+31	0.1	-	-	-	-
138.40 - 138.50	+19	1.0	-	-	-	-
142.70	-	-	+0.36	0.1	-	-
142.70	+23	0.5	-	-	-	-
137.70 - 138.30	+42	171.0	+0.29	<0.1	-	-
137.70 - 138.30	+111	28.0	-	-	-	-
135.40 - 135.90	+49	8.9	-	-	-	-
142.90 - 144.40	+48	69.4	+0.34	0.1	-	-
138.40 - 138.60	+54	10.3	-	-	-	-
138.50 - 140.00	+38	148.9	-	-	-	-
138.70 - 139.30	+33	15.7	+0.32	0.1	-	-
139.80 - 140.10	+160	0.8	-	-	-	-
140.30	+76	0.5	-	-	-	-
138.55 - 140.10	+117	279.0	-	-	-	-
147.30 - 147.80	+47	33.4	-	-	-	-
148.60	+90	0.1	-	-	-	-
147.60 - 148.20	-	-	+0.59	<0.1	-	-
146.10	+22	30.2	-	-	-	-
146.20	+15	0.6	-	-	-	-
146.50	+27	<0.1	-	-	-	-
145.90	+42	0.2	-	-	-	-
145.30	+48	0.7	-	-	-	-
144.90 - 145.20	+33	4.5	-	-	-	-
144.5 - 144.80	+61	5.8	-	-	-	-
146.90	+37	0.5	-	-	-	-
141.30 - 141.50	+25	45.0	-	-	-	-
140.10 - 141.30	+128	227.0	-	-	-	-
149.15 - 150.30	+139	12.0	+0.64	<0.1	-	-
146.70 - 147.10	+83	<0.1	-	-	-	-
143.50 - 144.60	+58	9.6	-	-	-	-

Approximate chainage (km)	Changes in peak water levels[1]		Changes in peak velocities		Changes in duration of inundation	
	Maximum change (mm)	Total area affected by change > 10 mm (ha)	Maximum change (m/s)	Total area affected by change (ha)	Maximum change (%)	Total area affected by change (ha)
145.20 - 146.10	+295	183.0	-	-	-	-
146.15 - 146.80	+138	20.0	-	-	-	-
142.90	+26	0.1	-	-	-	-
142.90 - 144.40	+25	152.4	+0.32	<0.1	-	-
143.10 - 144.50	+58	37.2	-	-	-	-
141.30	+74	10.1	-	-	-	-
141.30 - 142.90	+81	135.9	-	-	-	-
147.80	+28	10.9	-	-	-	-
147.40	+38	1.8	-	-	-	-
146.80 - 147.20	+24	0.1	-	-	-	-
146.80 - 147.20	+21	0.1	-	-	-	-
146.80 - 147.20	+21	0.1	-	-	-	-
146.80 - 147.20	+21	0.1	-	-	-	-
146.80 - 147.20	+21	0.1	-	-	-	-
146.80 - 147.20	+20	0.1	-	-	-	-
146.80 - 147.20	+20	0.1	-	-	-	-
146.80 - 147.20	+19	0.1	-	-	-	-
146.80 - 147.20	+18	0.1	-	-	-	-
146.80 - 147.20	+18	0.1	-	-	-	-
146.80 - 147.20	+17	0.1	-	-	-	-
146.80 - 147.20	+17	0.1	-	-	-	-
146.80 - 147.20	+18	0.1	-	-	-	-

Table notes:

1 Afflux on lots that exceed the flood impact objectives are summarised in the EIS Surface Water Chapter
2 Change in peak water levels at this location is localised and directly adjacent to the Project alignment

9.5.3.6 Flow distribution

A key landowner concern is changes to flow distribution. To understand the magnitude of these flowpaths, flows were extracted from the hydraulic model at key locations. The difference between the Existing Case and Developed Case was considered and reported in Table 9.49. The results indicate minor changes in peak flows in some locations directly upstream of the proposed rail embankment; however, the changes dissipate downstream of the proposed rail embankment.

Figure 58 presents the selected flowpath comparison locations. The flow is calculated across the length of the line. Therefore, the lines presented are either calculating the flow across the width of the floodplain (for the longer flow lines) or the main flowpath of the waterways (generally for smaller flow lines).

Table 9.49 Condamine River – flow comparison

Flow location ID	10% AEP			1% AEP		
	Existing Case flow (m³/s)	Developed Case flow (m³/s)	% Change	Existing Case flow (m³/s)	Developed Case flow (m³/s)	% Change
1	1,516	1,541	2%	3,739	3,703	-1%
2	317	311	-2%	1,021	1,014	-1%
3	1,262	1,236	-2%	2,724	2,714	-
4	434	438	1%	1,193	1,169	-2%
5	116	127	8%	247	276	10%
6	882	872	-1%	1,512	1,474	-3%
7	21	21	1%	140	150	6%
8	1,475	1,470	-	3,672	3,661	-
9	62	63	1%	200	192	-4%

9.5.4 Sensitivity analysis – Condamine River

The following sensitivity analyses were completed:

- Variability of crop distributions by type, maturity and planting orientation
- Differing culvert blockage factors (0% and 50%)
- Change in rainfall associated with climate change
- Back Creek local inflow.

9.5.4.1 Spatial variability in crops

The local community have raised concerns that historical floods have differed significantly from one another and that one of the factors that drives this variation is the different types of crops that are being cultivated at the time of flood. Furthermore, several community members have stated that the height of each crop (or its maturity) and its planting orientation has had visibly obvious impacts on flow direction and velocities at different times of a flood.

Research was undertaken into the types of crops grown in the area and their cycles – including times of fallow – to understand what range of ground conditions could be expected during the times of year when floods were likely. The spatial variation in crop type and maturity was then captured in the model by spatially varying roughness values, based on probability distributions produced from the crop cycle research. The distributions made it possible to stochastically generate crop patterns across the floodplain that were statistical representations of crop patterns likely to occur at the time of a flood. In total, ten different crop patterns were generated, and their potential hydraulic impacts were quantified in the TUFLOW hydraulic model using hydrographs from the 2,880-minute 1% AEP event.

In addition to the ten different crop patterns generated through probability distributions, a number of seasonal crop patterns were developed based on publications from the Department of Agriculture and Water Resources which describe the percentage breakdown of crops from various regions, providing both historical data and future forecasts. Using this information, roughness layers were developed for the Condamine River floodplain, so that the quantities of crops (i.e. 60% wheat) were represented within the model for various seasons. Consequently, four files were developed which represented conditions for both summer and winter, and seedlings and mature crops.

The two behaviours documented from this sensitivity scenario were the variation in flood extents, which was quantified as a likelihood of inundation, and the variation in flood depths across the study area. The variation in flood extents remains largely unchanged across the different crop pattern scenarios. While there are certain areas where extents widen, these areas are insignificant and unlikely to occur regularly. The variation in flood depths across the floodplain is quite pronounced in localised areas but it does not appear to significantly influence the overall flood extents. This is, in part, due to the size of the floodplain and the duration of the flood event, whereby any local fluctuations at certain points in time tend to be evened out by fluctuations at other times. And subsequently, this behaviour gets evened out when the other crop pattern scenarios are considered.

The variation in flood extents and depths is documented in maps Volume II - Appendix C, Figures C-6d and C-6e.

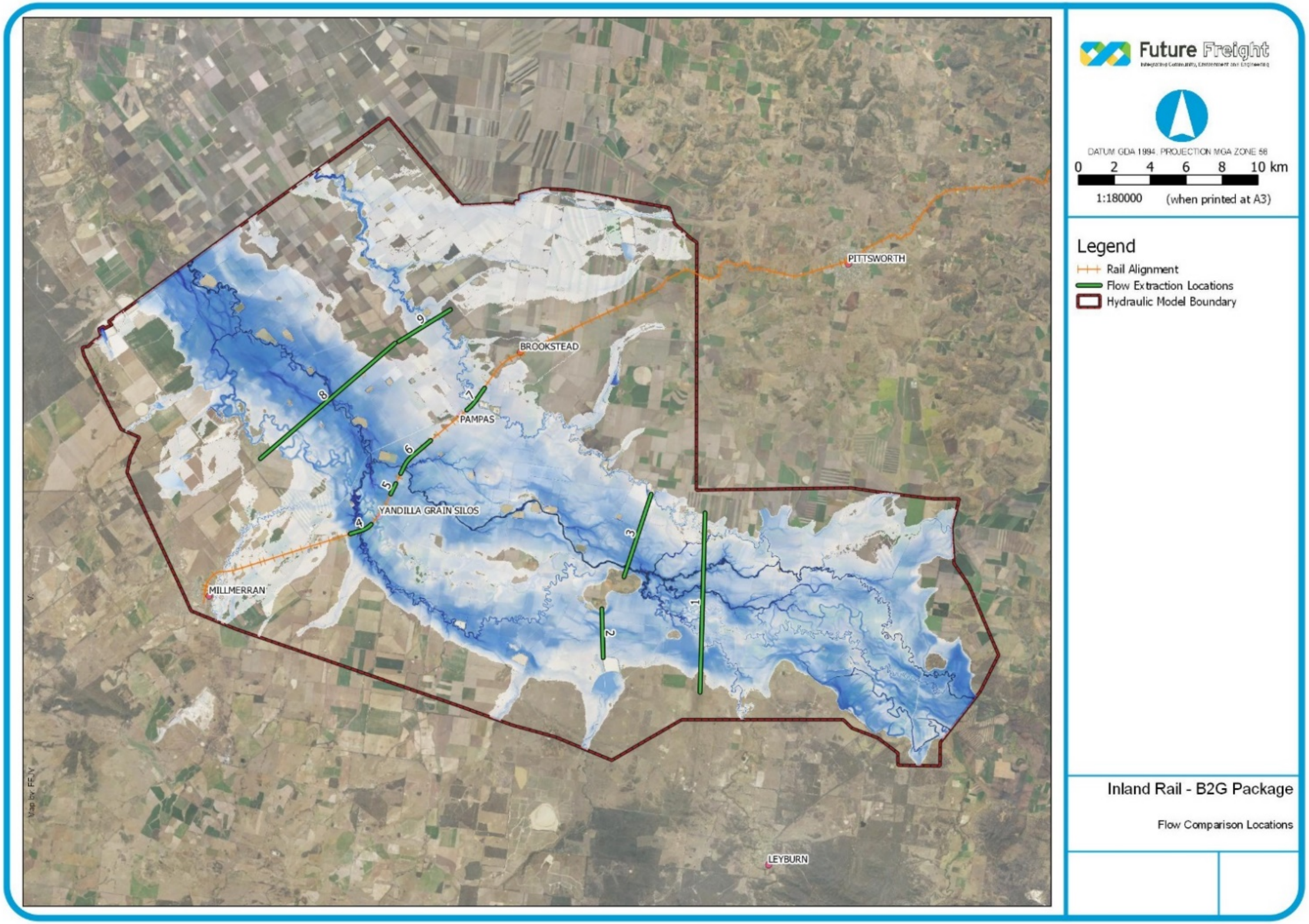


Figure 58 **Condamine River – flow comparison locations**

9.5.4.2 Culvert blockage

Blockage was assessed in accordance with ARR 2016. The blockage assessment undertaken resulted in a blockage factor of 25% being adopted for culverts. A minimum culvert size of 900 mm diameter was adopted to reduce potential for blockage and maintenance. A significant 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% and 50% blockage.

Information has been requested from the DTMR and QR with regards to maintenance records, and historical incidences of debris accumulation, culvert blockage or scour at their respective assets, namely the Gore Highway and the QR rail line through the Condamine River floodplain. The photographic evidence provided by QR is shown in Figure 59. DTMR South West Region do not have any records of debris management at the Gore Highway, but the department provided information on historical road closures due to flooding, including December 2010, January 2011, January 2013, February 2013, March 2014 and March 2017.

Figure 59 Evidence of historical debris deposition and scour provided by Queensland Rail (actual flood event unknown)

Table 9.50 provides a summary of 1% AEP peak flood levels at culverts for the blockage scenarios.

Table 9.50 Condamine River – 1% AEP event – culvert blockage assessment

Structure ID	Structure type	1 % AEP Peak water levels (m AHD)			Increase from Developed Case to 50% blockage scenario (mm)
		0% blockage	Developed Case (25% blockage)	50% blockage	
C131.39	RCP	401.74	401.79	401.88	+90
C131.49	RCP	401.74	401.79	401.88	+90
C137.83	RCP	380.64	380.65	380.66	+10
C137.88	RCP	380.61	380.62	380.64	+10
C137.92	RCP	380.63	380.64	380.65	+10
C139.37	RCP	380.62	380.63	380.64	+10
C139.44	RCP	380.63	380.64	380.66	+20
C139.5	RCP	380.63	380.64	380.65	+20
C139.56	RCP	380.62	380.63	380.65	+20
C139.71	RCP	380.63	380.65	380.67	+20
C139.73	RCBC	380.63	380.65	380.67	+30
C139.78	RCP	380.61	380.63	380.66	+30
C140.09	RCP	380.60	380.62	380.65	+30
C140.11	RCP	380.60	380.62	380.65	+30
C140.17	RCP	380.56	380.57	380.59	+30
C140.21	RCP	380.55	380.57	380.59	+30
C140.23	RCP	380.55	380.56	380.59	+30
C140.25	RCP	380.55	380.56	380.59	+30
C140.27	RCP	380.55	380.56	380.59	+30
C140.32	RCP	380.55	380.56	380.59	+30
C140.38	RCP	380.55	380.56	380.59	+30
C140.4	RCP	380.55	380.56	380.59	+30
C140.43	RCP	380.55	380.56	380.59	+30
C140.46	RCP	380.54	380.55	380.58	+30
C140.49	RCP	380.53	380.54	380.58	+30
C140.51	RCP	380.53	380.54	380.58	+30
C140.55	RCP	380.53	380.55	380.58	+30
C140.59	RCP	380.53	380.55	380.58	+40
C140.64	RCP	380.53	380.54	380.58	+40
C140.67	RCP	380.53	380.55	380.58	+40
C140.78	RCP	380.50	380.52	380.57	+50
C140.83	RCP	380.49	380.51	380.56	+50
C140.87	RCP	380.49	380.51	380.56	+50
C140.91	RCP	380.49	380.51	380.55	+50
C140.98	RCP	380.47	380.49	380.54	+50
C141.03	RCP	380.48	380.50	380.54	+50
C141.07	RCP	380.47	380.49	380.53	+50
C141.11	RCP	380.46	380.48	380.53	+50
C141.2	RCP	380.45	380.47	380.50	+40

Structure ID	Structure type	1 % AEP Peak water levels (m AHD)			Increase from Developed Case to 50% blockage scenario (mm)
		0% blockage	Developed Case (25% blockage)	50% blockage	
C141.24	RCP	380.44	380.46	380.49	+40
C141.29	RCP	380.43	380.45	380.48	+30
C141.32	RCP	380.42	380.43	380.46	+30
C142.02	RCP	380.38	380.39	380.40	+10
C142.04	RCP	380.39	380.40	380.42	+20
C142.08	RCP	380.39	380.39	380.42	+20
C142.13	RCP	380.38	380.39	380.42	+20
C142.15	RCP	380.38	380.39	380.41	+20
C142.19	RCP	380.38	380.39	380.41	+20
C142.22	RCP	380.38	380.39	380.41	+20
C142.25	RCP	380.39	380.40	380.42	+20
C142.28	RCP	380.39	380.41	380.43	+20
C142.36	RCP	380.39	380.40	380.42	+20
C142.41	RCP	380.40	380.41	380.43	+20
C142.44	RCP	380.40	380.41	380.43	+20
C142.48	RCP	380.40	380.41	380.43	+20
C142.5	RCP	380.41	380.42	380.43	+20
C142.54	RCP	380.42	380.43	380.44	+10
C142.58	RCP	380.41	380.41	380.42	+10
C145.16	RCBC	380.85	380.87	380.90	+30
C145.21	RCBC	380.96	380.98	381.01	+30
C145.25	RCBC	381.05	381.07	381.10	+20
C145.32	RCBC	381.09	381.10	381.12	+20
C145.4	RCBC	381.13	381.14	381.16	+20
C145.72	RCBC	381.28	381.32	381.36	+50
C145.83	RCBC	381.36	381.39	381.43	+40
C145.89	RCBC	381.33	381.38	381.43	+60
C145.92	RCBC	381.35	381.39	381.44	+50
C145.98	RCBC	381.37	381.41	381.46	+50
C146.03	RCBC	381.41	381.45	381.49	+50
C146.56	RCBC	381.79	381.80	381.80	-
C146.62	RCBC	381.81	381.82	381.82	+10
C147.58	RCP	382.47	382.47	382.46	-10
C147.63	RCP	382.50	382.51	382.52	+10
C147.66	RCP	382.45	382.45	382.44	-10
C147.73	RCP	382.40	382.40	382.39	-10
C149.39	RCP	382.65	382.65	382.65	-
C149.42	RCP	382.66	382.66	382.66	-
C149.45	RCP	382.67	382.67	382.67	-
C149.76	RCP	382.71	382.71	382.71	-
C149.8	RCP	382.71	382.71	382.71	-

Structure ID	Structure type	1 % AEP Peak water levels (m AHD)			Increase from Developed Case to 50% blockage scenario (mm)
		0% blockage	Developed Case (25% blockage)	50% blockage	
C149.83	RCP	382.71	382.71	382.71	-
C149.87	RCP	382.71	382.71	382.71	-
C149.91	RCP	382.71	382.71	382.71	-
C149.96	RCP	382.70	382.70	382.70	-
C150.01	RCP	382.70	382.70	382.70	-

Table 9.51 outlines the changes in peak water levels at flood sensitive receptors for the 50% blockage scenario where the increase exceeds 10 mm.

Table 9.51 Condamine River – summary of 50% blockage impacts at flood sensitive receptors

Flood sensitive receptor ID	Existing case flood depth (m)	Change in peak water level (mm)
CON_ID_4	0.74	+126
CON_ID_5	0.69	+178
CON_ID_6	0.72	+187
CON_ID_7	0.67	+183
CON_ID_8	0.92	+130
CON_ID_9	0.87	+127
CON_ID_10	0.99	+117
CON_ID_68	1.06	+41
CON_ID_78	0.95	+45
CON_ID_99	0.48	+20
CON_ID_100	0.65	+30
CON_ID_101	0.91	+31
CON_ID_102	0.95	+31
CON_ID_103	0.55	+29
CON_ID_104	0.57	+21
CON_ID_118	0.73	+180
CON_ID_119	0.98	+45
CON_ID_120	0.96	+50
CON_ID_146	0.29	+27
CON_ID_147	0.35	+27
CON_ID_148	0.35	+28
CON_ID_149	0.11	+23
CON_ID_158	0.01	+44
CON_ID_203	0.87	+21
CON_ID_204	1.17	+21
CON_ID_205	1.22	+21
CON_ID_206	0.97	+21
CON_ID_207	1.26	+21
CON_ID_229	0.27	+11
CON_ID_244	0.22	+14
CON_ID_246	0.35	+15

Flood sensitive receptor ID	Existing case flood depth (m)	Change in peak water level (mm)
CON_ID_247	0.46	+27
CON_ID_248	0.27	+23
CON_ID_275	0.03	+36
CON_ID_277	0.24	+41
Fysh Road	1.85	+91
Gilgai Lane	2.96	+75
Gore Highway	2.04	+26
Hall Road	4.00	+76
Lovell Road	0.52	+780
Millmerran - Leyburn Road	3.23	+102
Pampas Pit Road	1.30	+42

Maps demonstrating the effects of blockage are shown in Figures C-5a (0%) and C-5b (50%) in Volume II – Appendix C.

During detail 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.

9.5.4.3 Impacts during extreme events

Table 9.52 outlines the changes in peak water levels at flood sensitive receptors for the extreme events where the increase exceeds 10 mm under one of the events. The existing depth of flooding is also detailed and as can be seen the larger impacts that occur under the PMF event occur generally when there are already high flood depths as would be expected under such a rare event.

Flood immunity of the Project alignment is discussed in Section 9.5.2.3, and maps demonstrating the impacts during extreme events are shown in Volume II – Appendix C, Figures C-5f to C-5h.

Table 9.52 Condamine River – summary of extreme event impacts at flood sensitive receptors

Flood sensitive receptor ID	1 in 2,000 AEP event		1 in 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)
CON_ID_4	+224	1.09	+282	2.06	+91	3.46
CON_ID_5	+232	1.05	+304	2.02	+101	3.43
CON_ID_6	+232	1.08	+304	2.05	+98	3.47
CON_ID_7	+235	1.03	+304	2.0	+101	3.41
CON_ID_8	+225	1.28	+282	2.25	+91	3.66
CON_ID_9	+223	1.22	+280	2.19	+91	3.60
CON_ID_10	+221	1.34	+279	2.30	+92	3.71
CON_ID_68	+99	1.41	+168	2.38	+153	3.79
CON_ID_78	+110	1.31	+186	2.27	+169	3.69
CON_ID_79	+10	1.23	+19	2.09	+10	3.40
CON_ID_80	+12	1.16	+22	2.01	+10	3.32
CON_ID_81	+14	0.89	+23	1.72	+10	3.02
CON_ID_82	+12	1.37	+21	2.22	+10	3.53
CON_ID_96	+65	0.49	+121	1.46	+82	2.91

Flood sensitive receptor ID	1 in 2,000 AEP event		1 in 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)
CON_ID_97	+62	0.76	+120	1.72	+83	3.17
CON_ID_98	+62	0.62	+120	1.58	+82	3.03
CON_ID_99	+81	0.86	+133	1.86	+86	3.31
CON_ID_100	+81	1.03	+132	2.03	+86	3.48
CON_ID_101	+81	1.29	+132	2.29	+86	3.75
CON_ID_102	+81	1.33	+133	2.33	+87	3.79
CON_ID_103	+81	0.93	+133	1.93	+86	3.39
CON_ID_104	+82	0.95	+133	1.95	+86	3.41
CON_ID_118	+235	1.09	+314	2.06	+93	3.47
CON_ID_119	+106	1.34	+161	2.33	+92	3.77
CON_ID_120	+109	1.33	+164	2.32	+92	3.76
CON_ID_146	+133	0.48	+157	1.03	+73	1.87
CON_ID_147	+134	0.54	+152	1.08	+66	1.92
CON_ID_148	+131	0.54	+149	1.08	+63	1.92
CON_ID_149	+134	0.30	+184	0.86	+111	1.70
CON_ID_153	+12	1.15	+18	2.03	+10	3.35
CON_ID_154	-100	0.93	-67	1.77	+11	3.07
CON_ID_155	-108	0.97	-72	1.82	+11	3.11
CON_ID_156	-108	1.03	-73	1.87	+11	3.17
CON_ID_157	+70	0.08	+190	0.46	+107	1.27
CON_ID_158	+82	0.04	+187	0.42	+112	1.25
CON_ID_159	+2	1.92	+24	2.30	+50	3.21
CON_ID_160	+24	0.46	+84	0.89	+75	1.72
CON_ID_161	+25	0.63	+86	1.06	+77	1.89
CON_ID_162	+25	0.62	+85	1.05	+77	1.88
CON_ID_163	-	0.08	+20	0.28	+25	0.83
CON_ID_164	-	0.09	+19	0.28	+24	0.83
CON_ID_165	+1	0.09	+19	0.23	+24	0.80
CON_ID_166	-	0.09	+4	0.94	+10	2.12
CON_ID_167	-	0.03	+3	0.70	+10	1.88
CON_ID_168	-	0.01	+3	0.76	+10	1.94
CON_ID_169	-	0.09	+3	0.81	+10	1.98
CON_ID_170	-	0.12	+3	0.81	+10	1.99
CON_ID_171	-	0.25	+3	1.00	+10	2.18
CON_ID_172	-	0.30	+3	1.04	+10	2.21
CON_ID_173	-	0.28	+4	1.04	+10	2.23
CON_ID_186	-155	0.79	-86	1.64	+12	2.93
CON_ID_187	-169	0.81	-92	1.67	+11	2.96
CON_ID_188	-168	0.76	-91	1.61	+12	2.90
CON_ID_189	-163	0.83	-89	1.68	+12	2.97

Flood sensitive receptor ID	1 in 2,000 AEP event		1 in 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)
CON_ID_190	-165	0.78	-88	1.63	+12	2.91
CON_ID_191	-158	0.64	-86	1.48	+12	2.77
CON_ID_192	-155	0.67	-85	1.52	+12	2.81
CON_ID_193	+1	1.21	+10	1.82	+17	2.85
CON_ID_194	+1	1.32	+10	1.92	+17	2.95
CON_ID_195	+1	1.37	+9	1.97	+16	3.00
CON_ID_196	+1	1.28	+10	1.88	+17	2.89
CON_ID_197	+1	1.20	+9	1.80	+17	2.82
CON_ID_198	-	-	+12	0.13	+10	1.21
CON_ID_199	-	-	+13	0.29	+10	1.36
CON_ID_200	-	-	+13	0.32	+10	1.31
CON_ID_201	-	-	+14	0.38	+10	1.34
CON_ID_202	-	-	+14	0.20	+9	1.17
CON_ID_203	+71	1.24	+125	2.22	+70	3.65
CON_ID_204	+72	1.54	+126	2.52	+70	3.95
CON_ID_205	+72	1.59	+126	2.57	+70	4.00
CON_ID_206	+72	1.34	+127	2.32	+70	3.75
CON_ID_207	+72	1.63	+127	2.60	+70	4.03
CON_ID_208	+24	0.49	+72	1.45	+54	2.91
CON_ID_209	+27	0.53	+73	1.53	+54	3.00
CON_ID_210	+23	0.46	+71	1.43	+53	2.89
CON_ID_211	+12	0.33	+82	0.81	+152	1.68
CON_ID_212	+9	0.28	+74	0.67	+142	1.39
CON_ID_213	+10	0.51	+78	0.98	+147	1.85
CON_ID_214	+10	0.62	+76	1.09	+146	1.96
CON_ID_215	+20	1.14	+63	2.13	+46	3.58
CON_ID_216	+20	1.43	+64	2.41	+46	3.86
CON_ID_217	+20	1.21	+64	2.18	+46	3.63
CON_ID_218	+20	1.06	+63	2.04	+46	3.48
CON_ID_219	+20	1.10	+63	2.08	+46	3.52
CON_ID_220	+20	0.91	+62	1.90	+45	3.34
CON_ID_221	+20	0.91	+62	1.90	+45	3.35
CON_ID_222	+20	0.87	+61	1.86	+44	3.32
CON_ID_223	+20	1.00	+62	1.99	+45	3.44
CON_ID_224	+20	0.83	+61	1.84	+44	3.29
CON_ID_225	+20	1.07	+62	2.07	+45	3.52
CON_ID_231	-12	0.11	+34	0.31	+27	1.26
CON_ID_232	-12	0.09	+33	0.35	+29	1.27
CON_ID_233	-12	0.16	+31	0.42	+30	1.33
CON_ID_234	-10	0.11	+36	0.37	+22	1.36

Flood sensitive receptor ID	1 in 2,000 AEP event		1 in 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)
CON_ID_235	-12	0.23	+32	0.50	+31	1.42
CON_ID_236	-12	0.05	+28	0.30	+28	1.19
CON_ID_237	-	0.71	+4	1.47	+10	2.65
CON_ID_238	-	0.75	+4	1.50	+10	2.69
CON_ID_239	-	0.73	+4	1.48	+10	2.66
CON_ID_242	-	0.91	+4	1.67	+10	2.86
CON_ID_243	-	0.76	+4	1.52	+10	2.71
CON_ID_244	+205	0.40	+202	0.95	+70	1.83
CON_ID_245	+202	0.28	+189	0.84	+68	1.69
CON_ID_246	+167	0.54	+175	1.10	+71	1.97
CON_ID_247	+147	0.65	+167	1.20	+72	2.07
CON_ID_248	+145	0.46	+163	1.00	+70	1.85
CON_ID_256	-	0.85	+4	1.37	+12	2.30
CON_ID_257	-	1.17	+4	1.66	+12	2.56
CON_ID_258	-	0.82	+3	1.31	+12	2.20
CON_ID_259	-	0.90	+3	1.38	+11	2.26
CON_ID_260	-	0.96	+3	1.43	+11	2.30
CON_ID_261	-	0.87	+3	1.34	+11	2.21
CON_ID_268	-6	0.16	+31	0.45	+39	1.21
CON_ID_269	-6	0.36	+31	0.65	+39	1.41
CON_ID_270	-7	0.50	+31	0.80	+41	1.57
CON_ID_271	-15	0.23	+79	0.52	+75	1.47
CON_ID_272	-36	0.05	+36	0.39	+65	1.31
CON_ID_273	-18	0.16	+39	0.50	+70	1.44
CON_ID_274	-22	0.49	+20	0.86	+60	1.76
CON_ID_275	+61	0.06	+170	0.41	+110	1.26
CON_ID_276	+60	0.02	+161	0.27	+107	1.12
CON_ID_277	+77	0.30	+193	0.63	+118	1.47
CON_ID_278	-7	1.06	+48	1.38	+84	2.12
CON_ID_280	-14	0.13	+47	0.44	+54	1.45
CON_ID_281	-13	0.09	+77	0.45	+69	1.44
CON_ID_282	-16	0.04	+54	0.37	+71	1.38
CON_ID_283	-14	0.10	+48	0.44	+70	1.43
CON_ID_284	-13	0.12	+46	0.48	+73	1.48
CON_ID_300	-	-	-	-	+32	0.76
CON_ID_301	-	-	-	-	+33	0.17
CON_ID_302	+2	0.25	+15	0.91	+26	2.15
CON_ID_303	+2	0.41	+16	1.10	+27	2.35
Bellevue Road	+14	1.68	+44	2.53	+45	3.90
Brose Lane	+13	2.13	+19	3.11	+10	4.46

Flood sensitive receptor ID	1 in 2,000 AEP event		1 in 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)
Crank Road	-	1.66	+3	2.39	+10	3.57
Elsden Road	+145	2.63	+396	3.01	+587	3.79
Fysh Road	+112	2.09	+189	2.80	+113	3.90
Gibbs Road	+52	1.14	+169	1.59	+243	2.40
Gilgai Lane	+155	3.24	+239	4.03	+134	5.29
Gore Highway (Toowoomba-Millmerran)	+89	2.36	+313	3.25	+122	4.55
Grasstree Reserve Road	+2	2.37	+14	3.08	+24	4.32
Hall Road	+175	4.35	+234	5.30	+155	6.70
King Road	+1	2.43	1+0	3.14	+18	4.38
Lovell Road	+1,080	0.58	+1,209	0.67	+1,429	0.77
Mann Silo Road	-	0.88	+4	1.05	+98	1.34
Millmerran - Leyburn Road	+177	3.59	+268	4.42	+90	5.61
Missen Road	+1	1.38	+23	2.17	+33	3.37
Pampas - Horrane Road	-	1.80	+1	2.80	+14	3.99
Pampas Pit Road	+79	1.40	+204	1.78	+127	2.56
Pampas Road	+3	0.89	+124	1.97	+87	3.20
Reichle Road	+11	4.83	+49	5.59	+41	6.90
Yarramalong Road	-	6.20	+7	6.81	+57	7.76

9.5.4.4 Climate change

The potential impacts of climate change in the Condamine River floodplain were assessed for the 1% AEP design event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2016.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario has been adopted for the Project with an associated increase in rainfall intensity of 20.8% across the catchment area.

For the 1% AEP event, the change in peak water levels for the Representative Concentration Pathways 8.5 climate change scenario is presented in Volume II – Appendix C, Figure C-6c. The change in peak water levels is calculated from the difference between the Developed Case and the Existing Case with 20.8% increase to rainfall intensity applied to both cases.

The hydraulic model predicts that, with an increase in rainfall intensity of 20.8% across the catchment, peak water levels are likely to increase by up to 0.5 m directly upstream of Bridges 310-BR21 and 310-BR22. The Project alignment is expected to retain 1% AEP flood immunity to formation level under the climate change scenario.

Table 9.53 presents the structure performance under Representative Concentration Pathways 8.5 climate change conditions.

Table 9.53 Condamine River – 1% AEP event with Representative Concentration Pathways 8.5 conditions – structure performance

Structure ID	Structure type	U/S peak water level (m AHD)	Freeboard to formation level (m)	Outlet velocity (m/s)	Peak flow (m^3/s)
C131.39	RCP	401.79	401.93	0.14	4.11
C131.49	RCP	401.79	401.92	0.14	4.12
C137.83	RCP	380.65	381.08	0.43	1.49
C137.88	RCP	380.62	381.04	0.42	1.34
C137.92	RCP	380.64	381.06	0.42	1.18
310-BR21	Bridge	380.57	381.07	0.50	0.98
310-BR22	Bridge	380.60	381.11	0.51	0.94
C139.37	RCP	380.63	381.04	0.41	1.20
C139.44	RCP	380.64	381.06	0.42	1.18
C139.50	RCP	380.64	381.04	0.41	1.19
C139.56	RCP	380.63	381.04	0.41	1.20
C139.71	RCP	380.65	381.06	0.41	1.18
C139.73	RCBC	380.65	381.06	0.42	1.18
C139.78	RCP	380.63	381.02	0.39	1.21
C140.09	RCP	380.62	381.03	0.41	1.21
C140.11	RCP	380.62	381.03	0.42	1.20
C140.17	RCP	380.57	380.94	0.37	1.29
C140.21	RCP	380.57	380.96	0.39	1.28
C140.23	RCP	380.56	380.96	0.40	1.28
C140.25	RCP	380.56	380.96	0.40	1.28
C140.27	RCP	380.56	380.96	0.40	1.27
C140.32	RCP	380.56	380.96	0.40	1.27
C140.38	RCP	380.56	380.96	0.40	1.27
C140.4	RCP	380.56	380.96	0.40	1.27
C140.43	RCP	380.56	380.96	0.40	1.27
C140.46	RCP	380.55	380.95	0.40	1.29
C140.49	RCP	380.54	380.94	0.40	1.30
C140.51	RCP	380.54	380.94	0.40	1.30
C140.55	RCP	380.55	380.95	0.40	1.29
C140.59	RCP	380.55	380.95	0.40	1.39
C140.64	RCP	380.54	380.94	0.40	1.39
C140.67	RCP	380.55	380.94	0.40	1.39
C140.78	RCP	380.52	380.90	0.38	1.43
C140.83	RCP	380.51	380.91	0.40	1.42
C140.87	RCP	380.51	380.92	0.41	1.41
C140.91	RCP	380.51	380.91	0.40	1.42
C140.98	RCP	380.49	380.90	0.41	1.44
C141.03	RCP	380.50	380.90	0.40	1.43
C141.07	RCP	380.49	380.88	0.40	1.35
C141.11	RCP	380.48	380.88	0.40	1.36

Structure ID	Structure type	U/S peak water level (m AHD)	Freeboard to formation level (m)	Outlet velocity (m/s)	Peak flow (m^3/s)
C141.2	RCP	380.47	380.86	0.39	1.38
C141.24	RCP	380.46	380.85	0.39	1.39
C141.29	RCP	380.45	380.83	0.38	1.41
C141.32	RCP	380.43	380.79	0.36	1.45
310-BR24	Bridge	380.37	380.84	0.47	1.22
C142.02	RCP	380.39	380.73	0.35	1.50
C142.04	RCP	380.40	380.75	0.35	1.49
C142.08	RCP	380.39	380.76	0.36	1.48
C142.13	RCP	380.39	380.75	0.36	1.49
C142.15	RCP	380.39	380.75	0.36	1.49
C142.19	RCP	380.39	380.74	0.36	1.49
C142.22	RCP	380.39	380.74	0.35	1.50
C142.25	RCP	380.40	380.74	0.34	1.50
C142.28	RCP	380.41	380.76	0.35	1.48
C142.36	RCP	380.40	380.75	0.35	1.48
C142.41	RCP	380.41	380.75	0.34	1.48
C142.44	RCP	380.41	380.75	0.34	1.49
C142.48	RCP	380.41	380.75	0.34	1.49
C142.5	RCP	380.42	380.75	0.34	1.48
C142.54	RCP	380.43	380.76	0.33	1.47
C142.58	RCP	380.41	380.74	0.33	1.50
310-BR25	Bridge	380.66	380.85	0.19	1.21
310-BR26	Bridge	380.74	380.96	0.22	1.10
C145.16	RCBC	380.87	381.11	0.24	1.13
C145.21	RCBC	380.98	381.22	0.24	1.02
C145.25	RCBC	381.07	381.33	0.26	0.91
C145.32	RCBC	381.10	381.36	0.26	0.88
C145.4	RCBC	381.14	381.40	0.26	0.86
C145.72	RCBC	381.32	381.57	0.25	0.69
C145.83	RCBC	381.39	381.64	0.25	0.71
C145.89	RCBC	381.38	381.62	0.24	0.74
C145.92	RCBC	381.39	381.65	0.26	0.71
C145.98	RCBC	381.41	381.68	0.27	0.70
C146.03	RCBC	381.45	381.68	0.23	0.71
C146.56	RCBC	381.80	382.02	0.23	0.47
C146.62	RCBC	381.82	382.04	0.23	0.47
C147.58	RCP	382.47	382.50	0.03	0.87
C147.63	RCP	382.51	382.56	0.05	0.99
C147.66	RCP	382.45	382.57	0.12	1.11
C147.73	RCP	382.40	382.56	0.17	1.36
310-BR27	Bridge	382.64	382.77	0.13	1.02

Structure ID	Structure type	U/S peak water level (m AHD)	Freeboard to formation level (m)	Outlet velocity (m/s)	Peak flow (m^3/s)
C149.39	RCP	382.65	382.76	0.11	1.21
C149.42	RCP	382.66	382.77	0.11	1.20
C149.45	RCP	382.67	382.79	0.12	1.18
C149.76	RCP	382.71	382.83	0.12	1.14
C149.8	RCP	382.71	382.82	0.12	1.15
C149.83	RCP	382.71	382.82	0.11	1.16
C149.87	RCP	382.71	382.82	0.11	1.15
C149.91	RCP	382.71	382.82	0.11	1.15
C149.96	RCP	382.70	382.80	0.09	1.19
C150.01	RCP	382.70	382.80	0.10	1.39

Table 9.54 outlines the changes in peak water levels at flood sensitive receptors for the climate change scenario where the increase exceeds 10 mm.

Table 9.54 Condamine River – summary of climate change impacts at flood sensitive receptors

Flood sensitive receptor ID	1% AEP climate change event	
	Change in peak water level (mm)	Existing case flood depth (m)
CON_ID_4	+168	0.74
CON_ID_5	+181	0.69
CON_ID_6	+183	0.72
CON_ID_7	+185	0.67
CON_ID_8	+170	0.92
CON_ID_9	+167	0.87
CON_ID_10	+163	0.99
CON_ID_13	+17	0.11
CON_ID_14	+17	0.07
CON_ID_15	+13	0.20
CON_ID_16	+69	0.04
CON_ID_68	+57	1.06
CON_ID_78	+64	0.95
CON_ID_96	+28	0.17
CON_ID_97	+24	0.46
CON_ID_98	+24	0.31
CON_ID_99	+48	0.48
CON_ID_100	+47	0.65
CON_ID_101	+48	0.91
CON_ID_102	+48	0.95
CON_ID_103	+48	0.55
CON_ID_104	+49	0.57
CON_ID_118	+186	0.73
CON_ID_119	+66	0.98
CON_ID_120	+69	0.96
CON_ID_146	+66	0.29

Flood sensitive receptor ID	1% AEP climate change event	
	Change in peak water level (mm)	Existing case flood depth (m)
CON_ID_147	+67	0.35
CON_ID_148	+68	0.35
CON_ID_149	+59	0.11
CON_ID_157	+50	0.04
CON_ID_158	+51	0.01
CON_ID_160	+10	0.35
CON_ID_161	+11	0.51
CON_ID_162	+10	0.50
CON_ID_203	+40	0.87
CON_ID_204	+40	1.17
CON_ID_205	+40	1.22
CON_ID_206	+40	0.97
CON_ID_207	+41	1.26
CON_ID_244	+100	0.22
CON_ID_245	+91	0.11
CON_ID_246	+76	0.35
ID CON__247	+67	0.46
CON_ID_248	+68	0.27
CON_ID_275	+47	0.03
CON_ID_277	+52	0.24
Elsden Road[1]	+71	2.53
Fysh Road[1]	+99	1.85
Gibbs Road[1]	+26	0.99
Gilgai Lane[1]	+112	2.96
Gore Highway (Toowoomba-Millmerran) [1]	+66	2.04
Hall Road[1]	+92	4.00
Lovell Road[1]	+815	0.52
Millmerran- Leyburn Road[1]	+130	3.23
Pampas Pit Road[1]	+54	1.30

Table note:

1 These roads are affected by climate change regardless of the Project and so the amenity of the roads is not compromised by the Project

9.5.4.5 Back Creek sensitivity

Back Creek drains the 106 km^2 catchment upstream of Millmerran into the Condamine River. In small flood events, Back Creek meanders to the Condamine approximately 16 km downstream of the Gore Highway. However, in larger events, Back Creek breaks its banks earlier and floodwaters flow more directly to the Condamine at around the Centenary Bridge. As Back Creek's catchment size is significantly smaller than that of the Condamine River, the timing of its flood peak is much shorter. The peak 1% AEP local flood in Back Creek is a 9-hour event, whereas the peak 1% AEP regional flood is two to four days. This difference in timing implies that modelling the Condamine River solely from a regional perspective could overlook potential localised impacts from a Back Creek flood event. Therefore, to better understand this behaviour and any potential impacts of differences in flood peak timing, three sensitivity analyses were undertaken, as outlined below:

- Local peak 1% AEP Back Creek inflow, with no regional flooding in the Condamine River. This scenario represents a low tailwater condition.
- Local peak 1% AEP Back Creek inflow, with regional flooding in the Condamine River. This scenario represents a realistic high tailwater condition.
- Local peak 1% AEP Back Creek inflow, with regional flooding in the Condamine River and timing of both local and regional hydrographs aligned such that their peaks arrive simultaneously at the confluence of Back Creek and Condamine River. This scenario represents the worst-case scenario and hence its likelihood is less than the 1% AEP.

In all three sensitivity scenarios, the flood model was tested with and without the proposed rail embankment to ascertain its potential impacts on flood levels and velocities.

The results from these tests indicate that the impacts of the local Back Creek flood event with the differing tailwater conditions from the regional Condamine River flood event are negligible. This result is reflected with or without the proposed rail embankment.

10 Back Creek

The Project alignment crosses Back Creek as well as a tributary to the south of Millmerran. Back Creek is a narrow and meandering system which in the 1% AEP event has flood depths of up to 5.4 m in its main channel. The flood depths along the alignment range from 0.5 m to 1.5 m through the floodplain crossings with a depth of 4.7 m at the Project alignment crossing of the Back Creek channel. The Existing Case 1% AEP inundated floodplain varies between 400 m and 1 km wide.

Commodore Mine is approximately 2 km upstream of the proposed crossing on Back Creek. Future mine development may involve a diversion of Back Creek itself, which in turn may require additional flood modelling to ascertain any potential effects the diversion could have on the Project alignment.

The location of the Project rail alignment in relation to Back Creek is shown in Figure D-1a in Volume II – Appendix D.

10.1 Data collection and review – Back Creek

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:

- TRC – existing flood studies
- The BoM – rainfall data
- DTMR – existing infrastructure details.

10.1.1 Previous studies

A number of previous hydrology and hydraulic studies were sourced as part of this assessment. A review of each study was undertaken to determine suitability for use on the Project as documented in the following sections.

Work Package 3, Historical study for Millmerran Final Report, WRM/DHI 2014

This study focused on a small reach of Back Creek approximately extending from the southern Millmerran-Inglewood Road bridge to Yandilla. The study involved the development of a coupled 1D/2D MIKE FLOOD hydraulic model. The model attempted to recreate the anecdotally observed flood extents of the January 2011 event by scaling steady state inflows. The model could not be used for the B2G Hydrology and Flooding assessment because of the unsuitability of the hydrology underpinning the hydraulic model.

10.1.2 Survey

ARTC provided LiDAR data from 2015 as 1 m grid DEM tiles. Using GIS software, a DEM was generated with a 1 m grid resolution for use in the Project based on the 2015 dataset. This was used for modelling within the disturbance footprint and up to the full extent of the 2015 LiDAR where relevant.

In areas that were not covered by the LiDAR provided by ARTC, LiDAR tiles were sourced from Geoscience Australia. The DEM datasets utilised for modelling were based on surveys flown between 2009 and 2015. SRTM data was used for catchment delineation where no LiDAR data could be sourced to inform the hydrologic modelling.

The survey data sources and DEM developed for Back Creek is shown in Figure D-1b in Volume II – Appendix D.

10.1.3 Aerial imagery

Aerial imagery of the study area was provided by ARTC and was used to identify and confirm topographic and vegetative characteristics of the study area. Aerial imagery captured in 2015 was made available. Additional imagery outside the study area was sourced from QGIS imagery in an open source format.

10.1.4 Existing drainage structure data

DTMR as-constructed drawings were also sourced for culvert and bridge details. This information will be refined as the local survey is progressively completed.

10.1.5 Stream gauge data

No streamflow gauges exist within the Back Creek catchment.

10.1.6 Rainfall data

A number of daily and sub-daily rainfall stations are located in and around the Back Creek catchment. However, since there are no streamflow gauges to use for model calibration, no historical rainfall data was sought.

10.1.7 Anecdotal and observed flood data

Several local landowners have commented on the rapid catchment response to large rainfall events and the velocity of associated flood waters. One landowner was able to identify on an aerial image where flood levels reached during the 2011 and 2013 events on his property adjacent to Back Creek. Additional consultation with landowners may be necessary to discuss veracity of modelling results.

10.1.8 Site inspection

A site inspection was undertaken during February 2018. During the site inspection, all major waterway crossings were visited and inspected with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

10.2 Hydrologic model development – Back Creek

10.2.1 Model setup

A calibrated hydrologic model for the Upper Condamine River was established by FFJV as outlined in Section 9.2. The results from this model reflect the large-scale, regional characteristics of longer duration floods and tend to underestimate the expected flood flows arising from a local storm event in Back Creek. Hence, a new URBS hydrological model was established for the Back Creek catchment. The URBS hydrologic model covers approximately 103 km^2 of the Back Creek catchment upstream of Millmerran. The catchment area was delineated into 16 sub-catchments to represent the network of creeks and streams within the catchment.

The Back Creek URBS hydrologic model could not be calibrated due to the lack of observed stream gauge data in the catchment. The nearest gauge that Back Creek flood waters drain to is Lemon Tree Weir, which is a low-flow gauge. This gauge records flows up to 1 m^3/s only, which essentially negates its use for calibration. There are other gauges further downstream but, due to flat topography of the Condamine floodplain, floodwaters tend to disperse, making it difficult to determine how much of a Back Creek flood reaches a given gauge with any degree of confidence.

The URBS hydrologic model extent and sub-catchment boundaries are presented in Figure D-1c in Volume II – Appendix D.

The URBS model setup details are summarised in Table 10.1.

Table 10.1 Summary of Back Creek URBS model details

Input parameter	Details
URBS model type	Basic
Routing variables	Catchment area, stream lengths
Channel lag parameter, α	1.20
Catchment non-linearity parameter, m	0.8

Note that default values were adopted for all other URBS parameters.

10.2.2 Design event parameters

Hydrologic information to assist estimation of design event flows was sourced from the ARR 2016 Data Hub as summarised in Table 10.2.

Table 10.2 Back Creek – Summary of URBS model design event inputs

Input parameter	Remarks
Design rainfall	IFDs for each sub-catchment were downloaded from the BoM's Data Hub to account for variation in rainfall across the catchment.
Extreme event rainfall	PMP depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the method presented in the Bulletin 53 (BOM, 2003). The rainfall depths for the 1 in 10,000 AEP event were estimated using the interpolation method presented in ARR 2016 Book 8 Section 3.5.
Losses	The losses were initially adopted from the ARR Data Hub (Initial loss – 35 mm, Continuing loss – 0.7 mm/h). Adjustments were made to reconcile URBS flows to area-scaled FFA.
Areal reduction factor	Parameters were adopted for the Semi-Arid Inland Queensland region. The catchment area upstream of the proposed rail crossing on Back Creek is approximately 103 km^2, which yields an areal reduction factor of between 95.0% and 96.6% depending on design storm event AEP and duration.
Ensemble temporal patterns	Central Slopes regions. However, as the study catchment area exceeds 75 km^2, the standard ensemble rainfall patterns from ARR 2016 do not apply to this catchment for storm durations longer than 12h. These were replaced with the areal temporal patterns for the Central Slopes region.
Preburst depths	Median preburst depths were downloaded from the ARR 2016 Data Hub for each sub-catchment. Preburst depths vary by design storm event AEP and duration. Preburst depths were applied to the model by reducing the initial losses for each storm event.

10.2.3 Hydrologic model validation

In the absence of data to calibrate the Back Creek catchment, the hydrologic model was instead validated against an area-scaled FFA of an analogous catchment. Canal Creek was chosen for this exercise due to its proximity to Back Creek and its relatively similar topography and catchment area. The catchment area of Canal Creek is 395 km and its outlet is approximately 35 km from the outlet of Back Creek. Both catchments flow into the Condamine River basin.

The initial and continuing losses were adjusted until there was a reasonable match between the modelled flows and those from the scaled FFA. DTMR's Quantile Regression Technique (QRT) and the RFFE were included as an additional point of comparison. All URBS hydrologic model results between the 20% AEP and 1% AEP events fall within the 90% confidence limits of the RFFE and show a fair match with QRT. The estimated flood flows at the outlet of the Back Creek model are presented in Figure 60 and Table 10.3. The model outlet is approximately 2.15 km downstream of the Project alignment.

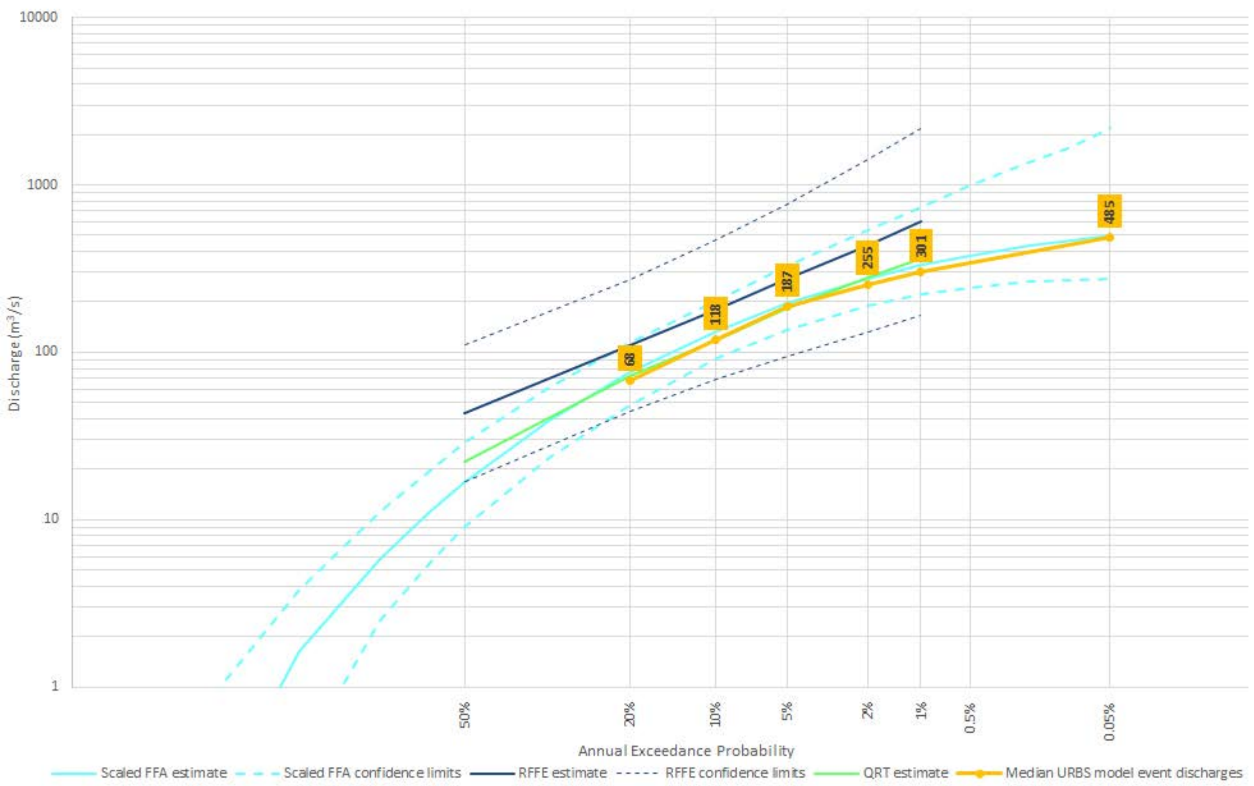


Figure 60 Estimate of flows at the outlet of Back Creek model

Table 10.3 Estimate of flows at the outlet of Back Creek model

AEP (%)	RFFE – lower bound 90% confidence level (m^3/s)	RFFE – estimate of flow (m^3/s)	RFFE – upper bound 90% confidence level (m^3/s)	DTMR quantile regression technique (m^3/s)[1]	URBS hydrologic model flows (m^3/s)
50	17	43	111	22	-
20	44	109	270	71	68
10	69	179	472	118	118
5	94	272	769	182	187
2	133	437	1,430	279	255
1	165	603	2,170	364	301
1 in 2,000	-	-	-	-	485
1 in 10,000	-	-	-	-	841
PMP	-	-	-	-	3,648

Table note:

1The QRT method estimates the 39.3%, 18.1%, 9.5% and 4.9% AEP instead of 50%, 20% and 10% and 5% respectively

10.3 Hydraulic model development – Back Creek

A two-dimensional modelling approach was adopted to appropriately simulate flood mechanisms around the Project rail alignment at Back Creek. The platform used for hydraulic modelling is the TUFLOW HPC software package. The processes and assumptions adopted throughout the development of the hydraulic model are described in the following sections.

10.3.1 Model setup

The setup of the TUFLOW model is summarised in Table 10.4.

Table 10.4 Back Creek hydraulic model summary

Parameter	Information
Completion date	June 1019
AEPs assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF
Hydraulic model build	TUFLOW HPC GPU – version 2017-09-AC-w64-iSP
Model extent	Refer to Figure D-1d in Volume II – Appendix D
Grid size	5m
DEM (year flown)	ARTC dataset (2015).
Roughness	Spatially varying roughness values compliant with industry norms.
Eddy viscosity	Smagorinsky (default)
Model calibration	N/A
D/S model boundary	Height-Discharge (HQ) Boundary with normal slope approximated based upon topography dataset.
Hydraulic model timestep	Adaptive Timestep
Hydraulic model wetting and drying depths	Cell centre set at 0.0002m Cell side set at 0.0001 m
Modelled scenarios	Existing Case, Developed Case
Sensitivity analysis	Blockage – 0%, 50% Climate change

The hydraulic model extent and the spatial distribution of land use in the 2D model domain is presented in Volume II – Appendix D, Figure D-1d and the landuse classification in Figure D-1e.

10.3.2 Hydraulic structures

The small number of existing minor culverts situated within the hydraulic model extents were omitted as structure details were not available and their expected impacts on overall flood conditions were negligible.

10.4 Existing Case modelling results – Back Creek

10.4.1 Hydrologic model peak flow assessment

A critical duration assessment was undertaken to determine which storm duration(s) produced peak flood flows where the Back Creek intersects the Project alignment and at the downstream outlet of the model.

To assess the critical storm duration the following methodology was adopted:

- The models were modelled for a range of AEP events:
  - Each AEP was modelled for a range of durations
  - Each duration was modelled for each of the ten associated temporal patterns
- A critical duration assessment was undertaken at the locations mentioned above to determine which duration produced the highest median flow of the ten temporal patterns for each event.

Table 10.5 presents the estimated peak flow applied to the hydraulic model for key location (Figure D-1d in Volume II – Appendix D).

Table 10.5 Peak flow at key locations as applied in the hydraulic model

AEP (%)	Peak flow (m^3/s)	Critical storm
20	68	36 hour - Pattern 4
10	118	24 hour - Pattern 5
5	187	9 hour - Pattern 6
2	255	6 hour - Pattern 7
1	301	9 hour - Pattern 9
1 in 2,000	485	9 hour - Pattern 9
1 in 10,000	841	6 hour – PMP Temporal Pattern
PMF	3,648	6 hour – PMP Temporal Pattern

10.4.2 Existing Case flood maps

Maps illustrating indicative flood extents and peak water levels were prepared and are presented in Volume II – Appendix D:

- 20% AEP: Figure D-2a
- 10% AEP: Figure D-2b
- 5% AEP: Figure D-2c
- 2% AEP: Figure D-2d
- 1% AEP: Figure D-2etable 374
- 1 in 2,000 AEP: Figure D-2f
- 1 in 10,000 AEP: Figure D-2g
- PMF: Figure D-2h.

Figure D-3a presents peak flood velocities expected in a 1% AEP event.

10.4.3 Flood inundation extent and flood levels

Figure D-2e in Volume II – Appendix D shows the 1% AEP indicative flood extent and peak water levels within the Back Creek floodplain for the Existing Case.

Under the 1% AEP event, the peak depth is about 5.4 m within the Back Creek channel. This depth reduces to an average depth of around 0.7 m in other areas of the floodplain. The peak depth on the floodplain is estimated to be up to 4.7 m (406.8 m AHD) where the proposed alignment crosses Back Creek. The peak depth where the proposed alignment crosses the unnamed tributary of Back Creek is 2.3 m (411.7 m AHD).

The model indicates that the time of inundation across the floodplain during the critical 1% AEP design flood is between 8 and 12 hours.

10.4.4 Flood immunity of existing infrastructure

Table 10.6 presents a summary of overtopping depths for key roads near the Project alignment under a range of design events. Modelling results show that Millmerran-Inglewood Road has an existing low flood immunity in the areas close to the Project alignment.

Table 10.6 Back Creek – Existing Case – Overtopping depths of key infrastructure

Infrastructure location	Overtopping depth (m)							
	PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Millmerran-Inglewood Road	3.23	2.58	2.03	1.87	1.88	1.86	1.71	1.68

10.4.5 Existing Case velocities

Peak flood velocities are expected to reach approximately 4.9 m/s in localised areas in the main creek channel, whereas the average velocity across the floodplain is approximately 0.6 m/s as shown in Figure C3-a in Volume II – Appendix D. This peak channel velocity corroborates with the accounts from local landowners of flood conditions.

10.5 Developed Case modelling results – Back Creek

10.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW hydraulic model (1D and 2D approach).

On the Back Creek floodplain, the Project includes the following floodplain (or regional structures):

- Two waterway bridges
- Two RCP locations (a total of 24 cells).

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and depending on their interaction with the Back Creek floodplain were incorporated in the hydraulic model.

The proposed drainage structures are summarised in Table 10.7 and Table 10.8 and are also shown in Figure D-1f in Volume II – Appendix D. The 1% AEP flood levels at each drainage structure is presented in Table 10.12. A minimum culvert size of 900 mm diameter was adopted to reduce potential for blockage and maintenance.

Bridges were modelled as an opening in the rail embankment. The optimisation of bridge lengths was balanced between minimising the changes to the hydraulic regime, primarily afflux and velocities, and the cost of replacing bridge spans by large earth embankments.

Blockage of hydraulic structures was assessed in accordance with ARR 2016. Despite the catchment being vegetated, ARR guidelines determined that the likelihood of significant amounts of debris accumulating against the piers of the bridge is low. Therefore, a zero blockage factor was applied at the Back Creek bridges.

Table 10.7 Back Creek – proposed floodplain culvert locations and details

Chainage (km)	Structure ID	Type	US invert (m AHD)	DS invert (m AHD)	Diameter (m)	Number of cells
126.76	C126.76	RCP	408.37	407.73	0.9	12
126.80	C126.80	RCP	405.56	405.21	0.9	12

Table 10.8 Back Creek – proposed bridge location and details

Chainage (km)	Structure ID	Approximate span (m)	Deck width (m)	Deck level (m AHD)	Deck superstructure type	Deck depth (mm)
126.97	310-BR37[1]	167	3.97	421.0	Type D1	2,000
128.06	310-BR38	230	3.97	411.0	Type D1	2,000

Table note:

1 310-BR37 is also a rail bridge, and spans Millmerran Inglewood Road

10.5.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

10.5.2.1 Rail immunity and structures results

Appendix C summarises the peak 1% AEP water levels on the upstream side of the proposed alignment. Local drainage structures (i.e. those not included in the flood model) and road culverts are not reported.

The results of flood modelling indicate that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the Back Creek floodplain. There is over 8.5 m freeboard above the culvert obvert levels to the rail formation in a 1% AEP event. Minimum freeboard to formation level is at proposed bridge 310-BR38 where there is 0.8 m freeboard.

10.5.2.2 Velocities and scour

Appendix C summarises the peak 1% AEP outflows and velocities at structures. The 1% AEP peak velocity through the proposed culverts is generally less than 2.1 m/s, whereas peak velocities through bridges 310-BR37 and 310-BR38 are 3.0 m/s and 2.2 m/s respectively.

Scour protection requirements for culverts were calculated based on the velocities predicted from the hydraulic modelling. The scour protection was 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.

Table 10.9 lists the soil types encountered along the Project alignment and the allowable soil velocity based on AGRD.

Table 10.9 Allowable soil velocities along the Project alignment

Soil type	Soil description	Maximum allowable soil velocity as per AGRD (m/s)
Sodosols	Firm loam or stiff clay	2 m/s
Vertosols, Dermosols, Rudosols and Kandosols	Sandy or silty clay	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.

Scour protection requirements for culverts have been calculated based on the velocities predicted from the hydraulic modelling. Appendix D presents the proposed scour protection at each structure.

A conservative scour estimation has been undertaken at the bridge site based on available information and will be refined during detailed design.

10.5.2.3 Flood immunity for extreme events

The risk of overtopping of the rail alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF) with Table 10.10 presenting the depth of water above the formation level and over the top of rail at each structure. It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the alignment. As such, overtopping of the rail is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario.

Table 10.10 Back Creek - extreme events – depth of water above formation and top of rail levels

Chainage (km)	Depth of water above formation level (m)			Depth of water over 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
125.65 to 126.75	0.03	0.04	0.04	-	-	-
126.85 to 127.05	0.89	1.30	1.86	0.19	0.60	1.16
127.95 to 128.15	2.08	2.93	4.49	1.38	2.23	3.79
128.15 to 129.25	-	-	1.42	-	-	0.72

10.5.3 Flood impact objectives outcomes – Back Creek

The impact of the Developed Case was assessed through inclusion of the proposed rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, duration of inundation, velocities and peak flows within the floodplain.

- Changes in peak water levels for the AEP's assessed are presented in Figures D-4a to D-4h in Volume II – Appendix D
- Changes in 1% AEP duration of inundation are presented in Figure D-4i in Volume II – Appendix D
- Changes in 1% AEP velocities are presented in Figure D-4j in Volume II – Appendix D.

The effects of all impacts are required to be agreed with relevant stakeholders and affected landowners as part of the EIS process.

The Project design outcomes relative to the flood impact objectives (Table 10.11) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Volume II – Appendix N.

The potential impacts to water levels across events up to and including the 1% AEP are summarised in Table 10.11.

Table 10.11 Afflux summary – Back Creek

Afflux outside rail disturbance footprint	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
Maximum afflux (mm)	127	139	205	285	289
Area afflux >10 mm experienced (ha)	7.9	28.7	53.3	61.5	60.8
Area afflux >200 mm experienced (ha)	-	<0.01	<0.01	1.2	1.6
Area afflux >400 mm experienced (ha)	-	-	-	-	-

10.5.3.1 Flood impacts at proposed hydraulic structures

The estimated potential impacts on peak water levels at each proposed structure are presented in Table 10.12. Peak water levels were extracted immediately upstream of each culvert and at the control line of each bridge.

The design achieved a freeboard of at least 300 mm between the 1% AEP peak water and formation levels.

Table 10.12 Back Creek – 1% AEP event – estimated impacts to peak water levels at proposed hydraulic structures

Chainage (km)	Structure ID	Structure type	Rail formation level (m AHD)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Change in peak water level (mm)
126.76	C126.76	RCP	421.0	412.5	412.6	+90
126.80	C126.80	RCP	421.0	412.5	412.6	+90
126.97	310-BR37	Bridge	421.0	411.7	411.9	+210
128.06	310-BR38	Bridge	409.8	406.8	407.0	+180

10.5.3.2 Flood impacts on flood sensitive receptors

Based on the available aerial imagery, no buildings or critical structures are located within the area impacted by afflux in the Back Creek floodplain for events up to the 1% AEP.

10.5.3.3 Flood impacts on state-controlled roads

The extent of the hydraulic model developed for Back Creek is shown in Figure 61. Within the extent of the hydraulic model, the only state-controlled road which is influenced by flooding and the Project alignment is the Millmerran-Inglewood Road. The location of the state-controlled road is shown in Figure 61.

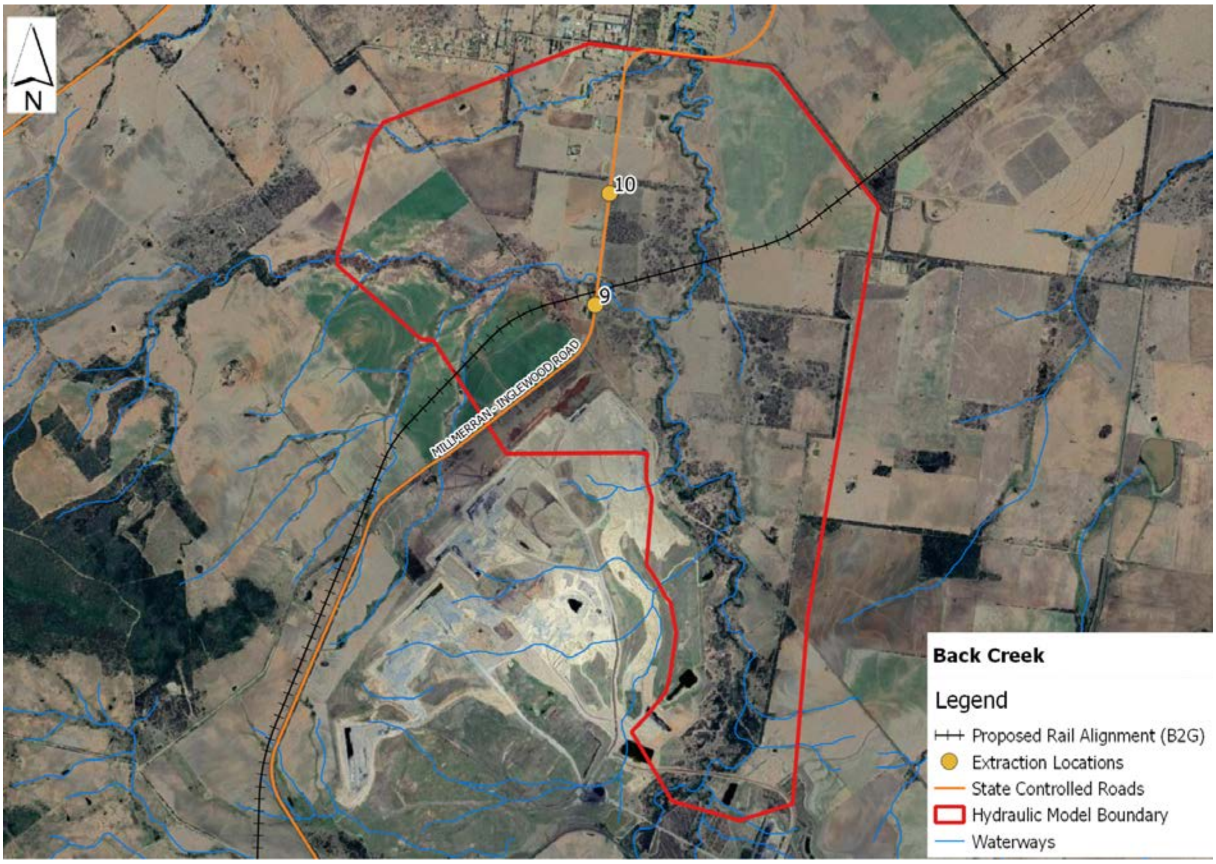


Figure 61 Back Creek - hydraulic model extent and associated state-controlled roads

The following sections describe the impacts to state-controlled roads in both the Existing Case and the Developed Case and summarises the differences between the two.

Within the Back Creek model extent, throughout the various AEP events modelled, there are typically two discrete areas where the road is overtopped up until the PMP event where the whole segment is impacted. Point 9 represents the southern segment of Millmerran-Inglewood road which is overtopped while point 10 represents the northern segment.

Millmerran-Inglewood road has low immunity in regard to Back Creek flooding. Existing Case model results indicate that large segments of the road are overtopped in a 20% AEP event.

Existing Case flooding conditions

Table 10.13 Back Creek – Existing Case flood depths

Reporting location	Road	Estimated depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
9	Millmerran-Inglewood Road	0.08	0.11	0.18	0.22	0.21	0.27	0.54	0.97
10	Millmerran-Inglewood Road	0.03	0.09	0.21	0.25	0.24	0.29	0.50	0.95

Table 10.14 Back Creek – Existing Case flood inundation length

Reporting location	Road	Approximate length of inundation (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
9	Millmerran-Inglewood Road	73	76	199	213	223	368	534	1,675
10	Millmerran-Inglewood Road	10	92	232	255	271	350	605	

Table 10.15 Back Creek – Existing Case time of submergence

Reporting location	Road	Estimated time of submergence (hrs)								AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
9	Millmerran-Inglewood Road	15.2	17.5	8.4	6.8	10.0	10.2	8.2	8.5	8.8
10	Millmerran-Inglewood Road	10.4	13.5	8.4	7.0	10.1	10.5	8.2	8.6	6.3

Developed Case flooding conditions

Table 10.16 Back Creek – Developed Case flood depths

Reporting location	Road	Estimated depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
9	Millmerran-Inglewood Road	0.09	0.12	0.19	0.23	0.22	0.28	0.53	1.06
10	Millmerran-Inglewood Road	0.04	0.09	0.21	0.25	0.24	0.29	0.50	0.96

Table 10.17 Back Creek – Developed Case flood inundation length

Reporting location	Road	Approximate length of inundation (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
9	Millmerran-Inglewood Road	63	76	200	240	243	243	560	1,675
10	Millmerran-Inglewood Road	12	70	231	243	269	348	607	

Table 10.18 Back Creek - Developed Case time of submergence

Reporting location	Road	Estimated time of submergence (hrs)								AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
9	Millmerran-Inglewood Road	20.2	22.3	8.4	6.9	10.0	10.3	8.2	8.5	11.4
10	Millmerran-Inglewood Road	11.2	14.0	8.5	7.0	10.2	10.5	8.3	8.6	6.7

Impacts of Project alignment

Table 10.19 Back Creek - change in flood depths

Reporting location	Road	Estimated change in depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
9	Millmerran-Inglewood Road	0.01	0.01	0.01	0.01	0.01	0.01	-0.01	0.09
10	Millmerran-Inglewood Road	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01

Table 10.20 Back Creek - change in time of submergence

Reporting location	Road	Estimated change in time of submergence (hrs)								Estimated change in AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
9	Millmerran-Inglewood Road	5.0	4.8	0.0	0.1	0.0	0.1	0.0	0.0	2.6
10	Millmerran-Inglewood Road	0.8	0.5	0.1	0.0	0.1	0.0	0.1	0.0	0.4

Change in flood hydrographs

Figure 62 presents the Developed Case and Existing Case water level time series for the 1% AEP event at extraction point 9, located along Millmerran-Inglewood Road. As shown in the figure, the Developed Case is consistently higher than the Existing Case throughout the scenario; however, the difference in level itself is negligible, being approximately 10 mm throughout the simulation. The shape of the time series results indicates near identical catchment response and negligible difference on flood behaviours.

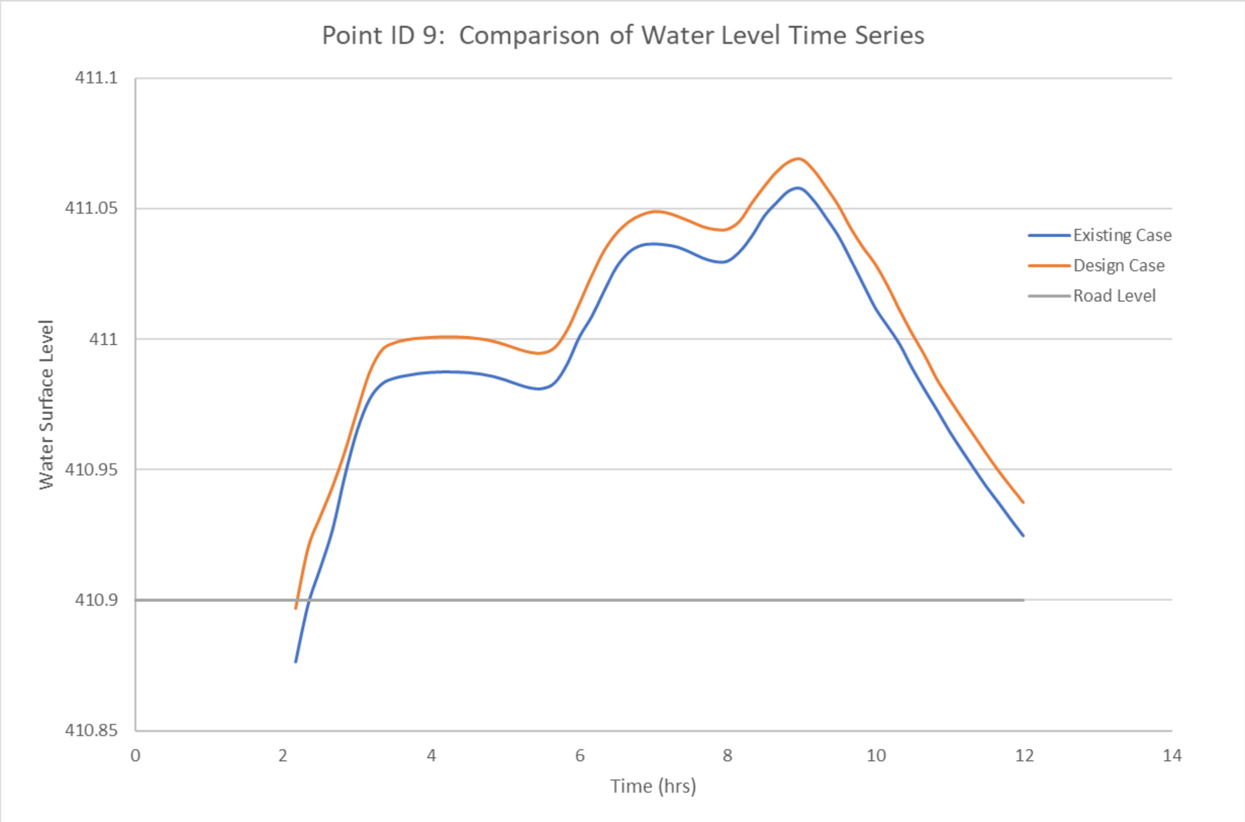


Figure 62 **Extraction Point 9 – comparison of water level time series, 1% AEP**

10.5.3.4 Flood impacts on local public roads

The change in peak water levels and flood hazard (velocity-depth) for the 1% AEP event were evaluated on local public roads within the hydraulic model domain. Local public roads that are expected to experience an increase in flood hazard and/or increases in peak flood levels are reported in Table 10.21.

Table 10.21 **Back Creek – changes in peak water levels and flood hazard for local public roads, 1% AEP**

Location	Existing flood hazard (m^2/s)	Design flood hazard (m^2/s)	Maximum existing flood depth (m)	Maximum design flood depth (m)	Maximum change in peak water levels (mm)[1]
Schwartens Road	0.817	0.817	0.594	0.594	+22
Unnamed Road	0.121	0.03	0.263	0.177	+34

Table note:

1 The maximum change in peak water level does not necessarily occur at the same location as where the existing and/or design maximum flood depth occur

Duration of inundation

Assessment of the time of submergence (ToS) and average annual time of submergence (AAToS) was undertaken for local public roads within the hydraulic model domain. Local public roads that are expected to experience an increase in ToS and/or AAToS are presented in Table 10.22.

Table 10.22 **Back Creek – ToS and AAToS for local public roads**

Location	Existing 1% AEP ToS (hrs)	1% AEP ToS diff. (hrs)	2% AEP ToS diff. (hrs)	5% AEP ToS diff. (hrs)	10% AEP ToS diff. (hrs)	AAToS Existing Case (hrs)	AAToS Developed Case (hrs)	AAToS diff. (hrs)
Unnamed Road	9.57	-6.03	-3.45	1.98	5.93	9.76	12.38	2.62

10.5.3.5 Flood impacts on private land outside the rail disturbance footprint

Most of the area where afflux is predicted to occur is agricultural land or open land on which nominal afflux is unlikely to cause any adverse impact. Table 10.23 presents the modelled changes in flood conditions during the 1% AEP event on a lot basis according to the following thresholds:

- Peak water levels increased by greater than +10 mm
- Peak velocities increased by greater than 0.25 m/s
- Duration of inundation changed by more than 25% of its original duration of inundation across the lot.

Table 10.23 Back Creek – summary of flood impacts on private land outside the rail disturbance footprint for 1% AEP

Approximate chainage (km)	Changes in peak water levels[1]		Changes in peak velocities		Changes in Duration of inundation (hrs)	
	Maximum change (mm)	Total area affected by change > 10 mm (ha)[2]	Maximum change (m/s)	Total area affected by change (ha)	Maximum change (%)	Total area affected by change (ha)
128.30 to 128.80	+15	3.86	-	-	-	-
128.10	-	-	+0.5	0.04	-	-
125.70 to 126.50	+83	0.01	+0.7	0.59	-	-
125.40	+44	0.05	-	-	-	-
127.90 to 128.30	+53	10.53	+0.5	9.34		-
123.90 to 126.40	-	-	+0.3	0.09	-	-
123.90 to 126.40	-	-	+0.6	0.71	-	-
126.50 to 126.90	+120	2.91	+0.7	2.95	-	-
128.40 to 129.40	-	-	+0.4	0.15		-
128.20	+260	3.85	+0.3	1.00	-	-
127.40 to 127.95	+290	28.00	+0.6	13.29	-	-
127.00	+87	0.50	+0.1	0.22	-	-

Table notes:

1 Afflux on lots that exceed the flood impact objectives are summarised in the EIS Surface Water Chapter
2 Only minor areas, usually directly upstream of culverts are affected by the maximum afflux as stated

10.5.3.6 Flow distribution

A key landowner concern is changes to flow distributions. However, Back Creek is well-defined, and the Project alignment crosses the creeks generally perpendicularly. There are no lateral breakouts of floodwaters in up to the 1% AEP event and hence there are negligible changes to flow distribution in those events.

10.5.4 Sensitivity analysis – Back Creek

The sensitivity of the model to various parameters was assessed using the following three scenarios:

- An increase in rainfall intensity, i.e. to reflect climate change scenario
- Increase in blockage of culverts from 25% to 50%
- Decrease in blockage of culverts from 25% to 0%.

10.5.4.1 Blockage

A significant 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% and 50% blockage.

Results of the blockage sensitivity analysis are presented in Table 10.24 and shown in Figure D-5a and D-5b (Volume II – Appendix D), respectively. The results indicate that by varying the level of blockage at culverts has a relatively minor impact on changes in peak water levels in Back Creek.

Table 10.24 1% AEP event – impacts on peak water levels due to different blockage factors

Afflux outside rail disturbance footprint	0% blockage	25% blockage (Developed Case)	50% blockage
Maximum afflux (mm)	+289	+289	+289
Area afflux >10 mm experienced (ha)	58.16	60.81	63.88

Table 10.25 provides a summary of 1 % AEP peak flood levels at cross drainage structures for the blockage scenarios.

Table 10.25 Back Creek – 1 % AEP event – culvert blockage assessment

Structure ID	Structure type	1 % AEP Peak water levels (m AHD)			Increase from Developed Case to 50% blockage scenario (mm)
		0% blockage	Developed Case (25% blockage)	50% blockage	
C126.76	RCP	412.6	412.6	412.6	+6
C126.80	RCP	412.6	412.6	412.6	+6

Table 10.26 outlines the changes in peak water levels at flood sensitive receptors for the 50% blockage scenario where the increase exceeds 10 mm.

Table 10.26 Back Creek – summary of 50% blockage impacts at flood sensitive receptors

Flood sensitive receptor ID	Existing case flood depth (m)	Change in peak water level (mm)
Millmerran-Inglewood Road[1]	1.90	+298
Unnamed road[1]	0.29	+34
Schwartens Road[1]	1.56	+77

Table note:

1 These roads are affected by climate change regardless of the Project and so the amenity of the roads is not compromised by the Project

During detail 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.

10.5.4.2 Impacts during extreme events

Table 10.27 outlines the changes in peak water levels at flood sensitive receptors for the extreme events where the increase exceeds 10 mm under one of the events. The existing depth of flooding is also detailed and as can be seen the larger impacts that occur under the PMF event occur generally when there are already high flood depths as would be expected under such a rare event.

Flood immunity of the Project alignment is discussed in Section 10.5.2.3, and maps demonstrating the impacts during extreme events are shown in Volume II – Appendix D, Figures D-4f to D-4h.

Table 10.27 Back Creek – summary of extreme event impacts at flood sensitive receptors

Flood sensitive receptor ID	1 in 2,000 AEP event		1 in 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)
BAC_ID_1	-	-	-	-	+51	0.72
BAC_ID_2	-	-	+4	0.15	+47	0.85
BAC_ID_3	-	-	+45	0.28	+215	0.98
BAC_ID_4	-	0.26	+35	0.53	+172	1.23
BAC_ID_5	-	0.19	+36	0.57	+164	1.32
Kooroongarra Road	-	2.80	+6	3.60	+221	5.20
Millmerran - Inglewood Road	+247	2.03	+173	2.58	+237	3.23
Unnamed road	+182	0.31	+249	0.44	+204	0.98
Schwartens Road	+69	1.66	+90	2.31	+39	3.25

10.5.4.3 Climate change

The potential impacts of climate change in the Back Creek floodplain were assessed for the 1% AEP design event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2016.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario has been adopted for the Project with an associated increase in rainfall intensity of 23.9% across the catchment area.

Table 10.28 presents the structure performance under Representative Concentration Pathways 8.5 climate change conditions. For the 1% AEP event, the change in peak water levels for the Representative Concentration Pathways 8.5 climate change scenario is presented in Figure D-5c in Volume II – Appendix D.

Climate change results are expected to increase peak water levels upstream of the Project alignment by up to 80 mm at structure locations for the 1% AEP event. The Project alignment is expected to retain 1% AEP flood immunity to formation level under the climate change scenario.

Table 10.28 Back Creek – 1% AEP event Representative Concentration Pathways 8.5 conditions – structure performance

Structure ID	Structure type	1% AEP peak water level (m AHD)	1% AEP +CC peak water level (m AHD)	Difference in peak water level (m)	Freeboard to rail formation with CC (m)
C126.76	RCP	412.6	412.7	+0.08	8.3
C126.80	RCP	412.6	412.7	+0.08	8.3
310-BR38	Bridge	411.9	412.0	+0.09	9.0

Table 10.29 outlines the changes in peak water levels at flood sensitive receptors for the climate change scenario where the increase exceeds 10 mm.

Table 10.29 Back Creek – summary of climate change impacts at flood sensitive receptors

Flood sensitive receptor ID	1% AEP climate change event	
	Change in peak water level (mm)	Existing case flood depth (m)
Kooroongarra Road[1]	+184	2.50
Millmerran-Inglewood Road[1]	+195	1.90
Unnamed road[1]	+64	0.29
Schwartens Road[1]	+128	1.56

Table note:

1 These roads are affected by climate change regardless of the Project and so the amenity of the roads is not compromised by the Project

11 Nicol Creek

The Project alignment crosses Nicol Creek approximately 350 m east of Millmerran- Inglewood Road. Nicol Creek is a narrow and meandering system with depths under the Existing Case 1% AEP event of between 2 and 3 m within the main channel, and between 0.3 and 1 m in breakout areas. At the Project alignment the flood depths are approximately 1.2 m.

Nicol Creek is a defined creek in terms of channel depths and banks and the flood extents under the 1% AEP event remain in close proximity to the creek alignment. The Existing Case 1% AEP inundated floodplain varies between 50 m and 200 m wide.

The location of the Project rail alignment in relation to Back Creek is shown in Figure E-1a in Volume II – Appendix E.

11.1 Data collection and review – Nicol Creek

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:

- Goondiwindi Regional Council (GRC) – existing flood studies
- The BoM – rainfall gauging data
- DTMR – existing infrastructure details.

11.1.1 Previous studies

A number of previous hydrology and hydraulic studies were sourced as part of this assessment. A review of each study was undertaken to determine suitability for use on the Project as documented in the following sections.

Goondiwindi Regional Council, Inglewood Flood Study, Engeny, 2015

Engeny was commissioned by GRC to undertake a flood study of Inglewood. The study objectives were "to define the nature, extent and risks of flooding in Inglewood in order to inform disaster management planning and response, as well as control future development." (Engeny, 2015). An URBS hydrologic model and TUFLOW (1D/2D) hydraulic model were developed.

11.1.2 Survey

ARTC provided LiDAR data from 2015 as 1 m grid DEM tiles. Using GIS software, a DEM was generated with a 1 m grid resolution for use in the Project based on the 2015 dataset. This was used for modelling within the disturbance footprint and up to the full extent of the 2015 LiDAR where relevant.

In areas that were not covered by the LiDAR provided by ARTC, LiDAR tiles were sourced from Geoscience Australia. The DEM datasets utilised for modelling were based on survey flown between 2009 and 2015. SRTM data was used for catchment delineation where no LiDAR data could be sourced, to inform the hydrologic modelling.

The survey data sources and DEM developed for Nicol Creek are shown in Figure E-1b in Volume II – Appendix E.

11.1.3 Aerial imagery

Aerial imagery of the study area was provided by ARTC and was used to identify and confirm topographic and vegetative characteristics of the study area. Aerial imagery captured in 2015 was made available. Additional imagery outside the study area was sourced from QGIS imagery in an open source format.

11.1.4 Existing drainage structure data

DTMR as-constructed drawings were also sourced for culvert and bridge details. This information will be refined as the local survey is progressively completed.

11.1.5 Stream gauge data

No streamflow gauges exist within the Nicol Creek catchment.

11.1.6 Rainfall data

Several daily and sub-daily rainfall stations are located in and around the Nicol Creek catchment. However, since there are no streamflow gauges to use for model calibration, no historical rainfall data was sought.

11.1.7 Anecdotal and observed flood data

No anecdotal or observed flood data was available for this area of Nicol Creek.

11.1.8 Site inspection

A site inspection was undertaken during February 2018. During the site inspection, all major waterway crossings were visited and inspected with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

11.2 Hydrologic model development – Nicol Creek

11.2.1 Model setup

Nicol Creek is one of four major waterways which cross the Project rail alignment before flowing into Canning Creek. The other waterways are Bringalily Creek, Native Dog Creek and Cattle Creek. Due to their proximity to one another, all four waterways were modelled in a unified Canning Creek model.

The hydrology of the Canning Creek catchment was previously modelled in URBS by Engeny as part of the Inglewood Flood Study (2015) for Goondiwindi Regional Council. This model was validated against the 1976 flood event at two stream gauges (416402B/C, 416415A). Two significant modifications to the hydraulic model were necessary to ensure it was fit for use in this assessment, being:

- Hydrologic inputs such as rainfall intensities, losses and temporal patterns were updated to ARR 2016 standards
- Modelled catchments were subdivided to extract flood flow estimates at key locations including the Project alignment crossings of Nicol Creek, Bringalily Creek, Native Dog Creek and Cattle Creek.

The refined URBS hydrologic model covers approximately 1,202 km^2 of the Canning Creek catchment upstream of Inglewood. The catchment comprises 35 sub-catchments to capture the variability of rainfall and to better represent the network of creeks and streams within the catchment. The catchment area upstream of the Project alignment crossing of Nicol Creek is approximately 38 km^2. The hydrologic model setup including extent and sub-catchments is presented in Volume II – Appendix E, Figure E-1c.

The URBS model setup details are summarised in Table 11.1.

Table 11.1 Summary of URBS model inputs

Input parameter	Remarks
URBS model type	Basic
Routing variables	Catchment area, stream lengths
Channel lag parameter, α	1.20
Catchment non-linearity parameter, m	0.8

Note that the default values were adopted for all other URBS parameters

Three key locations were used to validate the results of the new URBS model against the results from the Inglewood Flood Study model for the 1% AEP event. The QRT flow estimation method was also carried out to provide a further validation of the URBS hydrologic model results. The results of this comparison are presented in Table 11.2.

Table 11.2 Comparison of flows at key locations – 1% AEP event

Engeny URBS catchment ID	FFJV URBS catchment ID	Revised Engeny URBS discharge estimate (m^3/s)	FFJV URBS flow estimate (m^3/s)	QRT estimate (m^3/s)
S_2	08T	561	632	703
S_3	14T	579	618	707
S_5	28T	980	1,408	1,322

11.2.2 Design event parameters

Hydrologic information to assist estimation of design event flows was sourced from the ARR 2016 Data Hub as summarised in Table 11.3.

Table 11.3 Summary of URBS model design event inputs

Input parameter	Remarks
Design rainfall	IFDs for each sub-catchment were downloaded from the BoM's website to account for variation in rainfall across the catchment.
Extreme event rainfall	PMP depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the method presented in the Bulletin 53 (BOM, 2003). The rainfall depths for the 1 in 10,000 AEP event were estimated using the interpolation method presented in ARR 2016 Book 8 Section 3.5.
Losses	The losses were initially adopted from the Inglewood Flood Study (2015) URBS model (Initial loss – 15 mm, Continuing loss – 1 mm/h). Adjustments were made to reconcile URBS flows to area-scaled FFA.
Areal reduction factor	Parameters were adopted for the Semi-Arid Inland Queensland region. The catchment area of Canning Creek is approximately 138 km^2, which yields an ARF between 52.2% and 93.5% depending on design storm event AEP and duration.
Ensemble temporal patterns	Central Slopes regions. However, as the study catchment area exceeds 75 km^2, the standard ensemble rainfall patterns from ARR 2016 do not apply to this catchment for storm durations longer than 12h. These were replaced with the areal temporal patterns for the Central Slopes region.
Preburst depths	Median preburst depths were downloaded from the ARR 2016 Data Hub for each sub-catchment. Preburst depths vary by design storm event AEP and duration. Preburst depths were applied to the model by reducing the initial losses for each storm event.

11.2.3 Hydrologic model validation

In the absence of data to calibrate the Canning Creek catchment, the hydrologic model was instead validated against an area-scaled FFA of an analogous catchment. The catchment on Macintyre Brook at Inglewood was suitable for this exercise since Canning Creek drains to this location approximately 14 km downstream. The FFA from the Inglewood Flood Study was adopted as well as one prepared by FFJV incorporating the additional stream gauge data post-2015.

Initial and continuing losses in the URBS model were adjusted until there was a reasonable match between the modelled flows and those from the scaled FFAs. Emphasis was placed on matching flows to those specified in the Inglewood Flood Study. It was noted that the flows in the Inglewood Flood Study's FFA were routinely 20% higher than the FFA generated by FFJV.

DTMR's Quantile Regression Technique (QRT) and the RFFE were included as an additional points of comparison. All URBS hydrologic model results between the 20% AEP and 1% AEP events fall within the 90% confidence limits of the RFFE and show a fair match with QRT flow estimates. The URBS model flows for the 1% AEP design event corroborate well with the scaled FFA and QRT estimates. The estimated flood flows at the outlet of the Canning Creek model are presented in Figure 63 and Table 11.4.

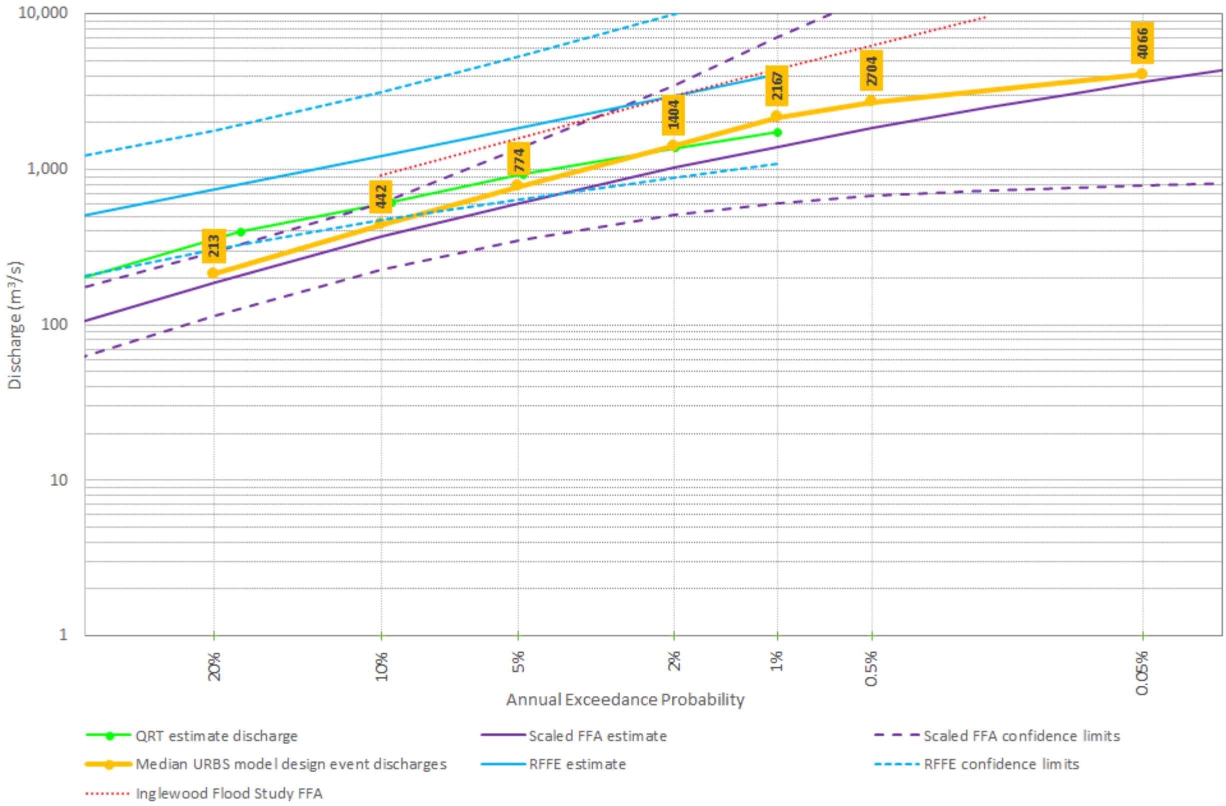


Figure 63 Estimate of flows at the Canning Creek model outlet

Table 11.4 Estimate of flows at the Canning Creek model outlet

AEP (%)	RFFE – lower bound 90% confidence level (m^3/s)	RFFE – estimate of flow (m^3/s)	RFFE – upper bound 90% confidence level (m^3/s)	DTMR quantile regression technique (m^3/s)[1]	URBS model flows (m^3/s)
20	18	43	105	34	213
10	27	70	184	60	442
5	37	107	303	94	774
2	52	172	563	146	1,404
1	65	237	860	193	2,167
1 in 2,000	-	-	-	-	4,066

Table note:

1 The QRT method estimates the 39.3%, 18.1%, 9.5% and 4.9% AEP instead of 50%, 20% and 10% and 5% respectively

The flood flows in Nicol Creek were estimated from the Canning Creek model with the areal reduction factor amended to suit the Nicol Creek catchment. All flows produced by the URBS between the 20% AEP and 1% AEP events reside within the 90% confidence limits of the RFFE and show a close match with QRT. The estimated flood flows at the proposed Nicol Creek crossing are presented in Figure 64 and Table 11.5.

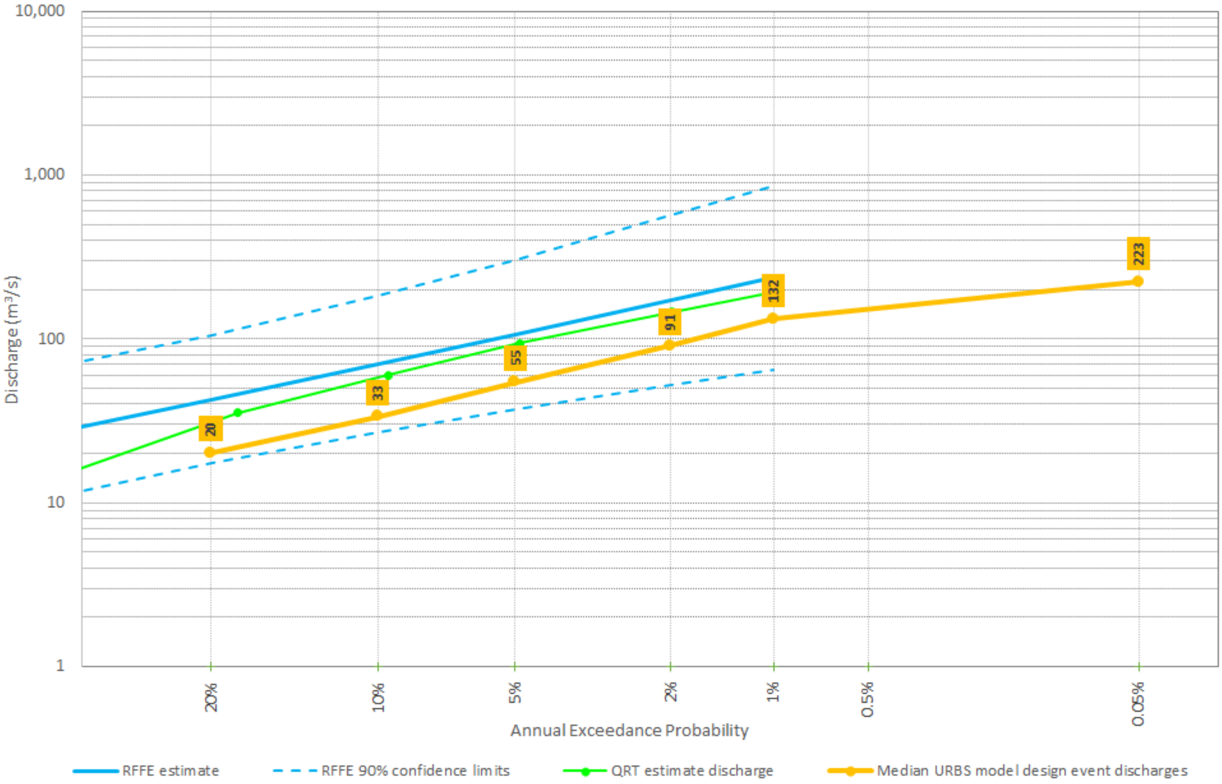


Figure 64 Estimate of flows at the Nicol Creek crossing

Table 11.5 Estimate of flows at the Nicol Creek crossing

AEP (%)	RFFE – lower bound 90% confidence level (m^3/s)	RFFE – estimate of flow (m^3/s)	RFFE – upper bound 90% confidence level (m^3/s)	DTMR quantile regression technique (m^3/s)[1]	URBS model flows (m^3/s)
20	18	43	105	34	20
10	27	70	184	60	33
5	37	107	303	94	55
2	52	172	563	146	91
1	65	237	860	193	132
1 in 2,000	-	-	-	-	223
1 in 10,000	-	-	-	-	394
PMF	-	-	-	-	1,884

Table note:

1 The QRT method estimates the 39.3%, 18.1%, 9.5% and 4.9% AEP instead of 50%, 20% and 10% and 5% respectively

11.3 Hydraulic model development – Nicol Creek

A two-dimensional modelling approach was adopted to simulate the flood regime around the proposed rail crossing at Nicol Creek. The platform used for hydraulic modelling was the TUFLOW HPC software package. The processes and assumptions adopted throughout the development of the hydraulic model are described in the following sections.

11.3.1 Model setup

The setup of the TUFLOW model is summarised in Table 11.6.

Table 11.6 Nicol Creek hydraulic model summary

Parameter	Information
Completion date	June 2019
AEPs assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF
Hydraulic model build	TUFLOW HPC GPU – version 2017-09-AC-w64-iSP
Model extent	Refer to Figure E-1d in Volume II – Appendix E
Grid size	5m
DEM (year flown)	ARTC dataset (2015).
Roughness	Spatially varying roughness values compliant with industry norms.
Eddy viscosity	Smagorinsky (default)
Model calibration	N/A
D/S model boundary	Height-discharge (HQ) boundary with normal slope approximated based upon topography dataset.
Hydraulic model timestep	Adaptive timestep
Hydraulic model wetting and drying depths	Cell centre set at 0.0002m Cell side set at 0.0001 m
Modelled scenarios	Existing Case, Developed Case
Sensitivity analysis	Climate change

The hydraulic model extent and the spatial distribution of land use in the 2D model domain is presented in Volume II – Appendix E, Figure E-1d, and the landuse classification in Figure E-1e.

11.3.2 Hydraulic structures

No existing hydraulic structures are situated within the extents of the hydraulic model.

11.4 Existing Case modelling results – Nicol Creek

11.4.1 Hydrologic model peak flow assessment

A critical duration assessment was undertaken to determine which storm duration(s) produced peak flood flows where the major waterways are intersected by the Project alignment and at the downstream outlet of the model. To assess the critical storm duration the following methodology was adopted:

- The models were modelled for a range of AEP events
  - Each AEP was modelled for a range of durations
  - Each duration was modelled for each of the ten associated temporal patterns
- A critical duration assessment was undertaken at the locations mentioned above to determine which duration produced the highest median flow of the ten temporal patterns for each event.

Table 11.7 presents the estimated 1% AEP event peak flows applied to the hydraulic model for a number of key locations.

Table 11.7 Peak flow at key locations as applied in the hydraulic model

AEP (%)	Peak flow (m^3/s)	Critical duration storm/temporal pattern
20	22	12 hour - Pattern 4
10	36	12 hour - Pattern 6
5	55	12 hour - Pattern 6
2	93	9 hour - Pattern 6
1	133	9 hour - Pattern 9
1 in 2,000	223	9 hour - Pattern 9
1 in 10,000	394	6 hour – PMP Temporal Pattern
PMF	1,884	6 hour – PMP Temporal Pattern

11.4.2 Existing Case flood maps

Maps illustrating indicative flood extents and peak water levels were prepared and are presented in Volume II – Appendix E:

- 20% AEP: Figure E-2a
- 10% AEP: Figure E-2b
- 5% AEP: Figure E-2c
- 2% AEP: Figure E-2d
- 1% AEP: Figure E-2e
- 1 in 2,000 AEP: Figure E-2f
- 1 in 10,000 AEP: Figure E-2g
- PMF: Figure E-2h.

Figure E-3a presents peak flood velocities under a 1% AEP event.

11.4.3 Flood inundation extent and flood levels

Figure E-2e in Volume II – Appendix E shows the 1% AEP indicative flood extent and peak water levels within the Nicol Creek floodplain for the Existing Case.

The peak flood depth is about 4.3 m in the Nicol Creek channel. This depth reduces to an average flood depth of approximately 0.6 m in other areas of the floodplain. The maximum flood depth on the floodplain is estimated to be up to 1.8 m (353.1m AHD) where the proposed alignment crosses the floodplain.

The model indicates that the time of inundation across the floodplain during the critical 1% AEP design flood is between 7 and 12 hours.

11.4.4 Flood immunity of existing infrastructure

Table 11.8 presents a summary of overtopping depths for key roads near the Project alignment under a range of design events. Modelling results show that Millmerran-Inglewood Road has an existing low flood immunity in the areas close to the Project alignment.

Table 11.8 Nicol Creek – Existing Case – overtopping depths of key infrastructure

Infrastructure	Location	Overtopping depth (m)							
		PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Millmerran-Inglewood Road		5.59	3.89	3.03	2.74	2.56	2.39	2.31	1.89

11.4.5 Existing Case velocities

Existing Case peak flood velocities are reach up to 4.6 m/s in localised areas with an average velocity across the floodplain of approximately 0.7 m/s as shown in Figure E3-a in Volume II – Appendix E.

11.5 Developed Case modelling results – Nicol Creek

11.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW model (1d and 2d approach).

In the Nicol Creek floodplain, the Project includes the following floodplain (or regional structures):

- One waterway bridge
- Four RCP locations (a total of 36 cells).

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and depending on their interaction with the Nicol Creek floodplain were incorporated in the hydraulic model.

The proposed drainage structures are summarised in Table 11.9 and Table 11.10. The 1% AEP flood level at the bridge is presented in Table 11.14 and shown in Figure E-1f in Volume II – Appendix E.

A minimum culvert size of 900 mm diameter was adopted to reduce potential for blockage and maintenance.

Bridges were modelled as an opening in the rail embankment. The optimisation of bridge lengths was balanced between minimising the changes to the hydraulic regime, primarily afflux and velocities, and the cost of replacing bridge spans by large earth embankments.

Blockage of hydraulic structures was assessed in accordance with ARR 2016. Despite the catchment being vegetated, ARR guidelines determined that the likelihood of significant amounts of debris accumulating against the piers of the bridge is low. Therefore, a zero blockage factor was applied at the Nicol Creek bridges.

Table 11.9 Nicol Creek – proposed floodplain culvert locations and details

Chainage (km)	Structure ID	Type	US invert (m AHD)	DS invert (m AHD)	Diameter (m)	Number of cells
104.94	C104.94	RCP	354.33	354.29	0.9	18
105.09	C105.09	RCP	354.34	354.14	0.9	6
105.11	C105.11	RCP	354.31	354.23	0.9	6
105.13	C105.13	RCP	354.39	354.25	0.9	6

Table 11.10 Nicol Creek - proposed bridge location and details

Chainage (km)	Structure ID	Approximate span (m)	Deck width (m)	Deck level (m AHD)	Deck superstructure type	Deck depth (mm)
104.39	310-BR11	92	3.97	356.5	Type D1	2,000

11.5.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

11.5.2.1 Rail immunity and structures results

Appendix C summarises the peak 1% AEP water levels on the upstream side of the proposed alignment. Local drainage structures (i.e. those not included in the flood model) and road culverts are not reported.

The results of flood modelling indicate that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the Nicol Creek floodplain. There is over 1.7 m freeboard to the rail formation.

11.5.2.2 Velocities and scour

Appendix C summarises the peak 1% AEP outlet flows and velocities at structures. The 1% AEP peak velocity through the proposed culverts is less than 1.3 m/s, whereas at the bridge, it is up to 2.6 m/s.

Scour protection requirements for culverts were calculated based on the velocities predicted from the hydraulic modelling. The scour protection was 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.

Table 11.11 lists the soil types encountered along the Project alignment and the allowable soil velocity based on AGRD.

Table 11.11 Allowable soil velocities along the Project alignment

Soil type	Soil description	Maximum allowable soil velocity as per AGRD (m/s)
Sodosols	Firm loam or stiff clay	2 m/s
Vertosols, Dermosols, Rudosols and Kandosols	Sandy or silty clay	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.

Scour protection requirements for culverts have been calculated based on the velocities predicted from the hydraulic modelling. Appendix D presents the proposed scour protection at each structure.

A conservative scour estimation has been undertaken at the bridge site based on available information and will be refined during detailed design.

11.5.2.3 Flood immunity for extreme events

The risk of overtopping of the rail alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF) with Table 11.12 presenting the depth of water above the formation level and over the top of rail at each structure. It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the alignment. As such, overtopping of the rail is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario.

Table 11.12 Nicol Creek – extreme events – depth of water above formation level and over top of rail level

Chainage (km)	Depth of water above formation level (m)			Depth of water over top of rail level (m)		
	1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
103.95 to 104.35	-	-	0.9	-	-	0.2
104.35 to 104.45	1.8	3.1	5.1	1.1	2.4	4.4
104.45 to 105.15	-	-	1.3	-	-	0.6

11.5.3 Flood impact objectives outcomes – Nicol Creek

The impact of the Developed Case was assessed through inclusion of the proposed rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, duration of inundation, velocities and peak flows within the floodplain.

- Changes in peak water levels for the AEP's assessed are presented in Figures E-4a to E-4h in Volume II – Appendix E
- Changes in 1% AEP duration of inundation are presented in Figure E-4i in Volume II – Appendix E
- Changes in 1% AEP velocities are presented in Figure E-4j in Volume II – Appendix E.

The effects of all impacts are required to be agreed with relevant stakeholders and affected landowners as part of the EIS process. One-one-one consultation with landowners who are expected to experience changes in flooding behaviour on the property was conducted by ARTC supported by FFJV.

The Project design outcomes relative to the flood impact objectives (refer Table 4.2) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Volume II – Appendix N.

The potential impacts to water levels across events up to and including the 1% AEP are summarised in Table 11.13.

Table 11.13 Afflux summary – Nicol Creek

Afflux outside rail disturbance footprint	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
Maximum afflux (mm)	-	62	39	100	126
Area afflux >10 mm experienced (ha)	-	<0.01	<0.01	1.1	2.6
Area afflux >200 mm experienced (ha)	-	-	-	-	-
Area afflux >400 mm experienced (ha)	-	-	-	-	-

11.5.3.1 Flood impacts at proposed hydraulic structures

The change in peak water levels at the proposed structures are presented in Table 11.14. Peak water levels were extracted upstream of culverts and at the control line of the bridge.

The design achieved a freeboard of at least 300 mm between the 1% AEP peak water and formation levels.

Table 11.14 Nicol Creek - 1% AEP event – estimated impacts to peak water levels at proposed hydraulic structures

Chainage (km)	Structure ID	Structure type	Rail formation level (m AHD)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Change in peak water level (mm)
104.39	310-BR11	Bridge	356.4	353.1	353.1	+50
104.94	C104.94	RCP	356.5	354.6	354.8	+210
105.09	C105.09	RCP	357.7	354.6	354.8	+250
105.11	C105.11	RCP	357.9	354.6	354.8	+200
105.13	C105.13	RCP	357.9	354.7	354.8	+150

11.5.3.2 Flood impacts on flood sensitive receptors

Based on the available aerial imagery, no buildings or critical infrastructure are located within the area impacted by afflux in the Nicol Creek floodplain for events up to the 1% AEP.

11.5.3.3 Flood impacts on state-controlled roads

The extent of the hydraulic model developed for Nicol Creek is shown in Figure 65. Within the extent of the hydraulic model, the only state-controlled road which is influenced by flooding and the Project alignment is the Millmerran-Inglewood Road. The location of the state-controlled road is shown in Figure 65.

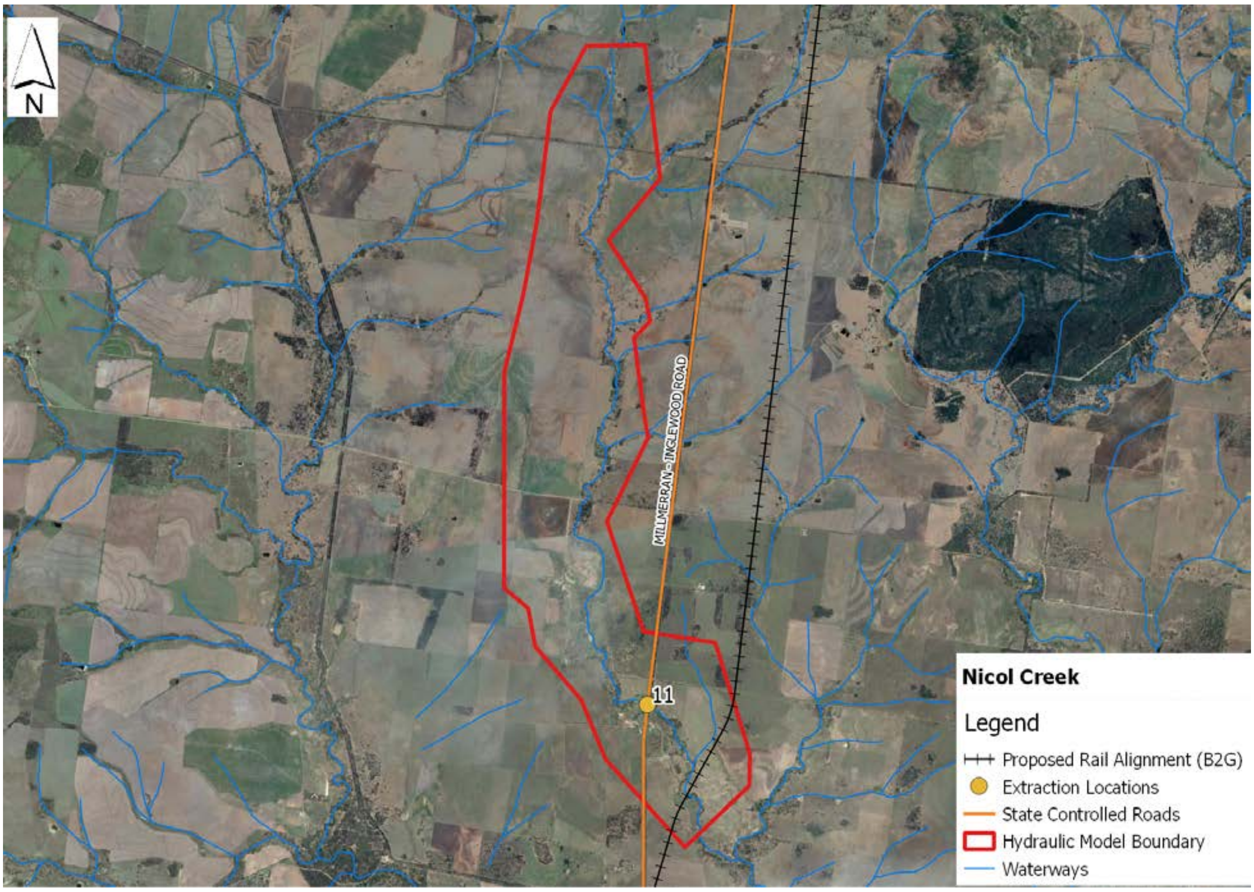


Figure 65 Nicol Creek Hydraulic Model Extent and Associated State-controlled Roads

The following sections describe the impacts to state-controlled roads in both the Existing Case and the Developed Case and summarises the differences between the two.

The flooding behaviour from Nicol Creek in regard to the impact on Millmerran-Inglewood road is confined to the crossing point. The cross drainage structure sufficiently passes the 20% AEP event; however, some overtopping is observed in the 10% AEP event. As such the road immunity in regard to Nicol Creek flooding is approximately 20% AEP.

Existing Case flooding conditions

Table 11.15 Nicol Creek – Existing Case flood depths

Reporting location	Road	Estimated depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
11	Millmerran-Inglewood Road	0.00	0.08	0.12	0.25	0.40	0.67	1.50	3.17

Table 11.16 Nicol Creek – Existing Case flood inundation length

Reporting location	Road	Approximate length of inundation (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
11	Millmerran-Inglewood Road	0	200	213	220	235	265	506	904

Table 11.17 Nicol Creek - Existing Case time of submergence

Reporting location	Road	Estimated time of submergence (hrs)								AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
11	Millmerran-Inglewood Road	0	3.4	3.9	6.3	8.3	9.3	6.4	7.5	0.7

Developed Case flooding conditions

Table 11.18 Nicol Creek - Developed Case flood depths

Reporting Location	Road	Estimated Depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
11	Millmerran-Inglewood Road	0.00	0.08	0.12	0.25	0.40	0.67	1.50	3.17

Table 11.19 Nicol Creek - Developed Case flood inundation length

Reporting location	Road	Approximate length of inundation (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
11	Millmerran-Inglewood Road	0	200	213	220	239	262	508	904

Table 11.20 Nicol Creek - Developed Case time of submergence

Reporting location	Road	Estimated time of submergence (hrs)								AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
11	Millmerran-Inglewood Road	0.0	3.4	3.9	6.3	8.3	9.3	6.4	7.5	0.7

Impacts of Project alignment

Table 11.21 Nicol Creek - change in flood depths

Reporting location	Road	Estimated change in depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
11	Millmerran-Inglewood Road	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table 11.22 Nicol Creek - change in time of submergence

Reporting location	Road	Estimated change in time of submergence (hrs)								Estimated change in AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
11	Millmerran-Inglewood Road	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Change in flood hydrographs

Figure 66 presents the Developed Case and Existing Case water level time series for the 1% AEP event at extraction point 11, located along Millmerran-Inglewood Road. The hydrographs reflect the same levels in both Existing Case and Developed Case, and any differences in flood behaviours are negligible.

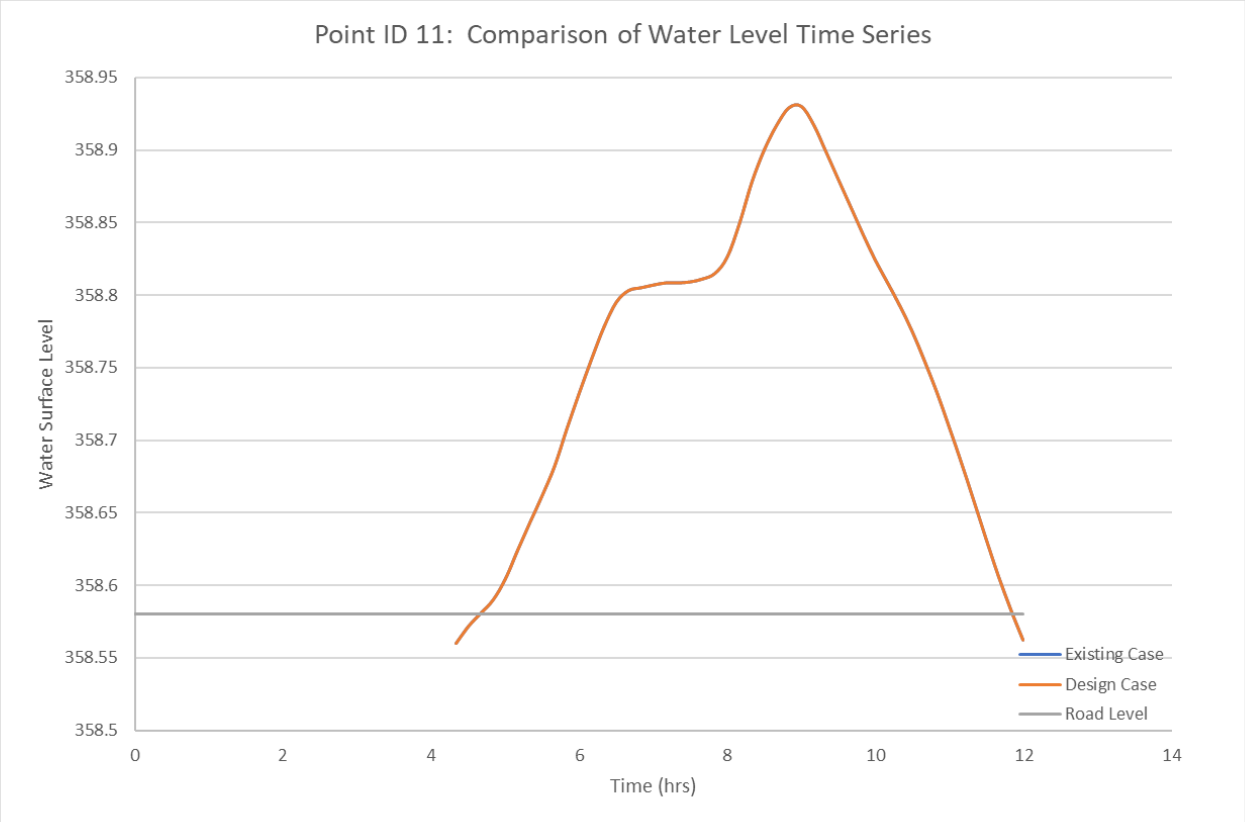


Figure 66 Extraction Point 11 - comparison of water level time series, 1% AEP

11.5.3.4 Flood impacts on local public roads

The change in peak water levels and flood hazard (velocity-depth) for the 1% AEP event were evaluated on local public roads within the hydraulic model domain. No local public roads are expected to experience an increase in flood hazard or increases in peak flood levels.

Duration of inundation

Assessment of the time of submergence (ToS) and average annual time of submergence (AAToS) was undertaken for local public roads. No local public roads within the hydraulic model domain are expected to experience increases in ToS or AAToS.

11.5.3.5 Flood impacts on private land outside the rail disturbance footprint

Most of the area where afflux is expected is agricultural land or open land on which nominal afflux is unlikely to cause any adverse impact. Table 11.23 presents the modelled changes in flood conditions during the 1% AEP event on a lot basis according to the following thresholds:

- Peak water levels increased by greater than +10 mm
- Peak velocities increased by greater than 0.25 m/s
- Duration of inundation changed by more than 25% of its original duration of inundation across the lot.

Table 11.23 Nicol Creek – summary of flood impacts on private land outside the rail disturbance footprint for 1% AEP

Approximate chainage (km)	Changes in peak water levels[1]		Changes in peak velocities		Changes in Duration of inundation (hrs)	
	Maximum change (mm)	Total area affected by change > 10 mm (ha)[2]	Maximum change (m/s)	Total area affected by change (ha)	Maximum change (%)	Total area affected by change (ha)
104.30	+66	2.2	-	-	-	-
104.90 to 105.20	+126	0.4	-	-	-	-

Table notes:

1 Afflux on lots that exceed the flood impact objectives are summarised in the EIS Surface Water Chapter
2 Only minor areas, usually directly upstream of culverts are affected by the maximum afflux as stated

11.5.3.6 Flow distribution

A key landowner concern is changes to flow distributions. However, Nicol Creek is well defined and there are no lateral breakouts of floodwater under events up to the 1% AEP events. Hence there are negligible changes to flow distribution in those events.

11.5.4 Sensitivity analysis – Nicol Creek

11.5.4.1 Blockage

A significant community concern is the potential impacts to flood conditions should the proposed culverts become blocked with debris. The primary concern is the blockage of culverts which 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% and 50% blockage.

Results of the blockage sensitivity analysis are presented in Table 11.24 and shown in Figure E-5a and E-5b in Volume II – Appendix E respectively. There is little difference between results with zero blockage and 25% blockage, but peak blockage increases significantly with 50% blockage.

Table 11.24 1% AEP Event – impacts on peak water levels due to different blockage factors

Afflux outside rail disturbance footprint	0% blockage	25% blockage (Developed Case)	50% blockage
Maximum afflux (mm)	112	126	157
Area afflux >10 mm experienced (ha)	2.6	2.6	2.7

Table 11.25 provides a summary of 1 % AEP peak flood levels at cross drainage structures for the blockage scenarios. There are no changes to impacts on flood sensitive receptors under the blockage scenarios.

Table 11.25 Nicol Creek – 1 % AEP event – culvert blockage assessment

Structure ID	Structure type	1 % AEP Peak water levels (m AHD)			Increase from Developed Case to 50% blockage scenario (mm)
		0% blockage	Developed Case (25% blockage)	50% blockage	
C104.94	RCP	354.8	354.8	354.9	+32
C105.09	RCP	354.8	354.8	354.8	+40
C105.11	RCP	354.8	354.8	354.9	+39
C105.13	RCP	354.8	354.8	354.9	+40

During detail 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.

11.5.4.2 Impacts during extreme events

Table 11.26 outlines the changes in peak water levels at flood sensitive receptors for the extreme events where the increase exceeds 10 mm under one of the events. The existing depth of flooding is also detailed and as can be seen the larger impacts that occur under the PMF event occur generally when there are already high flood depths as would be expected under such a rare event.

Flood immunity of the Project alignment is discussed in Section 11.5.2.3, and maps demonstrating the impacts during extreme events are shown in Volume II – Appendix E, Figures E-4f to E-4h.

Table 11.26 Nicol Creek – Summary of extreme event impacts at flood sensitive receptors

Flood sensitive receptor ID	1 in 2,000 AEP event		1 in 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)
NIC_ID_10	-	-	-	0.62	+15	2.10
NIC_ID_11	-	-	-	0.33	+15	1.74
NIC_ID_12	-	0.08	+16	0.67	+449	2.06
NIC_ID_13	-	-	+27	1.17	+424	2.65
NIC_ID_14	-	-	+8	0.70	+208	2.25
NIC_ID_15	-	-	-	-	+161	0.29
NIC_ID_16	-	-	-	0.34	+13	1.76
Millmerran - Inglewood Road	-	3.03	-	3.89	+22	5.59
Paton Road	-	0.16	-	0.50	+66	2.42

11.5.4.3 Climate change

The potential impacts of climate change in the Nicol Creek floodplain were assessed for the 1% AEP design event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2016. The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario has been adopted for the Project with an associated increase in rainfall intensity of 23.9% across the catchment area.

Table 11.27 presents the structure performance with Representative Concentration Pathways 8.5 climate change conditions. For the 1% AEP event, the change in peak water levels for the Representative Concentration Pathways 8.5 climate change scenario is presented in Figure E-5c in Volume II – Appendix E. Climate change results are expected to increase peak water levels upstream of the Project alignment by up to 0.2 m at structure locations for the 1% AEP event. The Project alignment is expected to retain 1% AEP flood immunity to formation level under the climate change scenario.

Table 11.27 Nicol Creek – 1% AEP event with Representative Concentration Pathways 8.5 conditions – structure performance

Structure ID	Structure type	1% AEP Peak water level (m AHD)	1% AEP +CC peak water level (m AHD)	Difference in peak water level (m)	Freeboard to rail formation with CC (m)
310-BR11	Bridge	353.1	353.3	+0.2	3.1
C104.94	RCP	354.8	354.9	+0.1	1.5
C105.09	RCP	354.8	355.0	+0.1	1.5
C105.11	RCP	354.8	355.0	+0.1	1.6
C105.13	RCP	354.8	355.0	+0.1	2.7

No flood sensitive receptors are detrimentally affected by the climate change scenario. The downstream extents of these impacts are similar to those under the 1% AEP event.

12 Bringalily Creek

Bringalily Creek is a well-defined water course with high sinuosity and is an upstream tributary of Canning Creek. Under the Existing Case 1% AEP event, the flood depth in Bringalily Creek channel is up to approximately 7 m. On the floodplain, in the vicinity of the Project alignment, flood depths range from 3 to 4.5 m. The Bringalily Creek flood inundation extent, under the 1% AEP event, varies between 500 m and 1 km wide.

The location of the Project rail alignment in relation to Bringalily Creek is shown in Figure F-1a in Volume II – Appendix F.

12.1 Data collation and review – Bringalily Creek

Bringalily Creek forms part of the Canning Creek system. Please refer to Section 11.1 for further details on data collation and review.

12.2 Hydrologic model development – Bringalily Creek

12.2.1 Model setup

Bringalily Creek forms part of the Canning Creek system. Please refer to Section 11.2 for further details on the development of the hydrologic model and associated modelling parameters.

The Project alignment crosses two major watercourses in the vicinity of Bringalily Creek. The catchment area upstream of the main Bringalily Creek crossing (310-BR10) is approximately 188 km^2. The catchment area upstream of the Project alignment crossing (310-BR08) of Bringalily Creek's minor tributary is approximately 14.5 km^2. This tributary is tailwater affected by flooding on Bringalily Creek generated by approximately 85 km^2 of upstream catchment.

12.2.2 Hydrologic model validation

The flood flows in Bringalily Creek were estimated from the Canning Creek model with the areal reduction factor amended to suit the Bringalily Creek catchment. All flows produced by the URBS between the 20% AEP and 1% AEP events reside within the 90% confidence limits of the RFFE and show a close match with QRT. The estimated flood flows at the proposed Bringalily Creek crossing (310-BR10) are presented in Figure 67 and Table 12.1.

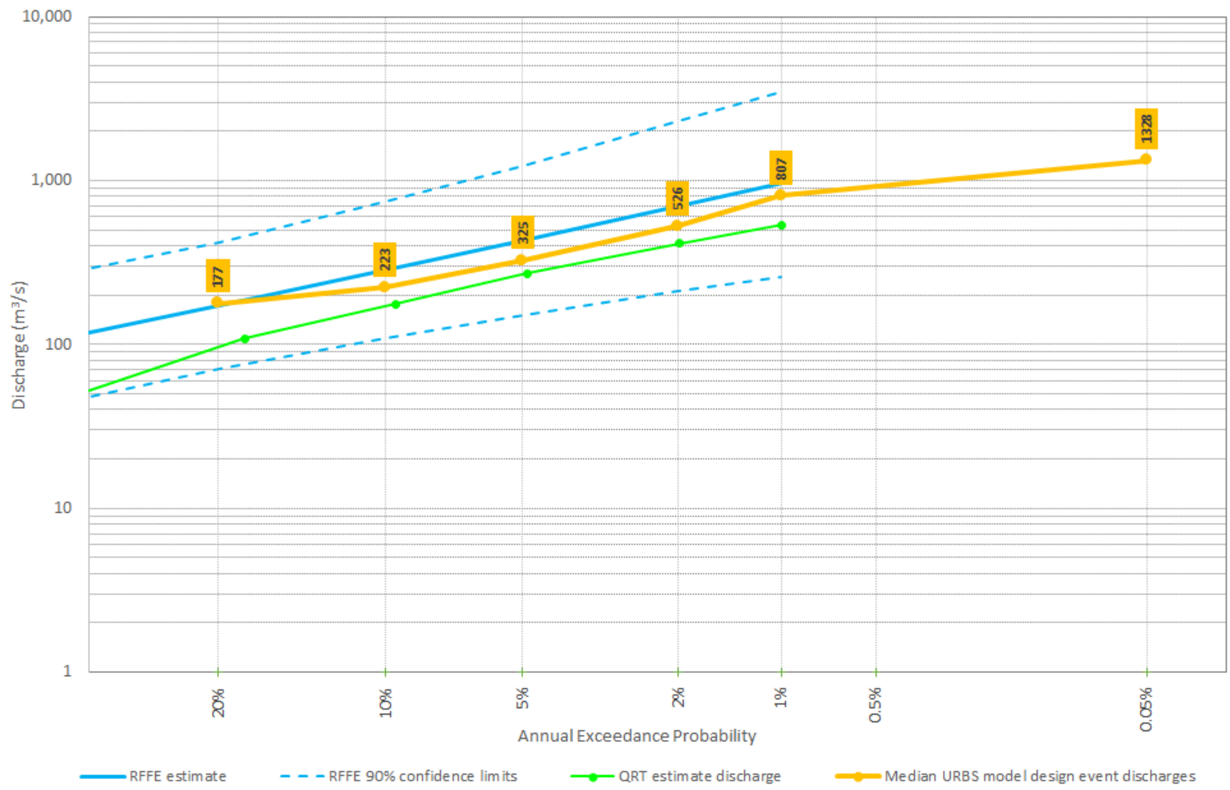


Figure 67 Estimate of flows at the Bringalily Creek crossing

Table 12.1 Estimate of flows at the Bringalily Creek crossing

AEP (%)	RFFE – lower bound 90% confidence level (m^3/s)	RFFE – estimate of flow (m^3/s)	RFFE – upper bound 90% confidence level (m^3/s)	DTMR quantile regression technique (m^3/s)[1]	URBS model flows (m^3/s)
20	71	172	422	109	177
10	109	284	741	176	223
5	150	431	1,220	271	325
2	210	694	2,280	411	526
1	259	957	3,490	534	807
1 in 2,000	-	-	-	-	1,328
1 in 10,000	-	-	-	-	2,678
PMF	-	-	-	-	8,552

Table note:

1 The QRT method estimates the 39.3%, 18.1%, 9.5% and 4.9% AEP instead of 50%, 20% and 10% and 5% respectively

12.3 Hydraulic model development – Bringalily Creek

A two-dimensional modelling approach was adopted to appropriately simulate flood mechanisms around the Project alignment crossing on Bringalily Creek. The platform used for hydraulic modelling is the TUFLOW HPC software package. The processes and assumptions adopted throughout the development of the hydraulic model are described in the following sections.

12.3.1 Model setup

The setup of the TUFLOW model is summarised in Table 12.2.

Table 12.2 Bringalily Creek hydraulic model summary

Parameter	Information
Completion date	June 2019
AEPs assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF
Hydraulic model build	TUFLOW HPC GPU – version 2017-09-AC-w64-iSP
Model extent	Refer to Figure F-1d in Volume II – Appendix F
Grid size	5m
DEM (year flown)	ARTC dataset (2015)
Roughness	Spatially varying roughness values compliant with industry norms.
Eddy viscosity	Smagorinsky (default)
Model calibration	N/A
D/S model boundary	Height-Discharge (HQ) Boundary with normal slope approximated based upon topography dataset.
Hydraulic model timestep	Adaptive Timestep
Hydraulic model wetting and drying depths	Cell centre set at 0.0002m Cell side set at 0.0001 m
Modelled scenarios	Existing Case, Developed Case
Sensitivity analysis	Blockage – 0%, 50% Climate change

The hydraulic model extent and the spatial distribution of land use in the 2D model domain is presented in Volume II – Appendix F, Figure F-1d and the landuse classification in Figure F-1e.

12.3.2 Hydraulic structures

The following existing structures were incorporated into the hydraulic model:

- One existing bridge on Millmerran-Inglewood Road (DTMR bridge 246)
- Eight existing culverts on Millmerran-Inglewood Road.

In the absence of as-constructed data, the dimensions and number of culvert barrels were estimated from aerial photography. Details of existing culverts are summarised in Table 12.3.

Table 12.3 Existing culverts within the hydraulic model extent

Structure ID	Type	Cells	Width (mm)	Height (mm)	Length (m)	US invert (m AHD)	DS invert (m AHD)
TMR_001	RCBC	6	1800	600	8.0	329.22	329.01
TMR_002	RCBC	3	1200	600	8.4	331.78	331.64
TMR_003	RCBC	1	1800	600	14.4	331.67	331.52
TMR_004	RCBC	2	1200	450	9.0	332.13	332.08
TMR_005	RCBC	2	1200	450	8.7	332.19	331.99

Approximated details of DTMR bridge are included in Table 12.4.

Table 12.4 Existing bridge at Bringalily Creek

Bridge ID	Approximate span (m)	Deck width (m)	Deck level (m AHD)	Deck superstructure type	Deck depth (mm)
TMR 246	30	8.60[1]	332.9	H20S16	1,050

Table note:

1 Estimated from aerial imagery

12.4 Existing Case modelling results – Bringalily Creek

12.4.1 Hydrologic model peak flow assessment

A critical duration assessment was undertaken to determine which storm duration(s) produced peak flood flows where the major waterways are intersected by the Project alignment and at the downstream outlet of the model. To assess the critical storm duration the following methodology was adopted:

- The models were modelled for a range of AEP events:
  - Each AEP was modelled for a range of durations
  - Each duration was modelled for each of the 10 associated temporal patterns
- A critical duration assessment was undertaken at the locations mentioned above to determine which duration produced the highest median flow of the ten temporal patterns for each event.

Table 12.5 presents the estimated peak flows applied to the hydraulic model for key locations (Figure F-1d in Volume II – Appendix F.

Table 12.5 Peak flow at key locations as applied in the hydraulic model

AEP (%)	Peak flow (m^3/s)	Critical storm duration/temporal pattern
20	177	12 hour – Pattern 7
10	223	9 hour – Pattern 8
5	325	3 hour – Pattern 3
2	526	2 hour – Pattern 0
1	807	0.75 hour – Pattern 5
1 in 2,000	1,328	0.75 hour – Pattern 5
1 in 10,000	2,678	6 hour – PMP Temporal Pattern
PMF	8,552	6 hour – PMP Temporal Pattern

12.4.2 Existing Case flood maps

Flood maps illustrating indicative flood extents and peak water levels were prepared and are presented in Volume II – Appendix F:

- 20% AEP: Figure F-2a
- 10% AEP: Figure F-2b
- 5% AEP: Figure F-2c
- 2% AEP: Figure F-2d
- 1% AEP: Figure F-2e
- 1 in 2,000 AEP: Figure F-2f
- 1 in 10,000 AEP: Figure F-2g
- PMF: Figure F-2h.

Figure F-3a presents peak flood velocities under a 1% AEP event.

12.4.3 Flood inundation extent and flood levels

Figure F-2e in Volume II – Appendix F shows the 1% AEP indicative flood extent and peak water levels within the Bringalily Creek floodplain for the Existing Case.

The peak modelled flood depth is about 7.6 m within the main Bringalily Creek channel. This depth reduces to an average modelled flood depth of around 1.0 m in other areas of the floodplain. The park flood depth on the floodplain is estimated to be up to 5.5 m (333.8m AHD) and 4.8 m (328.6m AHD) where the proposed alignment bridges 310-BR10 and 310-BR08 cross the floodplain respectively.

The model indicates that the time of inundation across the floodplain during the critical 1% AEP design flood is between 7 to 12 hours.

12.4.4 Flood immunity of existing infrastructure

Table 12.6 presents a summary of overtopping depths for key roads near the Project alignment under a range of design events. Modelling results show that Millmerran-Inglewood Road along with Heckels Road and Forestry Road have a low existing flood immunity in the areas close to the Project alignment and overtops in five key locations.

Table 12.6 Bringalily Creek – Existing Case – overtopping depths of key infrastructure

Infrastructure location	Overtopping depth (m)							
	PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Millmerran-Inglewood Road – middle	6.99	4.27	3.01	2.46	2.36	1.81	1.51	1.28
Millmerran-Inglewood Road - south	5.83	3.19	2.34	1.99	1.89	1.51	1.24	0.99
Millmerran-Inglewood Road - north	5.39	2.67	1.42	0.86	0.78	0.24	-	-
Heckels Road	4.85	2.28	1.20	0.76	0.71	0.54	0.46	0.38
Forestry Road	3.38	0.82	0.36	0.32	0.39	0.28	0.26	0.26

12.4.5 Existing Case flood velocities

Peak flood velocities are predicted to reach 5.0 m/s in localised areas with average velocities on the floodplain of approximately 0.6 m/s as shown in Figure F3-a in Volume II – Appendix F.

12.5 Developed Case modelling results – Bringalily Creek

12.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW model (1D and 2D approach).

In the Bringalily Creek floodplain, the Project includes the following floodplain (or regional structures):

- Two waterway bridges
- Eight RCP locations (a total of 55 cells)
- Three RCBC locations (a total of 28 cells).

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and depending on their interaction with the Bringalily Creek floodplain were incorporated in the hydraulic model.

The proposed drainage structures are summarised Table 12.7 and Table 12.8, and shown in Figure F-1f in Volume II – Appendix F. The 1% AEP flood levels at each drainage structure are presented in Appendix C.

The bridge was modelled as an opening in the rail embankment. The optimisation of bridge length was balanced between minimising the changes to the hydraulic regime, primarily change in peak water levels and velocities, and the economics of using bridge spans versus earth embankments.

Blockage of hydraulic structures was assessed in accordance with ARR 2016. Despite the predominant vegetation in the catchment being heavily forested, no allowance for blockage at bridges was made, as the likelihood of significant amounts of debris accumulating against the piers of the bridge is considered low. Furthermore, the bridge deck is highly unlikely to catch debris. Therefore, blockage has not been considered for the Bringalily Creek bridge crossings.

Table 12.7 Bringalily Creek – proposed floodplain culvert locations and details

Chainage (km)	Structure ID	Type	U/S invert (m AHD)	D/S invert (m AHD)	Diameter/width (m)	Height (m)	Number of cells
100.00	C100.00	RCP	332.31	332.21	1.5	-	8
99.84	C99.84	RCP	332.04	331.90	0.9	-	14
99.38	C99.38	RCP	331.15	330.95	0.9	-	17
97.29	C97.29	RCP	327.69	327.40	0.9	-	2
98.87	C98.87	RCP	330.00	329.76	1.5	-	1
99.77	C99.77	RCP	331.80	331.68	1.5	-	1
98.36	C98.36	RCP	328.80	328.54	0.9	-	10
97.38	C97.38	RCP	327.69	327.40	0.9	-	2
96.20	C96.20	RCBC	325.06	324.98	2.4	1.2	8
94.91	C94.91	RCBC	323.38	323.26	2.1	0.9	5
95.07	C95.07	RCBC	322.00	321.95	2.4	1.5	15

Table 12.8 Bringalily Creek - proposed bridge locations and details

Chainage (km)	Structure ID	Span (approx.) (m)	Deck width (m)	Deck level (m AHD)	Deck superstructure type	Deck depth (mm)
97.58	310-BR08	299	3.97	334.1	Type D1	2,000
100.39	310-BR10	621	3.97	335.2	Type D1	2,000

Table note:

1 Estimated from aerial imagery

12.5.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

12.5.2.1 Rail immunity and structures results

Appendix C summarises the peak 1% AEP water levels on the upstream side of the proposed alignment. Local drainage structures (i.e. those not included in the flood model) and road culverts are not reported.

The results of flood modelling indicate that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the Bringalily Creek floodplain, and that 1% AEP peak water levels remain below the proposed rail formation level. There is over 1.1 m freeboard above the culvert obvert levels to the rail formation level.

12.5.2.2 Velocities and scour

Appendix C summarises the peak 1% AEP outlet flows and velocities at structures. The 1% AEP peak velocity through the proposed drainage structures is less than 2.5 m/s.

Scour protection requirements for culverts were calculated based on the velocities predicted from the hydraulic modelling. The scour protection was 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.

Table 12.9 lists the soil types encountered along the Project alignment and the allowable soil velocity based on AGRD.

Table 12.9 Allowable soil velocities along the Project alignment

Soil type	Soil description	Maximum allowable soil velocity as per AGRD (m/s)
Sodosols	Firm loam or stiff clay	2 m/s
Vertosols, Dermosols, Rudosols and Kandosols	Sandy or silty clay	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.

Scour protection requirements for culverts have been calculated based on the velocities predicted from the hydraulic modelling. Appendix D presents the proposed scour protection at each structure.

A conservative scour estimation has been undertaken at the bridge site based on available information and will be refined during detailed design.

12.5.2.3 Flood immunity for extreme events

The risk of overtopping of the rail alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF) with Table 12.10 presenting the depth of water above the formation level and over the top of rail at each structure. It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the alignment. As such, overtopping of the rail is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario.

Table 12.10 Bringalily Creek – extreme events – depth of water above formation level and over top of rail level

Chainage (km)	Depth of water above formation level (m)			Depth of water over top of rail level (m)		
	1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
95.70	-	-	2.6	-	-	1.9
95.80	<0.1	0.2	2.5	-	-	1.8
95.90	-	<0.1	2.5	-	-	1.8
96.00	-	-	2.4	-	-	1.7
96.10	<0.1	0.6	3.2	-	-	2.5
96.20	-	-	2.4	-	-	1.7

Chainage (km)	Depth of water above formation level (m)			Depth of water over top of rail level (m)		
	1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
96.30	-	<0.1	2.3	-	-	1.6
96.40	-	-	2.3	-	-	1.6
96.50	-	0.1	2.3	-	-	1.6
96.60 to 96.80	-	-	1.9	-	-	1.2
96.90 to 97.00	-	-	0.6	-	-	-
97.50 to 97.70	2.8	3.6	6.0	2.1	2.9	5.6
99.00 to 99.20	-	-	0.4	-	-	-
99.30 to 100.00	-	-	1.5	-	-	0.8
100.00 to 100.60	5.0	6.3	9.0	4.3	5.6	8.4
100.70 to 101.20	-	-	1.9	-	-	1.2
101.30 to 101.70	-	-	0.7	-	-	-

12.5.3 Flood impact objectives outcomes – Bringalily Creek

The impact of the Developed Case was assessed through inclusion of the proposed rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, duration of inundation, velocities and peak flows within the floodplain.

- Changes in peak water levels for the AEP's assessed are presented in Figures F-4a to F-4h in Volume II – Appendix F
- Changes in 1% AEP duration of inundation are presented in Figure F-4i in Volume II – Appendix F
- Changes in 1% AEP velocities are presented in Figure F-4j in Volume II – Appendix F.

The effects of all impacts are required to be agreed with relevant stakeholders and affected landowners as part of the EIS process. One-one-one consultation with landowners who are expected to experience changes in flooding behaviour on the property was conducted by ARTC supported by FFJV.

The Project design outcomes relative to the flood impact objectives (refer Table 4.2) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Volume II – Appendix N.

The potential impacts to water levels across events up to and including the 1% AEP are summarised in Table 12.11.

Table 12.11 Afflux summary – Bringalily Creek

Afflux outside rail disturbance footprint	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
Maximum afflux (mm)	180	106	150	288	372
Area afflux >10 mm experienced (ha)	2.3	7.7	14.9	79.5	101.2
Area afflux >200 mm experienced (ha)	-	-	-	<0.01	0.1
Area afflux >400 mm experienced (ha)	-	-	-	-	-

12.5.3.1 Flood impacts at proposed hydraulic structures

The estimated impacts on peak water levels at each proposed structure are presented in Table 12.12. Peak water levels are taken immediately upstream of each culvert and at the control line of each bridge.

After preliminary model runs, it was determined that a 400 m long longitudinal channel is necessary between Chainages 97.90 km and 98.30 km to mitigate impacts on the flooding regime. In this area, the Project rail embankment passes through a substantial existing drainage path and the longitudinal channel will preserve the drainage path.

The design achieved a freeboard of at least 300 mm between the 1% AEP peak water and formation levels.

Table 12.12 Bringalily Creek - 1% AEP event – peak water levels at proposed hydraulic structures

Chainage (km)	Structure ID	Structure type	Rail formation level/bridge deck height (m AHD)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Difference (mm)
100.00	C100.00	RCP	335.2	333.0	333.1	+90
99.84	C99.84	RCP	335.7	332.7	332.9	+180
99.38	C99.38	RCP	335.5	331.7	331.7	-20
97.29	C97.29	RCP	335.1	328.0	328.3	+300
98.87	C98.87	RCP	333.4	-[1]	331.1	+300
99.77	C99.77	RCP	334.7	332.5	332.9	+380
98.36	C98.36	RCP	335.5	329.8	329.8	-
97.38	C97.38	RCP	334.2	328.2	328.3	+160
96.20	C96.20	RCBC	334.0	325.7	325.7	+40
94.91	C94.91	RCBC	327.1	-[1]	-[1]	-[1]
95.07	C95.07	RCBC	325.0	323.4	323.5	+50
97.58	310-BR08	Bridge	335.2	328.6	328.5	-40
100.39	310-BR10	Bridge	335.7	333.8	333.8	+10

Table note:

1 Local drainage culverts included in flood model. These culverts are necessary for minor drainage paths

12.5.3.2 Flood impacts on flood sensitive receptors

Flood sensitive receptors were identified from aerial imagery. Details of where afflux is greater than 10 mm, for events up to the 1% AEP are summarised in Table 12.13. Impacted flood sensitive receptors are labelled in the impact figures in Volume II - Appendix F, Figures F-4a to F-4j.

Impacts to flood sensitive receptors that exceed the flood impact objectives are reported in the EIS Surface Water Chapter.

Table 12.13 Bringalily Creek – estimated impacts to peak water levels at flood sensitive receptors

Flood sensitive receptor ID	Description	Afflux > +/- 10 mm				
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
BRI_ID_1	Shed	-	-	-	-9	-34
BRI_ID_2	House	-	-	-	+5	-2
BRI_ID_3	Shed	-	-	-	-28	-59
BRI_ID_4	Shed	-	-	-	-25	-63
BRI_ID_18	Shed	-	-	-	-	+16

12.5.3.3 Flood impacts on state-controlled roads

The extent of the hydraulic model developed for Bringalily Creek is shown in Figure 68. Within the extent of the hydraulic model, the only state-controlled road which is influenced by flooding and the Project alignment is the Millmerran-Inglewood Road. The location of the state-controlled road is shown in Figure 68.

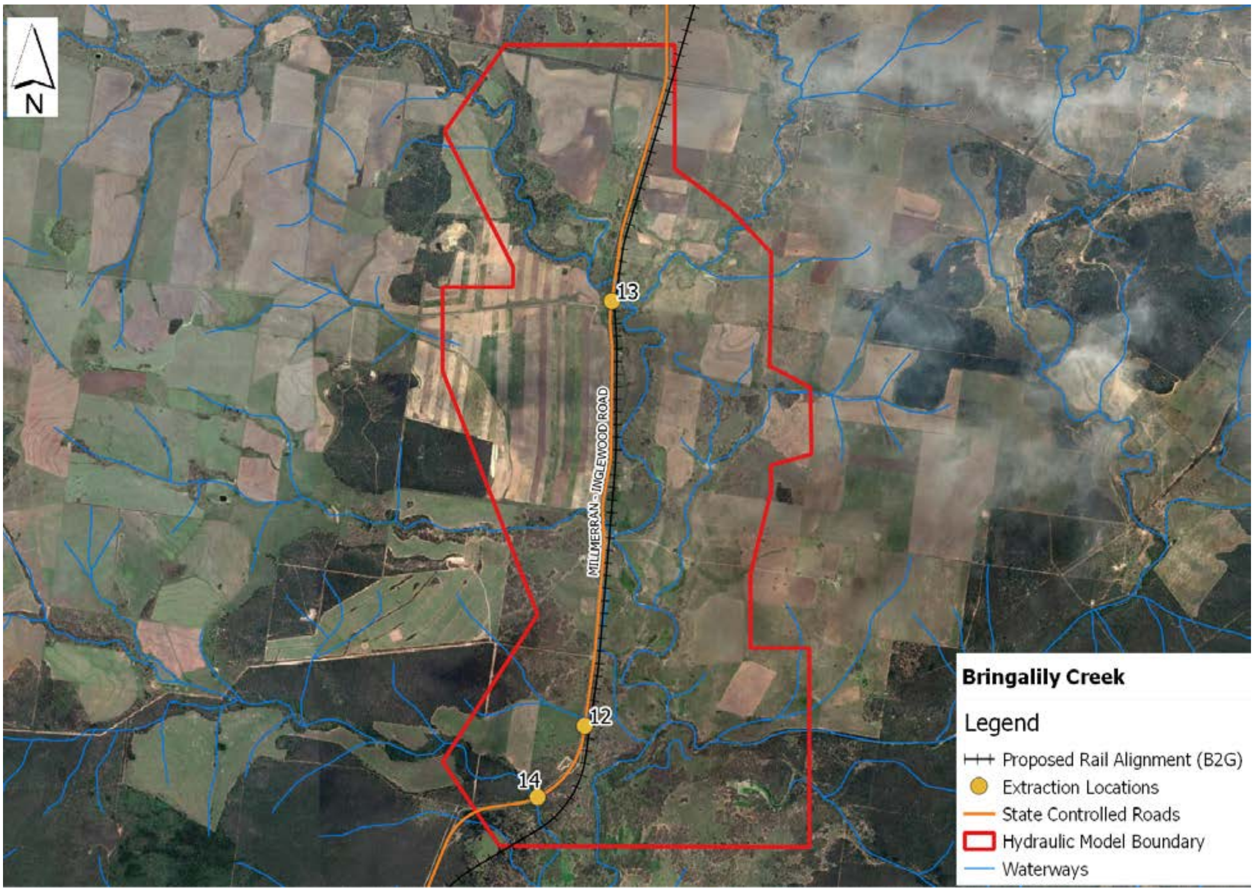


Figure 68 **Bringalily Creek Hydraulic Model Extent and Associated State-controlled Roads**

The following sections describe the impacts to state-controlled roads in both the Existing Case and the Developed Case and summarises the differences between the two. Millmerran-Inglewood road is typically overtopped in two locations. Point 12 in the following tables represents the southern section which is impacted, while point 13 represents the northern section.

The segment of Millmerran-Inglewood road through the Bringililly Creek extent has very low immunity in the Existing Case. Model results indicate that multiple sections of Millmerran-Inglewood road are overtopped in the 20% AEP event.

Existing Case flooding conditions

Table 12.14 **Bringalily Creek - Existing Case flood depths**

Reporting location	Road	Estimated depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
12	Millmerran-Inglewood Road	0.00	0.00	0.00	0.04	0.04	0.13	1.16	3.80
13	Millmerran-Inglewood Road	0.08	0.14	0.21	0.53	0.63	1.11	2.23	4.82

Table 12.15 Bringalily Creek - Existing Case flood Inundation length

Reporting location	Road	Approximate length of inundation (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
12	Millmerran-Inglewood Road	0	0	0	35	30	136	3,751	7,780
13	Millmerran-Inglewood Road	100	270	450	618	628	730	1,194	

Table 12.16 Bringalily Creek - Existing Case time of submergence

Reporting location	Road	Estimated time of submergence (hrs)								AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
12	Millmerran-Inglewood Road	0.0	0.0	0.0	11.8	9.1	9.8	6.5	8.2	0.4
13	Millmerran-Inglewood Road	2.1	4.4	10.7	15.8	10.0	10.6	8.6	8.8	2.2

Developed Case flooding conditions

Table 12.17 Bringalily Creek - Developed Case flood depths

Reporting location	Road	Estimated depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
12	Millmerran-Inglewood Road	0.00	0.00	0.00.	0.07	0.04	0.08	1.13	3.81
13	Millmerran-Inglewood Road	0.08	0.14	0.21	0.50	0.60	1.10	2.27	4.84

Table 12.18 Bringalily Creek - Developed Case flood inundation length

Reporting location	Road	Approximate length of inundation (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
12	Millmerran-Inglewood Road	0	0	0	36	25	66	3,758	7,780
13	Millmerran-Inglewood Road	100	270	450	610	628	746	1,990	

Table 12.19 Bringalily Creek - Developed Case time of submergence

Reporting location	Road	Estimated time of submergence (hrs)								AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
12	Millmerran-Inglewood Road	0.0	0.0	0.0	11.8	9.1	9.8	8.0	8.3	0.4
13	Millmerran-Inglewood Road	2.1	4.4	10.7	15.8	10.0	10.6	8.6	8.8	2.2

Impacts of Project alignment

Table 12.20 Bringalily Creek - change in flood depths

Reporting location	Road	Estimated change in depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
12	Millmerran-Inglewood Road	0.00	0.00	0.00	0.03	0.00	-0.05	-0.03	0.01
13	Millmerran-Inglewood Road	0.00	0.00	0.00	-0.03	-0.03	-0.01	0.04	0.02

Table 12.21 Bringalily Creek - change in time of submergence

Reporting location	Road	Estimated change in time of submergence (hrs)								Estimated change in AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
12	Millmerran-Inglewood Road	0.0	0.0	0.0	0.0	0.0	0.0	1.5	0.1	0.0
13	Millmerran-Inglewood Road	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Change in flood hydrographs

Figure 69 presents the Developed Case and Existing Case water level time series for the 1% AEP event at extraction point 13, located along Millmerran-Inglewood Road. The difference between the two scenarios is minimal, with the shape of the hydrograph being very similar and only minor differences in peak water levels being present.

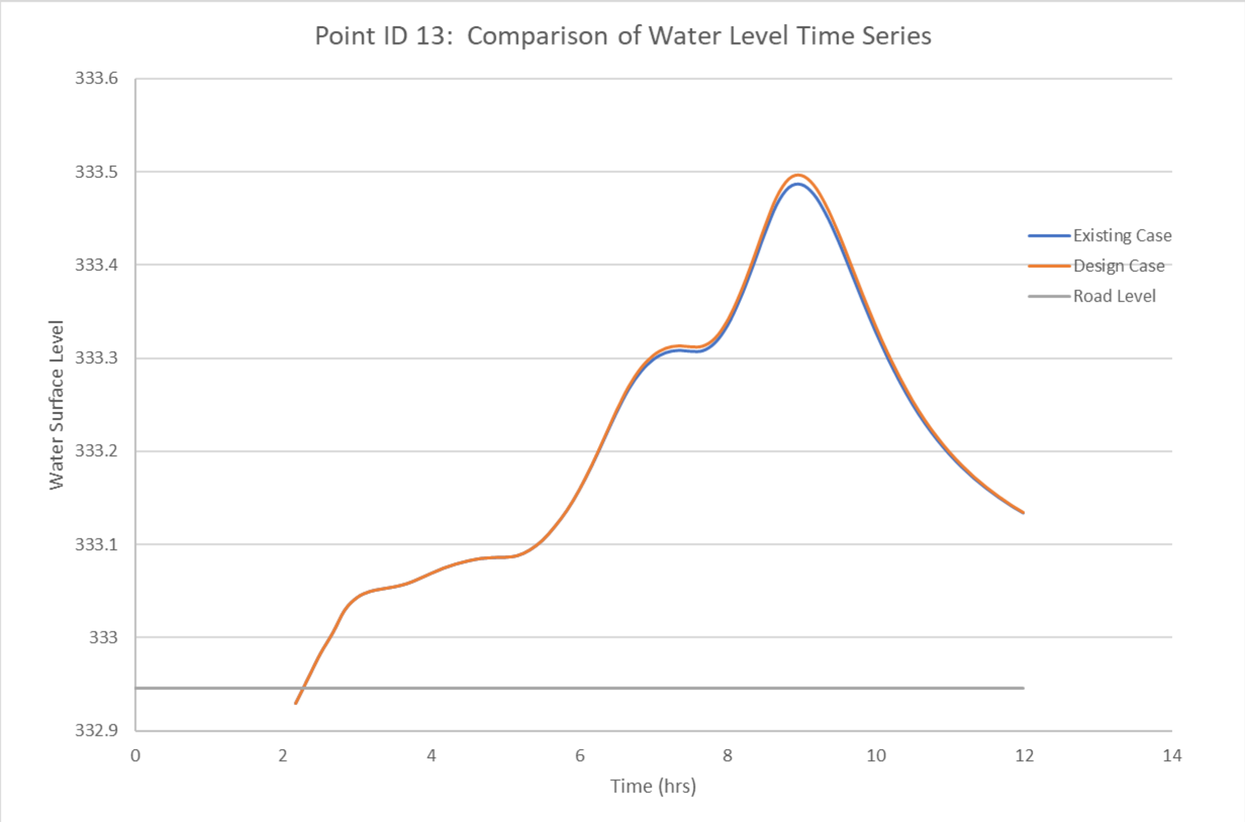


Figure 69 Extraction Point 13 – comparison of water level time series, 1% AEP

12.5.3.4 Flood impacts on local public roads

The change in peak water levels and flood hazard (velocity-depth) for the 1% AEP event were evaluated on local public roads within the hydraulic model domain. Local public roads that are expected to experience an increase in flood hazard and/or increases in peak flood levels are reported in Table 12.22.

Table 12.22 Bringalily Creek - changes in peak water levels and velocity depth and flood hazard for local public roads, 1% AEP

Location	Existing flood hazard (m^2/s)	Design flood hazard (m^2/s)	Maximum existing flood depth (m)	Maximum design flood depth (m)	Maximum change in peak water levels (mm)[1]
Forestry Road	0.15	0.15	0.28	0.28	-
Heckels Road	0.77	0.78	0.74	0.75	+12

Table note:

1 The maximum change in peak water level does not necessarily occur at the same location as where the existing and/or design maximum flood depth occur

Duration of inundation

Assessment of the time of submergence (ToS) and average annual time of submergence (AAToS) was undertaken for local public roads within the hydraulic model domain. Local public roads that are expected to experience an increase in ToS and/or AAToS are presented in Table 12.23.

Table 12.23 Bringalily Creek - ToS and AAToS for local public roads

Location	Existing 1% AEP ToS (hrs)	1% AEP ToS diff. (hrs)	2% AEP ToS diff. (hrs)	5% AEP ToS diff. (hrs)	10% AEP ToS diff. (hrs)	AAToS Existing Case (hrs)	AAToS Developed Case (hrs)	AAToS diff. (hrs)
Forestry Road	10.69	-	-	-	-	4.88	4.88	-
Heckels Road	11.47	-	-	-	-	5.25	5.25	-

12.5.3.5 Flood impacts on private land outside the rail disturbance footprint

Most of the area where changes in peak water levels are predicted to occurred is agricultural land or open land on which nominal afflux is unlikely to cause any adverse impact. Table 12.24 presents the modelled changes in flood conditions during the 1% AEP event on a lot basis according to the following thresholds:

- Peak water levels increased by greater than +10 mm
- Peak velocities increased by greater than 0.25 m/s
- Duration of inundation changed by more than 25% of its original duration of inundation across the lot.

Table 12.24 Bringalily Creek – summary of flood impacts on private land outside the rail disturbance footprint for 1% AEP

Approximate chainage (km)	Changes in peak water levels[1]		Changes in peak velocities		Changes in Duration of inundation (hrs)	
	Maximum change (mm)	Total area affected by change > 10 mm (ha)[2]	Maximum change (m/s)	Total area affected by change (ha)	Maximum change (%)	Total area affected by change (ha)
97.80 to 98.70	+372	25.6	+0.6	22.1	-	-
97.90 to 98.40	+255	34.1	-	-	-	-
97.70	+35	2.6	-	-	-	-
100.10 to 100.20	+59	3.2	-	-	-	-
96.10 to 97.40	+151	1.2	+0.3	2.7	-	-

Approximate chainage (km)	Changes in peak water levels[1]		Changes in peak velocities		Changes in Duration of inundation (hrs)	
	Maximum change (mm)	Total area affected by change > 10 mm (ha)[2]	Maximum change (m/s)	Total area affected by change (ha)	Maximum change (%)	Total area affected by change (ha)
100.30	+20	1.0	+0.3	0.1	-	-
100.30	+26	0.4	+0.3	0.1	-	-
97.80 to 98.10	+34	12.0	-	-	-	-
96.90 to 97.70	+150	1.4	+0.3	1.4	-	-
97.40 to 97.70	+30	18.9	-	-	-	-
95.10 to 95.30	-	-	+0.9	1.5	-	-
98.50 to 100.10	+15	0.2	-	-	-	-
95.50 to 96.00	+131	0.0	-	-	-	-
100.20	+27	1.5	-	-	-	-
100.10	+43	29.3	+0.4	14.9	-	-
100.00 to 100.20	+124	0.5	-	-	-	-

Table notes:

1 Afflux on lots that exceed the flood impact objectives are summarised in the EIS Surface Water Chapter

2 Only minor areas, usually directly upstream of culverts are affected by the maximum afflux as stated

12.5.3.6 Flow distribution

A key landowner concern is changes to flow distributions. However, Bringalily Creek is well defined, and the Project alignment crosses the creeks approximately perpendicularly. There are no lateral breakouts of floodwater in events up to the 1% AEP events and hence there are negligible changes to flow distribution in those events.

12.5.4 Sensitivity analysis – Bringalily Creek

12.5.4.1 Blockage

Blockage was assessed in accordance with ARR 2016. The blockage assessment was undertaken and resulted in a blockage factor of 25% adopted for culverts. A minimum culvert size of 900 mm diameter was adopted to reduce potential for blockage and maintenance. A significant community concern is the potential impacts to flood conditions should the proposed culverts become blocked with debris. The primary concern is the blockage of culverts which 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% and 50% blockage on culverts.

Results of the blockage sensitivity analysis are presented in Table 12.25 and shown in Figure F-5a and F-5b in Volume II – Appendix F respectively. There is little difference between results with zero blockage and baseline blockage, but peak blockage increases significantly with 50% blockage. However, the afflux footprint does not increase by a substantial amount, suggesting the topography constrains the afflux. Furthermore, the increased afflux is constrained to two properties.

Table 12.25 1% AEP event – impacts on peak water levels due to different blockage factors

Afflux outside rail disturbance footprint	0% blockage	25% blockage (Developed Case)	50% blockage
Maximum afflux (mm)	324	372	518
Area afflux >10 mm experienced (ha)	101.3	101.2	102.3
Area afflux >200 mm experienced (ha)	<0.1	0.1	0.2
Area afflux >400 mm experienced (ha)	-	-	0.01

Table 12.26 provides a summary of 1 % AEP peak flood levels at cross drainage structures for the blockage scenarios.

Table 12.26 Bringalily Creek – 1 % AEP event – culvert blockage assessment

Structure ID	Structure type	1 % AEP Peak water levels (m AHD)			Increase from Developed Case to 50% blockage scenario (mm)
		0% blockage	Developed Case (25% blockage)	50% blockage	
C100.00	RCP	333.1	333.1	333.1	+30
C99.84	RCP	332.9	332.9	333	+129
C99.38	RCP	331.7	331.7	331.7	+26
C97.29	RCP	328.3	328.3	328.3	+18
C98.87	RCP	Dry	Dry	Dry	N/A
C99.77	RCP	332.8	332.9	333	+146
C98.36	RCP	329.8	329.8	329.9	+36
C97.38	RCP	328.3	328.3	328.3	+21
C96.20	RCBC	325.7	325.7	325.7	-
C94.91	RCBC	Dry	Dry	Dry	N/A
C95.07	RCBC	323.5	323.5	323.5	-

Table 12.27 outlines the changes in peak water levels at flood sensitive receptors for the 50% blockage scenario where the increase exceeds 10 mm.

Table 12.27 Bringalily Creek – summary of 50% blockage impacts at flood sensitive receptors

Flood sensitive receptor ID	Existing case flood depth (m)	Change in peak water level (mm)
BRI_ID_18	0.54	+17
Heckels Road	0.94	+17
Millmerran - Inglewood Road	2.74	+342

During detail 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.

12.5.4.2 Impacts during extreme events

Table 12.28 outlines the changes in peak water levels at flood sensitive receptors for the extreme events where the increase exceeds 10 mm under one of the events. The existing depth of flooding is also detailed and as can be seen the larger impacts that occur under the PMF event occur generally when there are already high flood depths as would be expected under such a rare event.

Flood immunity of the Project alignment is discussed in Section 12.5.2.3, and maps demonstrating the impacts during extreme events are shown in Volume II – Appendix F, Figures F-4f to F-4h.

Table 12.28 Bringalily Creek – Summary of extreme event impacts at flood sensitive receptors

Flood sensitive receptor ID	1 in 2,000 AEP event		1 in 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)
BRI_ID_1	-29	0.55	+47	1.21	+193	3.54
BRI_ID_2	+4	0.68	+63	1.36	+217	3.66
BRI_ID_3	-17	0.63	+68	1.25	+176	3.64
BRI_ID_4	-33	0.59	+50	1.24	+187	3.60
BRI_ID_5	-	-	-	-	+26	2.48
BRI_ID_6	-	-	-	-	+27	2.01
BRI_ID_7	-	-	-	-	+28	1.39
BRI_ID_8	-	-	-	-	+28	1.70
BRI_ID_9	-	0.05	+2	0.18	+52	2.33
BRI_ID_10	-	-	-	0.18	+64	2.22
BRI_ID_11	-	-	-	0.11	+106	1.89
BRI_ID_12	-	-	-	-	+215	1.61
BRI_ID_13	-	-	-	-	+75	2.21
BRI_ID_14	-	-	-	-	+101	1.72
BRI_ID_15	-	-	-	-	+451	1.85
BRI_ID_16	-	-	-	-	+480	1.66
BRI_ID_17	-	-	-	0.41	+173	2.39
BRI_ID_18	+16	0.79	+8	1.87	+42	4.62
BRI_ID_19	-	0.00	+54	0.31	+141	3.05
BRI_ID_21	-17	0.45	-29	1.35	+12	3.90
Forestry Rd	+8	0.36	+165	0.82	+17	3.38
Heckels Rd	+56	1.20	+106	2.28	+328	4.85
Millmerran - Inglewood Road	+667	3.06	+1,215	4.34	+790	7.06

12.5.4.3 Climate change

The potential impacts of climate change in the Bringalily Creek floodplain were assessed for the 1% AEP design event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2016.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario has been adopted for the Project with an associated increase in rainfall intensity of 23.9% across the catchment area.

Table 12.29 presents the structure performance with Representative Concentration Pathways 8.5 climate change scenario is presented in Figure F-5c in Volume II – Appendix F.

Climate change results are expected to increase peak water levels upstream of the Project alignment by up to 0.46 m at structure locations for the 1% AEP event. The Project alignment is expected to retain 1% AEP flood immunity to formation level under the climate change scenario.

Table 12.29 Bringalily Creek – 1% AEP event Representative Concentration Pathways 8.5 conditions – structure performance

Structure ID	Structure type	1% AEP peak water level (m AHD)	1% AEP +CC peak water level (m AHD)	Difference in peak water level (m)	Freeboard to rail formation with CC (m)
C100.00	RCP	333.1	333.4	+0.29	2.3
C99.84	RCP	332.9	333.3	+0.42	2.2
C99.38	RCP	331.7	332.0	+0.31	3.2
C97.29	RCP	328.3	328.4	+0.12	4.9
C98.87	RCP	331.1	331.3	+0.19	3.4
C99.77	RCP	332.9	333.3	+0.46	2.1
C98.36	RCP	329.8	329.9	+0.12	4.3
C97.38	RCP	328.3	328.4	+0.12	5.6
C96.20	RCBC	325.7	326.1	+0.37	1.0
C94.91	RCBC	-[1]	323.6	+0.15	1.4
C95.07	RCBC	323.5	323.7	+0.22	0.9
310-BR08	Bridge	328.5	328.7	+0.13	5.45
310-BR10	Bridge	333.8	334.0	+0.25	1.81

Table note:

1 Dry

Table 12.30 outlines the changes in peak water levels at flood sensitive receptors for the climate change scenario where the increase exceeds 10 mm.

Table 12.30 Bringalily Creek – summary of climate change impacts at flood sensitive receptors

Flood sensitive receptor ID	1% AEP climate change event	
	Change in peak water level (mm)	Existing case flood depth (m)
BRI_ID_18	+19	0.54
Heckels Road[1]	+32	0.94
Millmerran - Inglewood Road[1]	+533	2.74

Table note:

1 These roads are affected by climate change regardless of the Project and so the amenity of the roads is not compromised by the Project

The downstream extents of these impacts are similar to those under the 1% AEP event.

13 Native Dog Creek

Native Dog Creek is a well-defined water course with well vegetated overbank areas. Native Dog Creek crosses Millmerran-Inglewood Road and is an upstream tributary of Canning Creek.

Under the Existing Case 1% AEP event, the flood depth in Native Dog Creek channel is up to approximately 3 m with depths of 1m on the floodplain area. The floodplain inundation extent is approximately 120 m wide.

The location of the Project rail alignment in relation to Native Dog Creek is shown in Figure G-1a in Volume II – Appendix G.

13.1 Data collation and review – Native Dog Creek

Native Dog Creek forms part of the Canning Creek system. Please refer to Section 11.1 for further details on data collation and review.

13.2 Hydrologic model development – Native Dog Creek

13.2.1 Model setup

Native Dog Creek forms part of the Canning Creek system. Please refer to Section 11.2 for further details on the development of the hydrologic model and parameters.

The catchment area upstream of the Project alignment crossing of Native Dog Creek is approximately 27 km^2.

13.2.2 Hydrologic model validation

The flood flows in Native Dog Creek were estimated from the Canning Creek model with the areal reduction factor amended to suit the Native Dog Creek catchment. All flows produced by the URBS between the 20% AEP and 1% AEP events reside within the 90% confidence limits of the RFFE and show a close match with QRT. The estimated flood flows at the proposed Native Dog crossing are presented in Figure 70 and Table 13.1.

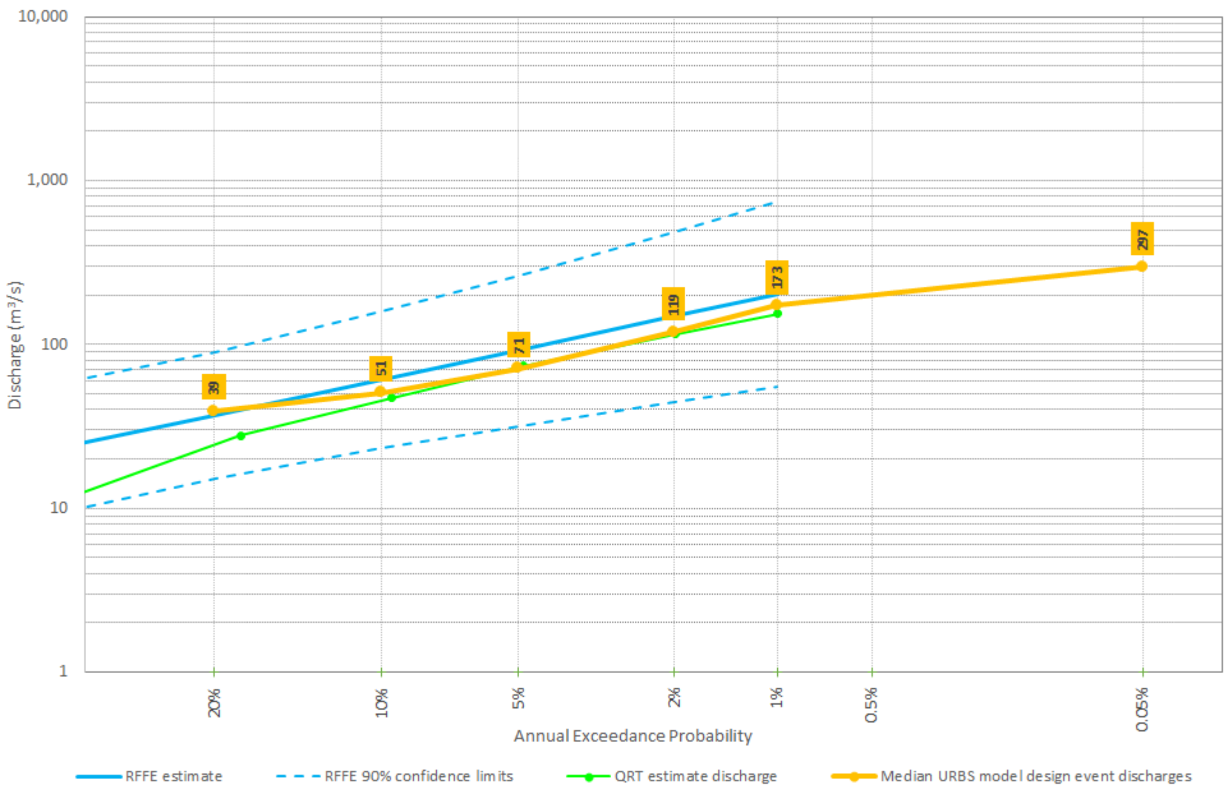


Figure 70 Estimate of flows at the Native Dog Creek crossing

Table 13.1 Estimate of flows at the Native Dog Creek crossing

AEP (%)	RFFE – lower bound 90% confidence level (m^3/s)	RFFE – estimate of flow (m^3/s)	RFFE – upper bound 90% confidence level (m^3/s)	DTMR quantile regression technique (m^3/s)[1]	URBS model flows (m^3/s)
20	15	37	90	28	39
10	23	60	158	47	51
5	32	92	261	74	71
2	45	148	486	116	119
1	55	204	745	154	173
1 in 2,000	-	-	-	-	297
1 in 10,000	-	-	-	-	600
PMF	-	-	-	-	2,544

Table note:

1 The QRT method estimates the 39.3%, 18.1%, 9.5% and 4.9% AEP instead of 50%, 20% and 10% and 5% respectively

13.3 Hydraulic model development – Native Dog Creek

A two-dimensional modelling approach was adopted to appropriately simulate flood mechanisms around the proposed rail crossing at Native Dog Creek. The platform used for hydraulic modelling is the TUFLOW HPC software package. The processes and assumptions adopted throughout the development of the hydraulic model are described in the following sections.

13.3.1 Model setup

The setup of the TUFLOW model is summarised in Table 13.2.

Table 13.2 Native Dog Creek hydraulic model summary

Parameter	Information
Completion date	June 2019
AEPs assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF
Hydraulic model build	TUFLOW HPC GPU – version 2017-09-AC-w64-iSP
Model extent	Refer to Figure G-1d in Volume II – Appendix G
Grid size	5m
DEM (year flown)	ARTC dataset (2015).
Roughness	Spatially varying roughness values compliant with industry norms.
Eddy viscosity	Smagorinsky (default)
Model calibration	N/A
D/S model boundary	Height-Discharge (HQ) Boundary with normal slope approximated based upon topography dataset.
Hydraulic model timestep	Adaptive Timestep
Hydraulic model wetting and drying depths	Cell centre set at 0.0002m Cell side set at 0.0001 m
Modelled scenarios	Existing Case, Developed Case
Sensitivity analysis	Climate change

The hydraulic model extent and the spatial distribution of land use in the 2D model domain is presented in Volume II – Appendix G, Figure G-1d and landuse classification in Figure G-1e.

13.3.2 Hydraulic structures

The following existing structure was incorporated into the hydraulic model:

- 1 existing bridge on Millmerran-Inglewood Road (DTMR bridge 24518)

Approximated details of DTMR bridge are included in Table 13.3.

Table 13.3 Existing bridge at Native Dog Creek

Bridge ID	Approximate span (m)	Deck width (m)	Deck level (m AHD)	Deck superstructure type	Deck depth (mm)
TMR 24518	56.9	8.60[1]	326.7	T44	1,050

Table note:

1 Estimated from aerial imagery

13.4 Existing Case modelling results – Native Dog Creek

13.4.1 Hydrologic model peak flow assessment

A critical duration assessment was undertaken to determine which storm duration(s) produced peak flood flows where the major waterways are intersected by the Project alignment and at the downstream outlet of the model. To assess the critical storm duration the following methodology was adopted:

- The models were modelled for a range of AEP events
  - Each AEP was modelled for a range of durations
  - Each duration was modelled for each of the ten associated temporal patterns
- A critical duration assessment was undertaken at the key locations to determine which duration produced the highest median flow of the ten temporal patterns for each event

Table 13.4 presents the estimated peak flow applied to the hydraulic model for a number of key locations (Figure G-1d in Volume II – Appendix G).

Table 13.4 Peak flow at key locations as applied in the hydraulic model

AEP (%)	Peak flow (m^3/s)	Critical storm duration/temporal pattern
20	39	12 hour – Pattern 7
10	51	9 hour – Pattern 2
5	71	9 hour – Pattern 8
2	119	3 hour – Pattern 5
1	173	2 hour – Pattern 7
1 in 2,000	297	1.5 hour – Pattern 7
1 in 10,000	600	6 hour – PMP Temporal Pattern
PMF	2,544	6 hour – PMP Temporal Pattern

13.4.2 Existing Case flood maps

Flood maps illustrating indicative flood extents and peak water levels were prepared and are presented in Volume II – Appendix G:

- 20% AEP: Figure G-2a
- 10% AEP: Figure G-2b
- 5% AEP: Figure G-2c
- 2% AEP: Figure G-2d
- 1% AEP: Figure G-2e
- 1 in 2,000 AEP: Figure G-2f
- 1 in 10,000 AEP: Figure G-2g
- PMF: Figure G-2h

Figure G-3a presents peak flood velocities under a 1% AEP event.

13.4.3 Flood inundation extent and flood levels

Figure G-2e in Volume II – Appendix G shows the 1% AEP indicative flood extent and peak water levels within the Native Dog Creek floodplain for the Existing Case.

The peak modelled flood depth is about 3.3 m within the main Native Dog Creek channel. This depth reduces to an average modelled flood depth of around 1 m in other areas of the floodplain. The maximum flood depth on the floodplain is estimated to be up to 1.6 m (321.5m AHD) where the proposed alignment crosses the floodplain.

The model indicates that the time of inundation across the floodplain during the critical 1% AEP design flood is between 2 to 4 hours.

13.4.4 Flood immunity of existing infrastructure

Table 13.5 presents a summary of overtopping depths for key roads near the Project alignment under a range of design events. Modelling results show that Millmerran-Inglewood Road has an existing low flood immunity in the areas close to the Project alignment.

Table 13.5 Native Dog Creek – Existing Case – overtopping depths of key infrastructure

Infrastructure location	Overtopping depth (m)							
	PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Millmerran-Inglewood Road	5.49	3.35	2.25	1.75	1.51	1.22	1.07	0.97

13.4.5 Existing Case velocities

Peak flood velocities are expected to reach 3.1 m/s in localised areas with the average velocity across the floodplain approximately 0.9 m/s as shown in Figure G3-a in Volume II – Appendix G.

13.5 Developed Case modelling results – Native Dog Creek

13.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW model (1d and 2d approach).

In the Native Dog Creek floodplain, the Project includes one waterway bridge.

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and depending on their interaction with the Native Dog Creek floodplain were incorporated in the hydraulic model.

The proposed bridge is summarised in Table 13.6 and shown in Figure G-1f in Volume II – Appendix G. The 1% AEP flood level at the proposed bridge is presented in Table 13.9.

Bridges were modelled as an opening in the rail embankment. The optimisation of bridge lengths was balanced between minimising the changes to the hydraulic regime, primarily afflux and velocities, and the cost of replacing bridge spans by large earth embankments.

Blockage of hydraulic structures was assessed in accordance with ARR 2016. Despite the catchment being vegetated, ARR guidelines determined that the likelihood of significant amounts of debris accumulating against the piers of the bridge is low. Therefore, a zero blockage factor was applied at the Native Dog Creek bridges.

Table 13.6 Native Dog Creek - proposed bridge location and details

Chainage (km)	Structure ID	Approximate span (m)	Deck width (m)	Deck level (m AHD)	Deck superstructure type	Deck depth (mm)
93.90	310-BR07	184	3.97	327.4	Type D1	2,000

13.5.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

13.5.2.1 Rail immunity and structures results

Appendix C summarises the peak 1% AEP water levels on the upstream side of the proposed alignment. Local drainage structures (i.e. those not included in the flood model) and road culverts are not reported.

The results of flood modelling indicate that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the Native Dog Creek floodplain, and that peak water levels remain below the proposed rail formation level. There is over 3.8 m freeboard to the bridge soffit.

13.5.2.2 Velocities and scour

Appendix C summarises the peak 1% AEP outlet flows and velocities at structures. The 1% AEP peak velocity through the proposed bridge is generally less than 1.2 m/s.

Scour protection requirements for culverts were calculated based on the velocities predicted from the hydraulic modelling. The scour protection was 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

Table 13.7 lists the soil types encountered along the Project alignment and the allowable soil velocity based on AGRD.

Table 13.7 Allowable soil velocities along the Project alignment

Soil type	Soil description	Maximum allowable soil velocity as per AGRD (m/s)
Sodosols	Firm loam or stiff clay	2 m/s
Vertosols, Dermosols, Rudosols and Kandosols	Sandy or silty clay	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.

Scour protection requirements for culverts have been calculated based on the velocities predicted from the hydraulic modelling. Appendix D presents the proposed scour protection at each structure.

A conservative scour estimation has been undertaken at the bridge site based on available information and will be refined during detailed design.

13.5.2.3 Flood immunity for extreme events

The risk of overtopping of the rail alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF) with Table 13.8 presenting the depth of water above the formation level and over the top of rail at each structure. It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the alignment. As such, overtopping of the rail is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario.

Table 13.8 Native Dog Creek – Extreme events – Depth of water above formation and top of rail levels

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
93.85 to 94.05	1.7	2.3	4.1	1.0	1.6	3.4
94.25 to 94.55	-	-	0.4	-	-	-
94.55 to 94.65	-	0.1	0.2	-	-	-
94.65 to 94.75	-	-	0.4	-	-	-
94.75 to 95.15	-	0.5	0.6	-	-	-
95.15 to 95.25	0.1	0.4	0.2	-	-	-
95.25 to 95.55	-	0.2	2.9	-	-	2.2
95.55 to 95.65	-	-	2.6	-	-	1.9

13.5.3 Flood impact objectives outcomes – Native Dog Creek

The impact of the Developed Case was assessed through inclusion of the proposed rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, duration of inundation, velocities and peak flows within the floodplain.

- Changes in peak water levels for the AEP's assessed are presented in Figures G-4a to G-4h in Volume II – Appendix G
- Changes in 1% AEP duration of inundation are presented in Figure G-4i in Volume II – Appendix G
- Changes in 1% AEP velocities are presented in Figure G-4j in Volume II – Appendix G.

All impacts are required to be agreed with relevant stakeholders and affected landowners as part of the EIS process. One-on-one consultation with landowners who are expected to experience changes in flooding behaviour on the property was conducted by ARTC supported by FFJV.

The Project design outcomes relative to the flood impact objectives (refer Table 13.9) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Volume II – Appendix N.

13.5.3.1 Flood impacts at proposed hydraulic structures

The estimated impacts on peak water levels at the proposed bridge structure is presented in Table 13.9. Peak water levels were extracted at the control line of the bridge.

The design achieved a freeboard of at least 300 mm between the 1% AEP peak water and formation levels.

Table 13.9 Native Dog Creek - 1% AEP event – peak water level at proposed structure

Chainage (km)	Structure ID	Structure type	Rail formation level (m AHD)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Change in peak water level (mm)
93.93	310-BR07	Bridge	327.4	321.5	321.5	+40

13.5.3.2 Flood impacts on flood sensitive receptors

Based on the available aerial imagery, no buildings or critical infrastructure are located within the area affected by afflux in the Native Dog Creek floodplain for events up to the 1% AEP.

13.5.3.3 Flood impacts on state-controlled roads

The extent of the hydraulic model developed for Native Dog Creek is shown in Figure 71. Within the extent of the hydraulic model, the only state-controlled road which is influenced by flooding and the Project alignment is the Millmerran-Inglewood Road. The location of the state-controlled road is shown in Figure 71.

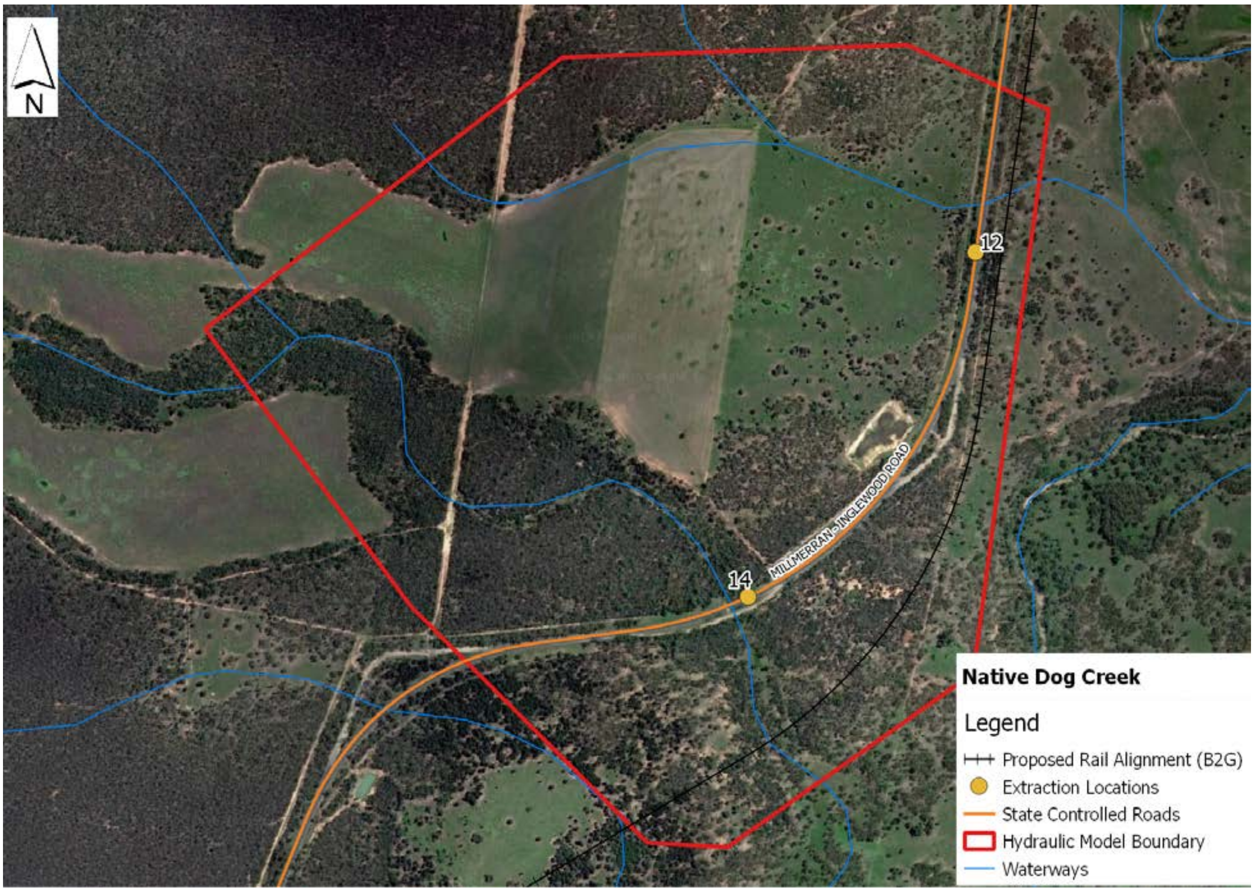


Figure 71 Native Dog Creek Hydraulic Model Extent and Associated State-controlled Roads

The following sections describe the impacts to state-controlled roads in both the Existing Case and the Developed Case and summarises the differences between the two.

Flooding from the Native Dog Creek catchment has minimal impact on the Millmerran-Inglewood road in both the existing or Developed Cases up until the extreme events. In both design and Existing Case, this segment of the road has greater than 1% AEP immunity.

Existing Case Flooding Conditions

Table 13.10 Native Dog Creek – Existing Case flood depths

Reporting location	Road	Estimated depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
14	Millmerran-Inglewood Road	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.56

Table 13.11 Native Dog Creek – Existing Case flood inundation length

Reporting location	Road	Approximate length of inundation (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
14	Millmerran-Inglewood Road	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1,635

Table 13.12 Native Dog Creek – Existing Case time of submergence

Reporting location	Road	Estimated time of submergence (hrs)								AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
14	Millmerran-Inglewood Road	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.7	0.0

Developed Case flooding conditions

Table 13.13 Native Dog Creek – Developed Case flood depths

Reporting location	Road	Estimated depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
14	Millmerran-Inglewood Road	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0. 65

Table 13.14 Native Dog Creek – Developed Case flood inundation length

Reporting location	Road	Approximate length of inundation (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
14	Millmerran-Inglewood Road	0	0	0	0	0	0	0	1,845

Table 13.15 Native Dog Creek – Developed Case time of submergence

Reporting location	Road	Estimated time of submergence (hrs)								AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
14	Millmerran-Inglewood Road	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.7	0.0

Impacts of Project alignment

Table 13.16 Native Dog Creek – change in flood depths

Reporting location	Road	Estimated change in depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
14	Millmerran-Inglewood Road	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table 13.17 Native Dog Creek – change in time of submergence

Reporting location	Road	Estimated change in time of submergence (hrs)								Estimated change in AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
14	Millmerran-Inglewood Road	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Change in flood hydrographs

As negligible change was present except for the extreme events, and the segment of Millmerran-Inglewood road within the Cattle Creek model extent is not inundated, no comparative plots of the water time series from the hydraulic model has been prepared.

13.5.3.4 Flood impacts on local public roads

The change in peak water levels and flood hazard (velocity-depth) for the 1% AEP event were evaluated on local public roads within the hydraulic model domain. No local public roads are expected to experience an increase in flood hazard or increases in peak flood levels.

Duration of inundation

Assessment of the time of submergence (ToS) and average annual time of submergence (AAToS) was undertaken for local public roads. No local public roads within the hydraulic model domain are expected to experience increases in ToS or AAToS.

13.5.3.5 Flood impacts on private land outside the rail disturbance footprint

There are negligible impacts to flood conditions on private properties outside the rail disturbance footprint.

13.5.3.6 Flow distribution

A key landowner concern is changes to flow distributions. However, Native Dog Creek is well defined and there are no lateral breakouts of flood waters under events up to the 1% AEP event. Hence there are negligible changes to flow distribution.

13.5.4 Sensitivity analysis – Native Dog Creek

13.5.4.1 Blockage

Blockage of hydraulic structures was assessed in accordance with ARR 2016. Despite the catchment being vegetated heavily forested, ARR guidelines determined that the likelihood of significant amounts of debris accumulating against the piers of the bridge is low. Therefore, a zero blockage factor was applied at the Native Dog Creek bridge. Additionally, there are no culverts in the Native Dog Creek floodplain, hence no sensitivity scenarios were conducted.

During detail 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.

13.5.4.2 Impacts during extreme events

Table 13.18 outlines the changes in peak water levels at flood sensitive receptors for the extreme events where the increase exceeds 10 mm under one of the events. The existing depth of flooding is also detailed and as can be seen the larger impacts that occur under the PMF event occur generally when there are already high flood depths as would be expected under such a rare event.

Flood immunity of the Project alignment is discussed in Section 13.5.2.3, and maps demonstrating the impacts during extreme events are shown in Volume II – Appendix G, Figures G-4f to G-4h.

Table 13.18 Native Dog Creek – summary of extreme event impacts at flood sensitive receptors

Flood sensitive receptor ID	1 in 2,000 AEP event		1 in 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)
Millmerran - Inglewood Road	-	2.25	+3	3.35	+1,230	5.49

13.5.4.3 Climate change

The potential impacts of climate change in the Native Dog Creek floodplain were assessed for the 1% AEP design event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2016.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario has been adopted for the Project with an associated increase in rainfall intensity of 23.9% across the catchment area.

Table 13.19 presents the structure performance with Representative Concentration Pathways 8.5 climate change scenario is presented in Figure G-5a in Volume II – Appendix G.

Climate change results are expected to increase peak water levels upstream of the Project alignment by up to 0.2 m at proposed bridge 310-BR07 for the 1% AEP event. The Project alignment is expected to retain 1% AEP flood immunity to formation level under the climate change scenario.

Table 13.19 Native Dog Creek – 1% AEP event with Representative Concentration Pathways 8.5 conditions – structure performance

Structure ID	Structure type	1% AEP peak water level (m AHD)	1% AEP +CC peak water level (m AHD)	Difference in peak water level (m)	Freeboard to rail formation with CC (m)
310-BR07	Bridge	321.5	321.7	+0.2	5.7

No flood sensitive receptors are detrimentally affected by the climate change scenario.

The downstream extents of these impacts are similar to those under the 1% AEP event.

14 Cattle Creek

Cattle Creek is a well-defined water course with minor breakout flow paths in the meandering sections of the creek. The creek system has well vegetated overbank areas which assists flow to remain within the main channel rather than breaking into overbank areas.

Under the Existing Case 1% AEP event, the flood depth in Cattle Creek channel is up to 4.5 m with approximately 1.5 m deep water on the floodplain area. The floodplain inundated extent is approximately 100 m wide.

The location of the Project rail alignment in relation to Cattle Creek is shown in Figure H-1a in Volume II – Appendix H.

14.1 Data collection and review – Cattle Creek

Cattle Creek forms part of the Canning Creek system. Please refer to Section 11.1 for further details on data collation and review.

14.2 Hydrologic model development – Cattle Creek

14.2.1 Model setup

Cattle Creek forms part of the Canning Creek system. Refer to Section 11.2 for further details on the development of the hydrologic model and parameters.

The catchment area upstream of the Project alignment crossing of Cattle Creek is approximately 65 km^2.

14.2.2 Hydrologic model validation

The flood flows in Cattle Creek were estimated from the Canning Creek model. The areal reduction factor was amended to suit the Cattle Creek catchment. All flows produced by the URBS between the 20% AEP and 1% AEP events reside within the 90% confidence limits of the RFFE and show a close match with QRT. The estimated flood flows at the proposed Cattle Creek crossing are presented in Figure 72 and Table 14.1.

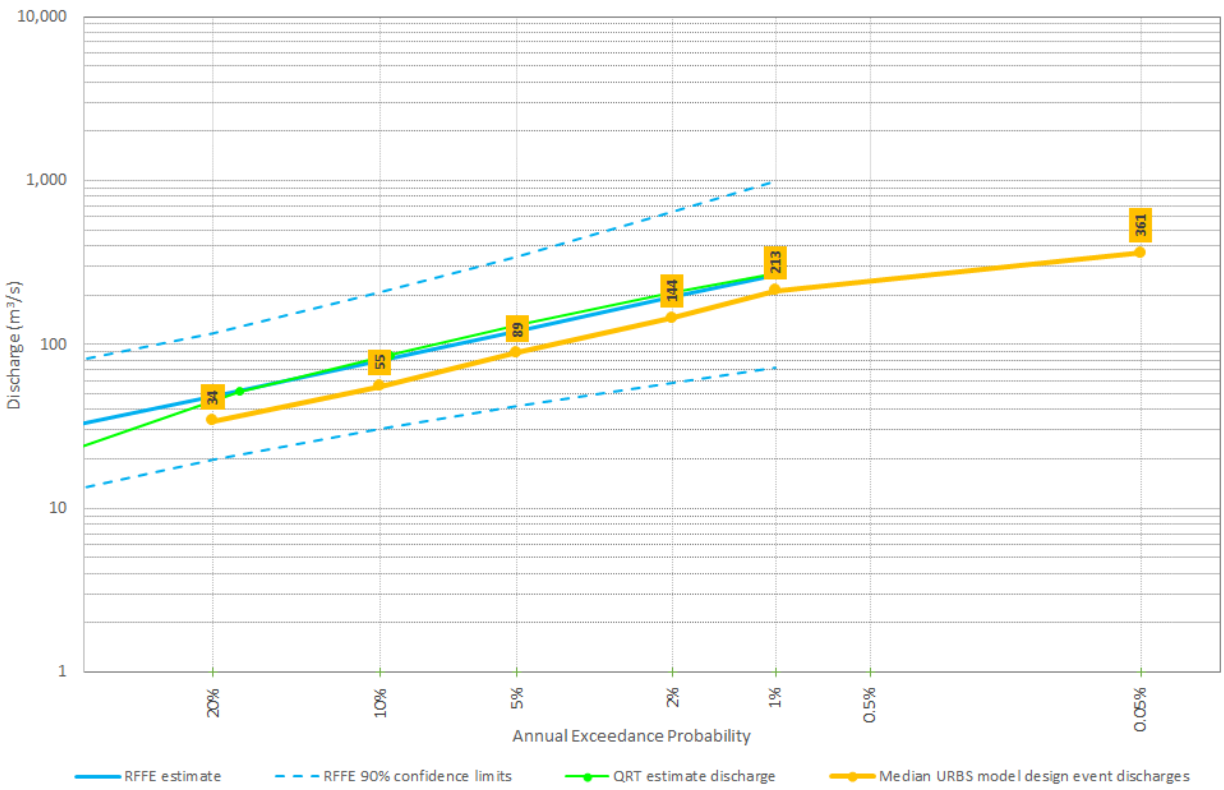


Figure 72 Estimate of flows at the Cattle Creek crossing

Table 14.1 Estimate of flows at the Cattle Creek crossing

AEP (%)	RFFE – lower bound 90% confidence level (m^3/s)	RFFE – estimate of flow (m^3/s)	RFFE – upper bound 90% confidence level (m^3/s)	DTMR quantile regression technique (m^3/s)[1]	URBS model flows (m^3/s)
20	20	48	118	52	34
10	31	79	207	86	55
5	42	120	343	134	89
2	58	194	640	206	144
1	72	267	982	271	213
1 in 2,000	-	-	-	-	361
1 in 10,000	-	-	-	-	941
PMF	-	-	-	-	3,770

Table note:

1The QRT method estimates the 39.3%, 18.1%, 9.5% and 4.9% AEP instead of 50%, 20% and 10% and 5% respectively

14.3 Hydraulic model development – Cattle Creek

A two-dimensional modelling approach was adopted to simulate the flood regime around the proposed rail crossing at Cattle Creek. The platform used for hydraulic modelling is the TUFLOW HPC software package. The processes and assumptions adopted throughout the development of the hydraulic model are described in the following sections.

14.3.1 Model setup

The setup of the TUFLOW model is summarised in Table 14.2.

Table 14.2 Cattle Creek hydraulic model summary

Parameter	Information
Completion date	June 2019
AEPs assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF
Hydraulic model build	TUFLOW HPC GPU – version 2017-09-AC-w64-iSP
Model extent	Refer to Figure H-1d in Volume II – Appendix H
Grid size	5m
DEM (year flown)	ARTC dataset (2015)
Roughness	Spatially varying roughness values compliant with industry norms.
Eddy viscosity	Smagorinsky (default)
Model calibration	N/A
D/S model boundary	Height-Discharge (HQ) Boundary with normal slope approximated based upon topography dataset.
Hydraulic model timestep	Adaptive Timestep
Hydraulic model wetting and drying depths	Cell centre set at 0.0002m Cell side set at 0.0001 m
Modelled scenarios	Existing Case, Developed Case
Sensitivity analysis	Climate change

The hydraulic model extent and the spatial distribution of land use in the 2D model domain is presented in Volume II – Appendix H, Figure H-1d and land use classification in Figure H-1e.

14.3.2 Hydraulic structures

The following existing structures were incorporated into the hydraulic model:

- One existing bridge on Millmerran-Inglewood Road (DTMR bridge 24519)

Assumed details of the DTMR bridge are included in Table 14.3.

Table 14.3 Existing bridge at Cattle Creek

Bridge ID	Approximate span (m)	Deck width (m)	Deck level (m AHD)	Deck superstructure type	Deck depth (mm)
TMR 24518	55	8.60[1]	326.7	T44	1,050

Table note:

1 Estimated from aerial imagery

14.4 Existing Case modelling results – Cattle Creek

14.4.1 Hydrologic model peak flow assessment

A critical duration assessment was undertaken to determine which storm duration(s) produced peak flood flows where the major waterways are intersected by the Project alignment and at the downstream outlet of the model. To assess the critical storm duration the following methodology was adopted:

- The models were modelled for a range of AEP events:
  - Each AEP was modelled for a range of durations
  - Each duration was modelled for each of the ten associated temporal patterns
- A critical duration assessment was undertaken at the locations mentioned above to determine which duration produced the highest median flow of the ten temporal patterns for each event.

Table 14.4 presents the estimated peak flow applied to the hydraulic model for a number of key locations (Figure H-1d in Volume II – Appendix H).

Table 14.4 Peak flow as applied in the hydraulic model

AEP (%)	Peak flow (m^3/s)	Critical storm duration/temporal pattern
20	34	12 hour - Pattern 4
10	55	12 hour - Pattern 6
5	89	12 hour - Pattern 4
2	144	12 hour - Pattern 6
1	213	9 hour - Pattern 9
1 in 2,000	361	9 hour - Pattern 9
1 in 10,000	941	6 hour – PMP Temporal Pattern
PMF	3,770	6 hour – PMP Temporal Pattern

14.4.2 Existing Case flood maps

Flood maps illustrating indicative flood extents and peak water levels were prepared and are presented in Volume II – Appendix H:

- 20% AEP: Figure H-2a
- 10% AEP: Figure H-2b
- 5% AEP: Figure H-2c
- 2% AEP: Figure H-2d
- 1% AEP: Figure H-2e
- 1 in 2,000 AEP: Figure H-2f
- 1 in 10,000 AEP: Figure H-2g
- PMF: Figure H-2h.

Figure H-3a presents peak flood velocities expected in a 1% AEP event.

14.4.3 Flood inundation extent and flood levels

Figure H-2e in Volume II – Appendix H shows the 1% AEP flood extent and peak water levels within the Cattle Creek floodplain for the Existing Case.

The peak modelled flood depth is about 4.9 m within the main Cattle Creek channel. This depth reduces to an average modelled flood depth of around 1.4 m in other areas of the floodplain. The park flood depth on the floodplain is estimated to be up to 3.0 m (324.1 m AHD) where the proposed alignment crosses the floodplain.

The model indicates that the time of inundation across the floodplain during the critical 1% AEP design flood is between 8 to 12 hours.

14.4.4 Flood immunity of existing infrastructure

Table 14.5 presents a summary of overtopping depths for key roads near the Project alignment under a range of design events. Modelling results show that Millmerran-Inglewood Road has an existing low flood immunity in the areas close to the Project alignment.

Table 14.5 Cattle Creek – Existing Case – overtopping depths of key infrastructure

Infrastructure	Overtopping depth (m)							
	PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Millmerran-Inglewood Road	6.17	4.10	3.17	2.55	2.22	1.87	1.65	1.46

14.4.5 Existing Case velocities

Peak flood velocities are expected to reach approximately 3.5 m/s in localised areas of the main creek channel, whereas the average velocity across the floodplain is approximately 0.9 m/s as shown in Figure H-3a in Volume II – Appendix H.

14.5 Developed Case modelling results – Cattle Creek

14.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW model (1D and 2D approach).

In the Cattle Creek floodplain, the Project includes the following floodplain (or regional structures):

- One waterway bridge
- One RCP location (a total of six cells)
- One RCBC location (a total of 15 cells)

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and depending on their interaction with the Cattle Creek floodplain were incorporated in the hydraulic model.

The proposed drainage structures are summarised in Table 14.6 and shown in Figure H-1f in Volume II – Appendix H. The 1% AEP flood levels at the structures are presented in Table 14.11.

Bridges were modelled as an opening in the rail embankment. The optimisation of bridge lengths was balanced between minimising the changes to the hydraulic regime, primarily afflux and velocities, and the cost of replacing bridge spans by large earth embankments.

Blockage of hydraulic structures was assessed in accordance with ARR 2016. Despite the catchment being vegetated, ARR guidelines determined that the likelihood of significant amounts of debris accumulating against the piers of the bridge is low. Therefore, a zero blockage factor was applied at the Cattle Creek bridges.

Table 14.6 Cattle Creek - proposed bridge locations and details

Chainage (km)	Structure ID	Approximate span (m)	Deck width (m)	Deck level (m AHD)	Deck superstructure type	Deck depth (mm)
88.28	310-BR06	138	3.97	331.0	Type D1	2,000

Table 14.7 Cattle Creek – proposed floodplain culvert locations and details

Chainage (km)	Structure ID	Type	US invert (m AHD)	DS invert (m AHD)	Diameter/ width (m)	Height(m)	Number of cells
87.37	C87.37	RCP	323.04	322.57	2.1	-	6
87.19	C87.19	RCBC	323.04	322.57	2.4	1.5	15

14.5.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

14.5.2.1 Rail immunity and structures results

Appendix C summarises the peak 1% AEP water levels on the upstream side of the proposed alignment. Local drainage structures (i.e. those not included in the flood model) and road culverts are not reported.

The results of flood modelling indicate that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the Cattle Creek floodplain, and that peak water levels remain below the proposed rail formation level. There is over 5.0 m freeboard to the bridge soffit level.

14.5.2.2 Velocities and scour

Appendix C summarises the peak 1% AEP outlet flows and velocities at structures. The 1% AEP peak velocity through the proposed bridge is generally less than 1.1 m/s.

Scour protection requirements for culverts were calculated based on the velocities predicted from the hydraulic modelling. The scour protection was 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.

Table 14.8 lists the soil types encountered along the Project alignment and the allowable soil velocity based on AGRD.

Table 14.8 Allowable soil velocities along the Project alignment

Soil type	Soil description	Maximum allowable soil velocity as per AGRD (m/s)
Sodosols	Firm loam or stiff clay	2 m/s
Vertosols, Dermosols, Rudosols and Kandosols	Sandy or silty clay	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.

Scour protection requirements for culverts have been calculated based on the velocities predicted from the hydraulic modelling. Appendix D presents the proposed scour protection at each structure.

A conservative scour estimation has been undertaken at the bridge site based on available information and will be refined during detailed design.

14.5.2.3 Flood immunity for extreme events

The risk of overtopping of the rail alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF) with Table 14.9 presenting the depth of water above formation level and over the top of rail at each structure.

Table 14.9 Cattle Creek - extreme events – depth of water above formation and top of rail levels

Chainage (km)	Depth of water above formation level (m)			Depth of water over 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
86.55 to 87.25	-	-	2.1	-	-	1.4
87.25 to 87.65	-	-	0.7	-	-	-
88.25 to 88.35	1.8	2.7	6.0	1.055	2.048	5.315

14.5.3 Flood impact objectives outcomes – Cattle Creek

The impact of the Developed Case was assessed through inclusion of the proposed rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, duration of inundation, velocities and peak flows within the floodplain:

- Changes in peak water levels for the AEP's assessed are presented in Figures H-4a to H-4h in Volume II – Appendix H
- Changes in 1% AEP duration of inundation are presented in Figure H-4i in Volume II – Appendix H
- Changes in 1% AEP velocities are presented in Figure H-4j in Volume II – Appendix H.

All impacts are required to be agreed with relevant stakeholders and affected landowners as part of the EIS process. One-one-one consultation with landowners who are expected to experience changes in flooding behaviour on the property was conducted by ARTC supported by FFJV.

The Project design outcomes relative to the flood impact objectives (refer Table 14.10) are presented in the following sections.

The potential impacts to water levels across events up to and including the 1% AEP are summarised in Table 14.10.

Flood sensitive receptors referred to in this section are shown in Volume II – Appendix N.

Table 14.10 Afflux summary Cattle Creek

Afflux outside rail disturbance footprint	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
Maximum afflux (mm)	234	231	252	309	338
Area afflux >10 mm experienced (ha)	0.1	0.1	0.1	0.1	0.1
Area afflux >200 mm experienced (ha)	<0.01	<0.01	<0.01	<0.01	<0.01
Area afflux >400 mm experienced (ha)	<0.01	<0.01	<0.01	<0.01	<0.01

14.5.3.1 Flood impacts at proposed hydraulic structures

The estimated impacts to peak water levels at each proposed structure are presented in Table 14.11. Peak water levels were extracted at the control line of each bridge.

The design achieved a freeboard of at least 300 mm between the 1% AEP peak water and formation levels.

Table 14.11 Cattle Creek – 1% AEP event – peak water level at proposed hydraulic structures

Chainage (km)	Structure ID	Structure type	Rail formation level (m AHD)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Change in peak water level (mm)
88.28	310-BR06	Bridge	331.1	324.1	324.1	-
87.37	C87.37	RCP	329.2	321.5	321.7	+210
87.19	C87.19	RCBC	328.7	323.2	323.2	+70

14.5.3.2 Flood impacts on flood sensitive receptors

Based on the available aerial imagery, no buildings or critical infrastructure are located within the area affected by afflux in the Cattle Creek floodplain for events up to the 1% AEP.

14.5.3.3 Flood impacts on state-controlled roads

The extent of the hydraulic model developed for Cattle Creek is shown in Figure 73. Within the extent of the hydraulic model, the only state-controlled road which is influenced by flooding and the Project alignment is the Millmerran-Inglewood Road. The location of the state-controlled road is shown in Figure 73.

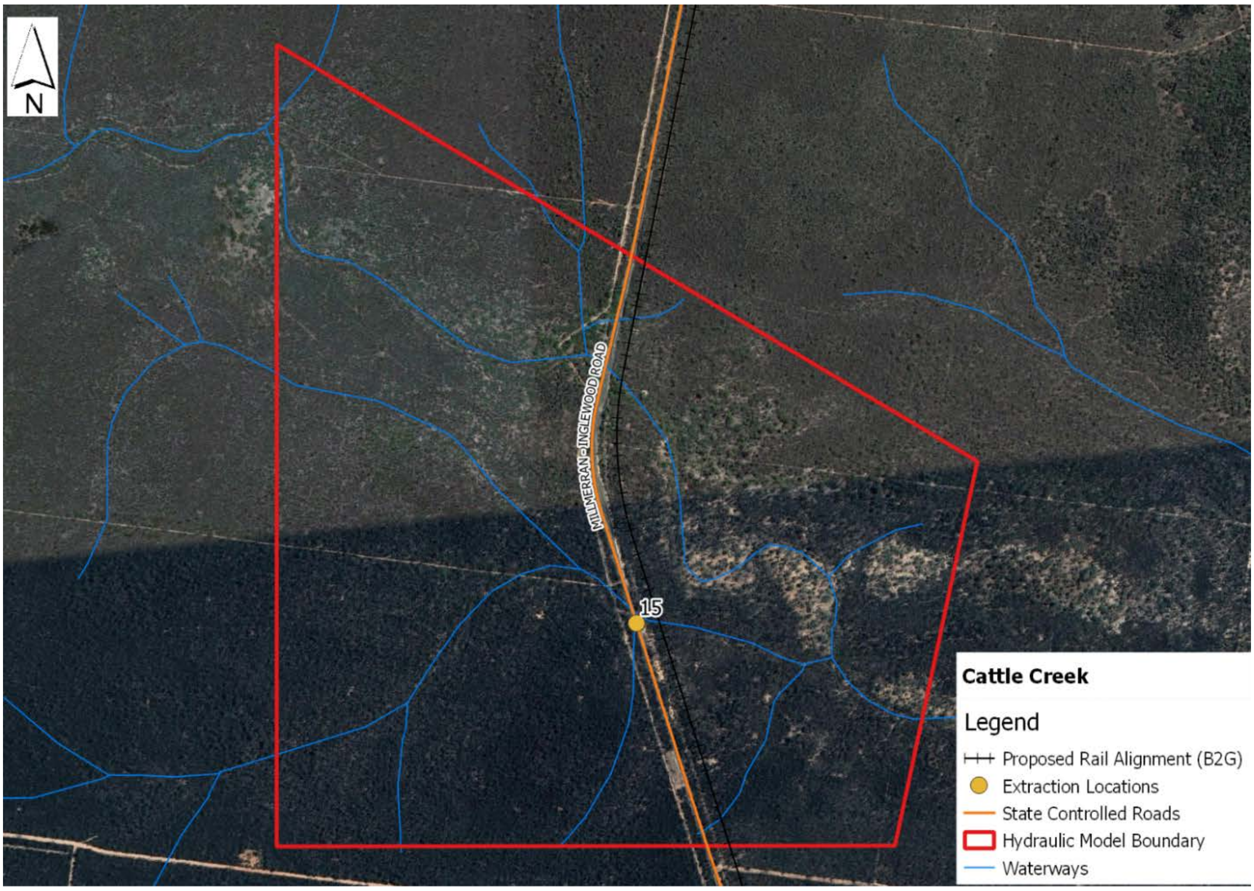


Figure 73 Cattle Creek Hydraulic Model Extent and Associated State-controlled Roads

The following sections describe the impacts to state-controlled roads in both the Existing Case and the Developed Case and summarises the differences between the two.

Flooding from the Cattle Creek catchment has minimal impact on the Millmerran-Inglewood road in both the existing or Developed Cases up until the extreme events. In both design and Existing Case, this segment of the road has greater than 1% AEP immunity.

Existing Case flooding conditions

Table 14.12 Cattle Creek - Existing Case flood depths

Reporting location	Road	Estimated depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
15	Millmerran-Inglewood Road	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.45

Table 14.13 Cattle Creek - Existing Case flood inundation length

Reporting location	Road	Approximate length of inundation (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
15	Millmerran-Inglewood Road	0	0	0	0	0	0	0	1,550

Table 14.14 Cattle Creek - Existing Case time of submergence

Reporting location	Road	Estimated time of submergence (hrs)								AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
15	Millmerran-Inglewood Road	0.0	0.0	0.0	0.0	0.0	0.0	0.0	3.0	0.0

Developed Case flooding conditions

Table 14.15 Cattle Creek - Developed Case flood depths

Reporting location	Road	Estimated depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
15	Millmerran-Inglewood Road	0.00	0.00	0.00	0.00	0.00	0.00	1.66	5.18

Table 14.16 Cattle Creek – Developed Case flood inundation length

Reporting location	Road	Approximate length of inundation (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
15	Millmerran-Inglewood Road	0	0	0	0	0	0	1,200	1,780

Table 14.17 Cattle Creek – Developed Case time of submergence

Reporting location	Road	Estimated time of submergence (hrs)								AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
15	Millmerran-Inglewood Road	0.0	0.0	0.0	0.0	0.0	0.0	2.2	6.1	0.0

Impacts of Project alignment

Table 14.18 Cattle Creek - change in flood depths

Reporting location	Road	Estimated change in depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
15	Millmerran-Inglewood Road	0.00	0.00	0.00	0.00	0.00	0.00	1.66	4.73

Table 14.19 Cattle Creek - change in time of submergence

Reporting location	Road	Estimated change in time of submergence (hrs)								Estimated change in AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
15	Millmerran-Inglewood Road	0.0	0.0	0.0	0.0	0.0	0.0	2.2	3.1	0.0

Change in flood hydrographs

As negligible change was present except for the extreme events, and the segment of Millmerran-Inglewood road within the Cattle Creek model extent is not inundated, no comparative plots of the water time series from the hydraulic model has been prepared.

14.5.3.4 Flood impacts on local public roads

The change in peak water levels and flood hazard (velocity-depth) for the 1% AEP event were evaluated on local public roads within the hydraulic model domain. No local public roads are expected to experience an increase in flood hazard or increases in peak flood levels.

Duration of inundation

Assessment of the time of submergence (ToS) and average annual time of submergence (AAToS) was undertaken for public roads. No local public roads within the hydraulic model domain are expected to experience increases in ToS or AAToS.

14.5.3.5 Flood impacts on private land outside the rail disturbance footprint

There are no impacts to flood conditions on private properties outside the rail disturbance footprint.

14.5.3.6 Flow distribution

A key landowner concern is changes to flow distributions. However, Cattle Creek is well defined and there are no lateral breakouts of flood water in events up to the 1% AEP event. Hence there are negligible changes to flow distribution.

14.5.4 Sensitivity analysis – Cattle Creek

14.5.4.1 Blockage

A significant community concern is the potential impacts to flood conditions should the proposed culverts become blocked with debris. The primary concern is the blockage of culverts which 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% and 50% blockage.

Results of the blockage sensitivity analysis are presented in Table 14.20 and shown in Figure H-5a and H-5b in Volume II – Appendix H respectively. There is little difference between results with zero blockage and baseline blockage, but peak blockage increases significantly with 50% blockage.

Table 14.20 1% AEP event – impacts on peak water levels due to different blockage factors

Afflux outside rail disturbance footprint	0% blockage	25% blockage (Developed Case)	50% blockage
Maximum afflux (mm)	+337	+338	+349
Area afflux >10 mm experienced (ha)	0.1	0.1	0.1

Table 14.21 provides a summary of 1 % AEP peak flood levels at cross drainage structures for the blockage scenarios. There are no changes to impacts on flood sensitive receptors under the blockage scenarios.

Table 14.21 Cattle Creek – 1 % AEP event – culvert blockage assessment

Structure ID	Structure type	1 % AEP Peak water levels (m AHD)			Increase from Developed Case to 50% blockage scenario (mm)
		0% blockage	Developed Case (25% blockage)	50% blockage	
C87.37	RCP	321.7	321.7	321.8	+44
C87.19	RCBC	323.2	323.2	323.2	+8

During detail 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.

14.5.4.2 Impacts during extreme events

Table 14.22 outlines the changes in peak water levels at flood sensitive receptors for the extreme events where the increase exceeds 10 mm under one of the events. The existing depth of flooding is also detailed and as can be seen the larger impacts that occur under the PMF event occur generally when there are already high flood depths as would be expected under such a rare event.

Flood immunity of the Project alignment is discussed in Section 14.5.2.3, and maps demonstrating the impacts during extreme events are shown in Volume II – Appendix H, Figures H-4f to H-4h.

Table 14.22 Cattle Creek – Summary of extreme event impacts at flood sensitive receptors

Flood sensitive receptor ID	1 in 2,000 AEP event		1 in 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)
Millmerran - Inglewood Road	-	3.17	+2,180	4.10	+4,953	6.17

14.5.4.3 Climate change

The potential impacts of climate change in the Cattle Creek floodplain were assessed for the 1% AEP design event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2016.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario has been adopted for the Project with an associated increase in rainfall intensity of 23.9% across the catchment area.

Table 14.23 presents the structure performance with Representative Concentration Pathways 8.5 climate change conditions. For the 1% AEP event, the change in peak water levels for the Representative Concentration Pathways 8.5 climate change scenario is presented in Figure H-5c in Volume II – Appendix H.

Climate change results are expected to increase peak water levels upstream of the Project alignment by up to 0.2 m at structure locations for the 1% AEP event. The Project alignment is expected to retain 1% AEP flood immunity to formation level under the climate change scenario.

Table 14.23 Cattle Creek – 1% AEP event with Representative Concentration Pathways 8.5 conditions – structure performance

Structure ID	Structure type	1% AEP peak water level (m AHD)	1% AEP +CC peak water level (m AHD)	Difference in peak water level (m)	Freeboard to rail formation with CC (m)
310-BR06	Bridge	324.1	324.3	+0.2	6.8
C87.37	RCP	321.7	321.9	+0.1	7.3
C87.19	RCBC	323.2	323.3	+0.1	5.4

No flood sensitive receptors are detrimentally affected by the climate change scenario.

The downstream extents of these impacts are similar to those under the 1% AEP event.

15 Pariagara Creek

Portions of Pariagara Creek and smaller tributaries are well defined channels, containing runoff from the adjacent hills. However closer to the Project alignment, the terrain flattens out and consequently more overland flow occurs. This is particularly prevalent between the Project alignment and Millmerran Inglewood road, where the topography is relatively flat and less vegetated.

Under the Existing Case 1% AEP event, the flood depth in the Pariagara Creek channel is up to approximately 5 m and approximately 1 m on the floodplain area. The Existing Case 1% AEP floodplain inundated extent is approximately 2.7 km wide where the Project alignment crosses.

The location of the Project rail alignment in relation to Pariagara Creek is shown in Figure I-1a in Volume II – Appendix I.

15.1 Data collection and review – Pariagara Creek

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:

- GRC – existing flood studies
- The BoM – rainfall data
- DTMR – existing infrastructure details.

15.1.1 Previous studies

A number of previous hydrology and hydraulic studies were sourced as part of this assessment. A review of each study was undertaken to determine suitability for use on the Project as documented in the following sections.

Goondiwindi Regional Council, Inglewood Flood Study, Engeny, 2015

Engeny was commissioned by GRC to undertake a flood study of Inglewood. The study objectives were "*to define the nature, extent and risks of flooding in Inglewood in order to inform disaster management planning and response, as well as control future development*" (Engeny, 2015). An URBS hydrologic model and TUFLOW (1D/2D) hydraulic model were developed.

15.1.2 Survey

ARTC provided LiDAR data from 2015 as 1 m grid DEM tiles. Using GIS software, a DEM was generated with a 1 m grid resolution for use in the Project based on the 2015 dataset. This was used for modelling within the disturbance footprint and up to the full extent of the 2015 LiDAR where relevant.

In areas that were not covered by the LiDAR provided by ARTC, LiDAR tiles were sourced from Geoscience Australia. The DEM datasets utilised for modelling were based on surveys flown between 2009 and 2015. SRTM data was used for catchment delineation where no LiDAR data could be sourced, to inform the hydrologic modelling.

The survey data sources and DEM developed for Pariagara Creek are shown in Figure I-1b in Volume II – Appendix I.

15.1.3 Aerial imagery

Aerial imagery of the study area was provided by ARTC and was used to identify and confirm topographic and vegetative characteristics of the study area. Aerial imagery captured in 2015 was made available. Additional imagery outside the study area was sourced from QGIS imagery in an open source format.

15.1.4 Existing drainage structure data

DTMR as-built drawings were also sourced for culvert and bridge details. This information will be refined as the local survey is complete.

15.1.5 Stream gauge data

No streamflow gauges exist within the Pariagara Creek catchment.

15.1.6 Rainfall data

A number of daily and sub-daily rainfall stations are located in and around the Pariagara Creek catchment. However, since there are no streamflow gauges to use for model calibration, no historical rainfall data was sought.

15.1.7 Anecdotal and observed flood data

No anecdotal or observed flood data was available for this area of Pariagara Creek.

15.1.8 Site inspection

A site inspection was undertaken during February 2018. During the site inspection, all major waterway crossings were visited and inspected with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

15.2 Hydrologic model development – Pariagara Creek

15.2.1 Model setup

A hydrologic model of Pariagara Creek was established in URBS using the latest procedures detailed in ARR 2016 and the latest rainfall data from BoM. The new ARR procedures cover revisions made to hydrologic parameters such as losses, pre-burst depths, temporal patterns and areal reduction factors.

The URBS model covers approximately 245 km^2 of the Pariagara Creek catchment upstream of its confluence with the Macintyre Brook. The catchment was delineated into 28 sub-catchments to capture the variability of rainfall and to better represent the network of creeks and streams within the catchment.

The hydrologic model setup including extent and sub-catchment map is presented in Volume II – Appendix I, Figure I-1c.

Model inputs are summarised in Table 15.1.

Table 15.1 Summary of URBS model inputs

Input parameter	Remarks
URBS model type	Basic
Routing variables	Catchment area, stream lengths
Channel lag parameter, α	1.20
Catchment non-linearity parameter, m	0.8

Note that the default values were adopted for all other URBS parameters.

15.2.2 Design event parameters

Hydrologic information to assist estimation of design event flows was sourced from the ARR 2016 Data Hub as summarised in Table 15.2.

Table 15.2 Summary of URBS model design event inputs

Input parameter	Remarks
Design rainfall	IFDs for each sub-catchment were downloaded from the BoM's website to account for variation in rainfall across the catchment.
Extreme event rainfall	PMP depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the method presented in the Bulletin 53 (BOM, 2003). The rainfall depths for the 1 in 10,000 AEP event were estimated using the interpolation method presented in ARR 2016 Book 8 Section 3.5.
Losses	The losses were adopted from the Inglewood Flood Study (2015) URBS model. Losses do not vary with AEP. Initial loss – 15 mm Continuing loss – 1 mm/h
Areal reduction factor	Parameters were adopted for the Semi-Arid Inland Queensland region. The catchment area U/S of the proposed rail crossing on Pariagara Creek is approximately 245 km^2, which yields an ARF between 64.5% and 93.3% depending on design storm event AEP and duration.
Ensemble temporal patterns	Central Slopes regions. However, as the study catchment area exceeds 75 km^2, the standard ensemble rainfall patterns from ARR 2016 do not apply to this catchment for storm durations longer than 12h. These were replaced with the areal temporal patterns for the Central Slopes region.
Preburst depths	Median preburst depths were downloaded from the ARR 2016 Data Hub for each sub-catchment. Preburst depths vary by design storm event AEP and duration. Preburst depths were applied to the model by reducing the initial losses for each storm event.

15.2.3 Hydrologic model validation

The hydrologic model for the Canning Creek catchment, which was used for the Pariagara Creek assessment was not calibrated due to unavailability of observed stream gauge data in the catchment. However, the routing parameter α was adjusted until there was a reasonable match between the URBS model flows and those derived using QRT and RFFE methods at the proposed Pariagara Creek crossing.

All URBS model results between the 50% AEP and 1% AEP events reside within the 90% confidence limits of the RFFE and show a close match with QRT. The estimated flood flows at the Project alignment crossing of Pariagara Creek are presented in Figure 74 and Table 15.3.

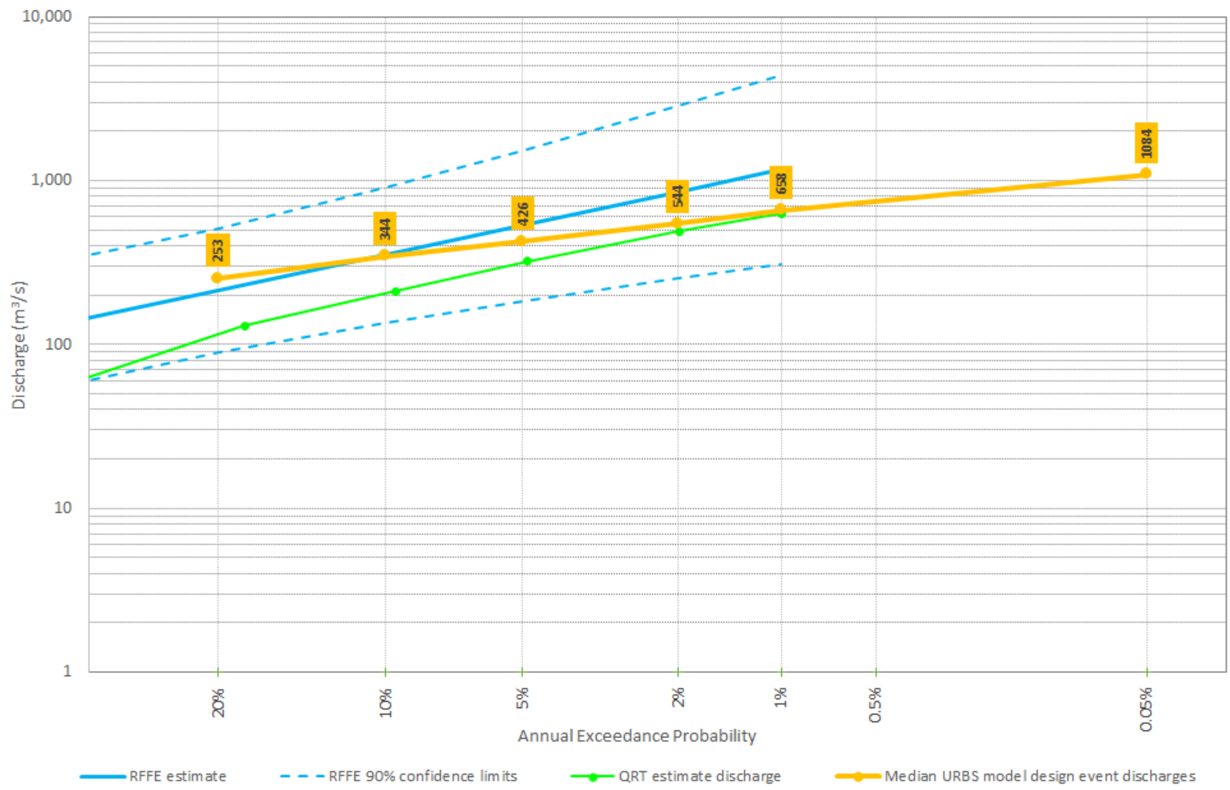


Figure 74 Estimate of flows at the Pariagara Creek crossing

Table 15.3 Estimate of flows at the Pariagara Creek crossing

AEP (%)	RFFE – lower bound 90% confidence level (m^3/s)	RFFE – estimate of flow (m^3/s)	RFFE – upper bound 90% confidence level (m^3/s)	DTMR quantile regression technique (m^3/s)[1]	URBS model flows (m^3/s)
20	89	213	512	130	253
10	136	351	906	210	344
5	185	532	1,520	323	426
2	255	855	2,850	488	544
1	311	1,180	4,400	631	658
1 in 2,000	-	-	-	-	1,084
1 in 10,000	-	-	-	-	2,247
PMF	-	-	-	-	8,055

Table note:

1 The QRT method estimates the 39.3%, 18.1%, 9.5% and 4.9% AEP instead of 50%, 20% and 10% and 5% respectively

15.3 Hydraulic model development – Pariagara Creek

A two-dimensional modelling approach was adopted to appropriately simulate flood mechanisms around the proposed rail crossing at Pariagara Creek. The platform used for hydraulic modelling is the TUFLOW HPC software package. The processes and assumptions adopted throughout the development of the hydraulic model are described in the following sections.

15.3.1 Model setup

The setup of the TUFLOW model is summarised in Table 15.4.

Table 15.4 Pariagara Creek hydraulic model summary

Parameter	Information
Completion date	June 2019
AEPs assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF
Hydraulic model build	TUFLOW HPC GPU – version 2017-09-AC-w64-iSP
Model extent	Refer to Figure I-1d in Volume II – Appendix I
Grid size	5m
DEM (year flown)	ARTC dataset (2015).
Roughness	Spatially varying roughness values compliant with industry norms.
Eddy viscosity	Smagorinsky (default)
Model calibration	N/A
D/S model boundary	Height-Discharge (HQ) Boundary with normal slope approximated based upon topography dataset.
Hydraulic model timestep	Adaptive Timestep
Hydraulic model wetting and drying depths	Cell centre set at 0.0002m Cell side set at 0.0001 m
Modelled scenarios	Existing Case, Developed Case
Sensitivity analysis	Blockage – 0%, 50% Climate change

The hydraulic model extent and the spatial distribution of land use in the 2D model domain is presented in Volume II – Appendix I, Figure I-1d, and landuse classification in Figure I-1e.

15.3.2 Hydraulic structures

No existing hydraulic structures are situated within the extent of the hydraulic model.

15.4 Existing Case modelling results – Pariagara Creek

15.4.1 Hydrologic model peak flow assessment

A critical duration assessment was undertaken to determine which storm duration(s) produced peak flood flows where the major waterways are intersected by the Project alignment and at the downstream outlet of the model. To assess the critical storm duration the following methodology was adopted:

- The models were modelled for a range of AEP events:
  - Each AEP was modelled for a range of durations
  - Each duration was modelled for each of the ten associated temporal patterns.

A critical duration assessment was undertaken at the locations mentioned above to determine which duration produced the highest median flow of the ten temporal patterns for each event

Table 15.5 presents the estimated peak flow applied to the hydraulic model for a number of key locations (Figure I-1d in Volume II – Appendix I).

Table 15.5 Peak flow as applied in the hydraulic model

AEP (%)	Peak flow (m^3/s)	Critical storm duration/temporal pattern
20	258	9 hour - Pattern 2
10	344	12 hour - Pattern 5
5	426	12 hour - Pattern 4
2	549	12 hour - Pattern 1
1	658	12 hour - Pattern 4
1 in 2,000	1,116	9 hour - Pattern 9
1 in 10,000	2,247	6 hour – PMP Temporal Pattern
PMF	8,055	6 hour – PMP Temporal Pattern

15.4.2 Existing Case flood maps

Flood maps illustrating indicative flood extents and peak water levels were prepared and are presented in Volume II – Appendix I:

- 20% AEP: Figure I-2a
- 10% AEP: Figure I-2b
- 5% AEP: Figure I-2c
- 2% AEP: Figure I-2d
- 1% AEP: Figure I-2e
- 1 in 2,000 AEP: Figure I-2f
- 1 in 10,000 AEP: Figure I-2g
- PMF: Figure I-2h.

Figure I-3a presents peak flood velocities under a 1% AEP event.

15.4.3 Flood inundation extent and flood levels

Figure I-2e in Volume II – Appendix I shows the 1% AEP flood extent and peak water levels within the Pariagara Creek floodplain for the Existing Case.

The peak flood depth is approximately 6.8 m within the Pariagara Creek channel and an of 1.0 m in other areas of the floodplain. The alignment crosses approximately 2.7 km of floodplain adjacent to Pariagara Creek. The flood depths on the floodplain where the alignment crosses the floodplain are estimated to be an average of 0.9 m, and up to 4.6 m (285.2m AHD) at the proposed bridge.

The model indicates that the time of inundation across the floodplain during the critical 1% AEP design flood is between 10 to 18 hours.

15.4.4 Flood immunity of existing infrastructure

Table 15.6 presents a summary of overtopping depth for key infrastructure near the Project alignment under a range of design events. Modelling results show that Millmerran-Inglewood Road and Thornton Road have an existing low flood immunity in the areas close to the Project alignment.

Table 15.6 Pariagara Creek – Existing Case – overtopping depths of key infrastructure

Infrastructure	Overtopping depth (m)							
	PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Millmerran-Inglewood Road	2.97	0.75	0.32	0.01	-	-	-	-
Thornton Road	6.57	4.98	4.38	4.09	3.98	3.78	3.59	3.39

15.4.5 Existing Case velocities

Peak flood velocities are expected to reach 5.9 m/s in localised areas with the average velocity across the floodplain approximately 0.5 m/s as shown in Figure I-3a in Volume II – Appendix I.

15.5 Developed Case modelling results – Pariagara Creek

15.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW model (1d and 2d approach).

In the Pariagara Creek floodplain, the Project includes the following floodplain (or regional structures):

- One waterway bridge
- Seventeen RCP locations (a total of 136 cells)
- Two RCBC locations (a total of 48 cells).

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and depending on their interaction with the Pariagara Creek floodplain were incorporated in the hydraulic model.

The proposed drainage structures are summarised in Table 15.7 and Table 15.8 and shown in Figure I-1f in Volume II – Appendix I. The 1% AEP flood levels at each drainage structure are presented in Table 15.12.

A minimum culvert size of 900 mm diameter was adopted to reduce potential for blockage and maintenance.

Bridges were modelled as an opening in the rail embankment. The optimisation of bridge lengths was balanced between minimising the changes to the hydraulic regime, primarily afflux and velocities, and the cost of replacing bridge spans by large earth embankments.

Blockage of hydraulic structures was assessed in accordance with ARR 2016. Despite the catchment being vegetated, ARR guidelines determined that the likelihood of significant amounts of debris accumulating against the piers of the bridge is low. Therefore, a zero blockage factor was applied at the Pariagara Creek bridge.

Table 15.7 Pariagara Creek – proposed floodplain culvert locations and details

Chainage (km)	Structure ID	Type	U/S invert (m AHD)	D/S invert (m AHD)	Diameter /width (m)	Height (m)	Number of cells
68.75	C68.75	RCBC	285.40	285.44	2.1	2.1	40
66.23	C66.23	RCBC	283.79	282.90	2.4	1.5	8
69.80	C69.80	RCP	285.67	285.67	1.8	-	5
69.67	C69.67	RCP	285.69	285.65	1.8	-	5
69.54	C69.54	RCP	285.72	285.70	1.8	-	5
69.41	C69.41	RCP	285.74	285.74	1.8	-	5
69.28	C69.28	RCP	285.87	285.86	1.8	-	2

Chainage (km)	Structure ID	Type	U/S invert (m AHD)	D/S invert (m AHD)	Diameter /width (m)	Height (m)	Number of cells
69.21	C69.21	RCP	285.91	285.88	1.8	-	2
69.14	C69.14	RCP	285.93	286.02	1.5	-	2
69.10	C69.10	RCP	286.07	285.97	1.2	-	2
69.02	C69.02	RCP	286.32	286.28	1.2	-	2
68.89	C68.89	RCP	286.45	286.47	1.2	-	2
67.57	C67.57	RCP	284.40	284.58	1.2	-	8
67.64	C67.64	RCP	284.39	284.56	1.2	-	8
67.70	C67.70	RCP	284.57	284.64	1.2	-	8
67.83	C67.83	RCP	284.90	284.99	1.2	-	20
67.96	C67.96	RCP	285.02	285.01	1.2	-	20
68.09	C68.09	RCP	285.10	285.06	1.2	-	20
68.41	C68.41	RCP	285.90	285.81	1.2	-	20

Table 15.8 Pariagara Creek - proposed bridge location and details

Chainage (km)	Structure ID	Approximate span (m)	Deck width (m)	Deck level (m AHD)	Deck superstructure type	Deck depth (mm)
67.35	310-BR05	345	3.97	287.7	Type D1	2,000

15.5.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

15.5.2.1 Rail immunity and structures results

Appendix C summarises the peak 1% AEP water levels on the upstream side of the proposed alignment. Local drainage structures (i.e. those not included in the flood model) and road culverts are not reported.

The results of flood modelling indicate that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the Pariagara Creek floodplain, and that peak water levels remain below the proposed rail formation level. There is over 0.4 m freeboard above the culvert obvert levels to the rail formation level.

15.5.2.2 Velocities and scour

Appendix C summarises the peak 1% AEP outlet flows and velocities at structures. The 1% AEP peak velocity through the proposed drainage structures is generally less than 2.0 m/s, except for the culvert at CH 66234 which has an outlet velocity of 3.2 m/s.

Scour protection requirements for culverts were calculated based on the velocities predicted from the hydraulic modelling. The scour protection was 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.

Table 15.9 lists the soil types encountered along the Project alignment and the allowable soil velocity based on AGRD.

Table 15.9 Allowable soil velocities along the Project alignment

Soil type	Soil description	Maximum allowable soil velocity as per AGRD (m/s)
Sodosols	Firm loam or stiff clay	2 m/s
Vertosols, Dermosols, Rudosols and Kandosols	Sandy or silty clay	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.

Scour protection requirements for culverts have been calculated based on the velocities predicted from the hydraulic modelling. Appendix D presents the proposed scour protection at each structure.

A conservative scour estimation has been undertaken at the bridge site based on available information and will be refined during detailed design.

15.5.2.3 Flood immunity for extreme events

The results of flood modelling indicate that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the Pariagara Creek floodplain.

The risk of overtopping of the rail alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF) with Table 15.10 presenting the depth of water above the formation level and over the top of rail at each structure. It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the alignment. As such, overtopping of the rail is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario.

Table 15.10 Pariagara Creek – extreme events – depth of water above formation and top of rail levels

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
66.25	-	-	0.5	-	-	-
66.65	-	-	0.3	-	-	-
66.85	-	0.4	0.8	-	-	<0.1
67.05	-	-	0.1	-	-	-
67.15	4.1	4.8	6.2	3.4	4.1	5.5
68.45	-	-	0.9	-	-	0.2
68.95	-	-	1.2	-	-	0.5
70.05	-	-	<0.1	-	-	-

15.5.3 Flood impact objectives outcomes – Pariagara Creek

The impact of the Developed Case was assessed through inclusion of the proposed rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, duration of inundation, velocities and peak flows within the floodplain:

- Changes in peak water levels for the AEP's assessed are presented in Figures I-4a to I-4h in Volume II – Appendix I
- Changes in 1% AEP duration of inundation are presented in Figure I-4i in Volume II – Appendix I
- Changes in 1% AEP velocities are presented in Figure I-4j in Volume II – Appendix I.

All impacts are required to be agreed with relevant stakeholders and affected landowners as part of the EIS process. One-one-one consultation with landowners who are expected to experience changes in flooding behaviour on the property was conducted by ARTC supported by FFJV.

The Project design outcomes relative to the flood impact objectives (Table 4.2) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Volume II – Appendix N.

The potential impacts to water levels across events up to and including the 1% AEP are summarised in Table 15.11.

Table 15.11 Afflux summary – Pariagara Creek

Afflux outside rail disturbance footprint	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
Maximum afflux (mm)	236	158	656	795	580
Area afflux >10 mm experienced (ha)	44.7	66.0	70.0	95.8	147.7
Area afflux >200 mm experienced (ha)	<0.01	<0.01	<0.01	0.3	0.2
Area afflux >400 mm experienced (ha)	<0.01	<0.01	<0.01	0.2	<0.01

15.5.3.1 Flood impacts at proposed hydraulic structures

The estimated potential impacts to peak water levels at each proposed structure are presented in Table 15.12. Peak water levels were extracted immediately upstream of each culvert and at the control line of each bridge.

The design achieved a freeboard of at least 300 mm between the 1% AEP peak water and formation levels.

Table 15.12 Pariagara Creek – 1% AEP event – impacts to peak water levels at proposed hydraulic structures

Chainage (m)	Structure ID	Structure type	Rail formation level (m AHD)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Change in peak water level (mm)
68.75	C68.75	RCBC	287.92	286.9	287.0	+60
66.23	C66.23	RCBC	290.81	284.8	285.5	+620
69.80	C69.80	RCP	288.40	287.3	287.4	+100
69.67	C69.67	RCP	288.35	287.3	287.4	+100
69.54	C69.54	RCP	288.31	287.2	287.3	+100
69.41	C69.41	RCP	288.29	287.2	287.3	+100
69.28	C69.28	RCP	288.21	287.2	287.3	+110
69.21	C69.21	RCP	288.17	287.2	287.3	+120
69.14	C69.14	RCP	288.20	287.2	287.3	+130
69.10	C69.10	RCP	288.14	287.1	287.3	+130
69.02	C69.02	RCP	288.06	287.1	287.2	+130
68.89	C68.89	RCP	288.02	287.0	287.1	+80
67.57	C67.57	RCP	287.62	285.3	285.3	+30

Chainage (m)	Structure ID	Structure type	Rail formation level (m AHD)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Change in peak water level (mm)
67.64	C67.64	RCP	287.64	285.4	285.4	+40
67.70	C67.70	RCP	287.67	285.4	285.5	+50
67.83	C67.83	RCP	287.67	285.5	285.5	+70
67.96	C67.96	RCP	287.75	285.5	285.7	+120
68.09	C68.09	RCP	287.79	285.6	285.8	+150
68.41	C68.41	RCP	287.93	286.2	286.4	+150
67.35	310-BR05	Bridge	287.66	285.2	285.3	+10

15.5.3.2 Flood impacts on flood sensitive receptors

Based on the available aerial imagery, no buildings or critical infrastructure are located within the area affected by afflux in the Pariagara Creek floodplain for events up to the 1% AEP.

15.5.3.3 Flood impacts on state-controlled roads

The extent of the hydraulic model developed for Pariagara Creek is shown in Figure 75. Within the extent of the hydraulic model, the only state-controlled road which is influenced by flooding and the Project alignment is the Millmerran-Inglewood Road. The location of the state-controlled road is shown in Figure 75.

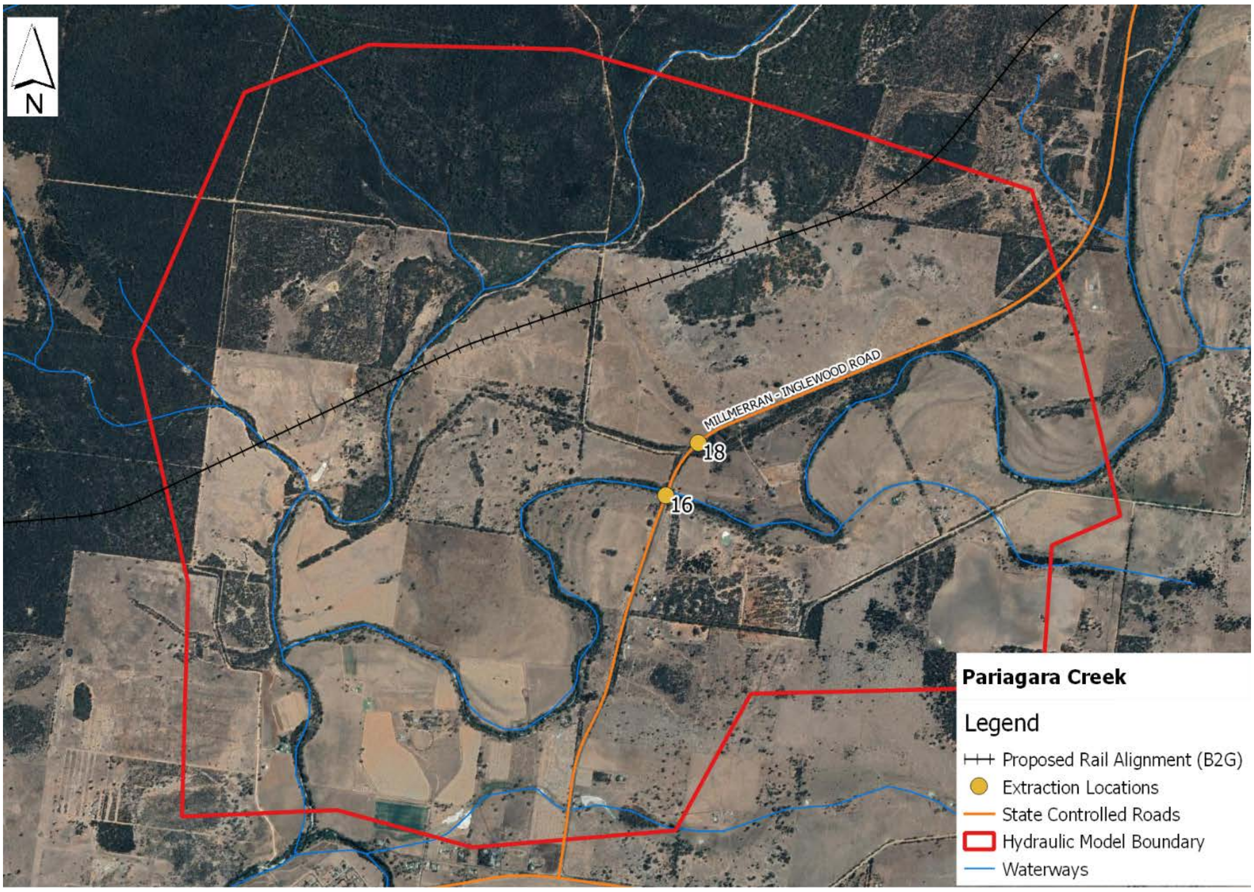


Figure 75 Pariagara Creek Hydraulic Model Extent and Associated State-controlled Roads

The following sections describe the impacts to state-controlled roads in both the Existing Case and the Developed Case and summarises the differences between the two.

Flooding from the Cattle Creek catchment has minimal impact on the Millmerran-Inglewood road in both the existing or Developed Cases up until the extreme events. In both design and Existing Case, this segment of the road has greater than 1% AEP immunity.

Existing Case flooding conditions

Table 15.13 Pariagara Creek - Existing Case flood depths

Reporting location	Road	Estimated depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
16	Millmerran-Inglewood Road	0.0	0.0	0.0	0.0	0.0	0.0	0.93	3.55

Table 15.14 Pariagara Creek - Existing Case flood inundation length

Reporting location	Road	Approximate length of inundation (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
16	Millmerran-Inglewood Road	0	0	0	0	0	0	108	2,560

Table 15.15 Pariagara Creek – Existing Case time of submergence

Reporting location	Road	Estimated time of submergence (hrs)								AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
16	Millmerran-Inglewood Road	0.0	0.0	0.0	0.0	0.0	0.0	5.6	7.0	0.0

Developed Case flooding conditions

Table 15.16 Pariagara Creek – Developed Case flood depths

Reporting location	Road	Estimated depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
16	Millmerran-Inglewood Road	0.0	0.0	0.0	0.0	0.0	0.0	5.9	3.41

Table 15.17 Pariagara Creek – Developed Case flood inundation length

Reporting location	Road	Approximate length of inundation (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
16	Millmerran-Inglewood Road	0	0	0	0	0	0	118	2,530

Table 15.18 Pariagara Creek – Developed Case time of submergence

Reporting location	Road	Estimated time of submergence (hrs)								AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
16	Millmerran-Inglewood Road	0.0	0.0	0.0	0.0	0.0	0.0	5.8	6.7	0.0

Impacts of Project alignment

Table 15.19 Pariagara Creek – change in flood depths

Reporting location	Road	Estimated change in depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
16	Millmerran-Inglewood Road	0.00	0.00	0.00	0.00	0.00	0.00	4.97	-0.14

Table 15.20 Pariagara Creek - change in time of submergence

Reporting location	Road	Estimated change in time of submergence (hrs)								Estimated change in AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
16	Millmerran-Inglewood Road	0.00	0.00	0.00	0.00	0.00	0.00	0.20	-0.30	0.0

Change in flood hydrographs

As negligible change was present except for the extreme events, and the segment of Millmerran-Inglewood road within the Pariagara Creek model extent is not inundated, no comparative plots of the water time series from the hydraulic model has been prepared.

15.5.3.4 Flood impacts on local public roads

The change in peak water levels and flood hazard (velocity-depth) for the 1% AEP event were evaluated on local public roads within the hydraulic model domain. Roads that are expected to experience an increase in flood hazard and/or increases in peak flood levels are reported in Table 15.21.

Table 15.21 Pariagara Creek – changes in peak water levels and velocity depth and flood hazard for local public roads, 1% AEP

Location	Existing flood hazard (m^2/s)	Design flood hazard (m^2/s)	Change in flood hazard (m^2/s)	Maximum existing flood depth (m)	Maximum design flood depth (m)	Maximum change in peak water levels (mm)[1]
Thornton Road	5.729	5.691	-0.038	4.086	4.141	+133
Unnamed Road	4.035	4.035	-	1.999	1.999	+56

Table note:

1 The maximum change in peak water level does not necessarily occur at the same location as where the existing and/or design maximum flood depth occur

Duration of inundation

Assessment of the time of submergence (ToS) and average annual time of submergence (AAToS) was undertaken for local public roads within the hydraulic model domain. Local public roads that are expected to experience increases in ToS and/or AAToS are presented in Table 15.22.

Table 15.22 Pariagara Creek – ToS and AAToS for local public roads

Location	Existing 1% AEP ToS (hrs)	1% AEP ToS diff. (hrs)	2% AEP ToS diff. (hrs)	5% AEP ToS diff. (hrs)	10% AEP ToS diff. (hrs)	AAToS Existing Case (hrs)	AAToS Developed Case (hrs)	AAToS diff. (hrs)
Lovells Crossing Road	23.46	-	-	-	-	14.41	18.09	3.68
Thornton Road	23.13	-	-	-	-	13.78	17.84	4.06
Unnamed Road	22.61	-0.01	-0.01	-0.01	-0.01	13.63	17.53	3.90
Unnamed Road	23.55	-	-	-	-	14.52	18.17	3.66
Unnamed Road	23.14	-	-	-	-	14.08	17.89	3.80

15.5.3.5 Flood impacts on private land outside the rail disturbance footprint

The majority of the area where afflux is expected is agricultural land or open land on which nominal afflux is unlikely to cause any adverse impact. Table 15.23 presents the modelled changes in flood conditions during the 1% AEP event on a lot basis according to the following thresholds:

- Peak water levels increased by greater than +10 mm; or
- Peak velocities increased by greater than 0.25 m/s; or.
- Duration of inundation changed by more than 25% of its original duration of inundation across the lot.

Table 15.23 Pariagara Creek – summary of flood impacts on private land outside the rail disturbance footprint for 1% AEP

Approximate chainage (km)	Changes in peak water levels[1]		Changes in peak velocities		Changes in Duration of inundation	
	Maximum change (mm)	Total area affected by change > 10 mm (ha)[2]	Maximum change (m/s)	Total area affected by change (ha)	Maximum change (%)	Total area affected by change (ha)
67.10	+114	20.7	+0.32	6.5	-	-
68.70 to 69.60	-	-	+0.48	0.02	-	-
66.20	+580	0.3	-	-	-	-
65.40 to 65.70	-	-	+0.26	0.2	-	-
65.50	-	-	+0.46	0.05	-	-
65.50 to 66.20	-	-	+0.64	0.6	-	-
68.40 to 69.40	+120	61.8	+0.34	13.0	-	-
69.70 to 70.30	+108	6.3	-	-	-	-
64.70 to 65.70	-	-	+0.93	1.1	-	-
67.30 to 68.70	+214	47.2	+0.70	22.9	-	-
66.70	-	-	-	-	+313%	0.2
70.10	+109	0.2	-	-	-	-
68.70 to 69.50	+130	10.3	+0.54	15.1	-	-
69.70 to 70.00	+110	7.0	+0.52	1.2	-	-
66.40	-	0.01	+0.46	1.0	-	-

Table notes:

1 Afflux on lots that exceed the flood impact objectives are summarised in the EIS Surface Water Chapter
2 Only minor areas, usually directly upstream of culverts are affected by the maximum afflux as stated

15.5.3.6 Flow distribution

A key landowner concern is changes to flow distribution. To understand the magnitude of these flowpaths, flows were extracted from the hydraulic model at key locations. The difference between the Existing Case and Developed Case was considered and is reported in Table 15.24. The results indicate moderate changes in one location within the floodplain as a result of the proposed rail embankment, but negligible changes within the main flow path of Pariagara Creek.

Figure 76 presents the selected flowpath comparison locations. The flow is calculated across the length of the line. Therefore, the lines presented are either calculating the flow across the width of the floodplain (for the longer flow lines) or the main flowpath of the waterways (generally for smaller flow lines).

Table 15.24 Pariagara Creek – Flow comparison

Flow location ID	10% AEP			1% AEP		
	Existing Case peak flow (m^3/s)	Developed Case peak flow (m^3/s)	% Change	Existing Case peak flow (m^3/s)	Developed Case peak flow (m^3/s)	% Change
A	249.8	246.0	-1.5%	398.2	404.0	+1.5%
B	34.2	28.6	-16.5%	105.8	91.5	-13.6%
C	294.1	296.0	+0.7%	526.7	536.5	+1.9%
D	294.1	295.9	+0.6%	525.0	534.5	+1.8%
E	301.7	307.7	+2.0%	565.2	569.9	+0.8%
F	300.9	304.7	+1.3%	613.4	606.0	-1.2%

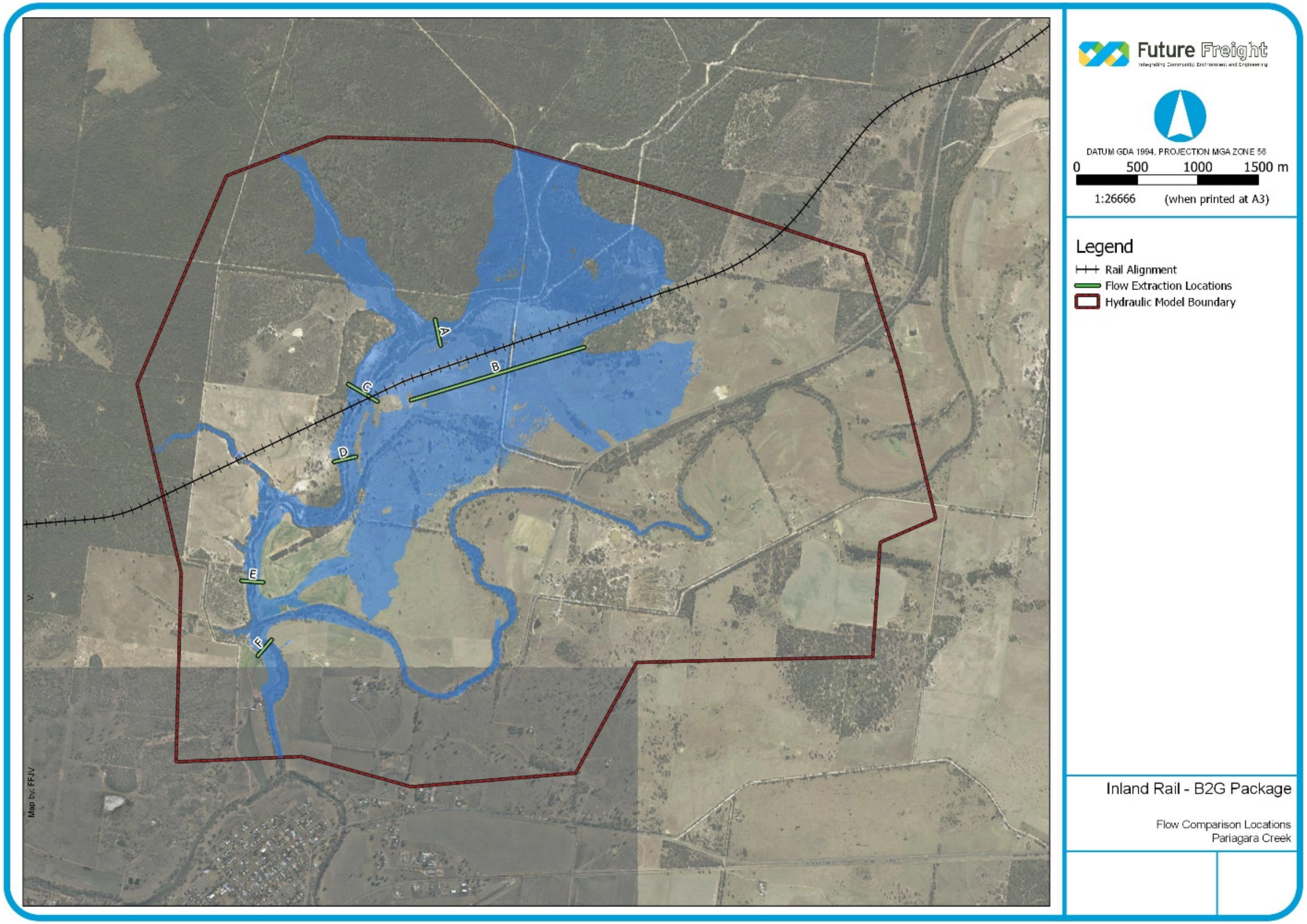


Figure 76 **Pariagara Creek – flow comparison locations**

15.5.4 Sensitivity analysis – Pariagara Creek

15.5.4.1 Blockage

A significant community concern is the potential impacts to flood conditions should the proposed culverts become blocked with debris. The primary concern is the blockage of culverts which 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% and 50% blockage.

Results of the blockage sensitivity analysis are presented in Table 15.25 and shown in Figure I-5a and I-5b in Volume II – Appendix I respectively. There are difference between results with zero blockage and baseline blockage, but peak blockage increases significantly with 50% blockage. However, the afflux footprint does not increase by a substantial amount, suggesting the topography constrains the afflux. Furthermore, the increased afflux is constrained to a single property.

Table 15.25 1% AEP event – impacts on peak water levels due to different blockage factors

Afflux outside rail disturbance footprint	0% blockage	25% blockage (Developed Case)	50% blockage
Maximum afflux (mm)	+369	+580	+1,083
Area afflux >10 mm experienced (ha)	146.4	148.0	151.1
Area afflux >200 mm experienced (ha)	<0.1	0.2	0.6
Area afflux >400 mm experienced (ha)	-	<0.01	0.3

Table 15.26 provides a summary of 1 % AEP peak flood levels at cross drainage structures for the blockage scenarios.

Table 15.26 Pariagara Creek – 1 % AEP event – culvert blockage assessment

Structure ID	Structure type	1 % AEP Peak water levels (m AHD)			Increase from Developed Case to 50% blockage scenario (mm)
		0% blockage	Developed Case (25% blockage)	50% blockage	
C68.75	RCBC	287.0	287.0	287.0	+18
C66.23	RCBC	285.2	285.5	286.0	+516
C69.80	RCP	287.3	287.4	287.4	+33
C69.67	RCP	287.3	287.4	287.4	+31
C69.54	RCP	287.3	287.3	287.4	+31
C69.41	RCP	287.3	287.3	287.4	+30
C69.28	RCP	287.3	287.3	287.3	+27
C69.21	RCP	287.3	287.3	287.3	+27
C69.14	RCP	287.3	287.3	287.3	+24
C69.10	RCP	287.3	287.3	287.3	+24
C69.02	RCP	287.2	287.2	287.3	+20
C68.89	RCP	287.1	287.1	287.1	+17
C67.57	RCP	285.3	285.3	285.3	+3
C67.64	RCP	285.4	285.4	285.4	+4
C67.70	RCP	285.5	285.5	285.5	+6
C67.83	RCP	285.5	285.5	285.5	+10
C67.96	RCP	285.6	285.7	285.7	+16

Structure ID	Structure type	1 % AEP Peak water levels (m AHD)			Increase from Developed Case to 50% blockage scenario (mm)
		0% blockage	Developed Case (25% blockage)	50% blockage	
C68.09	RCP	285.7	285.8	285.8	+18
C68.41	RCP	286.4	286.4	286.4	+9

Table 15.27 outlines the changes in peak water levels at flood sensitive receptors for the 50% blockage scenario where the increase exceeds 10 mm.

Table 15.27 Pariagara Creek – summary of 50% blockage impacts at flood sensitive receptors

Flood sensitive receptor ID	Existing case flood depth (m)	Change in peak water level (mm)
Thornton Road	4.25	+152
Unnamed road	2.11	+67

During detail 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.

15.5.4.2 Impacts during extreme events

Table 15.28 outlines the changes in peak water levels at flood sensitive receptors for the extreme events where the increase exceeds 10 mm under one of the events. The existing depth of flooding is also detailed and as can be seen the larger impacts that occur under the PMF event occur generally when there are already high flood depths as would be expected under such a rare event.

Flood immunity of the Project alignment is discussed in Section 15.5.2.3, and maps demonstrating the impacts during extreme events are shown in Volume II – Appendix I, Figures I-4f to I-4h.

Table 15.28 Pariagara Creek – Summary of extreme event impacts at flood sensitive receptors

Flood sensitive receptor ID	1 in 2,000 AEP event		1 in 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)
PAR_ID_18	-	-	-	-	+13	1.37
PAR_ID_19	-	-	-	-	+10	1.82
PAR_ID_20	-	-	-	-	+12	1.42
PAR_ID_21	-	-	+9	0.34	+10	2.04
Thornton Road	+223	4.38	+661	4.98	+657	6.57
Unnamed Road	+155	2.26	+602	3.02	+605	4.64
Lovells Crossing Road	-	0.83	+553	1.64	+777	3.25

15.5.4.3 Climate change

The potential impacts of climate change in the Pariagara Creek floodplain were assessed for the 1% AEP design event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2016.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario has been adopted for the Project with an associated increase in rainfall intensity of 23.9% across the catchment area.

Table 15.29 presents the structure performance with Representative Concentration Pathways 8.5 climate change scenario is presented in Figure I-5c in Volume II – Appendix I.

Climate change results are expected to increase peak water levels upstream of the Project alignment by up to 0.3 m at structure locations for the 1% AEP event. The Project alignment is expected to retain 1% AEP flood immunity to formation level under the climate change scenario.

Table 15.29 Pariagara Creek – 1% AEP event with Representative Concentration Pathways 8.5 conditions – structure performance

Structure ID	Structure type	1% AEP peak water level (m AHD)	1% AEP +CC peak water level (m AHD)	Difference in peak water level (m)	Freeboard to rail formation with CC (m)
C68.75	RCBC	287.0	287.2	+0.2	1.5
C66.23	RCBC	285.5	285.7	+0.2	4.9
C69.80	RCP	287.4	287.6	+0.2	1.6
C69.67	RCP	287.4	287.6	+0.2	1.5
C69.54	RCP	287.3	287.6	+0.3	1.5
C69.41	RCP	287.3	287.6	+0.3	1.4
C69.28	RCP	287.3	287.5	+0.2	1.4
C69.21	RCP	287.3	287.5	+0.2	1.4
C69.14	RCP	287.3	287.5	+0.2	1.3
C69.10	RCP	287.3	287.5	+0.2	1.3
C69.02	RCP	287.2	287.4	+0.2	1.4
C68.89	RCP	287.1	287.3	+0.2	1.4
C67.57	RCP	285.3	285.6	+0.3	2.4
C67.64	RCP	285.4	285.6	+0.2	2.3
C67.70	RCP	285.5	285.7	+0.2	2.3
C67.83	RCP	285.5	285.8	+0.3	2.3
C67.96	RCP	285.7	285.9	+0.2	2.2
C68.09	RCP	285.8	286.0	+0.2	2.2
C68.41	RCP	286.4	286.6	+0.2	1.8
310-BR05	Bridge	287.0	287.2	+0.2	1.5

Table 15.30 outlines the changes in peak water levels at flood sensitive receptors for the climate change scenario where the increase exceeds 10 mm.

Table 15.30 Pariagara Creek – summary of climate change impacts at flood sensitive receptors

Flood sensitive receptor ID	1% AEP climate change event	
	Change in peak water level (mm)	Existing case flood depth (m)
Thornton Road[1]	+222	4.25
Unnamed road[1]	+216	2.11
Lovells Crossing Road[1]	+87	0.72
Millmerran-Inglewood Road[1]	+107	0.07

Table note:

1 These roads are affected by climate change regardless of the Project and so the amenity of the roads is not compromised by the Project

The downstream extents of these impacts are similar to those under the 1% AEP event.

16 Macintyre Brook – Yelarbon to Inglewood

Macintyre Brook runs in an east to west direction through Inglewood and south of Yelarbon. The Macintyre Brook is fed by several creek systems as it flows from the east of Inglewood westwards towards Yelarbon. These include Mosquito Creek and Canning Creek to the north of Inglewood. Coolmunda Dam is situated on Macintyre Brook and is located upstream of Inglewood. Kippenbung Creek runs from east to west along the southern side of Yelarbon flowing into the Dumaresq River approximately 24 km downstream of the Macintyre Brook confluence with the Dumaresq River. Brigalow Creek, a tributary of the Weir River, runs from east to west to the north of Yelarbon. The hydraulic model for this assessment covers from the area immediately downstream of Yelarbon, east to and including Inglewood. The key areas of assessment are:

- Inglewood
- Whetstone
- Millmerran-Inglewood Road
- Yelarbon.

The location of the Project rail alignment in relation to Macintyre Brook at Yelarbon to Inglewood is shown in Figure J-1a in Volume II – Appendix J.

16.1 Data collection and review – Macintyre Brook – Yelarbon to Inglewood

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:

- GRC – existing flood studies and stream gauging data
- The BoM – rainfall and stream gauging data
- DNRME – stream gauging data
- Queensland Government – Flood Mapping Program
- QR – existing infrastructure details
- DTMR – existing infrastructure details.

16.1.1 Previous studies

Several previous hydrology and hydraulic studies were sourced as part of this assessment. A review of each study was undertaken to determine suitability for use on the Project as documented in the following sections. The models developed for these studies are also outlined below and were used for developing the models and comparison of the models developed for this assessment.

There are four key studies of the Macintyre Brook that were considered in this assessment. These are:

- Goondiwindi Regional Council, Inglewood Flood Study, Engeny, 2015
- Flood hazard mapping – Yelarbon (Bundle 8), SKM, March 2013
- Draft Floodplain Management Plan for the Borders River Valley Floodplain, Office of Environment and Heritage, 2018
- Inland Rail: Phase 2 – North Star to Border, 2018.

These four studies are both recent with a review undertaken to determine which study and models would be most applicable for the current assessment.

Goondiwindi Regional Council, Inglewood Flood Study, Engeny, 2015

In 2015 Engeny was commissioned by GRC to undertake the Inglewood Flood Study. The study objectives were "*to define the nature, extent and risks of flooding in Inglewood in order to inform disaster management planning and response, as well as control future development*" (Engeny, 2015).

An URBS hydrologic model and TUFLOW (1D/2D) hydraulic model were developed and an assessment of the following undertaken:

- 1976 historical event for model validation
- 10%, 5%, 1% and 0.5% AEP events
- Sensitivity analysis for Coolmunda Dam storage capacity.

Findings from the study were:

- Whilst Coolmunda Dam does not serve as a flood mitigation measure for Inglewood, the availability of storage within Coolmunda Dam does have a significant influence on flood events up to the 1% AEP event
- For flood events greater than 10% AEP, the obstruction caused by the existing railway line causes flows in excess of bank full capacity to be diverted towards Brook Street in a westerly direction
- The Macintyre Brook and Canning Creek channel banks have a 20% AEP to 10% AEP capacity
- The flood hazard within the Macintyre Brook and Canning Creek channel banks is classified at Extreme whilst the broader floodplain including the Inglewood Township is mostly classified as Significant
- The Inglewood Hospital is estimated to have a flood immunity of approximately 0.5% AEP
- The main evacuation route for Inglewood is via the Cunningham Highway Bridge in an easterly direction
- The Cunningham Highway Bridge is predicted to become flooded and closed after 12 hours and 8.5 hours in the 10% and 1% AEP events respectively
- The closure duration for the Cunningham Highway Bridge is approximately 8.5 hours and 26 hours in the 10% and 1% AEP events respectively.

Flood hazard mapping – Yelarbon (Bundle 8), SKM, 2013

A flood hazard study for Yelarbon was undertaken in 2013 as part of the Queensland Reconstruction Authority (QRA) Queensland Flood Mapping Program (QFMP). The assessment considered the impact of local catchment flooding.

The study included development of a TUFLOW model, simulation of the 1956 flood event, validation of the model and design assessment of the 2%, 1% and 0.2% AEP events. Flood extent, flood hazard, depth and velocity were presented for the three events.

A flood frequency assessment was utilised to predict flows for Kippenbung Creek. The assessment and modelling did not include Macintyre Brook flood flows.

The model predicted that during a 1% AEP event, flood depths between 0.8 and 2.0 m occurred in the town centre. Velocities of up to 2 m/s were experienced in the town centre, with velocities of 3.9 m/s along the Cunningham Highway just outside of the town.

As this model only considered local catchments (not Macintyre Brook), it was not considered further for use in this assessment. The assessment has utilized for comparative purposes only.

Draft Floodplain Management Plan for the Borders River Valley Floodplain 2018

The Floodplain Management Plan for the Borders River Valley Floodplain is currently being finalised. The plan provides a framework for coordinating and assessing development works on a whole of valley basis.

As part of the plan, hydrologic and hydraulic models (URBS, RAFTS and TUFLOW) were 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 16.1.

Table 16.1 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	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 models with stage storage and flow 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 modelled 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 with current floodplain conditions.

Table 16.2 and Table 16.3 present the calibration summary comparing modelled and recorded peak water levels for the two calibration events for Macintyre Brook.

Table 16.2 1976 event calibration summary

Catchment	Gauging station	Recorded peak level (m)	Modelled peak level (m)
Macintyre Brook	Terraine Inglewood CBM Inglewood	5.9 11.6 11.8	5.7 11.1 11.8

Table 16.3 1996 event calibration summary

Catchment	Gauging station	Recorded peak level (m)	Modelled peak level (m)
Macintyre Brook	Inglewood Booba Sands	9.8 8.9	9.2 9.0

The DPIE hydraulic model upper model boundary is downstream of Yelarbon, and therefore the model only covers a small area of interest from the upper boundary to the border. The following details are noted for understanding of model parameters used within the catchment. The model uses current conditions including existing and approved development in floodplain, with small (1996 flood event) and large (1976 flood event) historical rainfall events 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 water levels, changes in flow paths, flow rates and velocities). A TUFLOW GPU hydraulic model was 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 model area. Where data was not available, SRTM1-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). The DPIE hydraulic roughness is presented in Table 16.4.

Table 16.4 DPIE hydraulic model roughness

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

Boundary conditions were 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.

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 modelled with current topographic conditions for impact assessment (approximately 1% AEP flood in Macintyre River)
- No design event analysis was undertaken, and historical event modelling is used for impact assessment of development on the floodplain
- The modelling is currently being finalised.

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

Inland Rail: Phase 2 - North Star to Border, 2018

The NS2B section of Inland Rail will cross the Macintyre River and its floodplain which are a part of the Border Rivers catchment. The NS2B alignment runs through Moree Plains LGA, Gwydir LGA and Goondiwindi LGA. The Project alignment runs from North Star parallel to North Star road and Bruxner Way before tracking east on the southern side of Whalan Creek. The alignment then turns north to cross Whalan creek and the Macintyre River before turning east to Inglewood.

The Border Rivers floodplain has experienced many floods in recent years including the 1976 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 the study was to better understand and quantify the existing flooding characteristics of the Border Rivers floodplain and to assess and mitigate any potential impacts of the Project alignment.

The key objective of the report was to provide information on the data investigation, hydrologic and hydraulic calibration, design event modelling and provide comment on the performance on the Design.

The Office of Environment and Heritage (DPIE) model (discussed above) was utilised as a basis for the hydrology and hydraulic assessment.

To establish the reliability of the models the 2011 event was included in the calibration process in addition to the 1976 and 1996 events assessed by DPIE. The models were found to represent flows and levels across the floodplain well compared to the recorded 2011 event. Based on the performance of the hydraulic sub-model to predict the flood gauge heights at the Boggabilla gauge for all three events and the good correlation between the historical flood photographs and recorded flood levels for the 1976 and 2011 flood event, the Phase 2 hydrologic and hydraulic models were considered suitably calibrated for assessment of the Phase 2 Project alignment design.

Design event hydrology was developed from the calibrated models using ARR 2016 methods. The flows and levels were compared to FFA and previous flood studies and were found to be consistent with the other data.

A Log-Pearson III (LP3) FFA was carried out for the gauges including Inglewood gauge station. The FFA was used to classify the 1976 calibration event (refer Figure 77).

Analysis of the FFA graphs suggests that the flood event experienced by the Macintyre Brook at Inglewood for the 1976 event had a return period of approximately 1 in 87 year AEP.

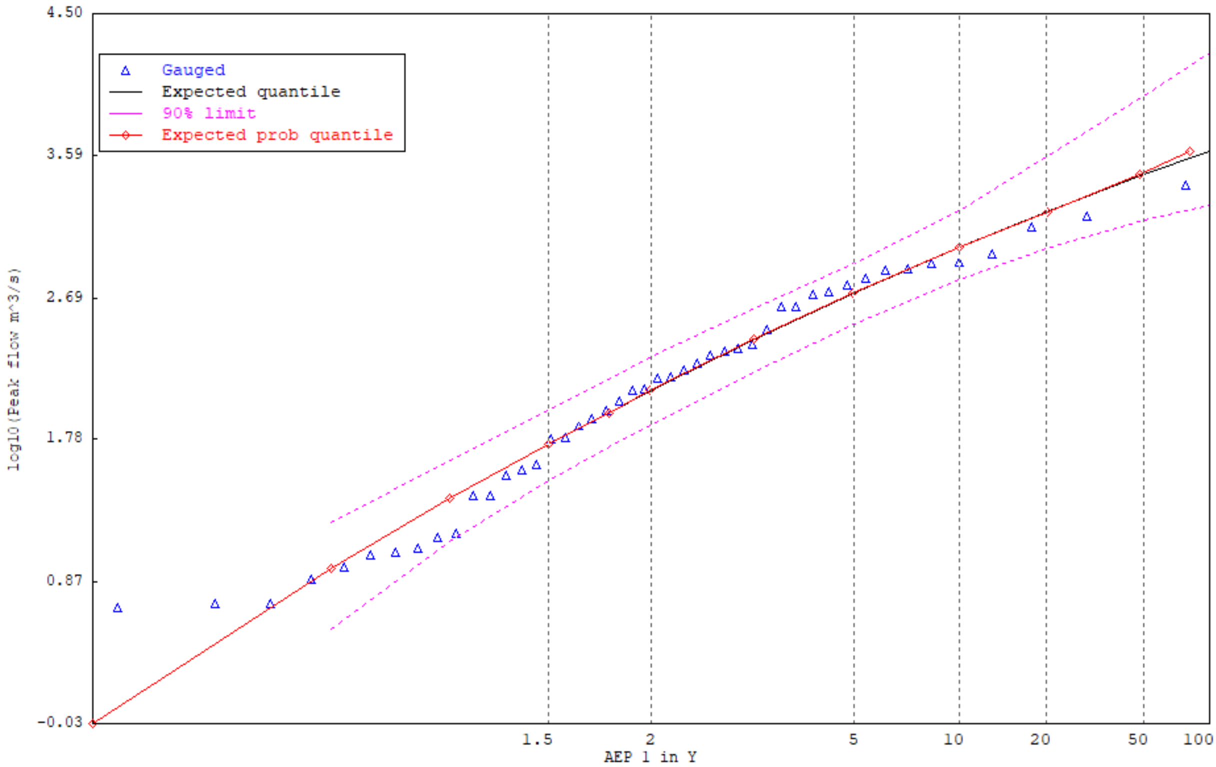


Figure 77 Inglewood Gauge flood frequency analysis

16.1.2 Comparison of the models

The DPIE models were adopted for use in the NS2B project modelling. As part of the NS2B assessment design flows were developed. The design hydrology from the NS2B study and the Inglewood Flood Study were compared as part of determining the modelling approach to adopt for the Project alignment assessment.

Table 16.5 presents the 1% AEP flows from both hydrology models and the corresponding flood frequency analysis results at the Inglewood Gauge.

Table 16.5 Comparison of hydrologic flows

Gauge	Bureau of Meteorology FFA flow (m^3/s)	FFJV FFA flow (m^3/s)	FFJV URBS flow (m^3/s)	Inglewood Flood Study FFA flow (m^3/s)	Inglewood Flood Study URBS flow (m^3/s)
Inglewood	3,700	2,750	2,087	4,390	3,448

The FFA considered the removal of smaller events which where influencing the fit to the curve and resulting in an exponential increase of flows in the prediction of larger events. It is considered that this provides a more realistic prediction of flows in the catchment during large events.

The plots of the other two FFA assessments are presented in Figure 78 and Figure 79.

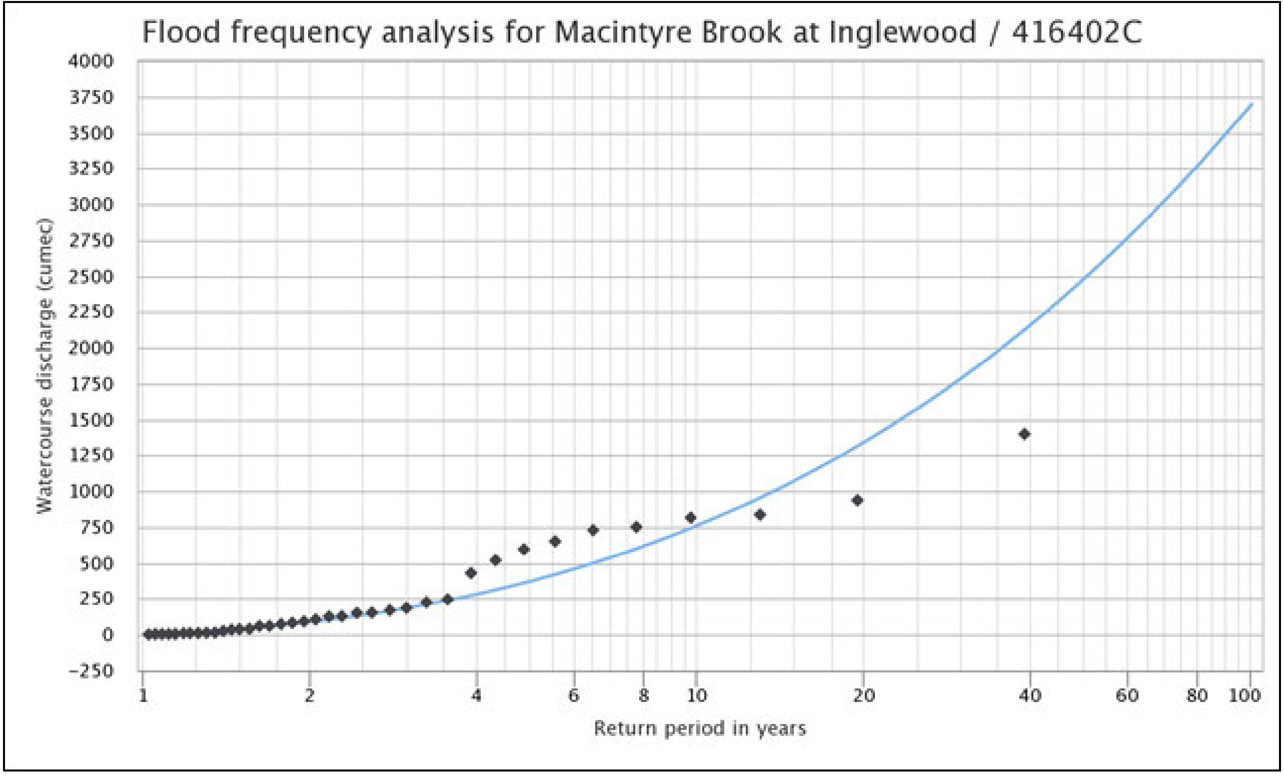


Figure 78 Bureau of Meteorology flood frequency analysis Inglewood

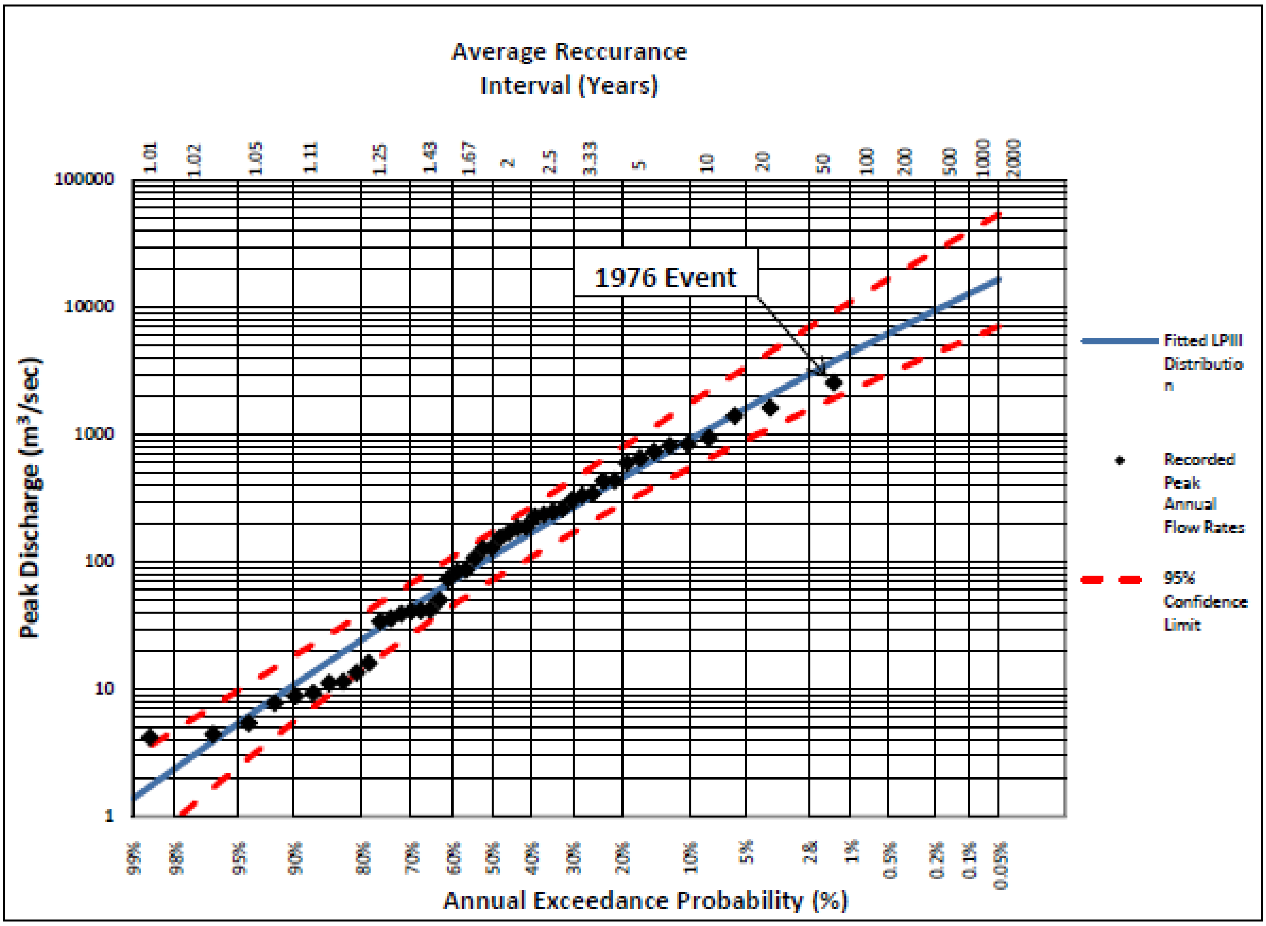


Figure 79 **Inglewood Flood Study flood frequency analysis Inglewood**

Source: Engeny (2015)

The BoM FFA provides a poor fit to the data, likely resulting in high predictions of the larger flood events. The Inglewood Flood Study provides a better fit; however, the study (Engeny, 2015) has concluded:

"*It is noted that the LP3 distribution gives a good fit to the recorded data for frequent and large events (i.e. up to 2% AEP); however, for rare flows (i.e. beyond the 1% AEP) the recorded annual flow series appears to flatten out more quickly than the LP3 distribution. Consequently, the FFA may over-estimate peak flows for rare and extreme events*".

Therefore the 1% AEP estimates for the flows from the Inglewood Flood Study (Engeny, 2015) are considered high. It is noted however that this is the Goondiwindi Regional Council adopted flood study. Therefore, for consistency the approach is to adopt the higher, more conservative flows for the Phase 2 assessment of the Macintyre Brook rather than the FFJV calculated flows. These flows have not been adopted for the Border Rivers (including Macintyre Brook) where there are multiple major water courses converging in the study area. It is considered adopting the higher flows for Macintyre Brook for inflow to the Border Rivers floodplain would be unreasonably conservative.

Based on this approach, the Inglewood Flood Study (Engeny, 2015) hydrology was adopted for inflows to the B2G Macintyre Brook model (current investigation). Downstream of Inglewood and below the extent of the Inglewood Flood Study (Engeny, 2015) hydrology, the NS2B URBS hydrology model was applied to provide inflows for the minor catchment. This is discussed further in the following sections.

16.1.3 Survey

ARTC provided LiDAR data from 2015 as 1m grid DEM tiles over the rail study area. Using GIS software FFJV generated a DEM with a 1m grid resolution for use in the Project. The DEM was based on the 2015 dataset.

Additional LiDAR data extents were required to appropriately model downstream boundary conditions and facilitate calibration against stream flow gauges. In areas that were not covered by the LiDAR provided by ARTC, LiDAR tiles were sourced from Geoscience Australia. The DEM datasets utilised for modelling were based on surveys flown between 2009 and 2015.

SRTM data was used for catchment delineation where no LiDAR data could be sourced, to inform the hydrologic modelling.

The survey data sources and DEM developed for Macintyre Brook is shown in Figure J-1b in Volume II – Appendix J.

16.1.4 Aerial imagery

Aerial imagery of the study area captured in 2015 was provided by ARTC and was used to identify and confirm topographic and vegetative characteristics of the study area. Additional imagery outside the study area was sourced from QGIS in an open source format.

16.1.5 Existing drainage structure data

Drainage structure geometry information was obtained from the following sources:

- Previous studies
- Site inspection
- Topographic survey (refer Section 16.1.3)

Structure geometry information contained within the previous hydraulic models was used in this assessment. QR as-constructed drawings were also sourced for culvert sizes along the rail where no other information was available. Existing floodplain infrastructure includes:

- Bybera Road Bridge
- Cunningham Highway
- Millmerran-Inglewood Road
- Local roads
- Existing QR rail line
- Levees and dams associated with farming practices.

The Bybera Road Bridge crosses the Macintyre Brook approximately 5 km downstream of Inglewood. The Bybera Road Bridge is approximately 30 m long and has an immunity less than 10% AEP (Inglewood Flood Study, 2015). A 40 km stretch of the Cunningham highway runs between Inglewood and Yelarbon and is a low-level road which includes two bridges and minor drainage structures. One of these bridges is the Cunningham Highway Bridge at Inglewood. The Inglewood Flood Study (Engeny, 2015) predicts this bridge has an immunity less than a 2% AEP and reports that this bridge is a main evacuation route.

Millmerran-Inglewood Road crosses Canning Creek approximately 4 km north east of Inglewood with a bridge structure. This bridge is 60 m long and has an immunity above 1% AEP (Inglewood Flood Study, Engeny, 2015). Key features were incorporated into the model topography, including the existing QR rail line and the Cunningham Highway to the east of Yelarbon.

Ground survey of the existing QR rail line was not available for this assessment. Determination of the formation level of the existing rail was through inspection of the available topographic data (LiDAR). This includes the existing rail line through Yelarbon.

16.1.6 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 flow. The most reliable source of data for deriving a rating curve are actual instream 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.

There are two gauges within the Macintyre Brook study area with suitable data for the assessment, being Inglewood and Booba Sands. Two other stream gauges, Macintyre Brook at Ben Dor Weir (4164064A) and Macintyre Brook at Whetstone (416401A), were disregarded due to the limited periods of operation, 1954-1988 and 1924-1953 respectively. The location of the adopted gauges are presented in Figure J-1c in Volume II – Appendix J and the gauge details are outlined in Table 16.6.

Table 16.6 Stream gauges within the Macintyre Brook study area

Station name	Station number	Ownership	Number of records (year)	Record commenced
Inglewood	416402B/C	NRME	50 years	(1969-ongoing) Coolmunda Dam commissioned 1968
Booba Sands	416415A	NRME	32 years	(1987-ongoing)

16.1.7 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 1976 event. This information was sourced from the BoM for both the Inglewood Flood Study (Engeny, 2015) and the DPIE models.

The adopted rainfall stations for the Inglewood Flood Study (Engeny, 2015) model were Woodspring (41391) and Inglewood Forestry (41034), while the adopted rainfall stations for the DPIE model are detailed below in Table 16.7 and Figure 80.

Table 16.7 Rainfall used for calibration

Gauge	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

16.1.8 Anecdotal and observed flood data

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

- Previous studies
- Landholders and stakeholders.

The anecdotal data was used to assess of the performance of the hydraulic model to replicate historical flood conditions.

16.1.9 Site inspection

A site inspection was undertaken on 17 October 2018. During the site inspection, all major waterway crossings were visited and inspected with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

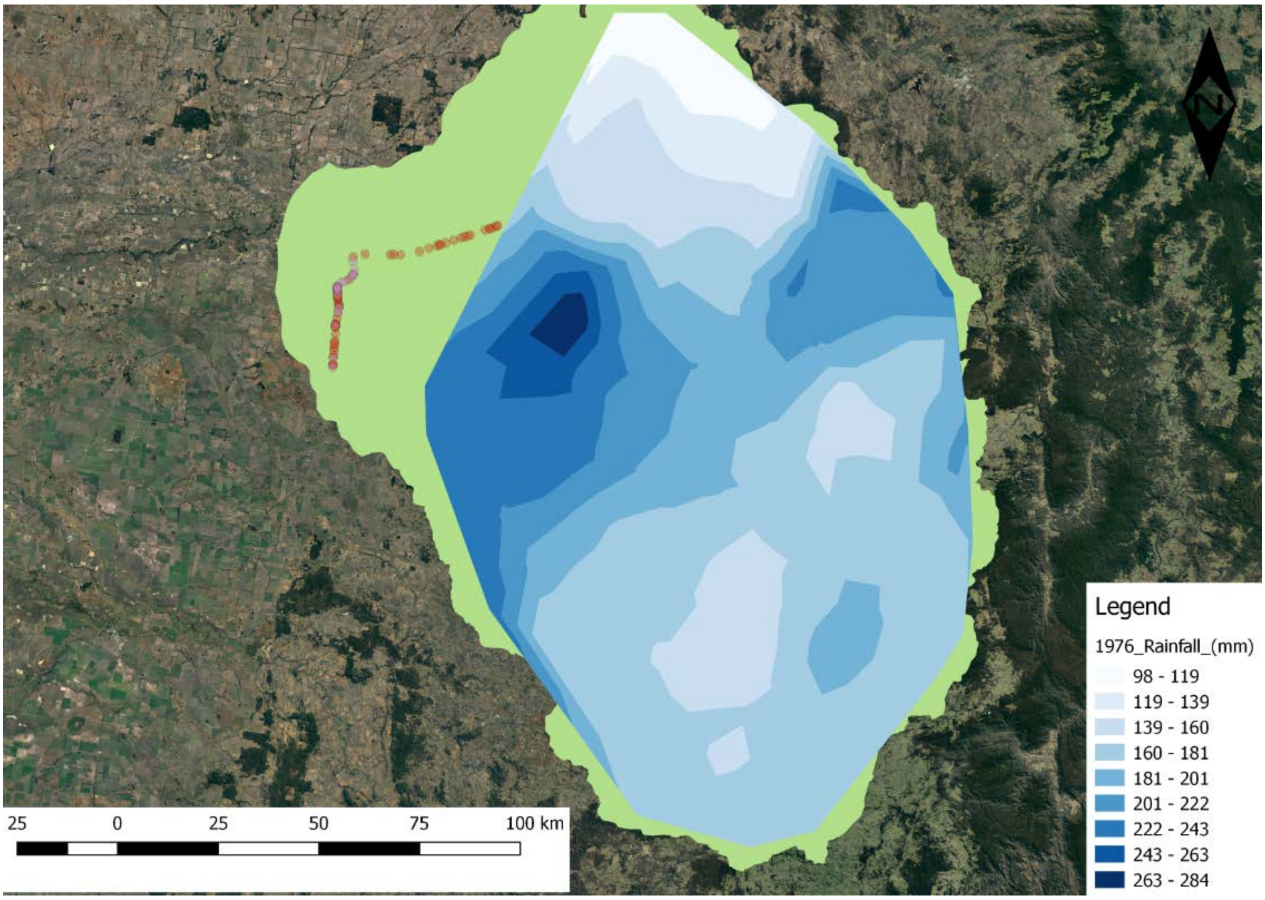


Figure 80 Spatial distribution of 1976 rainfall

16.2 Hydrologic model development – Macintyre Brook – Yelarbon to Inglewood

Two URBS hydrology models were utilised in this assessment to provide coverage to the entire hydraulic model study area. The URBS hydrologic models used for this assessment were sourced from the Inglewood Flood Study (Engeny, 2015), and the NS2B assessment (sourced from DPIE).

The Inglewood Flood Study (Engeny, 2015) URBS hydrologic model and spatial GIS files were supplied electronically for use in this assessment. The sub-catchment layout of the model is presented in Figure J-1c in Volume II – Appendix J. The hydrologic model covers the entire catchment upstream of Inglewood to immediately downstream of Inglewood.

For the sub-catchments downstream of Inglewood, the NS2B hydrologic model for Macintyre Brook was utilised. The model was sourced from DPIE who sourced it 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 were recreated in GIS, to present the general location of the sub-catchments and are presented in Figure J-1c in Volume II – Appendix J.

16.2.1 Model setup

For the Inglewood Flood Study hydrologic model, the catchment delineation was undertaken using CatchmentSIM with 30 m Shuttle Radar Topography Mission (SRTM). The catchment delineation and URBS parameters where not altered for the current URBS model. The URBS model was updated to ARR 2016 standards. Downstream of Inglewood the NS2B URBS model was applied. For the Inglewood Flood Study (Engeny, 2015) hydrologic model the total catchment area of 3,500 km^2 was divided into 21 sub-catchments. This was not altered for this current investigation.

For the NS2B hydrologic model the total catchment area of 3,320 km^2 was divided into 43 sub-catchments. Two additional sub-areas were included after the end of the system to model local flows from Brigalow Creek and Kippenbung Creek, both near Yelarbon.

The extent of the hydrologic models and sub-catchment boundaries are detailed in Volume II – Appendix J, Figure J-1c.

16.2.2 Fraction impervious and roughness

Fraction imperviousness values of 1.2% and 21.6% were applied for the Inglewood and upstream of Coolmunda Dam sub-areas, respectively, in the Inglewood Flood Study model (Engeny, 2015). It is noted that the high fraction imperviousness upstream of Coolmunda Dam sub-area results from Lake Coolmunda surface area at full capacity (16.45 km^2).

A fraction impervious value of 0% was applied for the Phase 2 NS2B Macintyre Brook model.

For both models the Muskingum coefficient, which is applied to the storage routing, was set to the default value of 1, while the Manning's roughness, which is used as a reach length scaling factor, was not activated.

16.2.3 Routing parameters

Routing parameters and losses are detailed in Sections 16.5.1 and 16.5.2.

16.3 Hydraulic model development – Macintyre Brook – Yelarbon to Inglewood

Two relevant hydraulic models were developed across the study area and provided to support this assessment. These are:

- Inland Rail: North Star to Border, 2018 TUFLOW model (based on the Draft Floodplain Management Plan for the Border Rivers Valley Floodplain, DPIE 2018 – TUFLOW model). This model covers the section of the B2G impact assessment area from Yelarbon to Inglewood.
- Goondiwindi Regional Council, Inglewood Flood Study, (Engeny, 2015) TUFLOW model. This model covers the section of the B2G impact assessment area in Inglewood.

The extents of the available hydraulic models are shown in Figure J-1d in Volume II – Appendix J. Neither of these models cover the full extent of the study area (from the Border to upstream of Inglewood). As such a hydraulic model based on the Inglewood Flood Study (Engeny, 2015) TUFLOW model was created, extending from upstream of Inglewood to Yelarbon in the west (B2G Macintyre Brook hydraulic model). The NS2B hydraulic model covers the section from Yelarbon to the NSW/QLD Border. The following sections provide details on the B2G Macintyre Brook hydraulic model, referred to as "TUFLOW hydraulic model" herein.

16.3.1 Model setup

The Macintyre Brook TUFLOW hydraulic model was set up on a 15 m grid and developed in TUFLOW HPC. The model covers an area of 610 km^2

The hydraulic model extent and adopted land use are presented in Volume II – Appendix J, Figure J-1d and Figure J-1e.

16.3.2 Hydraulic structures

Major structures were included in the TUFLOW hydraulic model. Bridge details were sourced from the Inglewood Flood Study (Engeny, 2015) TUFLOW Model for Inglewood and surrounds. Outside of these areas, details were obtained from site inspection and LiDAR inspection. Table 16.8 presents the existing structures included in the hydraulic model.

Table 16.8 Existing structure details

Structure modelling ID	Structure type	Road deck height (m AHD)	Representation in TUFLOW
Bybera Road Bridge	Bridge	270.5	Layered Flow Constriction (lfcsh)
Cunning Highway Bridge (Inglewood)	Bridge	282.7	Layered Flow Constriction (lfcsh)
Millmerran-Inglewood Road Bridge	Bridge	285	Layered Flow Constriction (lfcsh)
Potters Road Culvert	Culvert: 1/1.5 (h) x 1.8 (w) RCBC	272.2	1D Network
Lovells Crossing Road Causeway	Causeway	273.3	Z Line
QR Existing Rail Line	Railway embankment	N/A	Z Line
Yelarbon Levee	Levee	N/A	Z Line
Cunningham Highway Culverts (Yelarbon)	Culverts: 1/0.6 (h) x 1.2 (w) RCBC and 1/ 0.9 RCP	N/A	1D Network
QR Existing Rail Culverts (Yelarbon)	Culverts: 2/0.5 (h) x 3 (w) RCBC.	N/A	1D Network

16.3.3 Roughness

The hydraulic roughness reflects the types of development and ground cover that exists within the hydraulic model extents. The distribution of roughness categories adopted for this assessment was based on the parameters used in the NS2B model, the Inglewood Flood Study model, aerial imagery and confirmed during site inspection. Specific roughness values applied to the model are detailed in Table 16.9.

Table 16.9 Manning's n values

Land use	Manning's n
Floodplain area	0.06
Developed area	0.40
Waterways	0.03
Dense vegetation	0.08
Vegetated Waterways	0.12

Volume II – Appendix J, Figure J-1e shows the spatial breakdown of land use in the 2D model domain.

16.3.4 Boundary conditions

The Inglewood Flood Study URBS model outputs were applied as inflows into the TUFLOW hydraulic model. Total inflows from catchments upstream of the hydraulic model extents were applied at the upstream model boundary and local inflows from areas within the TUFLOW model were applied throughout the model. The flows from catchments downstream of Inglewood were sourced from the NS2B URBS model for the Macintyre Brook.

A normal depth boundary condition was applied at the downstream boundary.

16.4 Joint calibration – Macintyre Brook – Yelarbon to Inglewood

As part of the hydrologic assessment, the developed URBS hydrologic models for the Macintyre Brook catchment were calibrated against the 1976 historical event.

16.4.1 Historical events

The Inglewood Flood Study and NS2B URBS models have both been calibrated to the 1976 event. These calibrated 1976 flows were modelled in the B2G Macintyre Brook TUFLOW hydraulic model to test the ability of the model to replicate the results from the Inglewood Flood Study and the 1976 historical event.

16.4.2 Hydrologic model calibration

The provided URBS hydrologic models were calibrated to the 1976 flood event. The adopted model parameters for both models were not altered and were considered suitable for this assessment.

These values are:

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

Table 16.10 presents the parameters for both URBS models.

Table 16.10 Hydrologic model adopted parameters

Model	Alpha	Beta	m
Inglewood Flood Study URBS	1.7	N/A (Basic Model)	0.8
NS2B (DPIE) URBS	0.20	1.2	0.8

Initial and continuing losses for the 1976 rainfall event are presented in Table 16.11.

Table 16.11	Initial and continuing loss parameters 1976 event

Event	Sub-catchment	Initial loss (mm)	Continuing loss (mm/hour)
Inglewood Flood Study URBS	Macintyre Brook	15	1
NS2B (DPIE) URBS	Macintyre Brook	0.0	2.50

Figure 81 presents the URBS model calibration results for the 1976 rainfall event for the Macintyre Brook Gauge at Inglewood. The Booba Sands Gauge was not in operation in 1976.

The comparison of the Inglewood Flood Study, URBS model flows to the recorded stream gauge at Inglewood shows the model is adequately predicting flows at the Macintyre Brook stream gauge for the 1976 calibration event.

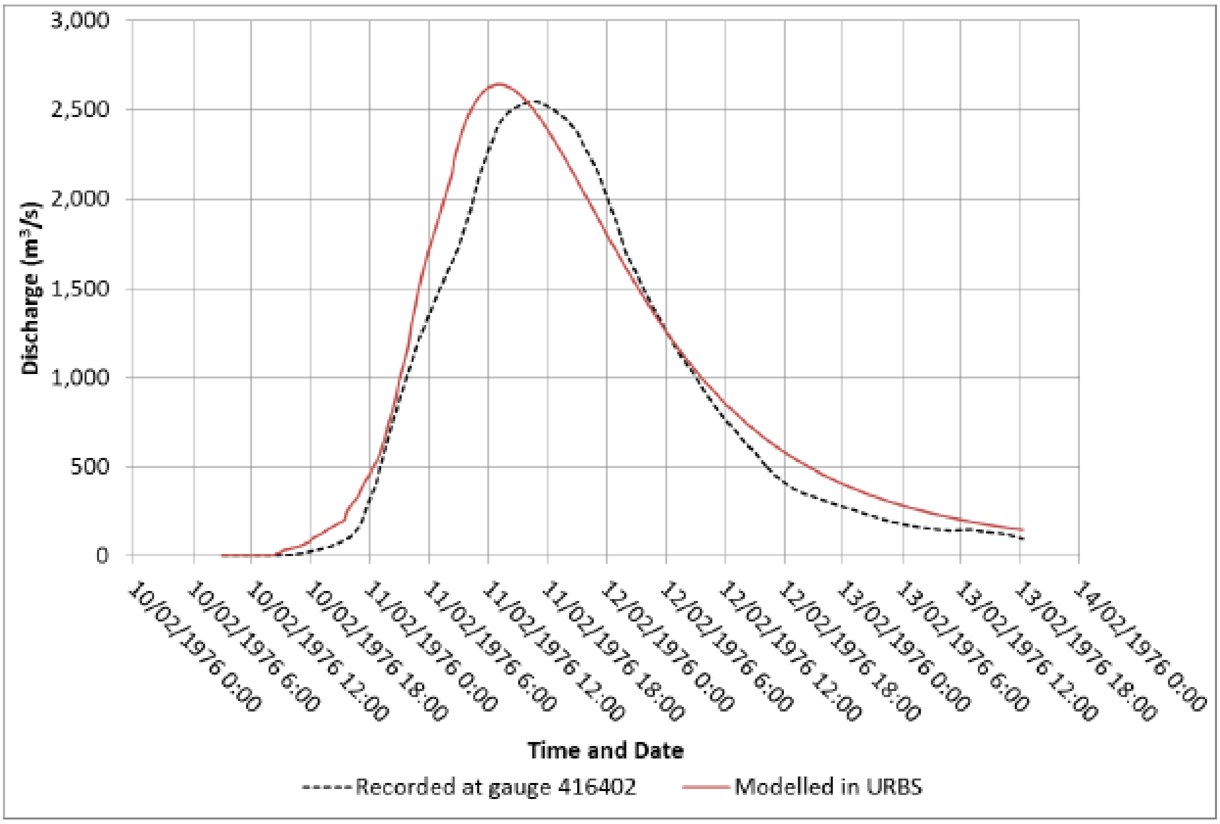


Figure 81	Macintyre Brook 1976 calibration result, Inglewood Flood Study, 2015 URBS model (Inglewood)

16.4.3 Hydraulic model calibration

The TUFLOW hydraulic model was calibrated to the 1976 flood event. The URBS hydrologic model flows were included in the TUFLOW model for the 1976 event and modelled to assess the ability of the hydraulic model to replicate peak water levels recorded during the historical events, at the gauges and from surveyed flood height recordings.

The Inglewood Flood Study, model was calibrated for the 1976 flood event. Therefore, the hydraulic calibration parameters set in the Inglewood Flood Study hydraulic model were adopted in the B2G Macintyre Brook TUFLOW hydraulic model for this event.

16.4.3.1 Recorded data

The Inglewood stream gauge was in place and operational for the 1976 event. The recorded gauge levels for the 1976 historical event is shown in Table 16.12. The stream gauge records calculate flows based on a rating curve derived for the gauge location.

Table 16.12 Inglewood gauge recorded levels and derived flows

Event	Recorded level (m AHD)	Rated flow (m^3/s)
1976	282.27	2,550

16.4.3.2 Anecdotal data

Anecdotal information for the 1976 historical event was obtained from many sources including:

- Previous studies – modelled and recorded flood heights, from landholders and local government
- Landholders.

It is noted that there was no available calibration data for Yelarbon for any event.

16.4.4 Joint calibration outcomes

16.4.4.1 Inglewood Gauge

The recorded and predicted flood levels and flows at the Inglewood stream gauge are presented in Table 16.13.

Table 16.13 Comparison of results at the Inglewood gauge

Event	Recorded level (m AHD)	TUFLOW modelled level (m AHD)	Difference in level (m)	Rated gauge flow (m^3/s)	TUFLOW modelled flow (m^3/s)	Difference in flow (m^3/s)
1976	282.27	282.43	+0.16	2,550	2,526	24

The plotted predicted versus recorded levels and flows for 1976 event are presented in Figure 82 to Figure 83 respectively.

The Inglewood stream gauge is located close to the proposed alignment. The hydraulic model was found to represent the peak levels well at the gauge with modelled levels being within 0.16m of the recorded level. The flows were found to be within +1% for the 1976 event. 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.

It is considered that the performance of the hydraulic model against the recorded data from the stream gauge is acceptable. The predicted results show that the Macintyre Brook TUFLOW hydraulic model is representing both the peak of the flood and the Volume of the event well, with the shape of the predicted hydrograph matching closely with the shape of the recorded hydrograph.

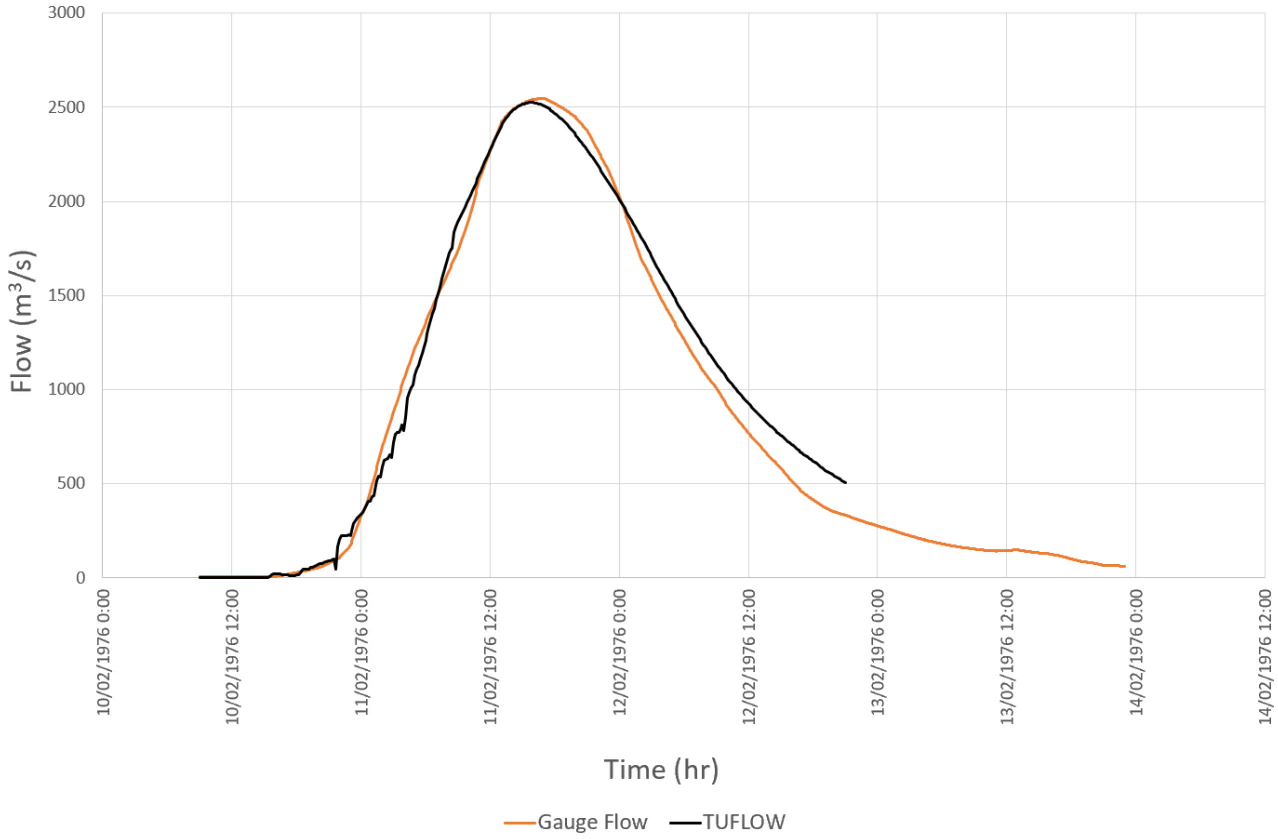


Figure 82 **1976 flows recorded and predicted, Inglewood gauge**

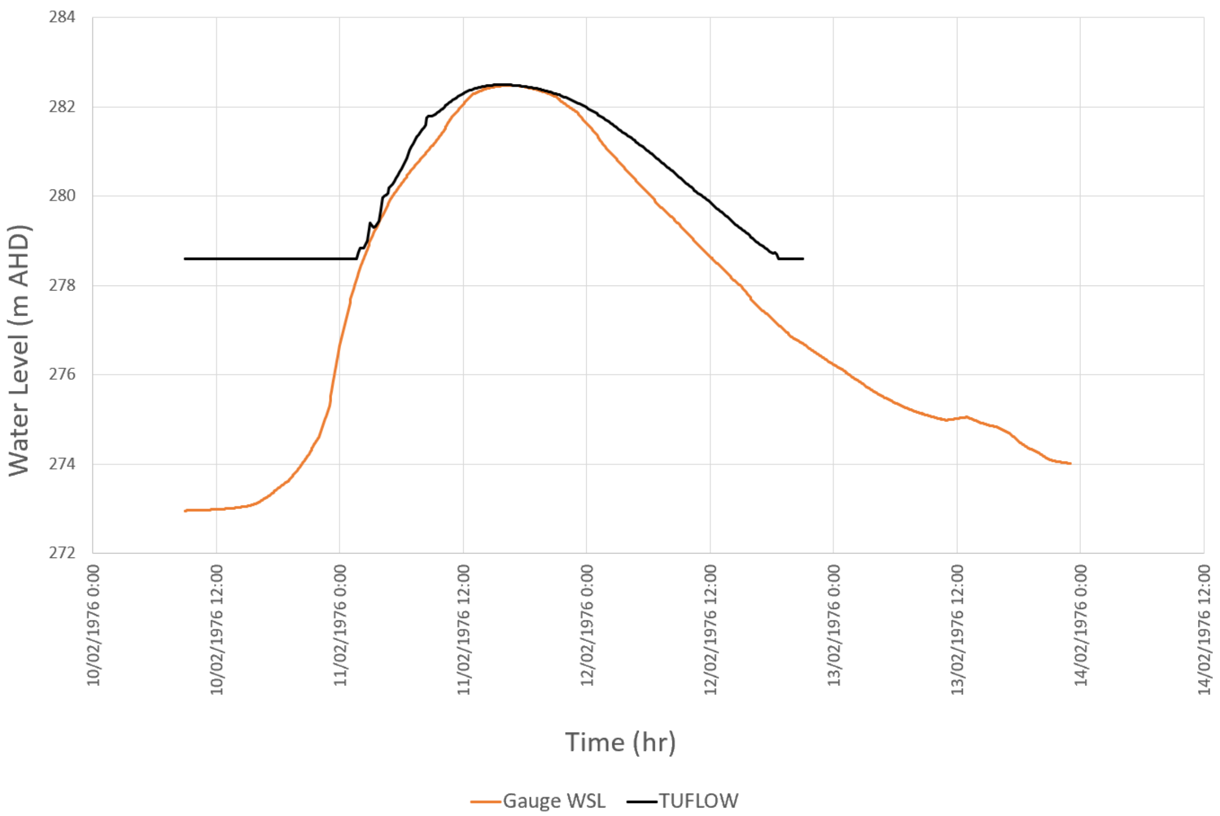


Figure 83 **1976 levels recorded and predicted, Inglewood gauge**

16.4.4.2 Historical flood levels

February 1976

There were 19 recorded flood marks collated by the Inglewood Flood Study (Engeny, 2015) and within the study area for the 1976 event. A comparison of the predicted flood levels to the recorded levels are presented in Table 16.14.

In general, the sub-model predicts levels within 0.3 m of the recorded flood levels across Inglewood. There are outliers where differences are higher than 0.5 m. These recorded levels appear inconsistent with the surrounding recorded levels and are likely be in error, or a result of wind and wave effects beyond the capability of the hydraulic model to replicate.

In the northern section of Inglewood, the TUFLOW hydraulic model is consistently higher than the recorded levels with differences up to 0.5 m. These differences were not predicted in the Inglewood Flood Study, where differences were up to 0.2 m. This is possibly due to the topography differences between the two models, with the Inglewood Flood Study, (Engeny, 2015) ground levels being approximately 0.1 to 0.2 m lower than the B2G Macintyre Brook model.

In addition, the application of the inflows to the model for Macintyre Brook is closer to Inglewood in the B2G model, and downstream of a breakout of the Macintyre Brook to convey flow across to Canning Creek and into Inglewood from the north. Whilst this is not expected to impact the levels predicted at the proposed alignment, it is noted that this may be elevating predicted levels at Points 2, 3 presented in Table 16.14.

Table 16.14 1976 recorded flood level comparison

Flood marker ID	Source	Recorded level (m AHD)	TUFLOW modelled level (m AHD)	Difference (m)
1	Inglewood Flood Study, June 2015	283.67	283.99	+0.32
2		283.60	284.12	+0.52
3		283.72	284.18	+0.46
4		283.61	284.00	+0.39
5		284.04	284.34	+0.30
6		284.15	284.33	+0.18
7		283.87	283.98	+0.11
8		283.69	283.73	+0.04
9		283.72	283.87	+0.15
10		283.94	284.10	+0.16
11		284.45	284.43	-0.02
12		284.13	283.66	-0.47
13		284.83	284.41	-0.42
14		283.87	283.87	+0.00
15		283.18	283.18	+0.00
16		285.14	284.45	-0.69
17		285.02	283.93	-1.09
18		282.98	282.88	-0.10
19		284.17	284.21	+0.04

16.4.5 Calibration summary

Available data and previous studies for the Macintyre Brook floodplain were collected and reviewed to support the development and calibration of the hydrologic and hydraulic models. The Inglewood Flood Study (Engeny, 2015) models were identified as the most detailed and suitable models for the assessment of floodplain conditions and impacts of the Project alignment in the Macintyre Brook. It is noted that these models also provide a conservative estimate of flows. The hydraulic model was extended downstream to Yelarbon, with flows for local downstream catchments extracted from the NS2B URBS models, adjusted to align with the parameters from the Inglewood Flood Study (Engeny, 2015).

The models were modelled for the 1976 flood event and results were compared to the Inglewood stream gauge data, recorded historical flood heights and flood photographs.

The following is concluded from the hydrologic and hydraulic calibration:

- The 1976 flood levels compare well to the recorded levels at the Inglewood stream gauge
- Flows are within 1% of the stream gauge recorded flows, noting that the flows (estimated from the recorded levels using rating curves)
- The 1976 hydraulic model predicts flood levels that generally compare well with the recorded flood heights at Inglewood
- There was no calibration data available for Yelarbon for any historical event. Feedback on the performance of the hydraulic model in replicating the 1976 event will be obtained during consultation.

It is noted that other historical flood events i.e. the 1996 or 2011 flood events are more recent events that may be suitable for calibration. Based on the performance of the hydraulic sub-model to predict the flood gauge heights at the Inglewood gauge for the 1976 event and the good correlation with recorded flood levels for the 1976 event, the Macintyre Brook hydrologic and hydraulic models are considered suitably calibrated to use for this assessment.

16.5 Existing Case modelling results – Macintyre Brook – Yelarbon to Inglewood

16.5.1 Hydrologic modelling

Design event hydrologic modelling for the Project alignment assessment was undertaken using the methodology consistent with ARR 2016. The calibrated hydrologic models were used to develop design event flows in accordance with the requirements of ARR 2016. The Inglewood Flood Study URBS model was developed based on ARR 1987. The model required updating to ARR 2016 for this assessment. The following sections outline the design rainfall assessment undertaken in accordance with ARR 2016.

16.5.1.1 Rainfall IFD

Design rainfall for each hydrologic model was derived from intensity-frequency-duration (IFD) curves extracted from the Bureau of Meteorology 2016 Rainfall IFD Data Hub. An example of this data is presented in Table 16.15 for the 24-hour duration.

Table 16.15 24-hour rainfall depth (mm)

Catchment area	50% AEP	10% AEP	1% AEP
Macintyre Brook at Inglewood	57	89	141
Macintyre Brook to Booba Sands	55	88	139

16.5.1.2 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.

The continuing loss rates were applied as a constant value across the catchments. For upstream of Inglewood the Initial loss was fixed. For downstream of Inglewood the initial loss was varied by duration per AEP. The design rainfall losses used for each event are presented in Table 16.16. For initial loss the upper values are presented.

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.

Table 16.16 ARR 2016 design rainfall losses

Catchment	ARR 2016 Data Hub		Adopted	
	Initial loss (mm)	Continuing loss (mm/hr)	Initial loss (mm)	Continuing loss (mm/hr)
Macintyre Brook (For total flows to Inglewood)	25.0	1.0	15.0	1.0
Macintyre Brook (For local flows D/S of Inglewood)	28.0	1.0	25.0	0.5

16.5.1.3 Flood frequency analysis

An FFA was undertaken for the Inglewood stream gauge using Log Pearson 3 (LP3) with 49 sample years (1969 to 2018) using FLIKE.

Figure 84 presents the FFA for the Inglewood stream gauge. The FFA results were compared to the hydraulic model flows at the gauge and FFA results from recent studies as shown in Figure 84.

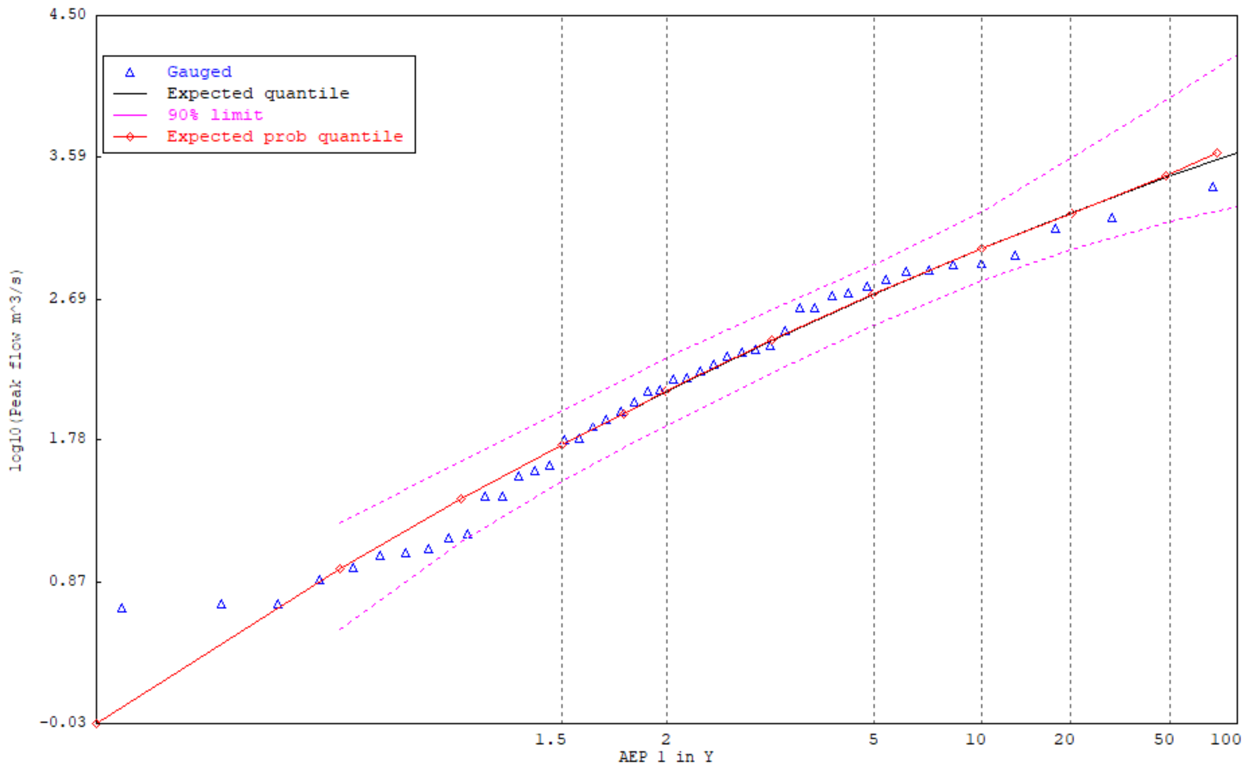


Figure 84 Flood frequency analysis at Inglewood Gauge

16.5.1.4 Design flows based on flood frequency analysis

The estimated design flows from the FFA are presented in Table 16.17, with the predicted flows from the TUFLOW model for Inglewood Gauge. The comparison shows that the TUFLOW model is over-estimating flows in comparison to the FFA, in particular for the smaller events. The hydrologic and hydraulic models assume the Coolmunda Dam storage is at full capacity for the design assessments. Therefore, is it expected that the flows through the models will be higher than the FFA, in particular in the smaller event flows where the impact of the dam storage is expected to be highest.

When compared to the Inglewood Flood Study, 2015 FFA, the TUFLOW flows are in closer agreement to the FFA in the smaller events (noting previous discussion that the Inglewood Flood Study, 2015 FFA appears to overestimate flows in the larger events).

Table 16.17 Design flows at Inglewood gauge

Event	Inglewood gauge FFA predicted flows (m^3/s), FFJV, 2018	Inglewood gauge FFA predicted flows (m^3/s), Inglewood Flood Study, 2015	Inglewood gauge TUFLOW model flows (m^3/s)
1% AEP	2,750	4,390	3,450
2% AEP	1,830	2,950	2,873
5% AEP	1,046	1,600	2,155
10% AEP	646	910	1,688
20% AEP	404	-	1,249

An assessment of the flows at Yelarbon was undertaken to determine the dominant flows for design consideration. Yelarbon experiences flooding from three main catchments, Macintyre Brook and Kippenbung Creek from the south and Brigalow Creek from the north. The peak flow estimates in the 1% AEP event in the three catchments near Yelarbon are 146 m^3/s, 114 m^3/s and 340 m^3/s for Macintyre Brook (flows that breakout from the Macintyre Brook and head north through Yelarbon only), Kippenbung Creek and Brigalow Creek respectively.

It is noted that the timing of these peaks would vary significantly. The Brigalow Creek and Macintyre Brook flows were considered in the TUFLOW model. The Macintyre Brook flows are larger than Kippenbung Creek and are expected to provide a worst-case assessment, therefore the Kippenbung Creek catchment flows were not incorporated into the TUFLOW model.

16.5.2 Hydraulic assessment

The design event flows were modelled in the hydraulic model for the suite of design events: 20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF.

16.5.2.1 Critical duration assessment

A critical duration assessment was undertaken to determine which storm duration produced peak water levels across the hydraulic sub-model and more specifically the study area. To assess the critical storm duration the following methodology was adopted:

- The 1% AEP event was run through the models for a range of durations from 540 to 5760 minutes for each of the ARR 2016 10 temporal patterns
- A critical duration assessment was undertaken at key locations across the hydraulic sub-model to determine which duration produced the peak flood flows.

The critical durations were determined to be 4320 minutes (temporal pattern 03b) for the 1% AEP event within the study corridor. The same process was undertaken for the other design events with the critical durations for the other design events presented below in Table 16.18.

Table 16.18 Critical durations within the study corridor

Event	Critical duration (min)/temporal pattern
20% AEP	4320m_03b
10% AEP	4320m_03b
5% AEP	4320m_03b
2% AEP	4320m_03b
1 in 2,000 AEP	2880m_02b
1 in 10,000 AEP	2160m_10b
PMF	1440m_04b

16.5.3 Existing Case flood maps

Flood maps illustrating indicative flood extents and peak water levels were prepared and are presented in Volume II – Appendix J:

- 20% AEP: Figure J-2a
- 10% AEP: Figure J-2b
- 5% AEP: Figure J-2c
- 2% AEP: Figure J-2d
- 1% AEP: Figure J-2e
- 1 in 2,000 AEP: Figure J-2f
- 1 in 10,000 AEP: Figure J-2g
- PMF: Figure J-2h.

Figure J-3a presents peak flood velocities predicted in the 1% AEP event.

16.5.4 Flood inundation extent and flood levels

The Existing Case model results for the 1% AEP event were compared against the results from the Inglewood Flood Study at Inglewood. The 1% AEP peak water levels are predicted to be up to 100 mm higher in the B2G investigation as compared to the Inglewood Flood Study. As the difference in the ground level data used in the two studies is of a similar magnitude of difference, and minor changes to peak flows from ARR 1987 to ARR 2016 guidelines, this is considered reasonable.

The B2G hydraulic model predicts 1% AEP depths of up to 13 m will occur in the Macintyre Brook main channel through Inglewood. On the floodplain, depths of up approximately 6m are predicted across the town of Inglewood. Flow remains mainly in the Macintyre Brook up to the 20% AEP event through Inglewood and breakouts occur between a 20% AEP and 10% AEP event.

At Yelarbon, flows are predicted to be mostly contained in the Macintyre Brook up to a 20% AEP event. Flow is predicted to breakout between a 20% AEP and 10% AEP and flow towards Yelarbon. Under the 1% AEP the flood depths at Yelarbon are predicted to be around 2 m deep.

Modelling results were presented to Goondiwindi Regional Council during February 2019, and although Council were unaware of any available calibration information for Yelarbon the modelling results were accepted as reasonable.

16.5.5 Flood immunity of existing infrastructure

Table 16.19 presents a summary of overtopping depths for key infrastructure near the Project alignment under a range of design events.

Table 16.19 Macintyre Brook – Existing Case – overtopping depths of key infrastructure

Infrastructure	Location	Maximum overtopping depth (m)							
		PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Cunningham Highway	Yelarbon	2.60	1.30	1.10	0.70	0.40	0.10	2.60	-
Existing Levee	Yelarbon	2.20	0.90	0.70	0.30	0.20	0.10	2.20	-
Existing Road Bridge	Cunningham Highway, Inglewood	7.75	4.68	3.88	2.39	2.03	1.37	0.70	0.22
Existing Road Bridge	Millmerran-Inglewood Road	8.13	6.33	5.68	4.59	4.35	3.99	3.68	3.05

16.5.6 Existing Case velocities

At Inglewood, velocities of approximately 1.9 m/s are predicted across the floodplain area under the 1% AEP event with higher velocities in the Macintyre Brook of up to 2.8 m/s. At Yelarbon velocities are predicted to be up to 1.3 m/s at the peak of the flood during a 1% AEP event. Generally, the average velocity across the floodplain is approximately 0.7 m/s as shown in Figure J-3a in Volume II – Appendix J.

16.6 Developed Case modelling results – Macintyre Brook – Yelarbon to Inglewood

16.6.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW model (1d and 2d approach).

In the Macintyre Brook floodplain from Yelarbon to Inglewood, the Project includes the following floodplain (or regional structures):

- Twenty-one RCP locations (a total of 509 cells)
- Thirteen RCBC locations (a total of 181 cells).

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and depending on their interaction with the Macintyre Brook floodplain were incorporated in the hydraulic model.

The proposed drainage structures are summarised in Table 16.20 and presented in Figure J-1f to J-1h in Volume II – Appendix J.

Table 16.20 Macintyre Brook Yelarbon to Inglewood – proposed floodplain culvert locations and details[1]

Chainage (km)	Structure ID	Structure type	U/S invert level (m AHD)	D/S invert level) m AHD)	Diameter/width (m)	Height (m)	No. of cells
25.15	C25.15	RCBC	243.51	243.45	3.0	0.6	1
25.19	C25.19	RCBC	243.51	243.45	3.0	0.6	1
25.46	C25.46	RCP	243.78	243.73	0.9	-	21
25.50	C25.50	RCP	243.62	243.56	0.9	-	21
25.80	C25.80	RCBC	244.16	244.11	2.4	0.9	24
25.87	C25.87	RCBC	243.96	243.91	2.4	0.9	24
25.95	C25.95	RCBC	243.67	243.60	3.0	0.5	1
25.97	C25.97	RCBC	243.69	243.62	3.0	0.5	1
27.05	C27.05	RCBC	244.97	244.93	1.5	1.2	15
27.15	C27.15	RCBC	245.05	245.01	1.5	1.2	15
27.24	C27.24	RCBC	245.21	245.16	1.5	1.2	25
27.33	C27.33	RCBC	245.38	245.33	1.5	1.2	25
27.42	C27.42	RCBC	245.44	245.39	1.5	1.2	20
27.53	C27.53	RCBC	245.58	245.53	1.5	1.2	20
42.87	C42.88	RCP	262.98	262.94	0.9	-	15
43.02	C43.02	RCP	262.36	262.29	1.2	-	15
43.08	C43.08	RCP	262.28	262.22	1.2	-	30
43.16	C43.16	RCBC	262.31	262.26	3.0	1.5	9
43.34	C43.34	RCP	262.59	262.52	1.2	-	45
43.56	C43.56	RCP	262.59	262.52	1.2	-	10
43.66	C43.66	RCP	262.52	262.45	1.2	-	15
43.77	C43.77	RCP	262.53	262.47	1.2	-	15
43.86	C43.86	RCP	262.59	262.52	1.2	-	15
43.97	C43.97	RCP	262.67	262.61	1.2	-	15
44.32	C44.32	RCP	262.84	262.77	1.2	-	15
44.67	C44.67	RCP	263.25	263.14	1.2	-	15
44.88	C44.88	RCP	263.70	263.66	0.9	-	30
44.99	C44.99	RCP	263.97	263.89	0.9	-	35
45.24	C45.24	RCP	264.65	264.58	0.9	-	35
45.30	C45.30	RCP	264.68	264.61	0.9	-	35
45.39	C45.39	RCP	264.67	264.60	0.9	-	40

Chainage (km)	Structure ID	Structure type	U/S invert level (m AHD)	D/S invert level) m AHD)	Diameter/width (m)	Height (m)	No. of cells
45.46	C45.46	RCP	264.81	264.74	0.9	-	40
45.53	C45.53	RCP	265.07	264.97	0.9	-	40
45.67	C45.67	RCP	265.36	265.31	0.9	-	7

Table note:

1 Details regarding cross drainage structures for Pariagara Creek (a tributary of Macintyre Brook) is reported in Section 15.5, and bridges at Bybera Road and Cremascos Road in Sections 17.5.1 and 18.5.1, respectively

16.6.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

16.6.2.1 Rail immunity and structures results

Appendix C summarises the peak 1% AEP water levels on the upstream side of the proposed alignment. Local drainage structures (i.e. those not included in the flood model) and road culverts are not reported.

16.6.2.2 Velocities and scour

Appendix C summarises the peak 1% AEP outlet flows and velocities at structures. The 1% AEP peak velocity through the proposed drainage structures is generally less than 2.2 m/s.

Scour protection requirements for culverts were calculated based on the velocities predicted from the hydraulic modelling. The scour protection was 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.

Table 16.21 lists the soil types encountered along the Project alignment and the allowable soil velocity based on AGRD.

Table 16.21 Allowable soil velocities along the Project alignment

Soil type	Soil description	Maximum allowable soil velocity as per AGRD (m/s)
Sodosols	Firm loam or stiff clay	2 m/s
Vertosols, Dermosols, Rudosols and Kandosols	Sandy or silty clay	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.

Scour protection requirements for culverts have been calculated based on the velocities predicted from the hydraulic modelling. Appendix D presents the proposed scour protection at each structure.

A conservative scour estimation has been undertaken at the bridge site based on available information and will be refined during detailed design.

16.6.2.3 Flood immunity for extreme events

The results of flood modelling indicate that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the Macintyre Brook floodplain, and that peak water levels remain below the proposed rail formation level.

The risk of overtopping along the Project alignment has been assessed for the modelled extreme events. During these extreme events the Project alignment is inundated at several locations. Table 16.22 outlines the overtopping locations and depths.

Table 16.22 Macintyre Brook (Yelarbon to Inglewood) – Project alignment – Extreme event rail overtopping details

Approximate chainage (km)[1]	1 in 2,000 AEP overtopping depth (m)	1 in 10,000 AEP overtopping depth (m)	PMF overtopping depth (m)
14.75 to 19.05	0.2	0.4	1.0
21.15 to 32.30	0.4	0.6	2.0
35.05 to 35.50	-	-	0.8
42.00 to 46.15	0.2	0.9	3.7

Table note:

1 The length of Project alignment overtopped around these areas varies between events

16.6.2.4 Proposed levee at Yelarbon

The flood impact assessment at Yelarbon has considered Existing Case regional flooding, developed case flood impacts and potential mitigation options at Yelarbon. In other locations along the alignment, a traditional cross-drainage approach was used to mitigate impact from raising the rail embankment. This approach is not practical through Yelarbon due to the interface with the existing Graincorp silos and rail siding that are proposed to remain operational at the existing ground levels.

Therefore, to minimise the use of cross-drainage culverts, the proposed mitigation option involves a combination of raising the existing Yelarbon Flood Levee and cross-drainage through the proposed rail embankment (where the interface with the existing Graincorp infrastructure is not an issue and where flood sensitive receptors are not impacted by cross-drainage flows).

In order to maintain the existing flood immunity, additional works are proposed to the Cunningham Highway to tie in with the proposed levee. The existing cross-drainage structure through the Cunningham Highway at Yelarbon will be maintained in the proposed works.

A number of different levee extents and heights were investigated for the proposed levee at Yelarbon. The two extents which were investigated were 1.3 km (East Levee only) and 1.8 km (East and West Levee connected). For these two extents, raises between 200 mm and "Glass Wall Raise" (approximately 1.5 m) were investigated. The investigation focused on the 1% AEP impacts to the Yelarbon Township and the trafficability of the Cunningham Highway but also considered reducing impacts under more frequent events.

The proposed levee extent and raise height is outlined in Figure 85, and shown in the impact maps (Figures J-4a to J-4j) in Volume II – Appendix J.

Figure 85 **Yelarbon levee – proposed raise heights**

16.6.3 Flood impact objectives outcomes – Macintyre Brook Yelarbon to Inglewood

The impact of the Developed Case was assessed through inclusion of the proposed rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, duration of inundation, velocities and peak flows within the floodplain:

- Changes in peak water levels for the AEP's assessed are presented in Figures J-4a to J-4h in Volume II – Appendix J
- Changes in 1% AEP duration of inundation are presented in Figure J-4i in Volume II – Appendix J
- Changes in 1% AEP velocities are presented in Figure J-4j in Volume II – Appendix J.

All impacts are required to be agreed with relevant stakeholders and affected landowners as part of the EIS process. One-on-one consultation with landowners who are expected to experience changes in flooding behaviour on the property was conducted by ARTC supported by FFJV.

Generally, the proposed alignment corridor runs outside of the 1% AEP event inundation extents on the Macintyre Brook floodplain from Yelarbon to Inglewood. There are only discreet locations where impacts are expected to occur. In Inglewood there are no impacts predicted as a result of the Project rail alignment. There are three locations where impacts are predicted above 10 mm in the 1% AEP:

- Yelarbon
- Whetstone
- Millmerran-Inglewood Road at Thornton Road at Pariagara Creek.

It is noted that the local catchment crossings of Cremascos Road and Bybera Road, and Pariagara Creek are discussed in separate sections of this report.

The Project design outcomes relative to the flood impact objectives (refer Table 4.2) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Volume II – Appendix N.

16.6.3.1 Flood impacts at proposed hydraulic structures

The estimated potential impacts to peak water levels at each proposed structure are presented in Table 15.12. Peak water levels were extracted immediately upstream of each culvert and at the control line of each bridge.

The design achieved a freeboard of at least 300 mm between the 1% AEP peak water and formation levels.

Table 16.23 Macintyre Brook Yelarbon to Inglewood – 1% AEP event – impacts to peak water levels at proposed hydraulic structures

Chainage (m)	Structure ID	Structure type	Rail formation level (m AHD)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Change in peak water level (mm)
25.15	C25.15	RCBC	245.61	244.35	244.33	-20
25.19	C25.19	RCBC	245.61	244.36	244.33	-25
25.46	C25.46	RCP	245.61	244.50	244.47	-28
25.50	C25.50	RCP	245.61	244.52	244.49	-30
25.80	C25.80	RCBC	245.64	244.52	244.60	+84
25.87	C25.87	RCBC	245.64	244.61	244.64	+26
25.95	C25.95	RCBC	245.66	244.70	244.73	+30
25.97	C25.97	RCBC	245.66	244.69	244.72	+30
27.05	C27.05	RCBC	246.53	245.78	245.81	+31
27.15	C27.15	RCBC	246.70	245.67	245.74	+68
27.24	C27.24	RCBC	246.78	245.85	246.06	+214
27.33	C27.33	RCBC	246.87	245.95	246.09	+136
27.42	C27.42	RCBC	246.95	246.02	246.05	+31
27.53	C27.53	RCBC	247.05	245.87	245.91	+38
42.87	C42.88	RCP	265.08	263.97	264.02	+55
43.02	C43.02	RCP	265.12	263.95	264.03	+83
43.08	C43.08	RCP	265.14	263.95	264.02	+70
43.16	C43.16	RCBC	265.17	263.97	264.03	+57
43.34	C43.34	RCP	265.21	264.05	264.08	+31
43.56	C43.56	RCP	265.28	264.15	264.17	+16
43.66	C43.66	RCP	265.31	264.20	264.21	+6
43.77	C43.77	RCP	265.34	264.26	264.27	+9
43.86	C43.86	RCP	265.37	264.31	264.34	+32
43.97	C43.97	RCP	265.40	264.38	264.43	+47
44.32	C44.32	RCP	265.50	264.55	264.61	+56
44.67	C44.67	RCP	265.50	264.72	264.77	+47
44.88	C44.88	RCP	265.78	264.84	264.95	+108
44.99	C44.99	RCP	265.94	264.90	265.02	+123
45.24	C45.24	RCP	266.36	265.10	265.23	+126
45.30	C45.30	RCP	266.45	265.14	265.26	+117

Chainage (m)	Structure ID	Structure type	Rail formation level (m AHD)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Change in peak water level (mm)
45.39	C45.39	RCP	266.68	265.25	265.34	+95
45.46	C45.46	RCP	266.68	265.35	265.44	+89
45.53	C45.53	RCP	266.80	265.45	265.52	+69
45.67	C45.67	RCP	267.25	265.69	266.04	+352

16.6.3.2 Flood impacts on flood sensitive receptors

Flood sensitive receptors were identified from aerial imagery. Details of where afflux is greater than 10 mm, for events up to the 1% AEP are summarised in Table 16.24. Impacted flood sensitive receptors are labelled in the impact figures in Volume II – Appendix J, Figures J-4a to J-4j.

Impacts to flood sensitive receptors that exceed the flood impact objectives are reported in the EIS Surface Water Chapter.

Table 16.24 Macintyre Brook Yelarbon to Inglewood – estimated impacts to peak water levels at flood sensitive receptors

Flood sensitive receptor ID	Description	Afflux > +/- 10 mm				
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
MCB_ID_82	Shed	-	-	-	-	+14
MCB_ID_246	Yelarbon Township (inside levee)	-	-	-	-200	-71
MCB_ID_228	Shed	-	-	-	-199	-72
MCB_ID_229	Shed	-	-	-	-119	-40
MCB_ID_230	House	-	-	-	+31	-14
MCB_ID_231	Shed	-	-	-	-200	-92
MCB_ID_232	Shed	-	-	-	-185	-89
MCB_ID_233	Shed	-	-	-	-177	-85
MCB_ID_234	Shed	-24	-23	-13	+35	-2
MCB_ID_235	Shed	-	-	-	-26	-93
MCB_ID_237	Silos	-	-	-	-	-170
MCB_ID_238	Shed	-	-	-	-35	-103
MCB_ID_239	Shed	-	-	-	-67	-135
MCB_ID_240	Shed	-	-	-	-59	-137
MCB_ID_241	House	-	-	-	+59	+20
MCB_ID_242	House	-	-	-	+72	+40
MCB_ID_243	House	-	-	-	+108	+50
MCB_ID_244	House	-	-	-	+52	+20

16.6.3.3 Yelarbon

The change in depth between the Existing Case Developed Case at Yelarbon under the 1% AEP event is shown in Figure J-4e in Volume II – Appendix J. Table 16.25 outlines that the change in peak water levels at the Yelarbon Township is generally -0.1 m in a 1% AEP event. On the eastern side (i.e. outside) of the levee increases of up to approximately +0.3 m are predicted in a 1% AEP event.

Table 16.25 also presents changes in peak water levels for extreme events within town at Yelarbon.

Table 16.25 Change in peak water levels

Location	Change in peak water levels (m)							
	PMF	1 in 10000 AEP	1 in 2000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Cunningham Highway (inside Levee)	-	+0.1	+0.1	-0.2	-0.3	-	-	-
Cunningham Highway (outside levee)	+0.1	+0.2	+0.2	+0.2	+0.2	+0.2	-	-
Yelarbon Township (inside levee)	-	+0.1	+0.1	-0.1	-0.3	-0.5	-	-

16.6.3.4 Whetstone

Under the 1% AEP event the change in peak water levels at Whetstone is predicted to up to 120 mm. The 120 mm increase is predicted on the north-east side of the level crossing adjacent to Morrish Road. Increases of up to 70 mm are predicted on the northern side of the Project alignment on Yelarbon-Kurrumbul Road.

16.6.3.5 Millmerran-Inglewood Road at Thornton Road, Pariagara Creek

Under the 1% AEP event the change in peak water levels is predicted up to 210 mm which is localised on the southern side of the Project alignment dissipating to less than 30 mm within 400 m upstream. There is a localised increase predicted at Thornton Road on the southern side of the alignment of 70 mm.

16.6.3.6 Flood impacts on private land outside the rail disturbance footprint

The majority of the area where afflux is expected is agricultural land or open land on which nominal afflux is unlikely to cause any adverse impact. Table 16.26 presents the modelled changes in flood conditions during the 1% AEP event on a lot basis according to the following thresholds:

- Peak water levels increased by greater than +10 mm
- Peak velocities increased by greater than 0.25 m/s
- Duration of inundation changed by more than 25% of its original duration of inundation across the lot.

Table 16.26 Macintyre Brook Yelarbon to Inglewood - summary of flood impacts on private land outside the rail disturbance footprint for 1% AEP

Approximate chainage (km)	Changes in peak water levels[1]		Changes in peak velocities		Changes in Duration of inundation (hrs)	
	Maximum change (mm)	Total area affected by change > 10 mm (ha)[2]	Maximum change (m/s)	Total area affected by change (ha)	Maximum change (%)[3]	Total area affected by change (ha)
70.10 to 70.30	+64	4.50	-	-	-	-
68.70 to 69.30	+320	11.6	+0.3	33.8	-	-
44.60 to 45.50	+135	23.10	-	-	-	-
45.75	+15	0.03	-	-	-	-
45.60 to 45.90	+288	11.64	+0.3	0.11	-	-
69.30	+12	53.08	-	-	-	-
69.80	+104	6.50	-	-	-	-
44.5	+78	136.63	-	-	-	-
26.42	+22	0.01	-	-	+70%	0.10
26.39	+22	0.03	-	-	+75%	0.11
26.37	+57	0.02	-	-	-	-

Approximate chainage (km)	Changes in peak water levels[1]		Changes in peak velocities		Changes in Duration of inundation (hrs)	
	Maximum change (mm)	Total area affected by change > 10 mm (ha)[2]	Maximum change (m/s)	Total area affected by change (ha)	Maximum change (%)[3]	Total area affected by change (ha)
17.99	+24	0.50	-	-	-	-
26.10	+110	24.04	-	-	-	-
23.51	+22	258.90	-	-	-	-
24.44	+11	1.14	-	-	-	-
24.35	+24	45.19	-	-	-	-
22.84	+14	2.23	-	-	-	-
16.61	+35	0.02	-	-	-	-
16.60	+17	0.02	-	-	-	-
22.28	+16	14.66	-	-	-	-
25.31	+134	13.84	-	-	-	-
17.28	+24	0.02	-	-	-	-
26.00 to 27.00	+166	166.10	-	-	-	-
26.02	+42	3.20	-	-	-	-
26.23	-	-	-	-	+70%	0.10
26.29	-	-	-	-	+71%	0.10
26.31	-	-	-	-	+71%	0.10
26.33	-	-	-	-	+72%	0.10
26.36	-	-	-	-	+70%	0.12
28.32	+86	1.73	-	-	-	-
26.19	-	-	-	-	+72%	0.10
26.04	-	-	-	-	+75%	0.08
26.06	-	-	-	-	+72%	0.08
26.89	-	-	-	-	+75%	0.16
26.20	-	-	-	-	+61%	0.02
26.22	-	-	-	-	+64%	0.05
26.72	+18	0.02	-	-	-	-
26.34	-	-	-	-	+74%	0.10
26.41	-	-	-	-	+72%	0.10
25.75	+114	12.00	-	-	-	-
26.32	+22	0.13	-	-	-	-
26.08	-	-	-	-	+66%	0.07
26.10	-	-	-	-	+73%	0.08
26.16	-	-	-	-	+72%	0.10
26.18	-	-	-	-	+72%	0.11
26.06	-	-	-	-	+71%	0.10
26.19	-	-	-	-	+69%	0.12
26.52	-	-	-	-	+72%	0.12
26.55	-	-	-	-	+71%	0.12
26.58	-	-	-	-	+70%	0.10

Approximate chainage (km)	Changes in peak water levels[1]		Changes in peak velocities		Changes in Duration of inundation (hrs)	
	Maximum change (mm)	Total area affected by change > 10 mm (ha)[2]	Maximum change (m/s)	Total area affected by change (ha)	Maximum change (%)[3]	Total area affected by change (ha)
31.46	-	-	-	-	+14%	27.57
26.19	-	-	-	-	+62%	0.04
26.19	-	-	-	-	+67%	0.08
26.16	-	-	-	-	+70%	0.11
26.14	-	-	-	-	+70%	0.10
26.08	-	-	-	-	+71%	0.10
26.54	-	-	-	-	+71%	0.13
27.10	+79	2.09	-	-	-	-
28.32	+34	32.16	-	-	-	-
27.80	+59	71.34	-	-	-	-
26.56	-		-	-	+72%	0.11
26.25	-		-	-	+71%	0.07
26.43	-		-	-	+72%	0.11
26.65	+18	0.03	-	-	-	-
26.14	-		-	-	+72%	0.10
26.16	-		-	-	+72%	0.10
25.46	+41	0.02	-	-	-	-
26.95	+73	10.70	-	-	-	-
25.44	+33	56.24	-	-	-	-
26.33	+47	0.20	-	-	-	-
26.28	+38	0.08	-	-	-	-
26.23	+35	0.13	-	-	-	-
31.00	+17	0.01	-	-	-	-
26.25	-	-	-	-	+130%	0.20
26.31	-	-	-	-	+62%	0.11
26.29	-	-	-	-	+67%	0.10
26.18	-	-	-	-	+70%	0.11
26.08	-	-	-	-	+74%	0.10
26.37	-	-	-	-	+68%	0.11
26.33	-	-	-	-	+72%	0.11
26.27	-	-	-	-	+64%	0.08
26.35	-	-	-	-	+61%	0.10
26.30	-	-	-	-	+63%	0.11
26.20	-	-	-	-	+74%	0.10
26.60	-	-	-	-	+72%	0.10
26.52	-	-	-	-	+72%	0.11
26.66	-	-	-	-	+74%	0.10
25.76	+14	0.01	-	-	-	-
26.43	-	-	-	-	+65%	0.11

Approximate chainage (km)	Changes in peak water levels[1]		Changes in peak velocities		Changes in Duration of inundation (hrs)	
	Maximum change (mm)	Total area affected by change > 10 mm (ha)[2]	Maximum change (m/s)	Total area affected by change (ha)	Maximum change (%)[3]	Total area affected by change (ha)
26.27	-	-	-	-	+64%	0.10
26.39	-	-	-	-	+67%	0.11
26.37	-	-	-	-	+64%	0.10
26.33	-	-	-	-	+66%	0.10
26.33	-	-	-	-	+60%	0.09
26.60	-	-	-	-	+72%	0.11
26.56	-	-	-	-	+72%	0.10
26.56		-	-	-	+72%	0.11
26.30	-	-	-	-	+140%	0.10
26.28	-	-	-	-	+127%	0.11
26.52	-	-	-	-	+62%	0.10
26.52	-	-	-	-	+60%	0.10
26.48	-	-	-	-	+56%	0.10
26.60	-	-	-	-	+55%	0.10
26.58	-	-	-	-	+67%	0.11
26.56	-	-	-	-	+68%	0.10
26.56	-	-	-	-	+65%	0.10

Table notes:

1 Afflux on lots that exceed the flood impact objectives are summarised in the EIS Surface Water Chapter
2 Only minor areas, usually directly upstream of culverts are affected by the maximum afflux as stated
3 Only minor areas affected by changes in duration, mostly restricted to a couple of cells in the hydraulic model

16.6.3.7 Flood impacts on Queensland Rail

From Yelarbon to Whetstone the proposal alignment is within the existing QR corridor. From Whetstone to Inglewood the Project alignment deviates from the existing QR rail. There is one location where impact is predicted along the QR rail alignment outside of the EIS disturbance footprint. This location is at Whetstone where the Project alignment tracks north and the QR existing line continues in an eastern direction. The location (approximately Chainage 45 km), is shown on Figure J-4e. The predicted 1% AEP afflux is up to 150 mm immediately to the east of the Project alignment and dissipates to less than 100 mm within 200 m along the QR existing line. The existing 1% AEP flood depth at this location is 650 mm in the 1% AEP existing flood event.

16.6.3.8 Flood impacts on state-controlled roads

The extent of the hydraulic model developed for Macintyre Brook is shown in Figure 86. Within the extent of the hydraulic model, the state-controlled roads which are influenced by flooding and the Project alignment are:

- Cunningham Highway
- Millmerran-Inglewood Road
- Yelarbon – Keetah Road.

The locations of the state-controlled roads are shown in Figure 86.

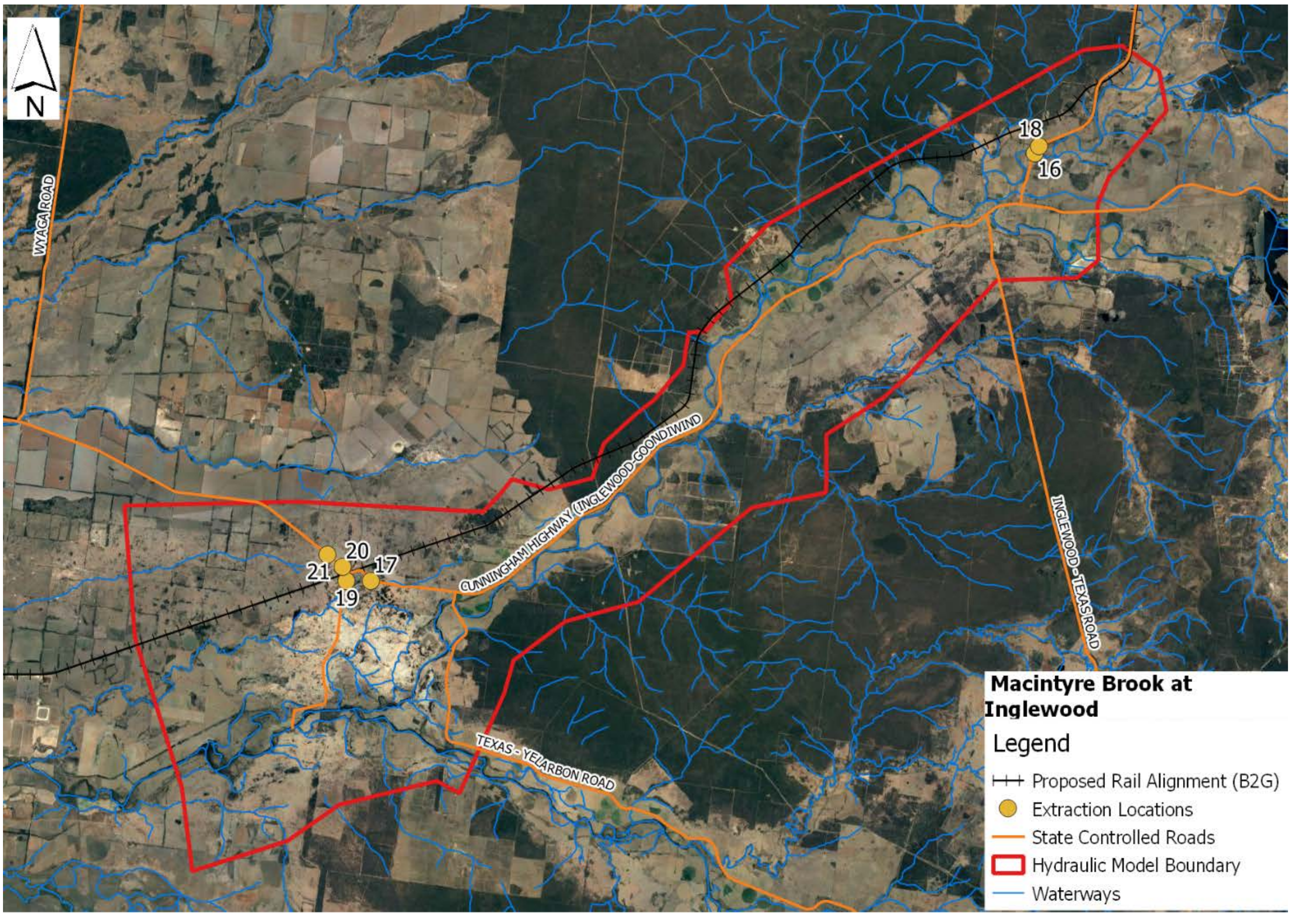


Figure 86 **Macintyre Brook Hydraulic Model Extent and Associated State-controlled Roads**

The following sections describe the impacts to state-controlled roads in both the Existing Case and the Developed Case and summarises the differences between the two.

All state-controlled roads in the Macintyre Brook model extent model have low flood immunity in the Existing Case scenario, with most roads having less than 20% AEP immunity.

Existing Case flooding conditions

Table 16.27 **Macintyre Brook - Existing Case flood depths**

Reporting location	Road	Estimated depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
17	Cunningham Highway	0.00	0.00	0.18	0.42	0.53	1.06	1.36	2.59
18	Millmerrran-Inglewood Road	0.00	0.00	0.00	0.00	0.02	0.58	1.02	3.77
19	Yelarbon - Keetah Road	0.00	0.00	0.53	1.00	1.21	1.90	2.28	3.64
20	Cunningham Highway	0.02	0.06	0.09	0.16	0.26	0.77	1.11	2.40
21	Cunningham Highway	0.18	0.22	0.27	0.32	0.35	0.57	0.87	2.14

Table 16.28 Macintyre Brook - Existing Case flood inundation length

Reporting location	Road	Approximate length of inundation (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
17	Cunningham Highway	0	0	390	4,604	4,616	43,615	43,735	44,480
18	Millmerrran-Inglewood Road	0	0	0	0	3,100	3,310	3,450	5,473
19	Yelarbon - Keetah Road	0	0	1,514	9,253	9,253	9,253	9,253	9,253
20	Cunningham Highway	13	51	92	481	3,812	4,079	4,015	4,295
21	Cunningham Highway	1,758	1,786	2,791	3,118				

Table 16.29 Macintyre Brook - Existing Case time of submergence

Reporting location	Road	Estimated time of submergence (hrs)								AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
17	Cunningham Highway	0.0	0.0	15.4	26.3	35.1	53.4	54.7	91.7	1.8
18	Millmerrran-Inglewood Road	39.3	59.4	47.8	49.3	70.0	68.5	64.7	102.3	26.1
19	Yelarbon - Keetah Road	56.0	82.5	55.4	61.1	66.5	80.8	82.8	103.6	35.9
20	Cunningham Highway	30.2	41.3	34.9	29.0	43.3	61.4	59.1	95.4	19.4
21	Cunningham Highway	74.2	95.0	89.9	83.3	97.6	89.1	88.7	115.5	47.2

Developed Case flooding conditions

Table 16.30 Macintyre Brook - Developed Case flood depths

Reporting location	Road	Estimated depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
17	Cunningham Highway	0.00	0.00	0.36	0.61	0.72	1.16	1.47	2.61
18	Millmerrran-Inglewood Road	0.00	0.00	0.00	0.00	0.02	0.62	1.08	3.78
19	Yelarbon - Keetah Road	0.00	0.05	0.35	0.92	1.17	1.97	2.38	3.66
20	Cunningham Highway	0.00	0.07	0.09	0.12	0.16	0.79	1.16	2.46
21	Cunningham Highway	0.19	0.22	0.27	0.32	0.35	0.58	0.75	2.12

Table 16.31 Macintyre Brook - Developed Case flood inundation length

Reporting location	Road	Approximate length of inundation (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
17	Cunningham Highway	0	0	380	3,500	4,560	43,615	43,735	44,480
18	Millmerrran-Inglewood Road	0	0	0	0	3,075	3,292	3,427	5,473
19	Yelarbon - Keetah Road	0	146	1,307	9,253	9,253	9253	9,253	9,253
20	Cunningham Highway	13	46	73	113	135	178	4,012	4,291
21	Cunningham Highway	1,745	1,773	2,782	3,134	3,193	3,567		

Table 16.32 Macintyre Brook - Developed Case time of submergence

Reporting location	Road	Estimated time of submergence (hrs)								AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
17	Cunningham Highway	0.0	0.0	26.7	38.8	44.4	56.7	63.1	92.1	2.6
18	Millmerrran-Inglewood Road	39.3	59.4	47.5	60.0	70.0	71.1	65.4	104.1	26.3
19	Yelarbon - Keetah Road	52.3	75.0	60.4	62.2	67.8	83.2	85.1	106.2	33.9
20	Cunningham Highway	29.7	40.7	51.7	60.7	63.3	68.5	67.6	100.1	20.7
21	Cunningham Highway	75.6	96.7	93.7	96.6	98.6	89.1	90.5	115.5	48.4

Impacts of Project alignment

Table 16.33 Macintyre Brook – change in flood depths

Reporting location	Road	Estimated change in depths (m)							
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP
17	Cunningham Highway	0.00	0.00	0.17	0.19	0.19	0.11	0.11	0.03
18	Millmerrran-Inglewood Road	0.00	0.00	0.00	0.00	0.00	0.05	0.06	0.01
19	Yelarbon - Keetah Road	0.00	0.05	-0.19	-0.08	-0.04	0.07	0.10	0.02
20	Cunningham Highway	-0.01	0.00	0.00	-0.04	-0.09	0.02	0.05	0.06
21	Cunningham Highway	0.00	0.00	0.00	0.00	0.00	0.01	-0.12	-0.02

Table 16.34 Macintyre Brook - change in time of submergence

Reporting location	Road	Estimated change in time of submergence (hrs)								Estimated change in AATOS (hrs)
		20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2000	1 in 10000	PMP	
17	Cunningham Highway	0.0	0.0	0.0	4.8	5.0	3.1	2.2	0.3	0.2
18	Millmerrran-Inglewood Road	0.0	0.0	-0.3	10.7	0.0	2.5	0.7	1.8	0.2
19	Yelarbon - Keetah Road	-3.7	-7.5	5.0	1.1	1.3	2.4	2.3	2.7	-2.0
20	Cunningham Highway	-0.5	-0.6	16.8	31.7	20.0	7.1	8.5	4.7	1.3
21	Cunningham Highway	1.5	1.7	3.8	13.3	1.0	0.0	1.8	0.0	1.2

Change in flood hydrographs

Two hydrographs have been prepared using the 1% AEP Existing Case and Developed Case results from the Macintyre Brook hydraulic model. The first hydrograph, Figure 87, was extracted at ID point 19, located along Yelarbon – Keetah Road. The second hydrograph, Figure 88, has been extracted at ID point 21, located along the Cunningham Highway (Inglewood to Goondiwindi).

Figure 87 indicates a decrease in peak elevation, but a longer duration of inundation. Figure 88 shows no change in peak elevations and a negligible change in duration of inundation along the Cunningham Highway.

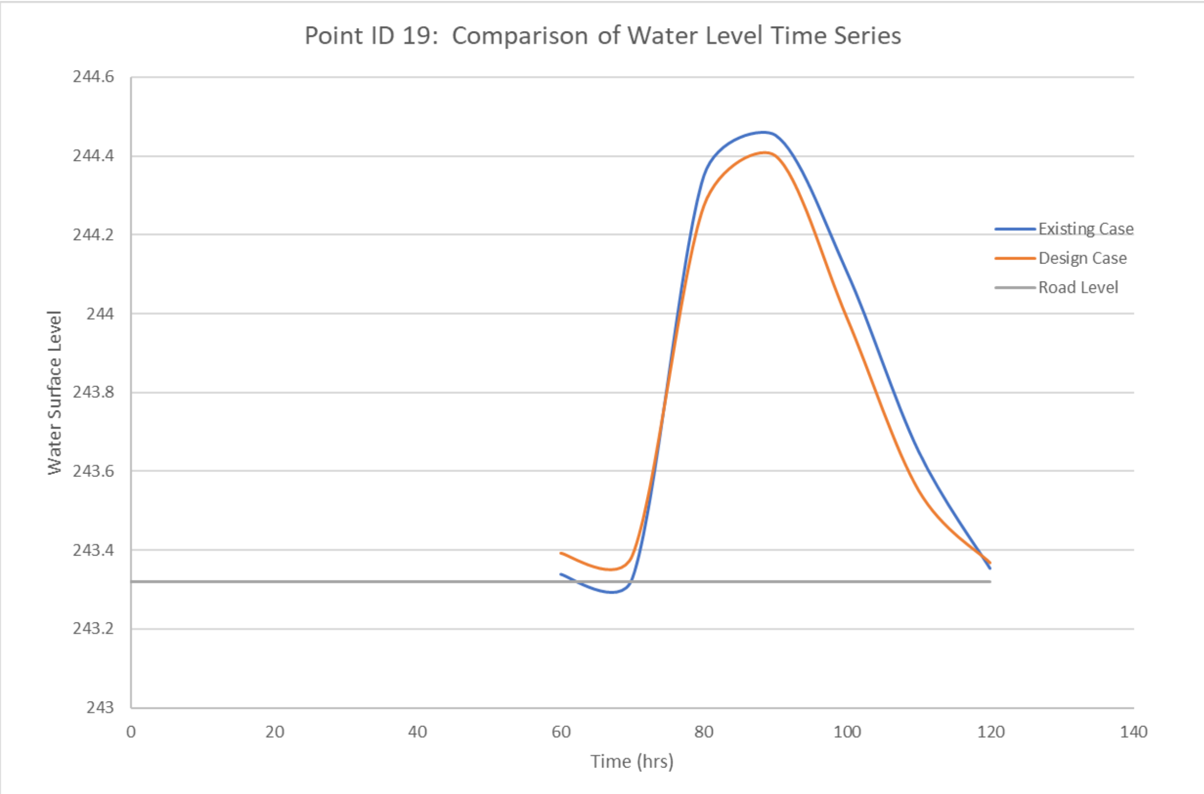


Figure 87 Extraction Point 19 - comparison of water level time series, 1% AEP

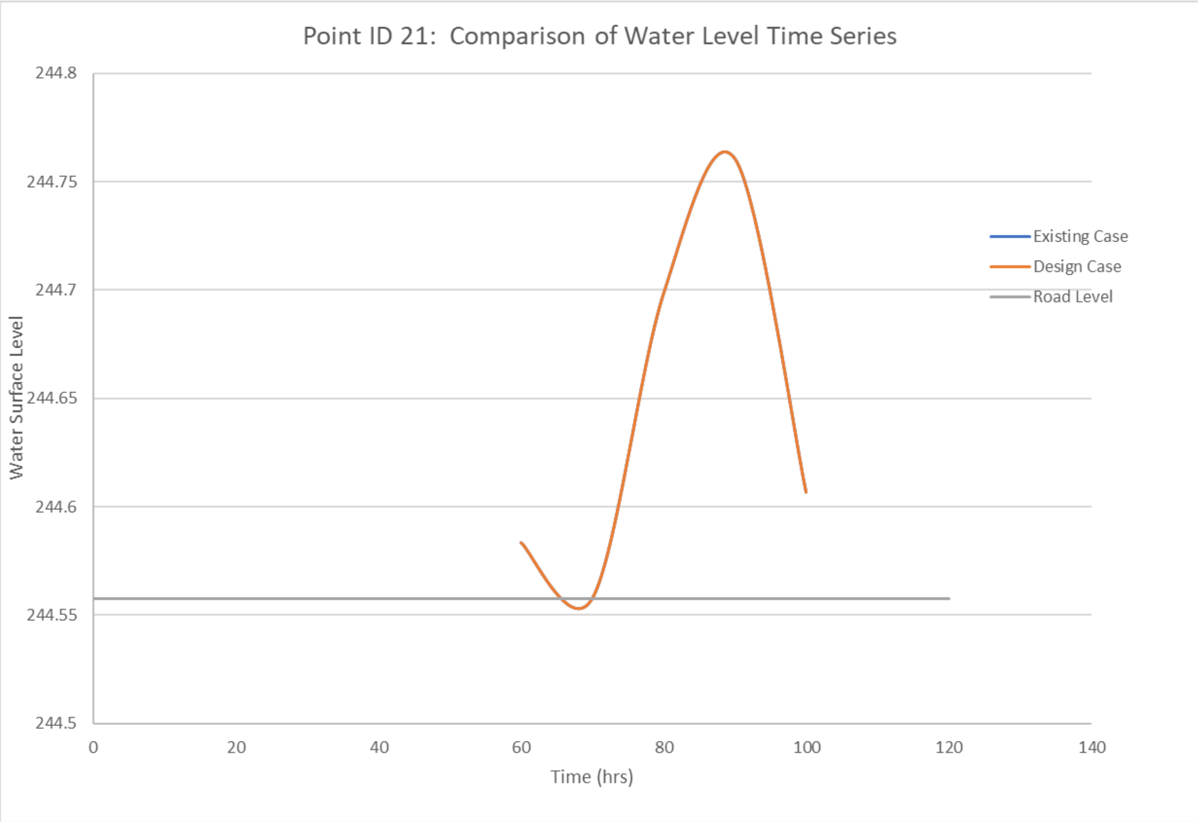


Figure 88 Extraction Point 21 - comparison of water level time series, 1% AEP

16.6.3.9 Flood impacts on local public roads

The change in peak water levels and flood hazard (velocity-depth) for the 1% AEP event were evaluated on local public roads within the hydraulic model domain. Local public roads that are expected to experience an increase in flood hazard and/or increases in peak flood levels are reported in Table 16.35.

Table 16.35 Macintyre Brook Yelarbon to Inglewood – changes in peak water levels and velocity depth and flood hazard for local public roads, 1% AEP

Location	Existing flood hazard (m^2/s)	Design flood hazard (m^2/s)	Maximum existing flood depth (m)	Maximum design flood depth (m)	Maximum change in peak water levels (mm)[1]
Aerodrome Road	0.151	0.151	0.53	0.53	+1
Albert Lane	0.228	0.228	1.77	1.77	+1
Alice Lane	0.714	0.714	3.37	3.37	+1
Alice Street	2.231	2.231	3.22	3.22	+1
Alice Street	0.229	0.58	0.692	0.86	+21
Babingtons Road	0.151	0.151	1.34	1.34	+1
Beechcraft Court	0.092	0.092	0.5	0.5	+1
Bethcar Road	0.925	0.926	1.08	1.09	+1
Bosnjaks Road	12.86	12.916	10.58	10.58	+6
Brook Street	0.958	0.958	2.66	2.66	+1
Brosnans Road	0.771	0.772	1.06	1.06	+1
Bybera Road	22.863	22.865	10.44	10.44	+1
Callandoon Lane	0.421	0.421	3.36	3.36	+1
Callandoon Street	1.728	1.729	4.05	4.05	+1
Campbells Lane	0.659	0.659	1.15	1.15	+1

Location	Existing flood hazard (m^2/s)	Design flood hazard (m^2/s)	Maximum existing flood depth (m)	Maximum design flood depth (m)	Maximum change in peak water levels (mm)[1]
Cemetery Road	0.351	0.351	0.57	0.57	+1
Cheshire Street	0.741	0.742	1.12	1.13	+1
Chilcott Street	0.449	0.45	1.4	1.4	+1
Clarkes Road	0.227	0.227	0.76	0.76	+1
Cremascos Road	14.884	14.885	10.73	10.73	+1
Denison Street	2.084	2.089	5.12	5.12	+1
East Sawmill Road	0.412	0.667	0.77	0.81	+179
East Street	0.614	0.614	2.46	2.46	+1
Elizabeth Street	0.504	0.504	2.02	2.02	+1
Frey Street	1.629	1.63	1.74	1.74	+1
George Lane	0.398	0.398	2.86	2.86	+1
George Street	0.558	0.558	2.91	2.91	+1
Girle Street	1.036	1.314	1.17	1.23	+60
Girle Street	1.036	1.314	1.17	1.23	+60
Gledsons Road	1.85	1.85	2.44	2.44	+1
Goodrich Street	0.721	0.722	1.29	1.29	+1
Great Road Street	1.034	1.035	3.25	3.25	+1
Grey Street	0.487	0.487	2.01	2.01	+1
Hansens Road	0.266	0.266	1.2	1.2	+1
Inglewood - Texas Road	0.92	0.92	1.324	1.324	+1
Killen Street	0.058	0.058	0.43	0.44	+1
King Lane	0.152	0.152	1.08	1.08	+1
King Street	0.198	0.198	1.12	1.12	+1
Lloyd Street	1.403	1.404	1.74	1.75	+1
Loupals Road	14.64	14.636	10.59	10.59	+6
Macintyre Street	0.417	0.417	1.4	1.4	+1
Mccorkells Road	0.791	0.792	1.18	1.18	+1
Mcdougalls Road	13.042	13.043	11.26	11.26	+1
Mcintyre Street	0.565	0.566	1.39	1.39	+1
Nicholas Street	0.164	0.164	1.18	1.18	+1
Park Lane	0.234	0.234	1.75	1.76	+1
Piper Court	0.156	0.156	0.43	0.43	+1
Potters Road	28.553	28.556	11.78	11.78	+1
Princess Lane	0.288	0.288	1.43	1.43	+1
Princess Street	0.221	0.221	1.58	1.58	+1
Queen Street	0.577	0.577	1.17	1.17	+1
Railway Lane	0.757	0.757	0.94	0.94	+1
Railway Parade	0.249	0.286	0.71	0.68	+7
Regent Lane	0.139	0.139	1.07	1.07	+1
Regent Street	0.252	0.252	1.71	1.71	+1
Reibelt Street	0.572	0.572	2.32	2.32	+1

Location	Existing flood hazard (m^2/s)	Design flood hazard (m^2/s)	Maximum existing flood depth (m)	Maximum design flood depth (m)	Maximum change in peak water levels (mm)[1]
Robinson Street	0.771	0.772	1.37	1.37	+1
School Lane	0.327	0.327	1.61	1.61	+1
Slack Street	0.462	0.462	1.11	1.11	+1
Tennis Lane	0.109	0.109	1.27	1.27	+1
Texas - Yelarbon Road	6.144	6.144	5.323	5.323	+0
Tobacco Road	1.729	1.729	2.37	2.37	+1
Tomkins Street	2.522	2.522	2.4	2.4	+1
Tomkins Street	0.173	0.173	1.08	1.08	+1
Tomlinsons Road	1.872	1.871	4.96	4.97	+4
Victoria Street	1.271	1.271	3.62	3.62	+1
Whetstone Access	2.137	2.139	2.77	2.78	+204
Wyemo Street	1.486	1.834	1.58	1.63	+63

Table note:

1 The maximum change in peak water level does not necessarily occur at the same location as where the existing and/or design maximum flood depth occur

Duration of inundation

Assessment of the time of submergence (ToS) and average annual time of submergence (AAToS) was undertaken for local public roads within the hydraulic model domain. Local public roads that are expected to experience increases in ToS and/or AAToS are presented in Table 16.36.

Table 16.36 Macintyre Brook Yelarbon to Inglewood – ToS and AAToS for local public roads

Location	Existing 1% AEP ToS (hrs)	1% AEP ToS diff. (hrs)	2% AEP ToS diff. (hrs)	5% AEP ToS diff. (hrs)	10% AEP ToS diff. (hrs)	AAToS Existing Case (hrs)	AAToS Developed Case (hrs)	AAToS diff. (hrs)
Albert Lane	63.92	0.01	-	-0.01	-0.01	7.92	7.92	-
Alice Lane	69.25	-	0.01	0.01	-0.03	30.75	30.74	-0.01
Alice Street	57.43	0.01	-	-0.01	-0.01	14.96	14.96	-
Babingtons Road	58.98	-0.01	-0.03	-	0.04	7.8	7.8	-
Beechcraft Court	18.88	-	2.2	-	-	0.59	0.64	0.04
Bengalla Street	69.66	1.57	2.59	11.51	-1.6	33.87	33.97	0.11
Bosnjaks Road	94.75	-	7.73	-	-	49.52	49.67	0.15
Brosnans Road	70.35	-	-	0.01	0.01	31.36	31.35	-
Bybera Road	103.78	-	9.43	-	-	59.28	59.47	0.19
Callandoon Street	67.21	0.01	-0.01	-0.03	-	17.68	17.68	-
Cremascos Road	99.91	-	8.68	-	-	57.19	57.37	0.17
Denison Street	87.84	-	0.4	-0.01	-	32.73	32.74	0.01
Frey Street	50.43	0.01	-	-0.02	0.01	5.22	5.22	-
Girle Street	47.07	0.64	0.59	0.14	0.04	4.31	4.34	0.03
Girle Street	47.07	0.64	0.59	0.14	0.04	4.31	4.34	0.03
Inglewood - Texas Road	62.53	-0.01	-0.01	-0.02	0.00	4.20	4.20	0.00
Kaboon Street	94.43	-	5.48	-	-	51.2	51.31	0.11

Location	Existing 1% AEP ToS (hrs)	1% AEP ToS diff. (hrs)	2% AEP ToS diff. (hrs)	5% AEP ToS diff. (hrs)	10% AEP ToS diff. (hrs)	AAToS Existing Case (hrs)	AAToS Developed Case (hrs)	AAToS diff. (hrs)
Lanes Lane	92.39	-	6.58	-	-	44.94	45.08	0.13
Largo North Road	73.37	-	7.57	-0.01	-0.01	27.89	28.04	0.15
Lloyd Street	61.73	-	-0.01	-0.01	0.02	7.13	7.13	-
Loupals Road	95.52	-	7.82	-	-	50.43	50.58	0.15
Mcdougalls Road	100.35	-	8.64	-	-	54.53	54.7	0.17
Oasis Lane	68.15	4.37	9.81	14.06	-24.59	35.89	34.29	-1.6
Park Lane	60.91	0.01	-	-0.02	-0.01	7.31	7.31	-
Potters Road	105.04	0.01	9.46	-	-	61.64	61.83	0.19
Princess Lane	29.89	-	2.15	-	-	2.26	2.3	0.04
Railway Parade	73.22	0.09	0.03	0.1	3.67	36.82	39.34	2.52
Regent Lane	35.71	0.01	-	-	-	2.14	2.14	-
Regent Street	67.14	-	-	-0.02	0.02	8.41	8.41	-
Reibelt Street	71.98	-	-0.01	-0.03	0.14	8.87	8.87	0.01
Springborg Road	70.77	-	-	-	0.02	29.79	29.79	-
Suttons Road	0	-	-	-	-	0.14	0.13	-0.01
Tennis Lane	32.51	0.01	0.92	-	-	1.89	1.91	0.02
Texas - Yelarbon Road	86.33	0.00	6.69	0.00	0.00	41.09	41.22	0.13
Tomlinsons Road	98.02	-	8.54	-	-	47.14	47.31	0.17
Victoria Street	54.55	-	-0.01	-0.01	0.02	17.09	17.08	-0.01
Wyemo Boundary Road	84.41	-	5.33	-	-	33.91	34.02	0.1
Wyemo Street	57.8	0.01	0.01	0.01	0.02	5.81	5.82	-

16.6.4 Sensitivity analysis – Macintyre Brook – Yelarbon to Inglewood

16.6.4.1 Blockage

Blockage was assessed in accordance with ARR 2016. The blockage assessment was undertaken and resulted in a blockage factor of 25% adopted for culverts. A minimum culvert size of 900 mm diameter was adopted to reduce potential for blockage and maintenance. A sensitivity analysis was undertaken with 0% and 50% blockage. The 1% AEP change in peak water levels for the 0% and 50% blockage scenarios are presented in Volume II – Appendix J, Figure J-5a and J-5b respectively.

No allowance for blockage at bridges was made, as the likelihood of significant blockage is low, given the nature of the catchments and the predominant vegetation of the waterways.

The model predicts that in the 0% blocked case the predicted changes in peak water levels meet the design criteria for the 1% AEP event. In the 50% blockage case localised increases are predicted at Yelarbon, Whetstone and Millmerran-Inglewood Road at Thornton Road, Pariagara Creek.

Table 16.37 provides a summary of 1 % AEP peak flood levels at cross drainage structures for the blockage scenarios.

Table 16.37 Macintyre Brook (Yelarbon to Inglewood) – 1 % AEP event – culvert blockage assessment

Structure ID	Structure type	1 % AEP Peak water levels (m AHD)			Increase from Developed Case to 50% blockage scenario (m)
		0 % blockage	Developed Case (25% blockage)	50% blockage	
C25.15	RCBC	244.32	244.33	244.33	+7
C25.19	RCBC	244.33	244.33	244.33	+7
C25.46	RCP	244.44	244.47	244.50	+31
C25.50	RCP	244.47	244.49	244.52	+26
C25.80	RCBC	244.59	244.60	244.62	+20
C25.87	RCBC	244.64	244.64	244.66	+11
C25.95	RCBC	244.71	244.73	244.75	+26
C25.97	RCBC	244.71	244.72	244.75	+25
C27.05	RCBC	245.77	245.81	245.88	+73
C27.15	RCBC	245.77	245.74	245.68	-64
C27.24	RCBC	245.98	246.06	246.15	+94
C27.33	RCBC	246.01	246.09	246.19	+101
C27.42	RCBC	245.97	246.05	246.18	+133
C27.53	RCBC	245.86	245.91	246.02	+108
C42.88	RCP	263.99	264.02	264.10	+71
C43.02	RCP	264.00	264.03	264.10	+67
C43.08	RCP	264.00	264.02	264.09	+68
C43.16	RCBC	264.01	264.03	264.10	+65
C43.34	RCP	264.08	264.08	264.13	+45
C43.56	RCP	264.16	264.17	264.20	+28
C43.66	RCP	264.21	264.21	264.23	+20
C43.77	RCP	264.27	264.27	264.29	+13
C43.86	RCP	264.34	264.34	264.35	+5
C43.97	RCP	264.43	264.43	264.42	-7
C44.32	RCP	264.62	264.61	264.58	-32
C44.67	RCP	264.76	264.77	264.77	+9
C44.88	RCP	264.93	264.95	264.97	+21
C44.99	RCP	265.00	265.02	265.05	+33
C45.24	RCP	265.21	265.23	265.27	+37
C45.30	RCP	265.24	265.26	265.31	+53
C45.39	RCP	265.32	265.34	265.39	+51
C45.46	RCP	265.43	265.44	265.47	+29
C45.53	RCP	265.52	265.52	265.53	+8
C45.67	RCP	266.02	266.04	266.09	+45

Table 16.38 outlines the changes in peak water levels at flood sensitive receptors for the 50% blockage scenario where the increase exceeds 10 mm.

Table 16.38 Macintyre Brook Yelarbon to Inglewood – Summary of 50% blockage impacts at flood sensitive receptors

Flood sensitive receptor ID	Existing case flood depth (m)	Change in peak water level (mm)
MCB_ID_230	0.39	-46
MCB_ID_241	0.33	+40
MCB_ID_242	0.33	+42
MCB_ID_243	0.33	+60
MCB_ID_82	0.96	+21

During detail 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.

16.6.4.2 Impacts during extreme events

Table 16.39 outlines the changes in peak water levels at flood sensitive receptors for the extreme events where the increase exceeds 10 mm under one of the events. The existing depth of flooding is also detailed and as can be seen the larger impacts that occur under the PMF event occur generally when there are already high flood depths as would be expected under such a rare event.

Flood immunity of the Project alignment is discussed in Section 16.6.2.1, and maps demonstrating the impacts during extreme events are shown in Volume II – Appendix J, Figures J-4f to J-4h.

Table 16.39 Macintyre Brook Yelarbon to Inglewood – Summary of extreme event impacts at flood sensitive receptors

Flood sensitive receptor ID	1 in 2,000 AEP event		1 in 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)
MCB_ID_10	+4	1.37	+16	1.81	+14	3.29
MCB_ID_11	+5	1.36	+15	1.80	+14	3.29
MCB_ID_12	+4	1.28	+14	1.72	+13	3.21
MCB_ID_13	0	1.28	+13	1.79	+12	3.29
MCB_ID_14	0	1.44	+12	1.96	+11	3.49
MCB_ID_15	0	1.41	+12	1.94	+11	3.48
MCB_ID_16	0	1.38	+5	1.77	+8	3.17
MCB_ID_230	-22	0.95	+17	1.32	+45	2.61
MCB_ID_233	+103	1.02	+63	1.39	+44	2.67
MCB_ID_241	+45	0.56	+69	0.91	+23	2.19
MCB_ID_242	+35	0.51	+66	0.85	+21	2.09
MCB_ID_243	+36	0.69	+67	1.03	+18	2.24
MCB_ID_244	+48	0.42	+72	0.77	+25	2.06
MCB_ID_246	+91	1.30	+69	1.67	+39	2.96
MCB_ID_75	+42	1.74	+33	2.44	+11	5.08
MCB_ID_76	+42	1.03	+33	1.73	+11	4.35
MCB_ID_78	-	-	-	-	+10	2.15
MCB_ID_79	-	-	-	-	+11	2.08
MCB_ID_80	-	-	-	-	+11	1.92
MCB_ID_82	+96	1.94	+85	2.63	+23	5.30

Flood sensitive receptor ID	1 in 2,000 AEP event		1 in 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)
MCB_ID_83	+66	1.79	+62	2.47	+20	5.12
MCB_ID_86	+30	2.38	+32	3.06	+13	5.75
MCB_ID_87	+29	0.43	+32	1.12	+13	3.82
Cunningham Highway North	+110	1.52	+85	1.87	+63	3.10
Cunningham Highway	+159	1.53	+126	1.91	+26	3.15
Existing QR Rail Line	+166	2.03	+86	2.80	+21	5.59
Access Road	+170	2.16	+142	2.86	+30	5.55

16.6.4.3 Climate change

The impacts of climate change were assessed for the Macintyre Brook floodplain for the 1% AEP 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.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario has been adopted for the Project with an associated increase in rainfall intensity of 23% across the catchment area.

Table 16.40 presents the structure performance with Representative Concentration Pathways 8.5 climate change conditions. For the 1% AEP event, the change in peak water levels for the Representative Concentration Pathways 8.5 climate change scenario is presented in Figure J-5c in Volume II – Appendix J.

Climate change results are expected to increase peak water levels upstream of the Project alignment by up to 0.5 m at structure locations for the 1% AEP event. The Project formation is not predicted to be overtopped as a result of the 23% increase in rainfall intensity.

Table 16.40 Macintyre Brook Yelarbon to Inglewood – 1% AEP event with Representative Concentration Pathways 8.5 conditions – structure performance

Approximate chainage (km)	Structure ID	Structure type	1 % AEP Peak water levels (m AHD)	1 % AEP + Climate Change Peak water levels (m AHD)	Difference in peak water levels (m)	Freeboard to rail formation with climate change (m)
25.15	C25.15	RCBC	244.33	244.62	+0.30	1.37
25.19	C25.19	RCBC	244.33	244.63	+0.30	1.38
25.46	C25.46	RCP	244.47	244.71	+0.24	1.26
25.50	C25.50	RCP	244.49	244.73	+0.24	1.25
25.80	C25.80	RCBC	244.60	244.89	+0.29	1.16
25.87	C25.87	RCBC	244.64	244.94	+0.30	1.12
25.95	C25.95	RCBC	244.73	245.09	+0.36	1.04
25.97	C25.97	RCBC	244.72	245.08	+0.36	1.05
27.05	C27.05	RCBC	245.81	246.11	+0.30	0.97
27.15	C27.15	RCBC	245.74	245.93	+0.19	1.06
27.24	C27.24	RCBC	246.06	246.19	+0.13	0.92
27.33	C27.33	RCBC	246.09	246.24	+0.15	0.97
27.42	C27.42	RCBC	246.05	246.27	+0.22	1.10
27.53	C27.53	RCBC	245.91	246.25	+0.34	1.30

Approximate chainage (km)	Structure ID	Structure type	1 % AEP Peak water levels (m AHD)	1 % AEP + Climate Change Peak water levels (m AHD)	Difference in peak water levels (m)	Freeboard to rail formation with climate change (m)
42.87	C42.88	RCP	264.02	264.52	+0.49	1.24
43.02	C43.02	RCP	264.03	264.52	+0.49	1.26
43.08	C43.08	RCP	264.02	264.51	+0.48	1.29
43.16	C43.16	RCBC	264.03	264.51	+0.48	1.30
43.34	C43.34	RCP	264.08	264.54	+0.46	1.28
43.56	C43.56	RCP	264.17	264.62	+0.45	1.26
43.66	C43.66	RCP	264.21	264.65	+0.44	1.26
43.77	C43.77	RCP	264.27	264.71	+0.44	1.22
43.86	C43.86	RCP	264.34	264.77	+0.43	1.18
43.97	C43.97	RCP	264.43	264.86	+0.43	1.12
44.32	C44.32	RCP	264.61	265.04	+0.43	1.04
44.67	C44.67	RCP	264.77	265.20	+0.44	1.07
44.88	C44.88	RCP	264.95	265.39	+0.44	1.22
44.99	C44.99	RCP	265.02	265.48	+0.46	1.34
45.24	C45.24	RCP	265.23	265.68	+0.45	1.50
45.30	C45.30	RCP	265.26	265.75	+0.49	1.59
45.39	C45.39	RCP	265.34	265.80	+0.46	1.65
45.46	C45.46	RCP	265.44	265.86	+0.42	1.63
45.53	C45.53	RCP	265.52	265.88	+0.36	1.67
45.67	C45.67	RCP	266.04	266.44	+0.40	1.38

Table 16.41 outlines the changes in peak water levels at flood sensitive receptors for the climate change scenario where the increase exceeds 10 mm.

Table 16.41 Macintyre Brook Yelarbon to Inglewood – Summary of climate change impacts at flood sensitive receptors

Flood sensitive receptor ID	1% AEP climate change event	
	Change in peak water level (mm)	Existing case flood depth (m)
MCB_ID_241	+10	0.33
MCB_ID_243	+11	0.33
MCB_ID_75	+14	0.69
MCB_ID_76	+15	-[2]
MCB_ID_82	+39	0.96
MCB_ID_83	+20	0.83
Cunningham Highway North [1]	+91	1.19
Cunningham Highway [1]	+226	1.20
Existing QR Rail Line	+135	1.38
Access Road	+87	1.51

Table notes:

1 These roads are affected by climate change regardless of the Project and so the amenity of the roads is not compromised by the Project

2 Not currently flooded

The downstream extents of these impacts are similar to those under the 1% AEP event.

17 Macintyre Brook at Bybera Road

The location of the Project rail alignment in relation to Macintyre Brook at Bybera Road is shown in Figure K-1a in Volume II – Appendix K.

17.1 Data collection and review – Macintyre Brook at Bybera Road

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:

- GRC – existing flood studies
- The BoM – rainfall data
- DTMR – existing infrastructure details.

17.1.1 Previous studies

Goondiwindi Regional Council, Inglewood Flood Study, Engeny, 2015

Engeny was commissioned by GRC to undertake a flood study of Inglewood. The study objectives were *"to define the nature, extent and risks of flooding in Inglewood in order to inform disaster management planning and response, as well as control future development*" (Engeny, 2015). An URBS hydrologic model and TUFLOW (1D/2D) hydraulic model were developed.

Inland Rail: Phase 2 - North Star to Border, 2018

The NS2B section of Inland Rail will cross the Macintyre River and its floodplain which are a part of the Border Rivers catchment. The NS2B alignment runs through Moree Plains LGA, Gwydir LGA and Goondiwindi LGA. To establish the reliability of the models the 2011 event was included in the calibration process in addition to the 1976 and 1996 events assessed by DPIE. The hydrologic and hydraulic models were found to represent flows and levels across the floodplain well compared to the recorded 2011 event.

17.1.2 Survey

ARTC provided LiDAR data from 2015 as 1 m grid DEM tiles. Using GIS software, a DEM was generated with a 1 m grid resolution for use in the Project based on the 2015 dataset. This was used for modelling within the disturbance footprint and up to the full extent of the 2015 LiDAR where relevant.

In areas that were not covered by the LiDAR provided by ARTC, LiDAR tiles were sourced from Geoscience Australia. The DEM datasets utilised for modelling were based on surveys flown between 2009 and 2015. SRTM data was used for catchment delineation where no LiDAR data could be sourced, to inform the hydrologic modelling.

The survey data sources and DEM developed for the unnamed creek catchment upstream of Bybera Road is shown in Figure K-1b in Volume II -Appendix K.

17.1.3 Aerial imagery

Aerial imagery of the study area was provided by ARTC and was used to identify and confirm topographic and vegetative characteristics of the study area. Aerial imagery captured in 2015 was made available. Additional imagery outside the study area was sourced from QGIS imagery in an open source format.

17.1.4 Existing drainage structure data

DTMR as-constructed drawings were also sourced for culvert and bridge details. This information will be refined as the local survey is complete.

17.1.5 Stream gauge data

No streamflow gauges exist in the unnamed creek catchment upstream of Bybera Road.

17.1.6 Rainfall data

A number of daily and sub-daily rainfall stations are located in and around the unnamed creek catchment upstream of Bybera Road. However, since there are no streamflow gauges to use for model calibration, no historical rainfall data was sought.

17.1.7 Anecdotal and observed flood data

No anecdotal or observed flood data was available for this area of the Macintyre Brook.

17.1.8 Site inspection

A site inspection was undertaken during February 2018. During the site inspection, all major waterway crossings were visited and inspected with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

17.2 Hydrologic model development – Macintyre Brook at Bybera Road

17.2.1 Model setup

A hydrologic model of the unnamed creek catchment upstream of Bybera Road was established in URBS using the latest procedures detailed in ARR 2016 and the latest rainfall data from BoM. The new ARR procedures cover revisions made to hydrologic parameters such as losses, pre-burst depths, temporal patterns and areal reduction factors.

The URBS model covers approximately 62 km^2 of the unnamed creek catchment upstream of Bybera Road and the confluence with Macintyre Brook. The catchment was delineated into 20 sub-catchments to capture the variability of rainfall and to better represent the network of creeks and streams within the catchment.

The hydrologic model setup including extent and sub-catchments are presented in Volume II – Appendix K, Figure K-1c.

Model inputs are summarised in Table 17.1.

Table 17.1 Summary of URBS model inputs

Input parameter	Remarks
URBS model type	Basic
Routing variables	Catchment area, stream lengths
Channel lag parameter, α	1.20
Catchment non-linearity parameter, m	0.8

Note that the default values were adopted for all other URBS parameters.

17.2.2 Design event parameters

Hydrologic information to assist estimation of design event flows was sourced from the ARR 2016 Data Hub as summarised in Table 17.2.

Table 17.2 Summary of URBS model design event inputs

Input parameter	Remarks
Design rainfall	IFDs for each sub-catchment were downloaded from the BoM's website to account for variation in rainfall across the catchment.
Extreme event rainfall	PMP depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the method presented in the Bulletin 53 (BOM, 2003). The rainfall depths for the 1 in 10,000 AEP event were estimated using the interpolation method presented in ARR 2016 Book 8 Section 3.5.
Losses	The losses were adopted from the Inglewood Flood Study (2015) URBS model. Losses do not vary with AEP. Initial loss – 15 mm Continuing loss – 1 mm/h
Areal reduction factor	Parameters were adopted for the Semi-Arid Inland Queensland region. The catchment area U/S of the proposed rail crossing adjacent Bybera Road is approximately 62 km^2, which yields an ARF between 76.3% and 95.9% depending on design storm event AEP and duration.
Ensemble temporal patterns	Central Slopes region
Preburst depths	Median preburst depths were downloaded from the ARR 2016 Data Hub for each sub-catchment. Preburst depths vary by design storm event AEP and duration. Preburst depths were applied to the model by reducing the initial losses for each storm event.

17.2.3 Hydrologic model validation

The hydrologic model for the Bybera Road catchment was not calibrated due to unavailability of observed stream gauge data in the catchment. However, the routing parameter α was adjusted until there was a reasonable match between the URBS model flows and those derived using QRT and RFFE methods.

All URBS results reside within the 90% confidence limits of the RFFE and show a close match with QRT. Adjusting losses for each AEP may improve the validation against QRT and RFFE and should be investigated at the next stage of reporting. The estimated flood flows at the outlet of the Bybera Road model are presented in Figure 89 and Table 17.3. The model outlet lies approximately 250 m downstream of where the unnamed creek crosses the Project alignment.

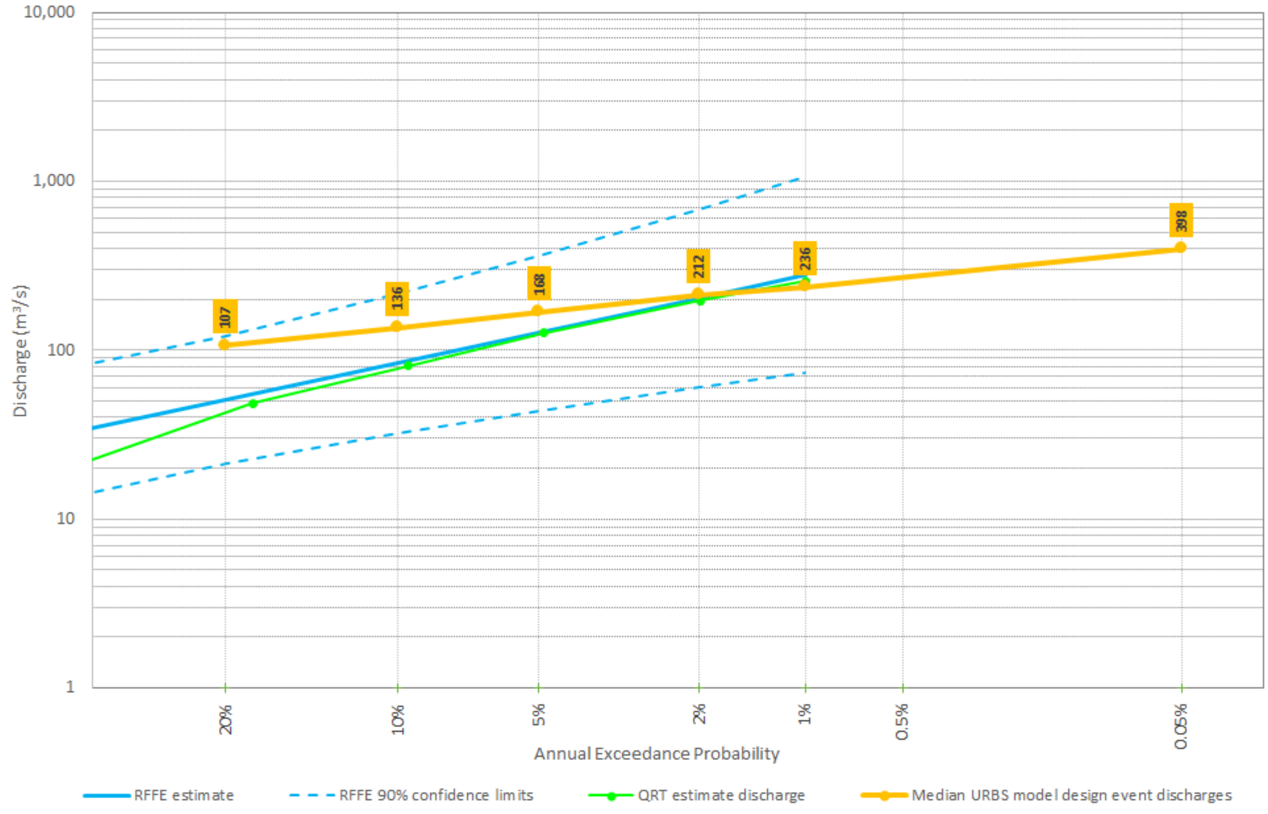


Figure 89 Estimate of flows at the outlet of Bybera Road model

Table 17.3 Estimate of flows at the outlet of Bybera Road model

AEP (%)	RFFE – lower bound 90% confidence level (m^3/s)	RFFE – estimate of flow (m^3/s)	RFFE – upper bound 90% confidence level (m^3/s)	DTMR quantile regression technique (m^3/s)[1]	URBS model flows (m^3/s)
20	21	51	122	48	107
10	32	84	216	81	136
5	44	127	365	126	168
2	61	204	685	195	212
1	74	281	1,060	257	236
1 in 2,000	-	-	-	-	398
1 in 10,000	-	-	-	-	1,010
PMF	-	-	-	-	4,463

Table note:

1 The QRT method estimates the 39.3%, 18.1%, 9.5% and 4.9% AEP instead of 50%, 20% and 10% and 5% respectively

17.3 Hydraulic model development – Macintyre Brook at Bybera Road

A two-dimensional modelling approach was adopted to appropriately simulate flood mechanisms around the Project rail crossing adjacent Bybera Road. The platform used for hydraulic modelling is the TUFLOW HPC software package. The processes and assumptions adopted throughout the development of the hydraulic model are described in the following sections.

17.3.1 Model setup

The setup of the TUFLOW model is summarised in Table 17.4.

Table 17.4 Bybera Road hydraulic model summary

Parameter	Information
Completion date	June 2019
AEPs assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF
Hydraulic model build	TUFLOW HPC GPU – version 2017-09-AC-w64-iSP
Model extent	Refer to Figure K-1d in Volume II – Appendix K
Grid size	5m
DEM (year flown)	ARTC dataset (2015).
Roughness	Spatially varying roughness values compliant with industry norms.
Eddy viscosity	Smagorinsky (default)
Model calibration	N/A. No stream gauge data available.
D/S model boundary	Height-Discharge (HQ) Boundary with normal slope approximated based upon topography dataset.
Hydraulic model timestep	Adaptive Timestep
Hydraulic model wetting and drying depths	Cell centre set at 0.0002m Cell side set at 0.0001 m
Modelled scenarios	Existing Case, Developed Case
Sensitivity analysis	Climate change

The hydraulic model extent and the spatial distribution of land use in the 2D model domain is presented in Volume II – Appendix K, Figure K-1d, and landuse classification in Figure K-1e.

17.3.2 Hydraulic structures

No existing hydraulic structures are situated within the extents of the hydraulic model.

17.4 Existing Case modelling results – Macintyre Brook at Bybera Road

17.4.1 Hydrologic model peak flow assessment

A critical duration assessment was undertaken to determine which storm duration(s) produced peak flood flows where the major waterways are intersected by the Project alignment and at the downstream outlet of the model. To assess the critical storm duration the following methodology was adopted:

- The models were modelled for a range of AEP events
  - Each AEP was modelled for a range of durations
  - Each duration was modelled for each of the ten associated temporal patterns.

A critical duration assessment was undertaken at the locations mentioned above to determine which duration produced the highest median flow of the ten temporal patterns for each event

Table 17.5 presents the estimated peak flow applied to the hydraulic model for a number of key locations (refer Figure K-1d in Volume II – Appendix K).

Table 17.5 Peak flow at key locations as applied in the hydraulic model

AEP (%)	Peak flow (m^3/s)	Critical storm duration/temporal pattern
20	107	9 hour – Pattern 4
10	136	6 hour – Pattern 7
5	168	6 hour – Pattern 7
2	212	6 hour – Pattern 7
1	236	12 hour – Pattern 9
1 in 2,000	398	6 hour – Pattern 0
1 in 10,000	1,010	6 hour – PMP Temporal Pattern
PMF	4,463	6 hour – PMP Temporal Pattern

17.4.2 Existing Case flood maps

Flood maps illustrating indicative flood extents and peak water levels were prepared and are presented in Volume II – Appendix K:

- 20% AEP: Figure K-2a
- 10% AEP: Figure K-2b
- 5% AEP: Figure K-2c
- 2% AEP: Figure K-2d
- 1% AEP: Figure K-2e
- 1 in 2,000 AEP: Figure K-2f
- 1 in 10,000 AEP: Figure K-2g
- PMF: Figure K-2h.

Figure K-3a presents peak flood velocities under a 1% AEP event.

17.4.3 Flood inundation extent and flood levels

Figure K-2e in Volume II – Appendix K shows the 1% AEP indicative flood extent and peak water levels within the Bybera Road floodplain for the Existing Case.

The peak flood depth is approximately 3.1 m within the Bybera Creek channel. This depth reduces to an flood depth of around 1.1 m in other areas of the floodplain. The peak flood depth on the floodplain is estimated to be up to 2.5 m (273.4 m AHD) where the proposed alignment crosses the floodplain.

The model indicates that the time of inundation across the floodplain during the 1% AEP flood event is between 14 and 24 hours.

17.4.4 Flood immunity of existing infrastructure

Within the area modelled, no flooding of existing infrastructure is observed up to the PMF event.

17.4.5 Existing Case velocities

Peak flood velocities are expected to reach 2.4 m/s in localised areas with the average velocity across the floodplain approximately 0.9 m/s, as shown in Figure K4-a in Volume II – Appendix K.

17.5 Developed Case modelling results – Macintyre Brook at Bybera Road

17.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW model (1d and 2d approach). At Bybera Road, the Project includes one waterway bridge.

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and depending on their interaction with the Macintyre Brook floodplain were incorporated in the hydraulic model.

The proposed bridge is summarised in Table 17.6, and presented in Figure K-1f in Volume II – Appendix K. The 1% AEP flood level at the bridge structure is presented in Appendix C.

Bridges were modelled as an opening in the rail embankment. The optimisation of bridge lengths was balanced between minimising the changes to the hydraulic regime, primarily afflux and velocities, and the cost of replacing bridge spans by large earth embankments.

Blockage of hydraulic structures was assessed in accordance with ARR 2016. Despite the catchment being vegetated heavily forested, ARR guidelines determined that the likelihood of significant amounts of debris accumulating against the piers of the bridge is low. Therefore, a zero blockage factor was applied at the Bybera Road bridge.

Table 17.6 Macintyre Brook at Bybera Road - proposed bridge location and details

Chainage (km)	Structure ID	Approximate span (m)	Deck width (m)	Deck level (m AHD)	Deck superstructure type	Deck depth (mm)
55.55	310-BR04	207	3.97	285.9	Type D1	2,000

17.5.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

17.5.2.1 Rail immunity and structures results

Appendix C summarises the peak 1% AEP water levels on the upstream side of the proposed alignment. Local drainage structures (i.e. those not included in the flood model) and road culverts are not reported.

Results indicate that peak water levels remain below the proposed rail formation level, and that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the Macintyre Brook at Bybera Road floodplain.

17.5.2.2 Velocities and scour

Appendix C summarises the peak 1% AEP outlet flows and velocities at structures. The 1% AEP peak velocity through the proposed bridge is generally less than 2.5 m/s.

Scour protection requirements for culverts were calculated based on the velocities predicted from the hydraulic modelling. The scour protection was 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.

Table 17.7 lists the soil types encountered along the Project alignmentand the allowable soil velocity based on AGRD.

Table 17.7 Allowable soil velocities along the Project alignment

Soil type	Soil description	Maximum allowable soil velocity as per AGRD (m/s)
Sodosols	Firm loam or stiff clay	2 m/s
Vertosols, Dermosols, Rudosols and Kandosols	Sandy or silty clay	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.

Scour protection requirements for culverts have been calculated based on the velocities predicted from the hydraulic modelling. Appendix D presents the proposed scour protection at each structure.

A conservative scour estimation has been undertaken at the bridge site based on available information and will be refined during detailed design.

17.5.2.3 Flood immunity for extreme events

The risk of overtopping of the rail alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF). The bridge at CH 55550 and formation do not overtop in extreme local flood events.

17.5.3 Flood impact objectives outcomes – Macintyre Brook at Bybera Road

The impact of the Developed Case was assessed through inclusion of the proposed rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, duration of inundation, velocities and peak flows within the floodplain:

- Changes in peak water levels for the AEP's assessed are presented in Figures K-4a to K-4h in Volume II – Appendix K
- Changes in 1% AEP duration of inundation are presented in Figure K-4i in Volume II – Appendix K
- Changes in 1% AEP velocities are presented in Figure K-4j in Volume II – Appendix K.

All impacts are required to be agreed with relevant stakeholders and affected landowners as part of the EIS process. One-on-one consultation with landowners who are expected to experience changes in flooding behaviour on the property was conducted by ARTC supported by FFJV.

The Project design outcomes relative to the flood impact objectives (refer Table 4.2) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Volume II – Appendix N.

The potential impacts to water levels across events up to and including the 1% AEP are summarised in Table 17.8.

Table 17.8 Afflux summary Macintyre Brook at Bybera Road

Afflux outside rail disturbance footprint	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
Maximum afflux (mm)	77	90	107	119	160
Area afflux >10 mm experienced (ha)	3.8	4.9	5.9	7.2	7.8
Area afflux >200 mm experienced (ha)	-	-	-	-	-
Area afflux >400 mm experienced (ha)	-	-	-	-	-

17.5.3.1 Flood impacts at proposed hydraulic structures

The estimated impact on peak water levels at the proposed bridge structure is presented in Table 17.9. Peak water levels were extracted at the control line of the bridge.

The design achieved a freeboard of at least 300 mm between the 1% AEP peak water and formation levels.

Table 17.9 1% AEP event – estimated impacts on peak water level at proposed hydraulic structures

Chainage (km)	Structure ID	Structure type	Rail formation level (m AHD)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Change in peak water level (mm)
55.55	310-BR04	Bridge	285.9	273.4	273.6	+130

17.5.3.2 Flood impacts on flood sensitive receptors

Based on the available aerial imagery, no buildings or critical infrastructure are located within the area affected by afflux in the Macintyre Brook at Bybera Road floodplain for events up to the 1% AEP.

17.5.3.3 Flood impacts on roads

No state-controlled or local public roads are affected by flooding within the Macintyre Brook at Bybera Road model for the 1% AEP event.

17.5.3.4 Flood impacts on private land outside the rail disturbance footprint

The majority of the area where afflux is expected is agricultural land or open land on which nominal afflux is unlikely to cause any adverse impact. Table 17.10 presents the modelled changes in flood conditions during the 1% AEP event on a lot basis according to the following thresholds:

- Peak water levels increased by greater than +10 mm
- Peak velocities increased by greater than 0.25 m/s
- Duration of inundation changed by more than 25% of its original duration of inundation across the lot.

Table 17.10 Macintyre Brook at Bybera Road – summary of flood impacts on private land outside the rail disturbance footprint for 1% AEP

Approximate chainage (km)	Changes in peak water levels[1]		Changes in peak velocities		Changes in Duration of inundation (hrs)	
	Maximum change (mm)	Total area affected by change > 10 mm (ha)[2]	Maximum change (m/s)	Total area affected by change (ha)	Maximum change (%)	Total area affected by change (ha)
55.40 to 55.90	+38	0.1	-	-	-	-
54.50 to 55.60	+160	7.5	+0.7	1.6	-	-

Table notes:

1 Afflux on lots that exceed the flood impact objectives are summarised in the EIS Surface Water Chapter
2 Only minor areas, usually directly upstream of culverts are affected by the maximum afflux as stated

17.5.3.5 Flow distribution

A key landowner concern is changes to flow distributions. However, the unnamed creek that drains the catchment upstream of Bybera Road is well defined. There are no lateral breakouts of floodwater and hence there are negligible changes to flow distributions in events up to the 1% AEP.

17.5.4 Sensitivity analysis – Macintyre Brook at Bybera Road

17.5.4.1 Blockage

Blockage of hydraulic structures was assessed in accordance with ARR 2016. Despite the catchment being vegetated heavily forested, ARR guidelines determined that the likelihood of significant amounts of debris accumulating against the piers of the bridge is low. Therefore, a zero blockage factor was applied at the Bybera Road bridge. Additionally, there are no culverts in the Bybera Road floodplain, hence no sensitivity scenarios were conducted.

During detail 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.

17.5.4.2 Climate change

The potential impacts of climate change in the Macintyre Brook at Bybera Road floodplain were assessed for the 1% AEP design event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2016.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario has been adopted for the Project with an associated increase in rainfall intensity of 23.9% across the catchment area.

Table 17.11 presents the structure performance with Representative Concentration Pathways 8.5 climate change conditions. For the 1% AEP event, the change in peak water levels for the climate change scenario is presented in Figure K-5a in Volume II – Appendix K.

Climate change results are expected to increase peak water levels upstream of the Project alignment by up to 0.3 m at proposed bridge 310-BR04 for the 1% AEP event. The Project alignment is expected to retain 1% AEP flood immunity to formation level under the climate change scenario.

Table 17.11 Macintyre Brook at Bybera Road – 1% AEP event with Representative Concentration Pathways 8.5 conditions – structure performance

Structure ID	Structure type	1% AEP peak water level (m AHD)	1% AEP +CC peak water level (m AHD)	Difference in peak water level (m)	Freeboard to rail formation with CC (m)
310-BR04	Bridge	273.6	273.9	0.3	12.1

18 Macintyre Brook at Cremascos Road

The location of the Project rail alignment in relation to Macintyre Brook at Cremascos Road is shown in Figure L-1a in Volume II – Appendix L.

18.1 Data collection and review – Macintyre Brook at Cremascos Road

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:

- GRC – existing flood studies
- The BoM – rainfall data
- DTMR – existing infrastructure details.

18.1.1 Previous studies

Goondiwindi Regional Council, Inglewood Flood Study, Engeny, 2015

Engeny was commissioned by GRC to undertake a flood study of Inglewood. The study objectives were "to define the nature, extent and risks of flooding in Inglewood in order to inform disaster management planning and response, as well as control future development" (Engeny, 2015). An URBS hydrologic model and TUFLOW (1D/2D) hydraulic model was developed.

Inland Rail: Phase 2 – North Star to Border, 2018

The NS2B section of Inland Rail will cross the Macintyre River and its floodplain which are a part of the Border Rivers catchment. The NS2B alignment runs through Moree Plains Local Government Area (LGA), Gwydir LGA and Goondiwindi LGA. To establish the reliability of the models the 2011 event was included in the calibration process in addition to the 1976 and 1996 events assessed by DPIE. The hydrologic and hydraulic models were found to represent flows and levels across the floodplain well compared to the recorded 2011 event.

18.1.2 Survey

ARTC provided LiDAR data from 2015 as 1 m grid DEM tiles. Using GIS software, a DEM was generated with a 1 m grid resolution for use in the Project based on the 2015 dataset. This was used for modelling within the disturbance footprint and up to the full extent of the 2015 LiDAR where relevant.

In areas that were not covered by the LiDAR provided by ARTC, LiDAR tiles were sourced from Geoscience Australia. The DEM datasets utilised for modelling were based on surveys flown between 2009 and 2015. SRTM data was used for catchment delineation where no LiDAR data could be sourced, to inform the hydrologic modelling.

The survey data sources and DEM developed for the unnamed creek catchment upstream of Cremascos Road are shown in Figure L-1b in Volume II – Appendix L.

18.1.3 Aerial imagery

Aerial imagery of the study area was provided by ARTC and was used to identify and confirm topographic and vegetative characteristics of the study area. Aerial imagery captured in 2015 was made available. Additional imagery outside the study area was sourced from QGIS imagery in an open source format.

18.1.4 Existing drainage structure data

DTMR as-constructed drawings were also sourced for culvert and bridge details. This information will be refined as the local survey is complete.

18.1.5 Stream gauge data

No streamflow gauges exist in the unnamed creek catchment upstream of Cremascos Road.

18.1.6 Rainfall data

A number of daily and sub-daily rainfall stations are located in and around the unnamed creek catchment upstream of Cremascos Road. However, since there are no streamflow gauges to use for model calibration, no historical rainfall data was sought.

18.1.7 Anecdotal and observed flood data

No anecdotal or observed flood data was available for this area of the Macintyre Brook.

18.1.8 Site inspection

A site inspection was undertaken during February 2018. During the site inspection, all major waterway crossings were visited and inspected with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

18.2 Hydrologic model development – Macintyre Brook at Cremascos Road

18.2.1 Model setup

A hydrologic model of the Cremascos Road catchment was established in URBS using the latest procedures detailed in ARR 2016 and the latest rainfall data from BoM. The new ARR procedures cover revisions made to hydrologic parameters such as losses, pre-burst depths, temporal patterns and areal reduction factors.

The URBS model covers approximately 57 km^2 of the unnamed creek catchment upstream of Cremascos Road and the confluence with Macintyre Brook. The catchment was delineated into 14 sub-catchments to capture the variability of rainfall and to better represent the network of creeks and streams within the catchment.

The hydrologic model setup including extent and sub-catchment map is presented in Volume II – Appendix L, Figure L-1c.

Model inputs are summarised in Table 18.1.

Table 18.1 Summary of URBS model inputs

Input parameter	Remarks
URBS model type	Basic
Routing variables	Catchment area, stream lengths
Channel lag parameter, α	1.20
Catchment non-linearity parameter, m	0.8

Note that the default values were adopted for all other URBS parameters.

18.2.2 Design event parameters

Hydrologic information to assist estimation of design event flows was sourced from the ARR 2016 Data Hub as summarised in Table 18.2.

Table 18.2 Summary of URBS model design event inputs

Input parameter	Remarks
Design rainfall	IFDs for each sub-catchment were downloaded from the BoM's website to account for variation in rainfall across the catchment.
Extreme event rainfall	PMP depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the method presented in the Bulletin 53 (BOM, 2003). The rainfall depths for the 1 in 10,000 AEP event were estimated using the interpolation method presented in ARR 2016 Book 8 Section 3.5.
Losses	The losses were adopted from the Inglewood Flood Study (2015) URBS model. Losses are fixed and do not vary with AEP. Initial loss – 15 mm Continuing loss – 1 mm/h
Areal reduction factor	Parameters were adopted for the Semi-Arid Inland Queensland region. The catchment area U/S of the proposed rail crossing adjacent Cremascos Road is approximately 57 km^2, which yields an ARF between 77.0% and 96.0% depending on design storm event AEP and duration.
Ensemble temporal patterns	Central Slopes region
Preburst depths	Median preburst depths were downloaded from the ARR 2016 Data Hub for each sub-catchment. Preburst depths vary by design storm event AEP and duration. Preburst depths were applied to the model by reducing the initial losses for each storm event.

18.2.3 Hydrologic model validation

The hydrologic model for the Cremascos Road catchment was not calibrated due to unavailability of observed stream gauge data in the catchment. However, the routing parameter α was adjusted until there was a reasonable match between the URBS model flows and those derived using QRT and RFFE methods.

All URBS results reside within the 90% confidence limits of the RFFE and show a close match with QRT. Adjusting losses for each AEP may improve the validation against QRT and RFFE and should be investigated at the next stage of reporting. The estimated flood flows at the outlet of the Cremascos Road model are presented in Figure 90 and Table 18.3. The model outlet lies approximately 250 m downstream of where the unnamed creek crosses the Project alignment.

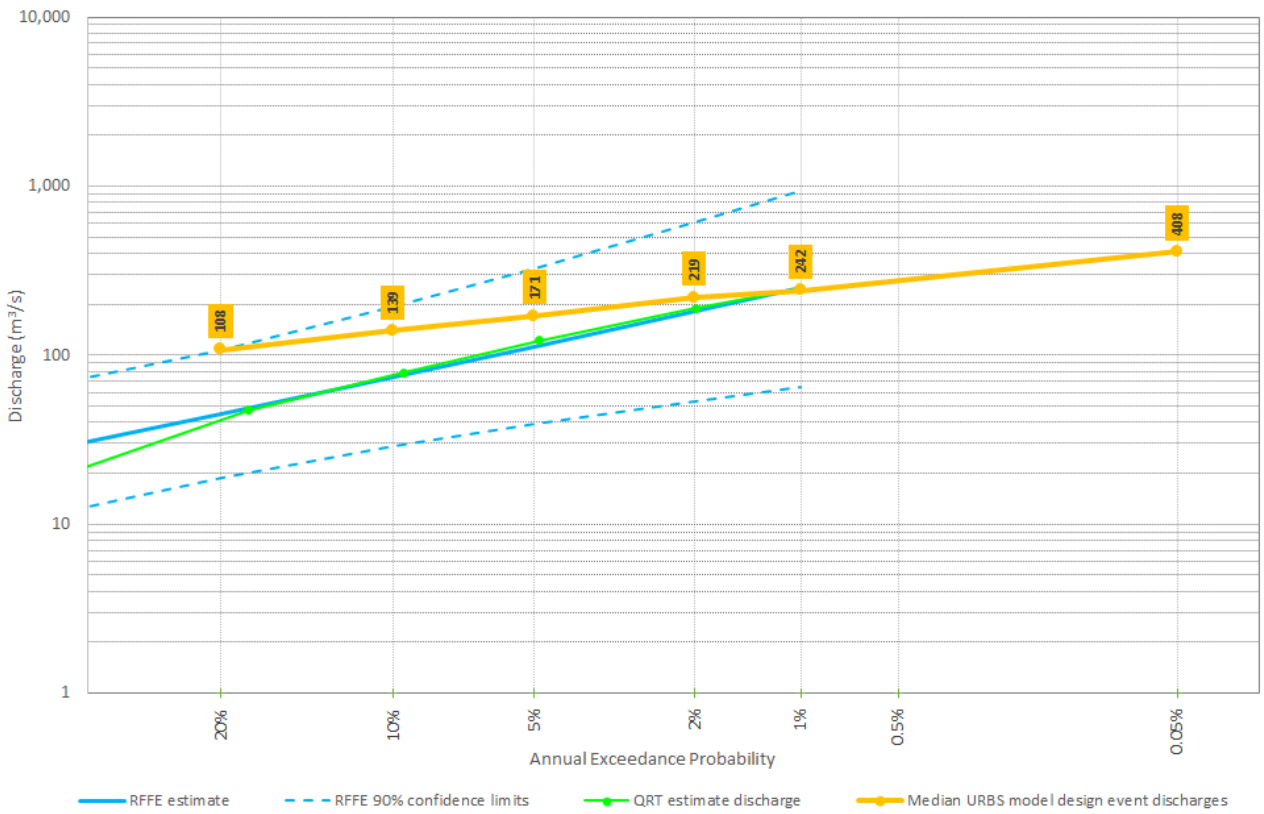


Figure 90 Estimate of flows at the outlet of Cremascos Road model

Table 18.3 Estimate of flows at the outlet of Cremascos Road model

AEP (%)	RFFE – lower bound 90% confidence level (m^3/s)	RFFE – estimate of flow (m^3/s)	RFFE – upper bound 90% confidence level (m^3/s)	DTMR quantile regression technique (m^3/s)[1]	URBS model flows (m^3/s)
20	19	45	108	47	108
10	29	74	192	79	139
5	39	113	324	122	171
2	54	181	609	189	219
1	65	249	941	249	242
1 in 2,000	-	-	-	-	408
1 in 10,000	-	-	-	-	1,116
PMF	-	-	-	-	4,793

Table note:

1 The QRT method estimates the 39.3%, 18.1%, 9.5% and 4.9% AEP instead of 50%, 20% and 10% and 5% respectively

18.3 Hydraulic model development – Macintyre Brook at Cremascos Road

A two-dimensional modelling approach was adopted to appropriately simulate flood mechanisms around the Project rail crossing adjacent Cremascos Road. The platform used for hydraulic modelling is the TUFLOW HPC software package. The processes and assumptions adopted throughout the development of the hydraulic model are described in the following sections.

18.3.1 Model setup

The setup of the TUFLOW model is summarised in Table 18.4.

Table 18.4 Cremascos Road hydraulic model summary

Parameter	Information
Completion date	June 2019
AEPs assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF
Hydraulic model build	TUFLOW HPC GPU – version 2017-09-AC-w64-iSP
Model extent	Refer to Figure L-1d in Volume II – Appendix L
Grid size	5 m
DEM (year flown)	ARTC dataset (2015).
Roughness	Spatially varying roughness values compliant with industry norms.
Eddy viscosity	Smagorinsky (default)
Model calibration	N/A
D/S model boundary	Height-Discharge (HQ) Boundary with normal slope approximated based upon topography dataset.
Hydraulic model timestep	Adaptive Timestep
Hydraulic model wetting and drying depths	Cell centre set at 0.0002 m Cell side set at 0.0001 m
Modelled scenarios	Existing Case, Developed Case
Sensitivity analysis	Climate change

The hydraulic model extent and the spatial distribution of land use in the 2D model domain is presented in Volume II – Appendix L, Figure L-1d, and landuse classification in Figure L-1e.

18.3.2 Hydraulic structures

No existing hydraulic structures are situated within the extents of the hydraulic model.

18.4 Existing Case modelling results – Macintyre Brook at Cremascos Road

18.4.1 Hydrologic model peak flow assessment

A critical duration assessment was undertaken to determine which storm duration(s) produced peak flood flows where the major waterways are intersected by the Project alignment and at the downstream outlet of the model. To assess the critical storm duration the following methodology was adopted:

- The models were modelled for a range of AEP events:
  - Each AEP was modelled for a range of durations
  - Each duration was modelled for each of the ten associated temporal patterns
- A critical duration assessment was undertaken at the locations mentioned above to determine which duration produced the highest median flow of the ten temporal patterns for each event.

Table 18.5 presents the estimated peak flow applied to the hydraulic model for a number of key locations (refer Figure L-1d in Volume II – Appendix L).

Table 18.5 Peak flow at key locations as applied in the hydraulic model

AEP (%)	Peak flow (m^3/s)	Critical storm duration/temporal pattern
20	108	9 hour – Pattern 9
10	139	6 hour – Pattern 7
5	171	4.5 hour – Pattern 7
2	217	4.5 hour – Pattern 7
1	238	4.5 hour – Pattern 8
1 in 2,000	408	4.5 hour – Pattern 8
1 in 10,000	1,116	6 hour – PMP Temporal Pattern
PMF	4,793	6 hour – PMP Temporal Pattern

18.4.2 Existing Case flood maps

Flood maps illustrating indicative flood extents and peak water levels were prepared and are presented in Volume II – Appendix L:

- 20% AEP: Figure L-2a
- 10% AEP: Figure L-2b
- 5% AEP: Figure L-2c
- 2% AEP: Figure L-2d
- 1% AEP: Figure L-2e
- 1 in 2,000 AEP: Figure L-2f
- 1 in 10,000 AEP: Figure L-2g
- PMF: Figure L-2h.

Figure L-3a presents peak flood velocities under a 1% AEP event.

18.4.3 Flood inundation extent and flood levels

Figure L-2e in Volume II – Appendix L shows the 1% AEP indicative flood extent and peak water levels within the Cremascos Road floodplain for the Existing Case.

The peak flood depth is approximately 4.4 m within the Cremascos Creek channel with an average flood depth of around 1.0 m in other areas of the floodplain. The flood depth on the floodplain is estimated to be up to 3.1 m (270.6m AHD) where the proposed alignment crosses the floodplain.

The model indicates that the time of inundation across the floodplain during the 1% AEP event is between 4 and 6 hours.

18.4.4 Flood immunity of existing infrastructure

Within the area modelled, no flooding of existing key infrastructure is observed up to the PMF event.

18.4.5 Existing Case velocities

Peak flood velocities are expected to reach 3.6 m/s in localised areas with the average velocity across the floodplain approximately 0.9 m/s, as shown in Figure L3-a in Volume II – Appendix L.

18.5 Developed Case modelling results – Macintyre Brook at Cremascos Road

18.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW model (1d and 2d approach). At Cremascos Road the Project includes one waterway bridge.

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and depending on their interaction with the Macintyre Brook floodplain were incorporated in the hydraulic model.

The proposed bridge is summarised in Table 18.6 and presented in Figure L-1f in Volume II – Appendix L. The 1% AEP flood level at the proposed bridge structure is presented in Table 18.9.

Bridges were modelled as an opening in the rail embankment. The optimisation of bridge lengths was balanced between minimising the changes to the hydraulic regime, primarily afflux and velocities, and the cost of replacing bridge spans by large earth embankments.

Blockage of hydraulic structures was assessed in accordance with ARR 2016. Despite the catchment being vegetated heavily forested, ARR guidelines determined that the likelihood of significant amounts of debris accumulating against the piers of the bridge is low. Therefore, a zero blockage factor was applied at the Cremascos Road bridge.

Table 18.6 Macintyre Brook at Cremascos Road - proposed bridge location and details

Chainage (km)	Structure ID	Approximate span (m)	Deck width (m)	Deck level (m AHD)	Deck superstructure type	Deck depth (mm)
52.58	310-BR03	184	3.97	281.7	Type D1	2,000

18.5.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

18.5.2.1 Rail immunity and structures results

Appendix C summarises the peak 1% AEP water levels on the upstream side of the proposed alignment. Local drainage structures (i.e. those not included in the flood model) and road culverts are not reported.

Results indicate that peak water levels remain below the proposed rail formation level. There is over 8.8 m freeboard above the bridge soffit level to the rail formation.

The results of flood modelling therefore indicate that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the Macintyre Brook at Cremascos Road floodplain.

18.5.2.2 Velocities and scour

Appendix C summarises the peak 1% AEP outlet flows and velocities at structures. The 1% AEP peak velocity through the proposed bridge is generally less than 2.0 m/s.

Scour protection requirements for culverts were calculated based on the velocities predicted from the hydraulic modelling. The scour protection was 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.

Table 18.7 lists the soil types encountered along the Project alignmentand the allowable soil velocity based on AGRD.

Table 18.7 Allowable soil velocities along the Project alignment

Soil type	Soil description	Maximum allowable soil velocity as per AGRD (m/s)
Sodosols	Firm loam or stiff clay	2 m/s
Vertosols, Dermosols, Rudosols and Kandosols	Sandy or silty clay	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.

Scour protection requirements for culverts have been calculated based on the velocities predicted from the hydraulic modelling. Appendix D presents the proposed scour protection at each structure.

A conservative scour estimation has been undertaken at the bridge site based on available information and will be refined during detailed design.

18.5.2.3 Flood immunity for extreme events

The risk of overtopping of the rail alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF). The formation and bridge at CH 52580 do not overtop in extreme local flood events.

18.5.3 Flood impact objectives outcomes – Macintyre Brook at Cremascos Road

The impact of the Developed Case was assessed through inclusion of the proposed rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, duration of inundation, velocities and peak flows within the floodplain:

- Changes in peak water levels for the AEP's assessed are presented in Figures L-4a to L-4h in Volume II – Appendix L
- Changes in 1% AEP duration of inundation are presented in Figure L-4i in Volume II – Appendix L
- Changes in 1% AEP velocities are presented in Figure L-4j in Volume II – Appendix L.

All impacts are required to be agreed with relevant stakeholders and affected landowners as part of the EIS process. One-on-one consultation with landowners who are expected to experience changes in flooding behaviour on the property was conducted by ARTC supported by FFJV.

The Project design outcomes relative to the flood impact objectives (refer Table 4.2) are presented in the following sections.
Flood sensitive receptors referred to in this section are shown in Volume II – Appendix N.

The potential impacts to water levels across events up to and including the 1% AEP are summarised in Table 18.8.

Table 18.8 Afflux summary – Macintyre Brook at Cremascos Road

Afflux outside rail disturbance footprint	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
Maximum afflux (mm)	16	31	-	25	46
Area afflux >10 mm experienced (ha)	<0.01	<0.01	<0.01	<0.01	<0.01
Area afflux >200 mm experienced (ha)	-	-	-	-	-
Area afflux >400 mm experienced (ha)	-	-	-	-	-

18.5.3.1 Flood impacts at proposed hydraulic structures

The estimated impacts on peak water levels at the proposed bridge structure are presented in Table 18.9. Peak water levels were extracted at the control line of the bridge.

The design achieved a freeboard of at least 300 mm between the 1% AEP peak water and formation levels.

Table 18.9 Macintyre Brook at Cremascos Road – 1% AEP event – change in on peak water levels

Chainage (km)	Structure ID	Structure type	Rail formation level (m AHD)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Change in peak water level (mm)
52.58	310-BR03	Bridge	281.7	270.6	270.6	+20

18.5.3.2 Flood impacts on flood sensitive receptors

Based on the available aerial imagery, no buildings or critical infrastructure are located within the area affected by afflux in the Macintyre Brook at Cremascos Road floodplain for events up to the 1% AEP.

18.5.3.3 Flood impacts on roads

No state-controlled or local public roads are affected by flooding within the Macintyre Brook at Cremascos Road model for the 1% AEP event.

18.5.3.4 Flood impacts on private land outside the rail disturbance footprint

The majority of the area where afflux is expected is agricultural land or open land on which nominal afflux is unlikely to cause any adverse impact.

Table 18.10 presents the modelled changes in flood conditions during the 1% AEP event on a lot basis according to the following thresholds:

- Peak water levels increased by greater than +10 mm
- Peak velocities increased by greater than 0.25 m/s
- Duration of inundation changed by more than 25% of its original duration of inundation across the lot.

Table 18.10 Macintyre Brook at Cremascos Road – summary of flood impacts on private land outside the rail disturbance footprint for 1% AEP

Approximate chainage (km)	Changes in peak water levels[1]		Changes in peak velocities		Changes in Duration of inundation (hrs)	
	Maximum change (mm)	Total area affected by change > 10 mm (ha)[2]	Maximum change (m/s)	Total area affected by change (ha)	Maximum change (%)	Total area affected by change (ha)
50.10 to 52.20	+27	1.2	-	-	-	-

Table notes:

1 Afflux on lots that exceed the flood impact objectives are summarised in the EIS Surface Water Chapter.
2 Only minor areas, usually directly upstream of culverts are affected by the maximum afflux as stated.

18.5.3.5 Flow distribution

A key landowner concern is changes to flow distributions. However, the unnamed creek that drains the catchment upstream of Cremascos Road is well defined. There are no lateral breakouts of floodwater and hence there are negligible changes to flow distributions for events up to the 1% AEP.

18.5.4 Sensitivity analysis – Macintyre Brook at Cremascos Road

18.5.4.1 Blockage

Blockage of hydraulic structures was assessed in accordance with ARR 2016. Despite the catchment being vegetated heavily forested, ARR guidelines determined that the likelihood of significant amounts of debris accumulating against the piers of the bridge is low. Therefore, a zero blockage factor was applied at the Cremascos Road bridge. Additionally, there are no culverts in the Cremascos Road floodplain, hence no sensitivity scenarios were conducted.

During detail 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.

18.5.4.2 Climate change

The potential impacts of climate change in the Macintyre Brook at Cremascos Road floodplain were assessed for the 1% AEP design event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2016.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario has been adopted for the Project with an associated increase in rainfall intensity of 23.9% across the catchment area.

Table 18.11 presents the structure performance with Representative Concentration Pathways 8.5 climate change conditions. For the 1% AEP event, the change in peak water levels for the Representative Concentration Pathways 8.5 climate change scenario is presented in Figure L-5a in Volume II – Appendix L.

Climate change results are expected to increase peak water levels upstream of the Project alignment by up to 0.23 m at proposed bridge 310-BR03 for the 1% AEP event. The Project alignment is expected to retain 1% AEP flood immunity to formation level under the climate change scenario.

Table 18.11 Macintyre Brook at Cremascos Road – 1% AEP event with Representative Concentration Pathways 8.5 conditions – structure performance

Structure ID	Structure type	1% AEP peak water level (m AHD)	1% AEP +CC peak water level (m AHD)	Difference in peak water level (m)	Freeboard to rail formation with CC (m)
310-BR03	Bridge	270.6	270.9	+0.23	10.9

19 Macintyre River

The hydrologic and hydraulic modelling of the Macintyre River was undertaken as part of the North Star to Border (NS2B) section of the Inland Rail project and documented in *North Star to Border: Hydrology & Flooding Report (2-0001-270-IHY-10-RP-0002, FFJV, 2019)*.

Results reported within this section relate to the NS2B project but was included to inform the B2G EIS. Chainages quoted in this section are NS2B chainages.

Widespread inundation is predicted under the 1% AEP event on the Macintyre River floodplain, with depths of approximately 10 m to 12 m in the Macintyre River, 6 m in Whalan Creek and up to 2 m on the floodplain area. The Macintyre River floodplain spans across the border, affecting areas in both NSW and QLD.

The location of the Project rail alignment in relation to Macintyre River is shown in Figure M-1a in Volume II – Appendix M.

This chapter provides a concise summary of the flood assessment work undertaken and key outcomes that are relevant to the Project. The section considered relevant to the Project is from the NSW/QLD border north. Only structures and flood sensitive receptors from the border north are reported in this section.

19.1 Data collection and review

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:

- GRC – existing flood studies
- The BoM – rainfall data
- DTMR – existing infrastructure details.

There were many studies undertaken by the local governments and stakeholders for the area; these are summarised below.

19.1.1 Previous studies

The Macintyre River catchment is located within the Moree Plains Local Government Area (LGA), Gwydir LGA and Goondiwindi LGA. FFJV was provided with modelling and historical data from these Councils for review. In addition, there are several previous studies of the Macintyre River catchment undertaken in earlier stages of Melbourne to Brisbane Inland Rail (MBIR) assessment, and other documents identified as potentially relevant to NS2B/B2G, these include:

- North Star to NSW/QLD Border – Hydrologic and Hydraulic Modelling Illabo to Stockinbingal and North Star to Yelarbon, (01-2700-PD-P00-DE-0010), SMEC, July 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, Office of Environment and Heritage, 2018
- Toomelah Flood Risk Assessment, Water Technology, 2016
- Dam Details – Pindari Dam, Coolmunda Dam, Glenlyon Dam.

Key studies were reviewed in detail and a proposed approach for modelling the Macintyre River catchment was developed for the hydrologic and hydraulic modelling based on the information and data available.

The DPIE Hydrology and hydraulic models were determined as the most appropriate for the assessment for the Macintyre River floodplain and were provided by DPIE for this purpose. The Draft Floodplain Management Plan for the Borders River Valley Floodplain is summarised below.

Draft Floodplain Management Plan for the Borders River Valley Floodplain 2018

The Floodplain Management Plan for the Borders River Valley Floodplain is currently being finalised. The plan provides a framework for coordinating and assessing development works on a whole of valley basis.

As part of the plan, hydrologic and hydraulic models (URBS, RAFTS and TUFLOW) were 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 19.1.

Table 19.1 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	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 models with stage storage and flow 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 modelled 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 19.2 and Table 19.3 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, Commoron and Ottleys Creek catchments.

Table 19.2 1976 event calibration summary

Catchment	Gauging station	Recorded peak flood height (m)	Modelled peak flood height (m)
Macintyre Brook	Terraine Inglewood CBM Inglewood	5.9 11.6 11.8	5.7 11.1 11.8
Dumaresq River	Bonshaw Weir Texas Oaky Creek Beebo	7.9 10.3 5.4 5.0	7.8 10.4 5.3 5.0
Macintyre River	Pindari Dam TW Ashford Wallangra Holdfast[1]	7.6 9.5 8.6 8.9	7.6 9.7 8.6 9.4

Table note:

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 19.3 1996 event calibration summary

Catchment	Gauging station	Recorded peak flood height (m)	Modelled peak flood height (m)
Macintyre Brook	Inglewood Booba Sands	9.8 8.9	9.2 9.0
Dumaresq River	Bonshaw Weir Texas Beebo Mauro	5.9 7.4 4.7 8.5	6.1 7.7 4.5 8.5
Macintyre River	Ashford Wallangra Holdfast	5.3 5.9 8.4	5.2 6.1 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 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 was 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 model area. Where data was not available, SRTM1-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). All topographic modifiers for the current topography are presented in Figure M-1b, and M-1d though M-1f in Volume II – Appendix M. DPIE hydraulic roughness is presented in Table 19.4.

Table 19.4 DPIE hydraulic model roughness

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

Boundary conditions were 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.

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

19.1.2 Existing Case hydrologic modelling

For the Borders River catchment, one key suite of hydrologic models was 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 were provided by DPIE and adopted for this assessment (referred to in this report as the DPIE hydrologic models).

19.1.3 Existing Case hydraulic modelling

Several hydraulic models were developed across the study area, these include:

- North Star to NSW/QLD Border, Hydrologic and Hydraulic Modelling Illabo to Stockinbingal 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.

19.1.4 Survey

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 M-1b in Volume II – Appendix M. 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 M-1b in Volume II – Appendix M.

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.

19.1.5 Existing drainage structure data

Drainage structure geometry information was obtained from the following sources:

- Previous studies
- Site inspection
- Field and validation survey.

19.1.6 Stream gauge data

Figure M-1c in Volume II – Appendix M presents the existing stream gauge stations available for historical events within the Border Rivers catchment. These stations are listed in Table 19.5.

Peak height records were obtained from the BoM for use in developing a series of partial peak flood flows for input into the FFA at the Boggabilla stream gauge.

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

Table 19.5 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%

Gauge	Location	Period	Catchment area (km²)	Rating ratio
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 km², with the upstream gauging stations accounting for 18,154 km², or just over 80% of the contributing catchment, which means that there is a residual catchment area of 4,446 km² 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.

19.1.7 Rainfall data

Historical rainfall data in the form of daily rainfall and total rainfall records was required for the calibration of the URBS hydrologic model. This information was sourced from the BoM, and from the SMEC 2016 RORB model. Data was obtained for the three historical flood events of 1976, 1996 and 2011.

Figure M-1c in Volume II – Appendix M presents the historical rainfall stations available within the Border Rivers catchment. These are listed in Table 19.6.

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 for the 2011 validation event were based on the quality of data available and suitability for the catchment model.

Table 19.6 Rainfall used for calibration

Gauge	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

Gauge	Location	Period of operation	Type
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

19.1.8 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.

19.1.9 Site inspection

A site inspection was undertaken on 9 to 10 April 2018. During the site inspection, all major waterway crossings were visited and inspected with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

19.2 Hydrologic model development – Macintyre River

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 assessment, 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 M-1c in Volume II – Appendix M. 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 were assessed as a separate drainage analysis.

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

19.3 Hydraulic model development – Macintyre River

19.3.1 Border River 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 19.7. Volume II – Appendix M, Figure M-1e shows the land use delineation.

Table 19.7 DPIE hydraulic model roughness

Land use type	Value
Waterway (Floodplain 3)	0.03
Floodplain (Floodplain 1)	0.06
Vegetated floodplain (Floodplain 2)	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.

19.3.2 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.

19.3.3 Joint calibration

Figure M-2a to Figure M-2c in Volume II – Appendix M present the calibration results for the three historical events. 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.

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.

19.4 Design event modelling

19.4.1 Hydrology

19.4.1.1 Overview

The calibrated hydrologic models were used to develop design event flows. Design flows were calculated in accordance with the requirements of ARR 2016.

19.4.1.2 Rainfall data

Design rainfall for each hydrologic model was derived from intensity-frequency-duration (IFD) curves extracted from the Bureau of Meteorology 2016 Rainfall IFD Data System webpage. An example of this data is presented in Table 19.8 for the 24-hour duration.

Table 19.8 24-hour rainfall depth (mm)

Catchment area	50% AEP (mm)	10% AEP (mm)	1% AEP (mm)
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. The rainfall depth was 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 Areal Reduction Factor (ARF) was applied to the rainfall intensities to account for the fact that rainfall is generally not equally extreme all over the catchment.

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

19.4.1.3 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.

The initial loss and continuing loss rates were applied as constant values across the catchments. The design rainfall losses used for each event are presented in Table 19.9.

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 assessment undertaken for the DPIE study.

Table 19.9 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

19.4.1.4 Flood frequency analysis

An FFA was undertaken using historical stream gauge data sourced from BoM for each stream gauge location. Details of the gauges are provided in Table 19.10.

Table 19.10 Gauge details

Gauge	Length of record	Location (catchment)	Comments
Booba Sands	32 years (1987-2018)	Macintyre Brook	All annual peaks were used. Max. value is 1,160 m^3/s (1988)
Farnbro	57 years (1962-2018)	Dumaresq River	All annual peaks were used. Max. value is 1,600 m^3/s (1976)
Roseneath	47 years (1972-2018)	Dumaresq River	All annual peaks were used. Max. value is 5,687 m^3/s (1976)
Holdfast	47 years (1972-2018)	Macintyre River	All annual peaks were used. Max. value is 2,612 m^3/s (1976)
Coolatai	41 years (1978-2018)	Ottleys Creek	All annual peaks were used. Max. value is 562 m^3/s (1994)

The FFA 1% AEP flow estimates were compared against that determined by the hydrologic models. Figure 91 to Figure 95 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.

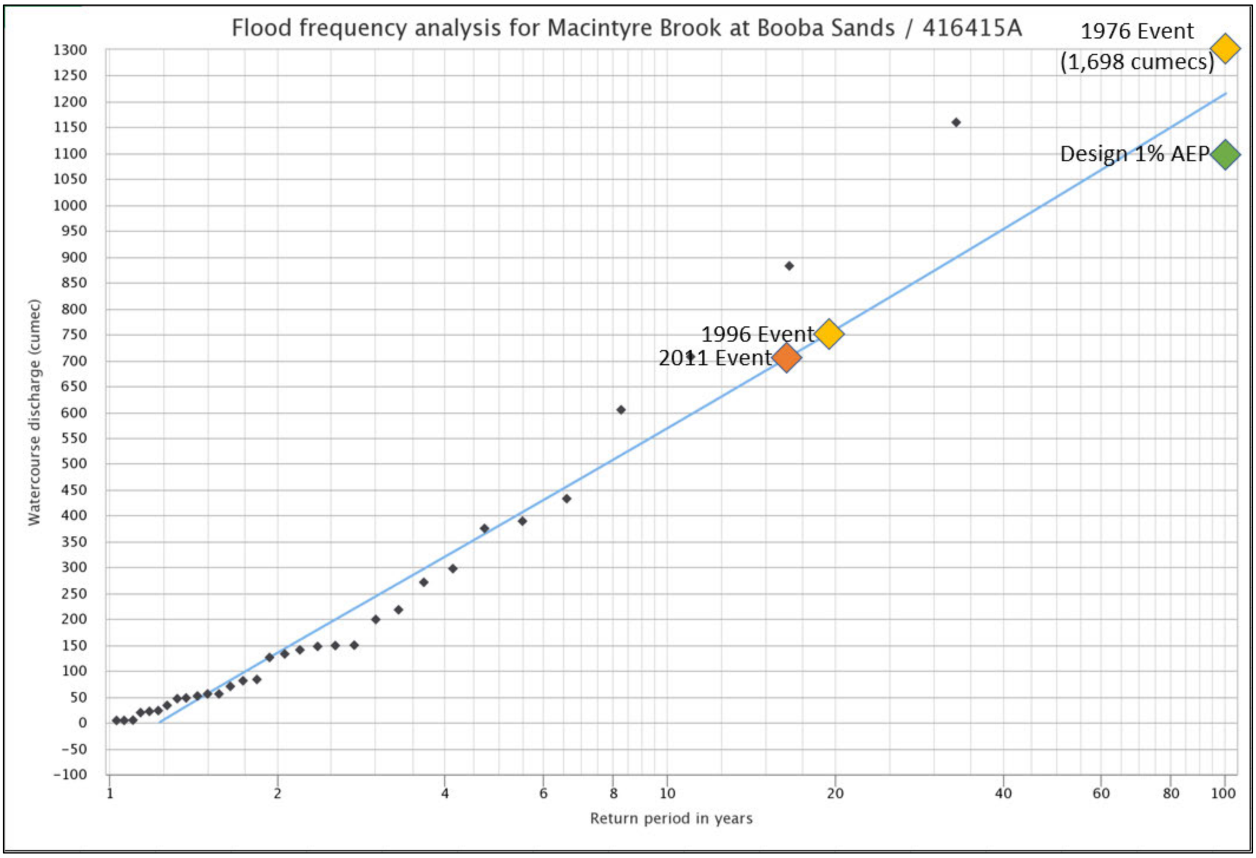


Figure 91 Flood frequency analysis at Booba Sands (GEV)

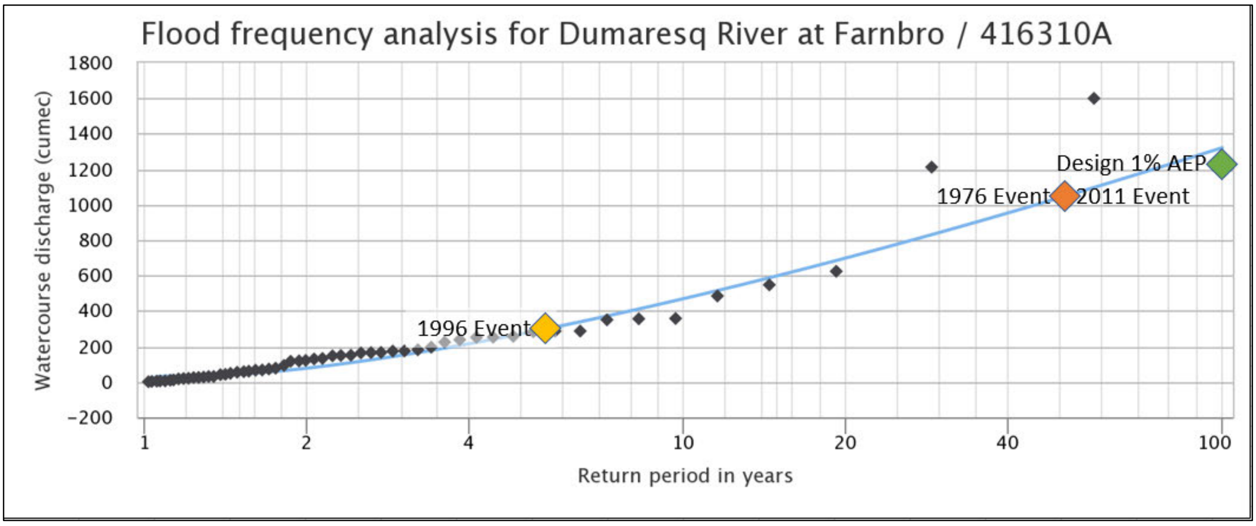


Figure 92 Flood frequency analysis at Farnbro (GEV)

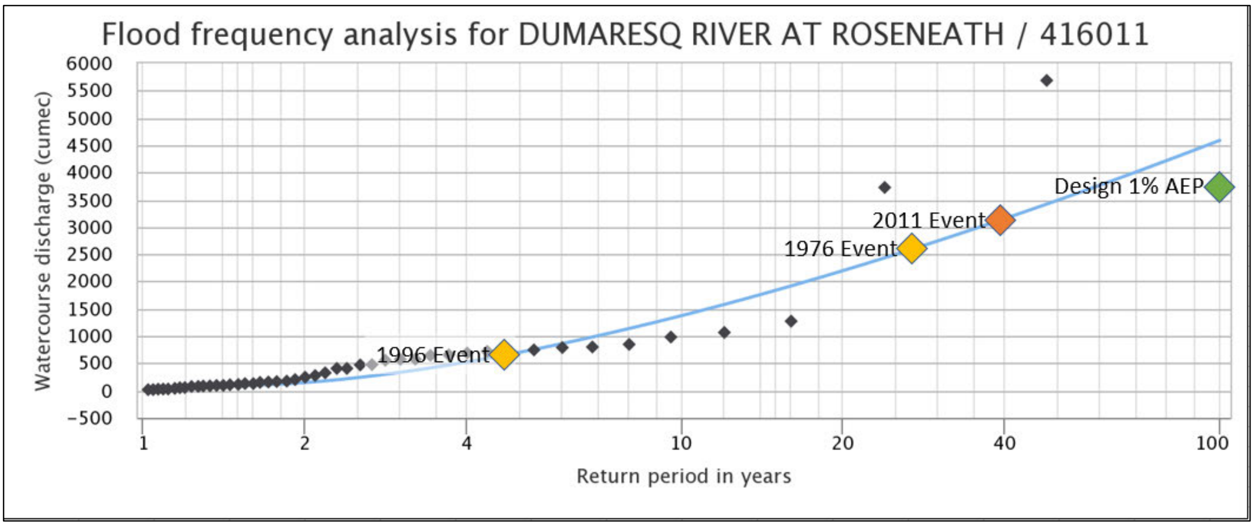


Figure 93 **Flood frequency analysis at Roseneath (LP3)**

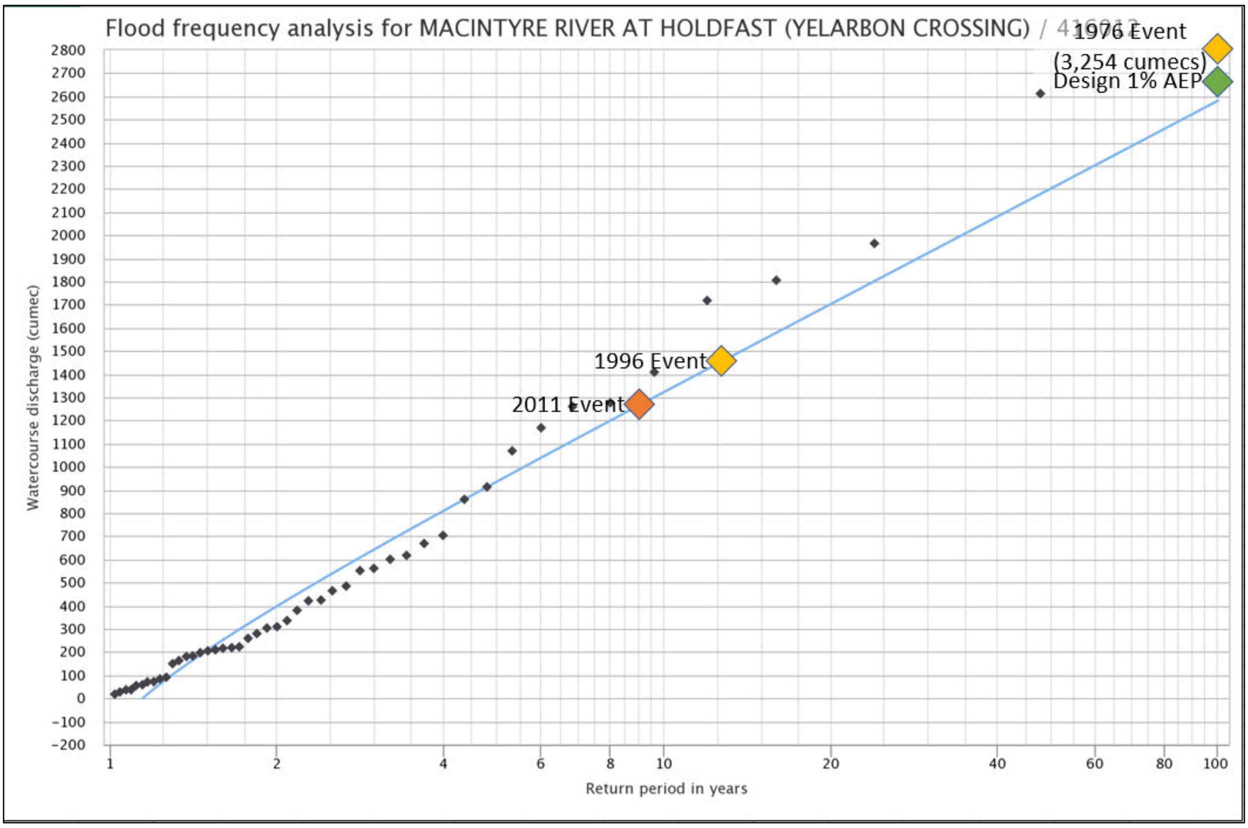


Figure 94 **Flood frequency analysis at Holdfast (GEV)**

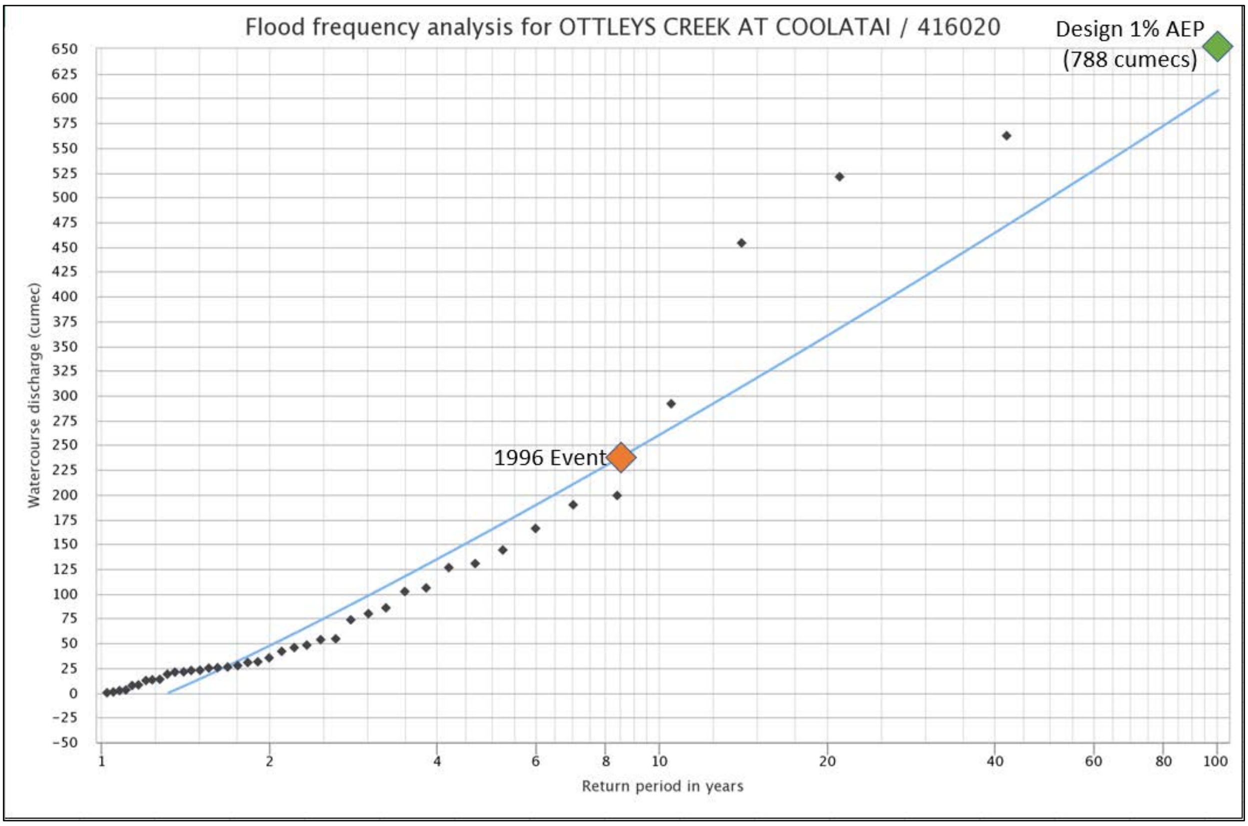


Figure 95 **Flood frequency analysis Coolatai (GEV)**

19.4.1.5 Extreme rainfall

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.

19.4.2 Hydraulic assessment

19.4.2.1 Introduction

Two design models were developed, representing the current state of development (Existing Case) and scenario where the Project alignment had been constructed (Developed Case).

Preparation of the Existing Case hydraulic sub-model to enable assessment of the proposal alignment and associated works was undertaken first. 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.

The Existing Case hydraulic sub-model for the assessment of the Project alignment was developed based on the 2019 LiDAR Existing Case topography. The Developed Case hydraulic sub-model was based on the 2019 LiDAR Existing Case model with the Project alignment, drainage structures and associated work included.

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

- A separate 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 rail alignment design. The larger structure was also included in the hydraulic sub-model to assess impacts associated with the proposed works.

19.4.2.2 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 presented below in Table 19.11.

Table 19.11 Critical durations within the study corridor

Design event	Duration (minutes)
20% AEP	1080m_08b 1080m_10b 1440m_01b 1440m_10b 2880m_01b 2880m_10b 4320m_04b 4320m_05b 4320m_08b
10% AEP	1080m_08b 1080m_10b 1440m_01b 1440m_02b 2880m_07b 2880m_10b 4320m_02b 4320m_04b
5% AEP	1080m_08b 1440m_01b 2880m_07b 2880m_10b 4320m_02b 4320m_07b
2% AEP	1080m_01b 1080m_08b 1440m_02b 1440m_04b 2880m_05b 2880m_10b 4320m_04b
1% AEP	1080m 07b 1080m 08b 1440m 02b 1440m 04b 1440m 09b 2880m 02b
1 in 2,000 AEP	2880m_01b 2880m_09b
1 in 10,000 AEP	1440m_09b 2160m_08b 2880m_05b 4320m_03b
PMF	1440m 09b 2160m 08b 2880m 05b 4320m 03b

19.4.2.3 Flood Frequency Analysis at 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.

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 gauge

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 19.12. 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 (refer discussion below). It is unknown whether the previous studies report total flow or flow at the gauge.

Table 19.12 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[1]	221.2

Table note:

1 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 19.13. 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 19.13 Stream gauge record

Item	Boggabilla Gauge	Goondiwindi Gauge
Years of record	117	76
Censor threshold	350 m³/s	110 m³/s
Censored records	39	8

Results of the FFA are compared with peak flows from the Design Event modelling in Figure 95. 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 m³/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. 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 95 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.

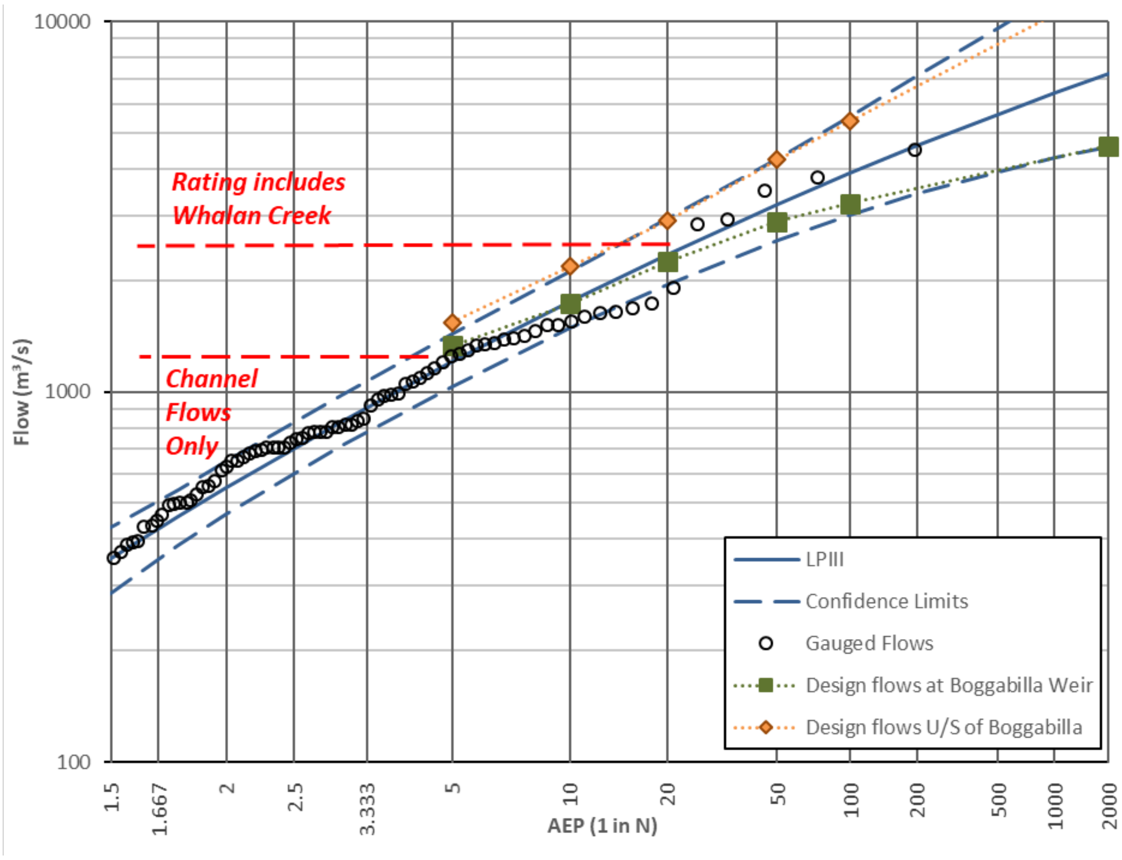


Figure 96 Flood frequency analysis at Boggabilla gauge

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 (Goondiwindi 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 97 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.

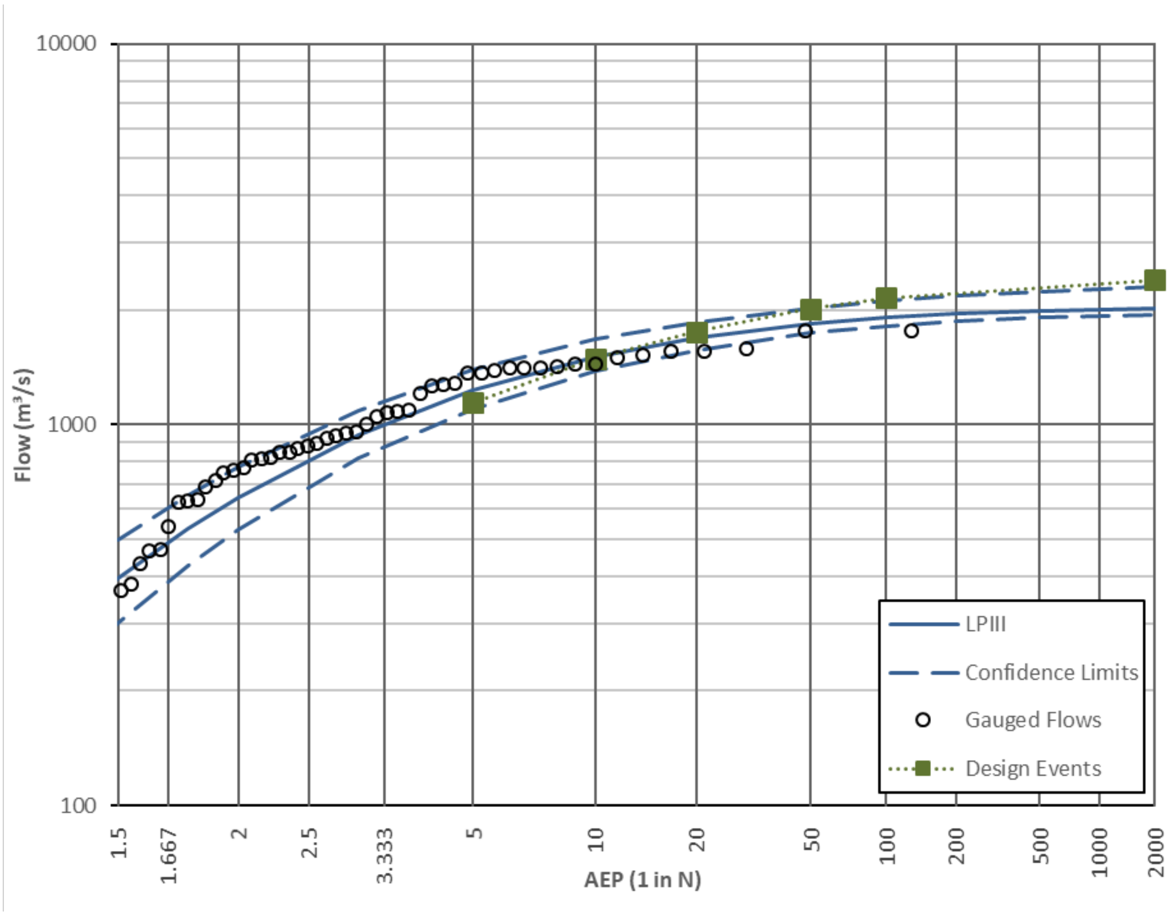


Figure 97 **Goondiwindi gauge flood frequency analysis results**

19.4.2.4 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. A uniform factor of 0.7 was selected for all inflows, but it is acknowledged that the application of a uniform factor is arbitrary. However, in the absence of a full joint probability assessment, it is 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 19.14 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 19.14 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

19.5 Existing Case modelling results – Macintyre River

19.5.1 Existing Case flood maps

Flood maps illustrating indicative flood extents and peak water levels were prepared and are presented in Volume II – Appendix M:

- 20% AEP: Figure M-3a
- 10% AEP: Figure M-3b
- 5% AEP: Figure M-3c
- 2% AEP: Figure M-3d
- 1% AEP: Figure M-3e
- 1 in 2,000 AEP: Figure M-3f
- 1 in 10,000 AEP: Figure M-3g
- PMF: Figure M-3h.

Figure M-4a presents peak flood velocities under a 1% AEP event.

19.5.2 Flood inundation extent and flood levels

Figure M-3e in Volume II – Appendix L shows the 1% AEP indicative flood extent and peak water levels within the Macintyre River floodplain for the Existing Case.

Widespread inundation is predicted under the 1% AEP event on the Macintyre River floodplain, with depths of approximately 10 to 12m in the Macintyre River, 6m in Whalan Creek and up to 2m on the floodplain area.

From the NSW/QLD border (Macintyre River) to the north, ground elevations are higher, and the 1% AEP flooding extends approximately 2 km from the border. This area is predicted to be inundated during extreme floods.

19.5.3 Flood immunity of existing infrastructure

Table 19.15 presents a summary of overtopping depths for key roads and the existing rail near the proposed alignment north of the NSW/QLD Border.

Table 19.15 Existing Case – overtopping depths of key infrastructure (north of the QLD/NSW border only)

Infrastructure	Location	Overtopping depth (m)							
		PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Kildonan Road	Downstream of the Project alignment	3.8	2.7	2.4	1.6	1.4	0.7	0.06	Dry
Kildonan Road	Upstream of the Project alignment	4.8	3.5	3	1.7	1.1	Dry	Dry	Dry

19.5.4 Existing Case velocities

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 township between a 10% and 5% AEP event.

19.6 Developed Case modelling results – Macintyre River

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

19.6.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW model (1d and 2d approach).

In the Macintyre River floodplain north of the QLD/NSW border, the NS2B Project includes the following floodplain (or regional structures):

- Three waterway bridges
- Four RCP locations (a total of 50 cells).

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and depending on their interaction with the Macintyre River floodplain were incorporated in the hydraulic model. North of the QLD/NSW border these are:

- Eight RCBC locations (a total of 56 cells)
- Two RCP locations (a total of 6 cells).

The locations of the structures are presented in Figure M-1f in Volume II – Appendix M.

Bridges were represented within the TUFLOW 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% blocked, and above the bridge deck (Layer 3), 50% blocked. It is recommended that following detailed design, these parameters be revisited.

The structures listed in Table 19.16 and Table 19.17 are assessed within the hydraulic sub-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).

Table 19.16 Macintyre River (north of QLD/NSW border only) - proposed bridge location and details

NS2B chainage (km)	Structure ID	Approximate span (m)	Deck level (m AHD)
30.63	270-BR11	1,748	231.12
31.52	270-BR12	144	230.12
32.55	270-BR13	521	229.00

Table 19.17 Macintyre River (north of QLD/NSW border only) – proposed floodplain culvert locations and details

NS2B chainage (km)	Structure ID	Structure type	U/S invert level (m AHD)	D/S invert level (m AHD)	Diameter/ width (m)	Height (m)	Number of cells
31.26	C31.26	RCP	226.19	226.11	1.8	-	10
31.32	C31.32	RCP	226.12	226.04	1.8	-	10
31.87	C31.87	RCP	226.75	226.61	0.9	-	15
31.97	C31.97	RCP	226.69	226.60	0.9	-	15

19.6.2 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.

19.6.2.1 Rail immunity and structures results

Appendix C presents hydraulic model results at each structure for the 1% AEP event. Local drainage structures (those not included in the flood model) and road culverts are not reported.

The formation level of the rail alignment is driven by several factors including achieving flood immunity and meeting geometric requirements (e.g. allowing for grade separations). Therefore, the freeboard achieved varies along the alignment with the 1% AEP event flood immunity achieved with at least a minimum freeboard of 300 mm, which is driven by non-flood related constraints.

The results of flood modelling indicate that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the Macintyre River floodplain.

19.6.2.2 Velocities and scour

Appendix C summarises the peak 1% AEP outlet flows and velocities at structures. The hydraulic model predicts culvert outlet velocities to be less than 2.5 m/s in accordance with the design requirements. Scour protection requirements for culverts were calculated based on the velocities predicted from the hydraulic modelling. The scour protection was 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.

Table 19.18 lists the soil types encountered along the Project alignment and the allowable soil velocity based on AGRD.

Table 19.18 Allowable soil velocities along the Project alignment

Soil type	Soil description	Maximum allowable soil velocity as per AGRD (m/s)
Sodosols	Firm loam or stiff clay	2 m/s
Vertosols, Dermosols, Rudosols and Kandosols	Sandy or silty clay	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.

Scour protection requirements for culverts have been calculated based on the velocities predicted from the hydraulic modelling. Appendix D presents the proposed scour protection at each structure.

A conservative scour estimation has been undertaken at the bridge site based on available information and will be refined during detailed design.

19.6.2.3 Flood immunity for extreme events

The risk of overtopping of the top of rail was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and the PMF events) with Table 19.19 presenting the overtopping locations by chainage and the depth of water above the formation level and over the top of rail level.

It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the alignment. As such, overtopping of the rail is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario.

Table 19.19 Extreme event overtopping of the proposed alignment

NS2B chainage (km)	Depth of water above formation level (m)			Depth of water over 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
30.36 – 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

19.6.3 Flood impact objectives outcomes – Macintyre River

The impact of the Developed Case was assessed through inclusion of the Project and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, duration of inundation, velocities and peak flows within the floodplain.

- Changes in peak water levels for the AEP's assessed are presented in Figures M-5a to M-5h in Volume II – Appendix M
- Changes in 1% AEP duration of inundation are presented in Figure M-5i in Volume II – Appendix M
- Changes in 1% AEP velocities are presented in Figure M-5j in Volume II – Appendix M

All impacts are required to be agreed with relevant stakeholders and affected landowners as part of the EIS process. One-on-one consultation with landowners who are expected to experience changes in flooding behaviour on the property was conducted by ARTC supported by FFJV.

The Project design outcomes relative to the flood impact objectives (refer Table 4.2) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Volume II – Appendix N.

19.6.3.1 Flood impacts at proposed hydraulic structures

The change in peak water levels at the proposed structures are presented in Table 19.20. Peak water levels were extracted upstream of culverts and at the control line of the bridge.

The design achieved a freeboard of at least 300 mm between the 1% AEP peak water and formation levels.

Table 19.20 Macintyre River (north of the QLD/NSW border only) - 1% AEP event – estimated impacts to peak water levels at proposed hydraulic structures

Chainage (km)	Structure ID	Structure type	Rail formation level (m AHD)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Change in peak water level (mm)
30.35	270-BR11	Bridge	-	227.95	227.96	10
31.26	C31.26	RCP	232.52	227.35	227.54	190
31.32	C31.32	RCP	232.04	227.24	227.47	230
31.52	270-BR12	Bridge	-	227.2	227.43	230
31.87	C31.87	RCP	229.05	226.98	227.14	160
31.97	C31.97	RCP	229.05	227.03	227.14	110
32.55	270-BR13	Bridge	-	227.06	227.14	80

19.6.3.2 Flood impacts on flood sensitive receptors

Flood sensitive receptors were identified from aerial imagery. Locations of flood sensitive receptors are presented in the impact figures in Volume II - Appendix M, Figure M-5a to M-5j. For the 1% AEP event there is no afflux above 10 mm predicted at identified flood sensitive receptors in the Macintyre River floodplain to north of the NSW/QLD border.

Impacts to flood sensitive receptors that exceed the flood impact objectives are reported in the EIS Surface Water Chapter.

19.6.3.3 Flood impacts on Queensland Rail

No impacts on the existing Queensland Rail line to the west of the Project alignment are expected in a 1% AEP event.

19.6.3.4 Flood impacts on key public roads

The extent of the hydraulic model developed for Macintyre River and relevant state-controlled roads are shown in Figure 98.

The change in peak water levels and flood hazard (velocity-depth) for the 1% AEP event were evaluated on key public roads within the Project hydraulic model domain north of the NSW/QLD border. No key roads are expected to experience an increase in flood hazard or increases in peak flood levels.

Duration of inundation

Assessment of the time of submergence (ToS) and average annual time of submergence (AAToS) was undertaken for key public roads within the Project hydraulic model domain, and the results are presented in Table 19.21.

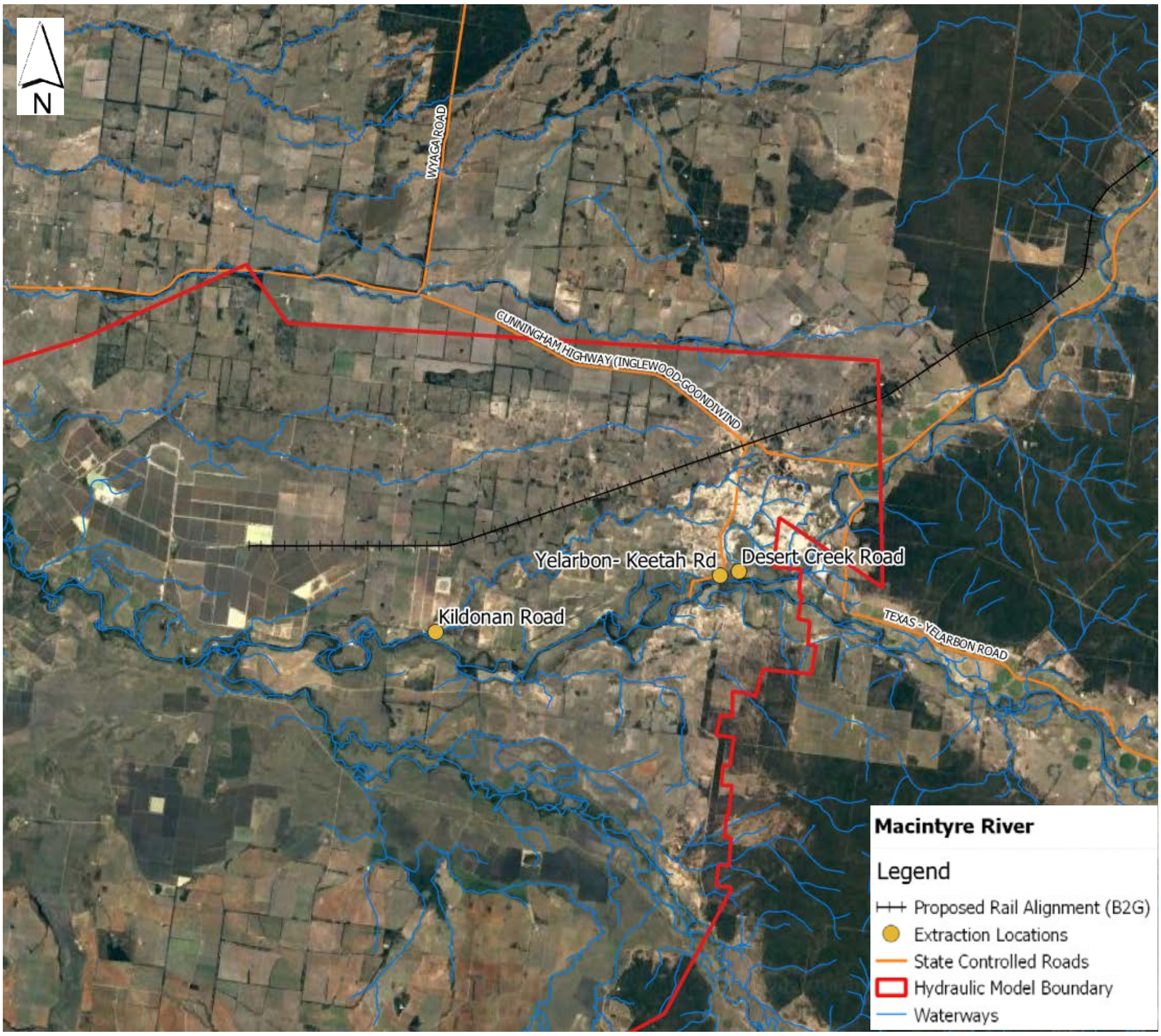


Figure 98 **Macintyre River Hydraulic Model Extent and Associated State-controlled Roads**

Table 19.21 Macintyre River – ToS and AAToS for local public roads (north of QLD/NSW border only)

Location	Existing 1% AEP ToS (hrs)	1% AEP ToS diff. (hrs)	2% AEP ToS diff. (hrs)	5% AEP ToS diff. (hrs)	10% AEP ToS diff. (hrs)	AAToS Existing Case (hrs)	AAToS Developed Case (hrs)	AAToS diff. (hrs)
Desert Creek Road	87.30	0.01	-0.01	-0.01	0.01	64.17	64.17	0.00
Kildonan Road	72.25	0.01	-0.01	0.01	-0.01	34.68	34.69	0.01
Yelarbon - Keetah Road	77.03	0.01	0.01	-0.01	0.01	44.67	44.67	0.00

19.6.3.5 Flood impacts on private land outside the rail disturbance footprint

The majority of the area where afflux is expected is agricultural land or open land on which nominal afflux is unlikely to cause any adverse impact.

Table 19.22 presents the modelled changes in flood conditions during the 1% AEP event on a lot basis according to the following thresholds:

- Peak water levels increased by greater than +10 mm; or
- Peak velocities increased by greater than 0.25 m/s; or
- Duration of inundation changed by more than 25% of its original duration of inundation across the lot.

Table 19.22 Macintyre River – summary of flood impacts on private land outside the rail disturbance footprint for 1% AEP (north of NQLD/NSW border only)

NS2B approximate chainage (km)	Changes in peak water levels[1]		Changes in peak velocities		Changes in Duration of inundation (hrs)	
	Maximum change (mm)	Total area affected by change > 10 mm (ha)[2]	Maximum change (m/s)	Total area affected by change (ha)	Maximum change (%)	Total area affected by change (ha)
31.60 to 33.0	125	77	-	-	35	1.5
31.00 to 31.4	222	7.4	0.35	0.9	-	-
30.35 to 31.0 (area north of the NSW/QLD Border)	30	99	-	-	-	-

Table notes:

1 Afflux on lots that exceed the flood impact objectives are summarised in the EIS Surface Water Chapter.
2 Only minor areas, usually directly upstream of culverts are affected by the maximum afflux as stated.

19.6.4 Sensitivity analysis – Macintyre River

19.6.4.1 Blockage

A blockage factor of 25% was incorporated into the culvert design as per ARR 2016 guidelines. In addition, two blockage sensitivity scenarios were tested; 0% and 50% blockage of all culverts. The results are presented in Figure M-6a and Figure M-6b in Volume II – Appendix M for the 0% and 50% blockage respectively.

The model predicts that in both the 0% and 50% blocked cases the predicted changes in peak water levels meet the guiding design criteria for the 1% AEP event. Table 19.23 provides a summary of 1 % AEP peak flood levels at cross drainage structures for the blockage scenarios.

Table 19.23 Macintyre River – 1 % AEP event – culvert blockage assessment (north of QLD/NSW border only)

Structure ID	Structure type	1 % AEP Peak water levels (m AHD)			Increase from Developed Case to 50% blockage scenario (mm)
		0 % blockage	Developed Case (25 % blockage)	50 % blockage	
C31.26	RCP	227.53	227.54	227.58	+37
C31.32	RCP	227.45	227.47	227.51	+43
C31.87	RCP	227.14	227.14	227.14	+5
C31.97	RCP	227.13	227.14	227.14	+3

There is negligible change to the 1% AEP afflux caused by the Project within the Macintyre River floodplain under the blockage scenarios given the significant bridge spans allowed for in the design. There are no changes to impacts on flood sensitive receptors.

During detail 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.

19.6.4.2 Impacts during extreme events

Table 19.24 outlines the changes in peak water levels at flood sensitive receptors for the extreme events where the increase exceeds 10 mm under one of the events. The existing depth of flooding is also detailed and as can be seen the larger impacts that occur under the PMF event occur generally when there are already high flood depths as would be expected under such a rare event.

For the 1% AEP event there is no afflux above 10 mm predicted at identified habitable dwellings. At non habitable dwellings there is one shed (MRC_ID_41) above 10 mm afflux with an afflux predicted of 47 mm in the 1% AEP event and an existing depth of 174 mm.

Flood immunity of the Project alignment is discussed in Section 19.6.2.3, and maps demonstrating the impacts during extreme events are shown in Volume II – Appendix M, Figures M-5f to M-5h.

Table 19.24 Macintyre River (north of QLD/NSW border) – Summary of extreme event impacts at flood sensitive receptors

Flood sensitive receptor ID	1 in 2,000 AEP event		1 in 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)
MRC_ID_41	+212	1.12	+235	1.46	+336	2.60
MRC_ID_36	+4	2.63	+22	2.88	+51	3.84
MRC_ID_93	-	-	-	-	+34	0.42
MRC_ID_95	+5	0.04	+21	0.08	+34	0.46
MRC_ID_98	-	-	-	-	+94	0.20
MRC_ID_104	+5	1.31	+22	1.49	+44	2.25
MRC_ID_105	+6	1.28	+23	1.48	+44	2.22
MRC_ID_106	+6	1.92	+23	2.12	+42	2.86
MRC_ID_108	+6	0.07	+24	0.15	+53	1.05
MRC_ID_111	+75	0.84	+233	1.18	+215	2.32
MRC_ID_115	+5	0.47	+25	0.68	+53	1.58
MRC_ID_118	+8	1.41	+29	1.66	+57	2.61
MRC_ID_120	+50	1.38	+154	1.79	+117	3.06
MRC_ID_123	-	-	+89	0.08	+68	1.62

Flood sensitive receptor ID	1 in 2,000 AEP event		1 in 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)
MRC_ID_126	+1	2.12	+7	2.55	+29	4.21
MRC_ID_128	0	1.09	+2	1.43	+19	3.11
MRC_ID_129	0	0.85	+2	1.19	+19	2.87
MRC_ID_130	0	0.69	+3	1.04	+19	2.73
MRC_ID_131	0	0.72	+3	1.06	+19	2.75
Kildonan Road - Downstream of the Project alignment	+6	2.39	+28	2.67	+53	3.75
Kildonan Road - Upstream of the Project alignment	+17	2.98	+56	3.44	+61	4.74

19.6.4.3 Climate change

The potential impacts of climate change in the Macintyre River floodplain were assessed for the 1% AEP design event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2016.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario has been adopted for the Project with an associated increase in rainfall intensity of 23% across the catchment area.

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.

The 1% AEP change in peak water levels for the Representative Concentration Pathways 8.5 climate change scenario is presented in Figure M-6c in Volume II – Appendix M.

The model predicts that with an increase in rainfall intensity of 23% across catchment the flood levels increase by up to 200 mm in the vicinity of the alignment. The Project alignment is not predicted to be overtopped as a result of the 23% increase in rainfall intensity with water levels predicted to be below formation level. Table 19.25 presents the structure performance with Representative Concentration Pathways 8.5 climate change conditions.

Table 19.25 Macintyre River - 1% AEP event with Representative Concentration Pathways 8.5 conditions – structure performance (north of QLD/NSW border only)

Structure ID	Structure type	1 % AEP Peak water levels (m AHD)	1 % AEP + Climate Change Peak water levels (m AHD)	Difference in peak water levels (m)	Freeboard to rail formation with climate change (m)
270-BR11	Bridge	227.96	228.21	0.25	2.31
C31.26	RCP	227.54	227.84	0.30	4.68
C31.32	RCP	227.47	227.77	0.30	4.27
270-BR12	Bridge	227.43	227.70	0.27	3.71
C31.87	RCP	227.14	227.66	0.52	1.39
C31.97	RCP	227.14	227.64	0.51	1.40
270-BR13	Bridge	227.14	227.50	0.36	1.54

One (1) flood sensitive receptor is impacted by the climate change. Flood sensitive receptor MCR_ID_41 has a predicted impact of +120 mm in the 1% AEP flood event with climate change, with an existing 1% AEP with climate change predicted flood depth of 453 mm. This flood sensitive receptor is a shed.

The downstream extents of these impacts are similar to those under the 1% AEP event.

20 Limitations

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 Project. 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 2015 (or as detailed in each catchment section). Therefore, any development or topographical change occurring within the catchment after the surveys taken is not included in this investigation, unless directly 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.

21 Conclusions

The key objectives of the Hydrology and Flooding Technical Report are to provide information on the data investigation, hydrology and hydraulic calibration, impact assessment and mitigation and to provide comment on the performance on the Project design. This report outlines the methodology followed, the outcomes of this investigation and the assessment of the Project design.

There are several major waterways within the vicinity of the Project, with the key waterways being the Macintyre River, Macintyre Brook, the Condamine River and Gowrie Creek. Other major creek crossings include Pariagara Creek, Cattle Creek, Native Dog Creek, Bringalily Creek, Nicol Creek, Back Creek and Westbrook Creek. The Project alignment crosses 16 major waterways (stream order ≥ 3) and 66 minor waterways (stream order <3).

Hydrologic and hydraulic modelling was undertaken for each of these catchments with the models calibrated to multiple historical events using stream gauges records and anecdotal data where available. Based on this performance, the hydrologic and hydraulic models were considered validated and appropriate to use to assess the potential impacts associated with the Project.

Design event hydrology was developed using the calibrated hydrologic models using ARR 2016 flood flow estimation techniques. The hydraulic models were run for a suite of design events from the 20% AEP event to the 1 in 10,000 AEP and PMF events. The flows and levels predicted by the hydrologic and hydraulic models were compared to the results of an FFA at stream gauges within each catchment as well as results from previous flood studies.

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 proposed works associated with the Project were incorporated into the hydraulic models to form the Developed Case. Assessment of the potential impacts upon the existing flood regime was undertaken and refinement of the Project 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 Project design has been guided and refined using hydraulic design criteria Table 4.1 and flood impact objectives (refer Table 4.2). The resulting design outcomes relative to the hydraulic design criteria are detailed in Table 21.1.

Table 21.1 Project hydraulic design criteria outcomes

Performance criteria	Design outcomes
Flood immunity	Rail line – 1 % AEP flood immunity with minimum of 300 mm freeboard 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. The Project design includes significant rail drainage structures under the Project alignment to convey flood flows on floodplains and minimise impacts under the full range of design events, being: ▪ Twenty seven (27) rail bridges ▪ One hundred and twenty (120) rail reinforced concrete box culvert (RCBC) banks ▪ Two hundred and twelve (212) rail reinforced concrete pipe culvert (RCP) banks ▪ Inclusion of road drainage structures under local roads adjacent to the Project alignment
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. A conservative scour estimation has been undertaken at each bridge site based on available information and will be refined during detailed design.

Performance criteria	Design outcomes
Structural design	1 in 2,000 AEP event has been modelled with details used for bridge design purposes.
Extreme events	Overtopping of the Project alignment under extreme events occurs at limited locations being: ▪ Gowrie Creek – Above formation level and top of rail level at Ch 205.87 km for PMF event. ▪ Westbrook and Dry Creeks – Above formation at Ch 188.72 km in 10,000 AEP and PMF events; above formation at Ch 193.4 in PMF event; above formation and top of rail at Ch 197.5 to Ch 197.7 km in PMF event. ▪ Condamine River – Above formation at several locations in 10,000 AEP and PMF events; above top of rail at several locations in a PMF event. ▪ Back Creek – Above formation and top of rail in 2,000, 10,000 AEP and PMF events at Ch 126.85 to 127.05 and 127.95 to 128.15 km. ▪ Nicol Creek – Above formation and top of rail at Ch 104.35 to 104.45 km for the 2,000, 10,000 AEP and PMF events. ▪ Bringalily Creek – Above formation and top of rail in several locations for the 2,000, 10,000 AEP and PMF events. ▪ Native Dog Creek – Above formation and top of rail at Ch 93.85 to 94.05 km for the 2,000, 10,000 AEP and PMF events. ▪ Cattle Creek – Above formation and top of rail at Ch 88.25 to 88.35 km for the 2,000, 10,000 AEP and PMF events. ▪ Pariagara Creek - Above formation and top of rail at Ch 67.05 for the 2,000, 10,000 AEP and PMF events. ▪ Macintyre Brook – Not overtopped. ▪ Macintyre River – Above formation and top of rail at Ch 31.30 to 35.00 km for the PMF event.
Flood flow distribution	Structures have been located along the Project alignment to maintain existing flood conveyance and spread of floodwaters.
Sensitivity testing	The risk to the Project design from climate change and blockage has been assessed in accordance with Australian Rainfall and Runoff 2016. Key outcomes are: ▪ The Project design maintains 1 % AEP flood immunity under 2090 climate change conditions ▪ Based on ARR 2016, a blockage factor of 25% has been applied to culverts and no blockage factor has been applied to bridges ▪ Varying the level of blockage to culverts between 0 % and 50 % does not impact upon the Project design

Flood impact objectives, as presented in Table 4.2, have been established and used to guide the Project design including mitigation of impacts through refinement of the hydraulic design, including adjustment of the numbers, dimensions and location of major drainage structures. Table 21.2 summarises how the Project design performs against each of the flood impact objectives.

Table 21.2 Flood impact objectives and outcomes

Parameter	Objectives and outcomes				
Change in peak water levels	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 (cropping) land	Agricultural (grazing) land/forest areas and other non-agricultural land
	≤ 10 mm	≤ 50 mm	≤ 100 mm	≤ 100 mm with localised areas up to 400 mm	≤ 200 mm with localised areas up to 400 mm

Parameter	Objectives and outcomes
	Objective: Changes in peak water levels are to be assessed against the above proposed limits. **Outcome:** Generally, the Project design meets the above limits with number of localised areas along the Project alignment where these limits are exceeded. These areas are generally on agricultural land. Flood sensitive receptors that are impacted by changes in peak water levels under the 1% AEP event that exceed the flood impact objectives include: - Nine dwellings (five between Pampas and Yandilla, and four at Yelarbon) - One shed at Pampas - Three commercial buildings (grain silos) at Yandilla - One state-controlled road (Cunningham Highway at Yelarbon) - One local public road (Leesons Road between Kingsthorpe and Gowrie Junction)
Change in duration of inundation	**Objective:** Identify changes to duration of inundation through determination of ToS. For roads, determine Average Annual Time of Submergence (AAToS) (if applicable) and consider impacts on accessibility during flood events. **Outcome:** There are localised increases in Time of Submergence (ToS) at the same locations where peak water levels are increased. These changes in inundation duration do not affect flood sensitive receptors except for one local public road local being Draper Road and one state-controlled road being the Cunningham Highway. The Cunningham Highway has a +0.8 hrs/yr increase in AAToS which is a negligible change with Draper Road experiencing an even lower 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. **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. Determine appropriate scour mitigation measures taking into account existing soil conditions. **Outcome:** In general, changes in velocities are minor, with most changes in velocities experienced immediately adjacent to the Project alignment and no flood sensitive receptors impacted. 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.
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. **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. Considering the flood depths that occur, particularly under the PMF event, indicates that the changes in peak water levels would be unlikely to exacerbate flood conditions during extreme events.
Sensitivity testing	**Objective:** Consider risks posed by climate change and blockage in accordance with ARR 2016. Undertake assessment of impacts associated with Project alignment for both scenarios. **Outcomes:** Climate change – climate change has been assessed in accordance with ARR 2016 requirements with the representative concentration pathway 8.5 (2090 horizon) scenario adopted. 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, with some additional impacts on flood sensitive receptors. 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 localised but impact on some flood sensitive receptors.

A comprehensive consultation exercise has been undertaken to provide the community with detailed information and certainty around the flood modelling and the Project design. 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 Project
- Continue to work with directly impacted landowners affected by the alignment throughout the detailed design, construction and operational phases of the Project
- Continue to work with local Councils and state government departments throughout the detailed design, construction and operational phases of the Project.

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Appendices

APPENDIX Q1

Hydrology and Flooding Technical Report—Volume I

Appendix A Project Figures

INLAND RAIL—BORDER TO GOWRIE ENVIRONMENTAL IMPACT STATEMENT

IR_1534

Appendix A
Project figures

- A1: Project Alignment
- A2: Creek and River Systems
- A3: Hydraulic Model Extents

Future Freight
Integrating Community, Environment and Engineering

DATUM GDA 1994, PROJECTION MGA ZONE 56

0 7.5 15 22.5 30 km

(when printed at A3)

Legend

- Project Alignment
- State Controlled Roads
- Local Government Areas

Inland Rail - Border to Gowrie Project:

Project Alignment

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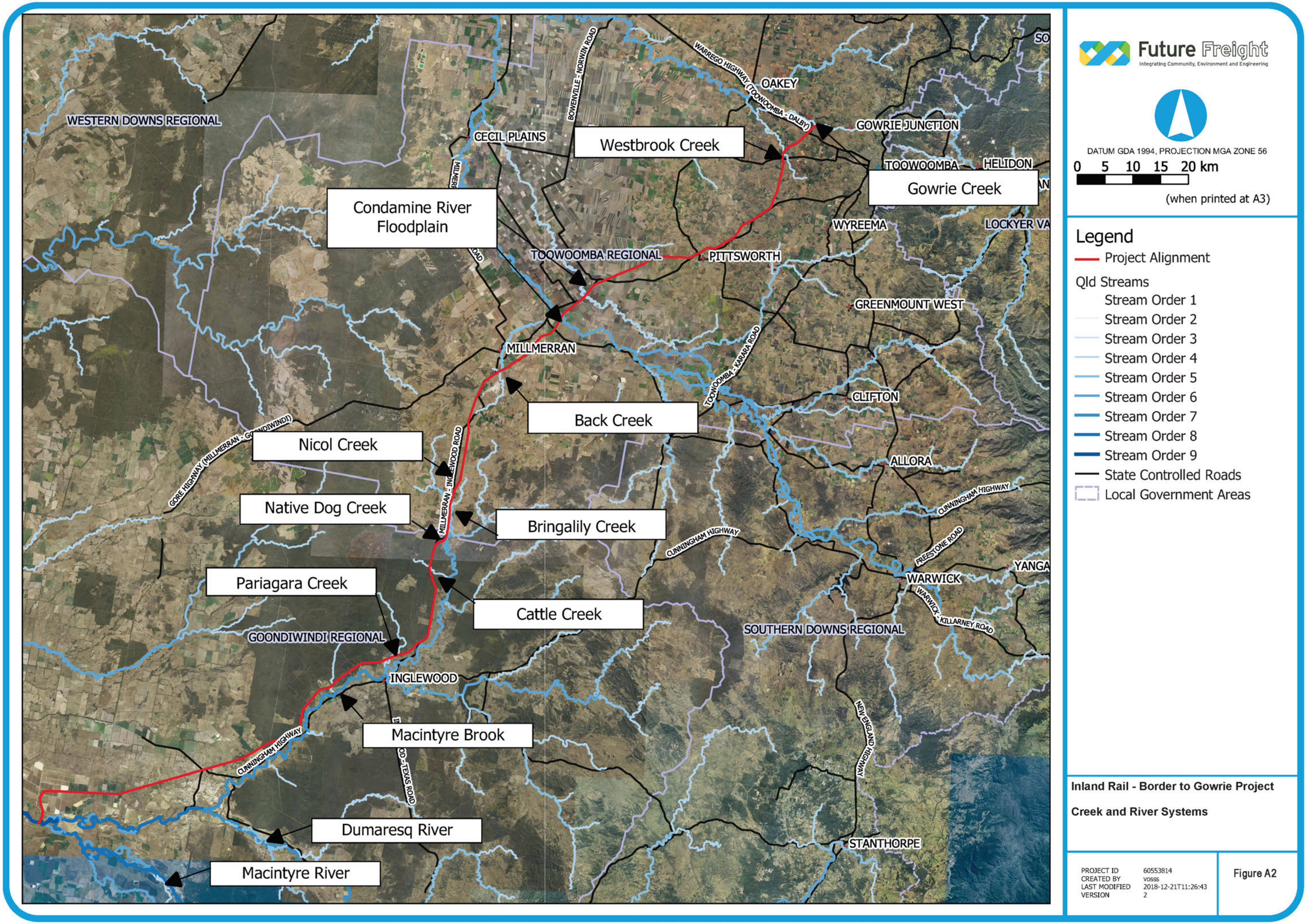
LAST MODIFIED	2018-12-21T11:20:47
VERSION	2

Figure A1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Gowrie Creek
Condamine River Floodplain
Back Creek
Nicol Creek
Native Dog Creek
Bringalily Creek
Pariagara Creek
Cattle Creek
Macintyre Brook
Dumaresq River
Macintyre River
WESTERN DOWNS REGIONAL
CECIL PLAINS
OAKEY
GOWRIE JUNCTION
TOOWOOMBA
HELIDON
WYREEMA
LOCKYER VA
TOOWOOMBA REGIONAL
PITTSWORTH
GREENMOUNT WEST
MILLMERRAN
CLIFTON
ALLORA
WARWICK
YANGA
SOUTHERN DOWNS REGIONAL
GOONDIWINDI REGIONAL
INGLEWOOD
STANTHORPE
WARREGO HIGHWAY (TOOWOOMBA - DALBY)
BOWENVILLE - NORWIN ROAD
GORE HIGHWAY (MILLMERRAN - GOONDIWINDI)
MILLMERRAN - INGLEWOOD ROAD
TOOWOOMBA - KARARA ROAD
CUNNINGHAM HIGHWAY
FREESTONE ROAD
WARWICK - KILLARNEY ROAD
NEW ENGLAND HIGHWAY
Future Freight
Integrating Community, Environment and Engineering
DATUM GDA 1994, PROJECTION MGA ZONE 56
0 5 10 15 20 km
(when printed at A3)
Legend
Project Alignment
Qld Streams
Stream Order 1
Stream Order 2
Stream Order 3
Stream Order 4
Stream Order 5
Stream Order 6
Stream Order 7
Stream Order 8
Stream Order 9
State Controlled Roads
Local Government Areas
Inland Rail - Border to Gowrie Project
Creek and River Systems
PROJECT ID 60553814
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VERSION 2
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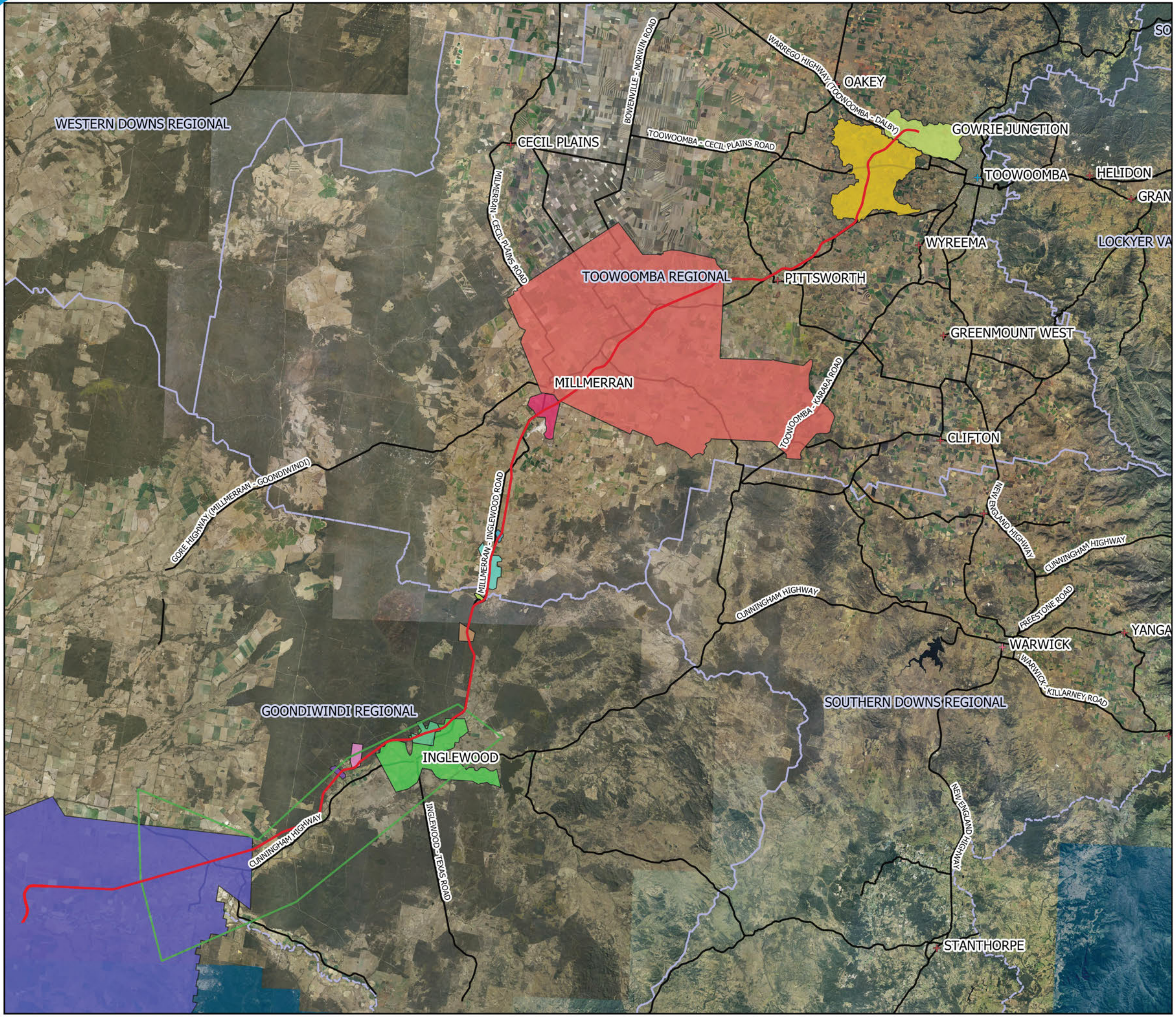

Future Freight
Integrating Community, Environment and Engineering

DATUM GDA 1994, PROJECTION MGA ZONE 56

0 5 10 15 20 km

(when printed at A3)

Legend

- Project Alignment

Model Extents

- Back Creek
- Bringalily Creek
- Bybera Road
- Cattle Creek
- Condamine River
- Cremascos Road
- Gowrie Creek
- Native Dog Creek
- Nicol Creek
- Pariagara Creek
- Westbrook
- Border Rivers
- Inglewood
- Macintyre Brook
- State Controlled Roads
- Local Government Areas

Inland Rail - Border to Gowrie Project

Hydraulic Model Extents

PROJECT ID	60553814
CREATED BY	vosss
LAST MODIFIED	2018-12-21T11:51:05
VERSION	2

Figure A3

APPENDIX Q1

Hydrology and Flooding Technical Report—Volume I

Appendix B Local Drainage Structures

INLAND RAIL—BORDER TO GOWRIE ENVIRONMENTAL IMPACT STATEMENT

IR_1534

Appendix B
Local drainage structures

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – culverts Span (m) - bridges	Height (m) – culverts Soffit level (m AHD) - bridges	Length (m)	Catchment area (ha)
C6.60	6.600	RCBC	3	1.5	1.2	16.56	69.8
C8.39	8.393	RCBC	8	1.2	1.2	9.66	109.8
C13.00	13.000	RCBC	13	2.4	1.2	10.24	627.4
C17.89	17.885	RCP	2	1.05	-	22.21	9.5
C18.51	18.506	RCP	2	1.2	-	20.33	12.7
C18.87	18.870	RCP	4	1.2	-	20.34	44.3
C20.00	19.995	RCBC	6	1.2	1.2	11.11	61.0
C22.42	22.424	RCBC	8	1.2	1.2	9.09	102.3
C23.05	23.046	RCBC	6	1.2	1.2	9.58	36.0
C23.53	23.530	RCBC	7	1.2	1.2	8.79	53.5
C24.41	24.410	RCBC	3	1.2	1.2	8.92	26.1
C24.84	24.839	RCBC	1	1.2	1.2	10.39	21.8
C30.60	30.595	RCBC	7	3	1.2	12.19	448.3
C31.40	31.397	RCP	3	1.2	-	11.44	16.7
C31.44	31.438	RCP	5	1.35	-	11.27	80.0
C32.03	32.030	RCP	10	0.9	-	10.59	71.0
C32.80	32.800	RCP	4	0.9	-	10.65	12.5
C33.66	33.664	RCP	9	1.5	-	13.07	53.8
C35.18	35.175	RCP	12	2.1	-	11.14	492.5
C41.20	41.195	RCBC	18	3	1.5	14.33	349.6
C46.46	46.464	RCP	10	1.2	-	10.58	81.1
C48.41	48.406	RCP	21	2.4	-	75.97	802.1
C49.83	49.825	RCBC	3	3	1.5	7.49	71.7
C49.97	49.972	RCBC	8	2.1	0.9	7.22	86.1
C51.50	51.495	RCP	7	1.35	-	37.55	84.2
C53.20	53.201	RCP	7	1.2	-	15.23	31.9

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – culverts Span (m) - bridges	Height (m) – culverts Soffit level (m AHD) - bridges	Length (m)	Catchment area (ha)
C53.62	53.618	RCBC	2	1.2	0.9	9.98	13.6
C54.44	54.439	RCBC	2	0.9	0.9	10.01	7.7
C55.06	55.056	RCP	7	1.05	-	50.26	52.9
C60.18	60.175	RCP	1	0.9	-	43.19	2.8
C60.49	60.490	RCP	11	2.4	-	90.73	196.2
C61.60	61.600	RCP	5	2.4	-	82.47	224.6
C61.90	61.900	RCP	5	2.4	-	70.15	222.3
C62.52	62.524	RCBC	2	1.2	0.9	21.25	9.3
C62.94	62.940	RCP	2	2.4	-	59.10	40.9
C63.15	63.150	RCP	5	2.4	-	75.63	167.7
C64.50	64.495	RCP	5	0.9	-	11.14	34.8
C65.11	65.114	RCBC	13	2.1	1.2	9.34	84.5
C66.81	66.813	RCP	16	1.2	-	17.81	87.7
C70.50	70.500	RCP	2	1.2	-	12.12	20.0
C71.51	71.510	RCBC	3	2.4	0.9	9.00	32.1
C73.33	73.330	RCP	2	1.2	-	15.54	15.2
C73.43	73.430	RCP	1	1.2	-	17.51	4.1
C73.52	73.520	RCP	1	1.35	-	24.63	9.3
C73.61	73.605	RCP	1	0.9	-	22.50	1.2
C73.71	73.705	RCBC	3	3	1.5	35.80	167.5
C74.97	74.970	RCP	3	1.2	-	21.86	22.2
C76.57	76.570	RCBC	16	1.2	0.9	12.13	52.9
C77.20	77.195	RCP	6	1.5	-	13.66	48.5
C77.47	77.465	RCP	4	0.9	-	12.27	12.3
C77.77	77.770	RCBC	8	1.2	1.2	10.87	48.0
C78.28	78.280	RCBC	4	2.1	0.9	10.00	10.0

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – culverts Span (m) - bridges	Height (m) – culverts Soffit level (m AHD) - bridges	Length (m)	Catchment area (ha)
C79.02	79.015	RCBC	5	2.4	1.5	30.58	105.1
C79.53	79.525	RCP	7	1.35	-	19.27	35.7
C79.98	79.980	RCBC	7	0.9	0.9	12.33	19.9
C80.65	80.645	RCBC	6	1.8	1.5	10.55	47.5
C81.19	81.185	RCBC	7	2.1	2.1	17.67	82.6
C82.35	82.350	RCBC	18	2.1	2.1	16.46	193.2
C83.51	83.505	RCBC	8	2.4	1.5	8.44	68.2
C84.38	84.380	RCBC	35	2.4	2.4	21.71	498.7
C87.54	87.540	RCBC	3	1.8	1.8	24.24	24.4
C88.11	88.110	RCP	2	1.2	-	27.89	6.3
C90.96	90.960	RCBC	31	2.4	2.4	36.36	478.4
C92.08	92.080	RCBC	16	2.4	1.2	9.20	199.2
C92.94	92.940	RCP	17	1.5	-	12.45	137.8
C93.61	93.610	RCBC	8	1.8	1.8	15.27	65.9
C94.91	94.910	RCBC	5	2.1	0.9	9.14	38.7
C95.07	95.065	RCBC	15	2.4	1.5	10.30	389.9
C96.20	96.195	RCBC	8	2.4	1.2	9.82	162.1
C98.87	98.865	RCP	1	1.5	-	23.19	20.3
C101.49	101.485	RCBC	2	1.5	1.2	14.50	5.9
C102.55	102.545	RCBC	2	1.5	0.9	8.34	17.9
C106.54	106.543	RCP	5	1.2	-	12.69	30.7
C107.22	107.222	RCBC	4	2.4	1.5	21.82	86.3
C107.81	107.808	RCBC	2	0.9	0.9	7.63	5.2
C107.97	107.965	RCP	4	1.2	-	14.34	30.8
C108.46	108.455	RCP	4	1.2	-	37.42	22.7
C109.43	109.430	RCBC	12	1.5	1.5	16.18	85.1

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – culverts Span (m) - bridges	Height (m) – culverts Soffit level (m AHD) - bridges	Length (m)	Catchment area (ha)
C110.91	110.913	RCBC	3	1.2	1.2	9.46	10.3
C111.17	111.165	RCBC	3	1.2	1.2	8.42	4.0
C111.26	111.260	RCBC	4	1.2	0.9	8.33	15.4
C112.33	112.325	RCBC	3	1.2	1.2	11.92	10.5
C113.00	113.000	RCBC	4	1.2	1.2	21.97	28.5
C113.28	113.280	RCP	7	1.2	-	17.49	55.1
C114.27	114.270	RCP	11	0.9	-	12.79	6.0
C114.36	114.360	RCBC	9	1.8	1.5	9.35	212.2
C114.90	114.899	RCBC	3	1.2	1.2	15.26	14.0
C115.00	115.003	RCP	3	0.9	-	21.02	7.2
C115.33	115.329	RCP	3	0.9	-	33.15	6.5
310-BR28	115.530	BRIDGE		94	459.2		61.6
C117.39	117.385	RCBC	13	3	1.5	16.32	159.1
C117.59	117.585	RCBC	6	1.8	1.8	11.95	83.0
C117.69	117.693	RCBC	1	1.2	1.2	10.52	3.2
C118.09	118.085	RCBC	10	2.1	1.2	9.06	47.7
C118.42	118.415	RCBC	6	3	1.5	8.81	164.4
C118.59	118.590	RCBC	17	1.2	1.2	9.56	3.5
C118.89	118.890	RCP	10	0.9	-	10.10	14.9
C119.02	119.023	RCBC	1	2.4	1.5	10.49	16.0
C119.29	119.285	RCBC	15	1.2	1.2	8.41	5.8
C119.37	119.365	RCBC	7	3	1.5	11.57	142.7
C119.74	119.740	RCBC	2	2.4	1.2	9.34	17.5
C119.86	119.860	RCBC	8	1.5	1.2	8.09	18.5
C120.07	120.065	RCBC	3	1.5	0.9	10.98	2.2
C120.24	120.240	RCBC	29	2.4	1.5	10.00	661.6

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – culverts Span (m) - bridges	Height (m) – culverts Soffit level (m AHD) - bridges	Length (m)	Catchment area (ha)
C120.75	120.750	RCBC	11	2.4	1.5	11.54	196.3
C124.44	124.435	RCBC	7	1.5	1.5	28.30	37.9
C125.47	125.470	RCBC	6	2.4	2.1	43.12	202.6
C125.82	125.820	RCP	1	1.8	-	17.98	11.2
C128.88	128.880	RCP	23	1.35	-	16.70	119.2
C129.63	129.625	RCP	5	1.2	-	12.00	30.9
C131.39	131.385	RCP	18	2.1	-	30.96	307.1
C133.53	133.530	RCBC	4	1.8	1.2	8.46	82.0
C133.90	133.900	RCBC	1	1.5	1.2	9.10	9.9
C134.37	134.370	RCBC	3	1.5	1.2	8.93	48.1
C135.28	135.275	RCBC	4	1.2	0.9	9.28	35.6
C135.82	135.815	RCBC	4	1.2	0.9	9.27	41.3
C151.11	151.108	RCBC	3	0.9	0.9	7.30	6.1
C152.15	152.150	RCBC	2	0.9	0.9	8.59	11.5
C153.22	153.222	RCBC	7	2.1	0.9	8.22	211.3
C154.31	154.305	RCP	11	1.2	-	12.66	306.4
C157.96	157.960	RCBC	3	2.4	1.2	7.45	59.0
C159.13	159.130	RCP	5	1.8	-	10.35	189.6
C159.87	159.865	RCP	5	1.5	-	24.81	32.9
C161.02	161.015	RCP	2	1.5	-	23.32	9.5
310-BR29	161.255	BRIDGE		90	431.3		47.0
C161.53	161.530	RCP	3	1.8	-	33.81	37.9
C163.01	163.010	RCP	5	1.8	-	71.59	32.8
C163.09	163.085	RCP	11	2.1	-	71.27	142.5
C163.79	163.785	RCP	3	1.8	-	61.23	36.7
C164.83	164.825	RCBC	2	2.4	1.2	8.46	20.1

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – culverts Span (m) - bridges	Height (m) – culverts Soffit level (m AHD) - bridges	Length (m)	Catchment area (ha)
C165.81	165.805	RCBC	2	1.2	1.2	12.21	18.1
C167.32	167.322	RCBC	4	2.4	0.9	12.05	23.1
C167.74	167.735	RCBC	2	1.2	1.2	10.67	8.9
C168.59	168.585	RCP	3	2.4	-	49.47	104.1
C169.27	169.265	RCBC	5	2.4	2.4	22.59	161.4
C169.74	169.735	RCBC	4	1.8	1.5	27.04	41.5
C170.62	170.620	RCP	9	2.4	-	70.70	138.3
310-BR30	170.945	BRIDGE		141	514.2		39.4
C172.27	172.270	RCP	3	1.35	-	73.04	6.5
310-BR40	172.440	BRIDGE		95	530.7		48.6
C175.61	175.605	RCP	4	1.8	-	32.13	54.2
C176.36	176.355	RCP	1	2.1	-	34.00	27.6
C176.74	176.735	RCP	5	2.1	-	42.67	58.7
C177.35	177.350	RCP	7	1.8	-	14.89	82.3
C179.93	179.925	RCP	9	2.1	-	66.79	124.3
C180.50	180.500	RCP	4	1.8	-	48.01	42.9
C181.71	181.705	RCBC	6	2.4	2.1	31.17	167.7
C182.28	182.275	RCP	6	2.1	-	28.39	48.0
310-BR42	183.630	BRIDGE		89	531.6		524.6
C184.87	184.872	RCBC	10	2.4	1.8	14.08	169.2
C185.91	185.910	RCP	14	1.8	-	54.99	214.0
C186.88	186.875	RCBC	3	1.5	0.9	8.53	14.5
C187.00	186.995	RCP	3	1.2	-	10.19	16.1
C190.81	190.807	RCP	5	1.5	-	43.51	62.7
C194.66	194.657	RCP	3	1.2	-	10.79	27.3
C195.19	195.185	RCP	3	1.5	-	70.61	26.7

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – culverts Span (m) - bridges	Height (m) – culverts Soffit level (m AHD) - bridges	Length (m)	Catchment area (ha)
C199.55	199.547	RCBC	6	0.9	0.9	8.63	18.0
C199.96	199.955	RCBC	3	0.9	0.9	9.56	7.9
C200.24	200.235	RCBC	3	0.9	0.9	8.23	4.7
C200.70	200.695	RCP	2	1.8	-	24.77	23.2
C201.25	201.246	RCP	5	1.5	-	26.62	32.1
C201.52	201.524	RCBC	4	1.8	1.5	33.52	69.4
C206.43	206.427	RCBC	1	1.8	0.9	11.46	7.6

APPENDIX Q1

Hydrology and Flooding Technical Report—Volume I

Appendix C 1% AEP Hydraulic Results At B2G Floodplain Structures

including Macintyre River north of the QLD/NSW border

INLAND RAIL—BORDER TO GOWRIE ENVIRONMENTAL IMPACT STATEMENT

IR_1534

Appendix C

1% AEP hydraulic results at B2G floodplain structures, including Macintyre River north of the QLD/NSW border

Chainage (km)	Waterway	Structure type	Rail formation level (m AHD)	U/S peak water level (m AHD)	Freeboard to formation level (m)	Outlet velocity (m/s)	Peak flow (m^3/s)
203.05	Gowrie Creek	Road Bridge	483.15	473.87	9.3	0.7	2.6
203.17		RCP	482.86	471.99	10.9	1.4	1.0
204.55		Road Bridge	468.24	449.80	18.4	1.4	11.6
204.89		RCP	462.46	447.40	15.1	1.3	0.7
205.08		RCP	460.14	447.19	13.0	1.6	8.0
205.15		RCP	459.38	447.27	12.1	1.2	0.7
205.30		RCP	457.53	447.72	9.8	1.5	2.4
205.37		RCP	456.67	447.93	8.7	1.6	9.6
205.47		RCP	455.56	448.55	7.0	1.2	1.5
205.60		RCP	453.98	449.33	4.7	1.8	1.9
205.87		RCP	450.97	450.36	0.6	2.0	8.6
206.95		RCBC	459.72	458.36	1.4	1.4	49.9
195.72	Westbrook and Dry Creeks	Bridge	444.64/442.64	433.32	9.3	2.8	57.0
196.85		Bridge	434.09/432.09	426.75	5.3	2.6	591.1
197.55		Bridge	430.89/428.89	425.80	3.1	3.1	198.7
198.33		Bridge	438.86/436.86	430.01	6.9	1.4	14.8
188.32		RCBC	518.49	512.02	6.5	3.1	43.5
191.43		RCP	487.15	466.14	21.0	2.6	54.8
192.98		RCBC	471.61	470.25	1.4	2.3	2.4
193.01		RCP	471.27	470.25	1.0	2.8	4.8
195.31		RCP	448.96	433.58	15.4	2.1	12.1
195.47		RCP	446.34	433.52	12.8	1.4	1.1
195.63		RCP	445.03	433.41	11.6	1.7	0.9
197.01		RCP	431.29	426.59	4.7	2.1	105.6
197.09		RCP	430.50	426.47	4.0	2.2	27.0
197.13		RCP	430.08	426.47	3.6	2.0	12.1

Chainage (km)	Waterway	Structure type	Rail formation level (m AHD)	U/S peak water level (m AHD)	Freeboard to formation level (m)	Outlet velocity (m/s)	Peak flow (m^3/s)
197.31		RCP	428.71	426.41	2.3	2.1	17.9
197.86		RCP	433.84	427.08	6.8	1.6	11.7
131.39	Condamine River	RCP	406.04	401.79	4.25	1.57	1.3
131.49		RCP	406.04	401.79	4.25	2.10	3.3
137.83		RCP	382.57	380.65	1.92	1.31	5.6
137.88		RCP	382.38	380.62	1.76	0.79	10.3
137.92		RCP	382.24	380.64	1.61	1.01	11.8
138.18		Bridge	382.05	380.57	1.48	0.69	204.4
138.88		Bridge	382.05	380.60	1.45	1.50	965.3
139.37		RCP	382.24	380.63	1.61	1.06	12.8
139.44		RCP	382.24	380.64	1.60	0.46	0.1
139.5		RCP	382.24	380.64	1.60	0.78	13.3
139.56		RCP	382.24	380.63	1.61	0.84	14.6
139.71		RCP	382.24	380.65	1.59	0.99	13.5
139.73		RCBC	382.24	380.65	1.59	1.04	14.2
139.78		RCP	382.24	380.63	1.61	1.59	14.1
140.09		RCP	382.24	380.62	1.62	1.73	11.3
140.11		RCP	382.24	380.62	1.62	1.74	11.6
140.17		RCP	382.24	380.57	1.67	0.93	8.4
140.21		RCP	382.24	380.57	1.67	1.19	9.1
140.23		RCP	382.24	380.56	1.68	1.14	9.1
140.25		RCP	382.24	380.56	1.68	1.12	8.9
140.27		RCP	382.24	380.56	1.68	1.47	9.7
140.32		RCP	382.24	380.56	1.67	1.24	9.6
140.38		RCP	382.24	380.56	1.68	1.02	9.3
140.4		RCP	382.24	380.56	1.68	1.00	10.2

Chainage (km)	Waterway	Structure type	Rail formation level (m AHD)	U/S peak water level (m AHD)	Freeboard to formation level (m)	Outlet velocity (m/s)	Peak flow (m^3/s)
140.43		RCP	382.24	380.56	1.68	0.85	10.9
140.46		RCP	382.24	380.55	1.69	0.82	9.2
140.49		RCP	382.24	380.54	1.70	0.76	10.2
140.51		RCP	382.24	380.54	1.70	0.83	11.3
140.55		RCP	382.24	380.55	1.69	1.11	10.8
140.59		RCP	382.34	380.55	1.79	1.16	11.1
140.64		RCP	382.33	380.54	1.79	1.47	14.7
140.67		RCP	382.33	380.55	1.78	1.58	13.4
140.78		RCP	382.33	380.52	1.81	1.83	16.2
140.83		RCP	382.33	380.51	1.82	1.84	16.6
140.87		RCP	382.33	380.51	1.82	1.91	12.0
140.91		RCP	382.33	380.51	1.82	1.88	17.6
140.98		RCP	382.34	380.49	1.85	1.80	16.9
141.03		RCP	382.33	380.50	1.83	1.81	12.1
141.07		RCP	382.24	380.49	1.75	1.84	17.8
141.11		RCP	382.24	380.48	1.76	1.86	17.5
141.2		RCP	382.24	380.47	1.77	1.70	14.9
141.24		RCP	382.24	380.46	1.78	1.78	15.3
141.29		RCP	382.24	380.45	1.79	1.56	13.3
141.32		RCP	382.24	380.43	1.81	1.73	9.0
141.67		Bridge	382.06	380.37	1.69	0.77	276.0
142.02		RCP	382.24	380.39	1.85	1.25	7.8
142.04		RCP	382.24	380.40	1.84	1.40	10.3
142.08		RCP	382.24	380.39	1.84	1.45	11.1
142.13		RCP	382.24	380.39	1.84	1.37	10.2
142.15		RCP	382.24	380.39	1.85	1.30	9.8

Chainage (km)	Waterway	Structure type	Rail formation level (m AHD)	U/S peak water level (m AHD)	Freeboard to formation level (m)	Outlet velocity (m/s)	Peak flow (m^3/s)
142.19		RCP	382.24	380.39	1.85	1.38	9.9
142.22		RCP	382.24	380.39	1.85	1.26	9.2
142.25		RCP	382.24	380.40	1.84	1.34	8.9
142.28		RCP	382.24	380.41	1.83	1.58	9.3
142.36		RCP	382.24	380.40	1.84	1.48	10.4
142.41		RCP	382.24	380.41	1.83	1.36	9.4
142.44		RCP	382.24	380.41	1.83	1.38	9.5
142.48		RCP	382.24	380.41	1.83	1.32	9.0
142.5		RCP	382.24	380.42	1.82	1.39	8.2
142.54		RCP	382.24	380.43	1.81	1.49	7.4
142.58		RCP	382.24	380.41	1.83	1.38	7.0
143.58		Bridge	382.06	380.66	1.40	2.21	1,322.4
144.88		Bridge	382.06	380.74	1.32	0.59	152.4
145.16		RCBC	382.24	380.87	1.37	1.90	2.5
145.21		RCBC	382.24	380.98	1.26	1.58	2.3
145.25		RCBC	382.24	381.07	1.17	1.68	2.6
145.32		RCBC	382.24	381.10	1.14	1.81	1.0
145.4		RCBC	382.26	381.14	1.12	1.01	1.1
145.72		RCBC	382.26	381.32	0.94	1.59	4.2
145.83		RCBC	382.35	381.39	0.96	0.83	0.3
145.89		RCBC	382.36	381.38	0.98	1.79	6.0
145.92		RCBC	382.37	381.39	0.98	1.09	1.0
145.98		RCBC	382.38	381.41	0.97	1.03	0.8
146.03		RCBC	382.39	381.45	0.94	1.39	5.5
146.56		RCBC	382.50	381.80	0.70	0.70	0.2
146.62		RCBC	382.51	381.82	0.69	1.01	0.3

Chainage (km)	Waterway	Structure type	Rail formation level (m AHD)	U/S peak water level (m AHD)	Freeboard to formation level (m)	Outlet velocity (m/s)	Peak flow (m^3/s)
147.58		RCP	383.37	382.47	0.90	0.32	2.3
147.63		RCP	383.55	382.51	1.04	0.49	5.7
147.66		RCP	383.68	382.45	1.23	0.72	5.0
147.73		RCP	383.92	382.40	1.52	0.89	5.1
148.70		Bridge	383.79	382.64	1.15	0.67	150.5
149.39		RCP	383.97	382.65	1.32	0.57	2.6
149.42		RCP	383.97	382.66	1.31	0.55	2.7
149.45		RCP	383.97	382.67	1.30	0.83	1.2
149.76		RCP	383.97	382.71	1.26	0.59	2.2
149.8		RCP	383.97	382.71	1.26	0.63	2.0
149.83		RCP	383.97	382.71	1.26	0.62	1.6
149.87		RCP	383.97	382.71	1.26	0.19	1.0
149.91		RCP	383.97	382.71	1.26	0.58	1.0
149.96		RCP	383.98	382.70	1.28	0.34	1.3
150.01		RCP	384.19	382.70	1.48	0.64	1.2
126.76	Back Creek	RCP	421.0	412.5	8.5	0.6	0.2
126.80		RCP	421.0	412.5	8.5	1.1	3.9
126.97		Bridge	421.0	411.9	7.1	3.0	71.5
128.06		Bridge	409.8	407.0	0.8	2.2	337.0
104.39	Nicol Creek	Bridge	356.4	353.1	3.3	2.6	160.0
104.94		RCP	356.5	354.8	1.7	1.0	1.5
105.09		RCP	357.7	354.8	2.9	1.3	1.8
105.11		RCP	357.9	354.8	3.0	1.3	2.0
105.13		RCP	357.9	354.8	3.1	1.2	1.4

Chainage (km)	Waterway	Structure type	Rail formation level (m AHD)	U/S peak water level (m AHD)	Freeboard to formation level (m)	Outlet velocity (m/s)	Peak flow (m^3/s)
100.00	Bringalily Creek	RCP	335.7	333.1	2.6	1.3	7.1
99.84		RCP	335.5	332.9	2.6	1.5	7.6
99.38		RCP	335.1	331.7	3.5	1.2	4.9
97.29		RCP	333.3	328.3	5.0	1.5	6.0
98.87		RCP	334.7	-1	-1	-1	-1
99.77		RCP	335.5	332.9	2.6	2.1	1.8
98.36		RCP	334.2	329.8	4.4	1.7	7.7
97.38		RCP	334.0	328.3	5.7	1.6	7.6
96.20		RCBC	327.1	325.7	1.4	<0.1	0.3
94.91		RCBC	325.0	-1	-1	-1	-1
95.07		RCBC	324.6	323.5	1.1	0.8	32.1
97.58		Bridge	334.1	328.5	5.6	1.9	56.4
100.39		Bridge	335.8	333.8	0.06	2.4	568.3
93.90	Native Dog Creek	Bridge	327.4	321.5	5.9	1.2	168.0
88.28	Cattle Creek	Bridge	331.1	324.1	5.0	2.2	239.7
87.37		RCP	329.2	321.7	7.5	1.1	22.2
87.19		RCBC	328.7	323.2	5.5	0.7	0.4
68.75	Pariagara Creek	RCBC	288.6	287.0	1.6	0.6	46.0
66.23		RCBC	290.7	285.5	5.2	3.2	43.1
69.80		RCP	289.2	287.4	1.8	1.4	13.1
69.67		RCP	289.1	287.4	1.8	1.4	13.3
69.54		RCP	289.1	287.3	1.8	1.4	12.8
69.41		RCP	289.0	287.3	1.7	1.5	13.0
69.28		RCP	288.9	287.3	1.6	1.6	4.9
69.21		RCP	288.9	287.3	1.6	1.6	4.8
69.14		RCP	288.9	287.3	1.6	1.3	3.3

Chainage (km)	Waterway	Structure type	Rail formation level (m AHD)	U/S peak water level (m AHD)	Freeboard to formation level (m)	Outlet velocity (m/s)	Peak flow (m^3/s)
69.10		RCP	288.8	287.3	1.5	1.8	2.8
69.02		RCP	288.8	287.2	1.6	1.7	1.9
68.89		RCP	288.7	287.1	1.6	1.3	1.0
67.57		RCP	287.9	285.3	2.6	0.2	0.1
67.64		RCP	288.0	285.4	2.6	0.2	0.9
67.70		RCP	288.0	285.5	2.6	0.7	3.0
67.83		RCP	288.1	285.5	2.5	0.6	5.6
67.96		RCP	288.2	285.7	2.5	1.0	8.7
68.09		RCP	288.3	285.8	2.5	1.1	10.2
68.41		RCP	288.4	286.4	2.1	1.4	6.8
67.35		Bridge	287.7	285.3	0.4	2.3	530.0
25.15	Macintyre Brook Yelarbon to Inglewood	RCBC	245.61	244.3	1.3	1.98	2.4
25.19		RCBC	245.61	244.3	1.3	2.51	2.4
25.46		RCP	245.61	244.4	1.2	1.13	6.2
25.50		RCP	245.61	244.5	1.1	1.13	5.4
25.80		RCBC	245.64	244.6	1.1	1.03	9.0
25.87		RCBC	245.64	244.5	1.1	0.83	9.0
25.95		RCBC	245.66	244.7	1.0	1.80	1.8
25.97		RCBC	245.66	244.7	1.0	1.84	1.9
27.05		RCBC	246.53	245.8	0.8	1.09	12.8
27.15		RCBC	246.70	245.9	0.8	1.19	14.5
27.24		RCBC	246.78	246.1	0.7	2.21	27.9
27.33		RCBC	246.87	246.1	0.8	2.03	21.5
27.42		RCBC	246.95	246.0	0.9	1.36	13.2
27.53		RCBC	247.05	245.9	1.2	0.85	3.4
42.87		RCP	265.08	264.0	1.1	1.01	6.9

Chainage (km)	Waterway	Structure type	Rail formation level (m AHD)	U/S peak water level (m AHD)	Freeboard to formation level (m)	Outlet velocity (m/s)	Peak flow (m^3/s)
43.02		RCP	265.12	264.0	1.1	0.90	11.5
43.08		RCP	265.14	264.0	1.2	0.80	20.3
43.16		RCBC	265.17	264.0	1.2	0.72	13.4
43.34		RCP	265.21	264.0	1.2	0.51	19.5
43.56		RCP	265.28	264.1	1.2	0.60	3.6
43.66		RCP	265.31	264.2	1.2	0.25	3.2
43.77		RCP	265.34	264.2	1.1	0.39	4.9
43.86		RCP	265.37	264.3	1.1	0.54	6.8
43.97		RCP	265.40	264.4	1.0	0.77	9.7
44.32		RCP	265.50	264.5	1.0	0.80	6.0
44.67		RCP	265.50	264.7	0.8	1.13	11.1
44.88		RCP	265.78	264.9	0.9	1.12	15.9
44.99		RCP	265.94	265.0	0.9	1.22	19.6
45.24		RCP	266.36	265.2	1.2	1.29	10.4
45.30		RCP	266.45	265.2	1.2	1.31	12.3
45.39		RCP	266.68	265.3	1.4	1.36	17.0
45.46		RCP	266.68	265.4	1.2	0.87	13.9
45.53		RCP	266.80	265.5	1.3	1.11	8.4
45.67		RCP	267.25	266.0	1.3	1.26	2.3
55.55	Bybera Road	Bridge	285.9	273.6	12.4	1.5	263.0
52.58	Cremascos Road	Bridge	281.7	270.6	10.8	2.0	256.0
30.35 (NS2B)	Macintyre River (north of QLD/NSW border only)	Bridge	230.52	227.96	2.6	3.1	3,875.0
31.26 (NS2B)		RCP	232.52	227.54	45.0	1.4	24.0
31.32 (NS2B)		RCP	232.04	227.47	4.6	1.4	25.0
31.52 (NS2B)		Bridge	232.41	227.43	4.0	1.0	69.0
31.87 (NS2B)		RCP	229.05	227.14	1.9	1.1	4.0

Chainage (km)	Waterway	Structure type	Rail formation level (m AHD)	U/S peak water level (m AHD)	Freeboard to formation level (m)	Outlet velocity (m/s)	Peak flow (m^3/s)
31.97 (NS2B)		RCP	229.05	227.14	1.9	1.0	3.0
32.55 (NS2B)		Bridge	229.04	227.14	2.9	1.8	274.0

Table note:

1 New local drainage culverts included in flood model. These culverts are necessary for minor drainage paths

APPENDIX Q1

Hydrology and Flooding Technical Report—Volume I

Appendix D Proposed Outlet Scour Protection Works—All Cross-Drainage Structures

INLAND RAIL—BORDER TO GOWRIE ENVIRONMENTAL IMPACT STATEMENT

IR_1534

Appendix D

Proposed outlet scour protection works - all cross-drainage structures

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – Culverts Span (m) - bridges	Height (m) – Culverts Soffit Level (m AHD) - bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream scour protection			
							Material	Scour protection required (Y)	Length	Rock d50
C6.60	6.600	RCBC	3	1.5	1.2	c	Vertosol	Y	7.2	200
C8.39	8.393	RCBC	8	1.2	1.2	1.98	Sodosol	N		
C13.00	13.000	RCBC	13	2.4	1.2	1.87	Sodosol	N		
C17.89	17.885	RCP	2	1.05		2.61	Sodosol	Y	6.3	200
C18.51	18.506	RCP	2	1.2		2.59	Sodosol	Y	9.6	300
C18.87	18.870	RCP	4	1.2		2.71	Sodosol	Y	9.6	300
C20.00	19.995	RCBC	6	1.2	1.2	1.83	Sodosol	N		
C22.42	22.424	RCBC	8	1.2	1.2	1.80	Sodosol	N		
C23.05	23.046	RCBC	6	1.2	1.2	1.71	Sodosol	N		
C23.53	23.530	RCBC	7	1.2	1.2	1.71	Sodosol	N		
C24.41	24.410	RCBC	3	1.2	1.2	1.80	Sodosol	N		
C24.84	24.839	RCBC	1	1.2	1.2	2.97	Sodosol	Y	9.6	300
C25.15	25.153	RCBC	1	3	0.6	1.98	Sodosol	N		
C25.19	25.185	RCBC	1	3	0.6	2.51	Sodosol	Y	3.6	200
C25.46	25.459	RCP	21	0.9		1.13	Sodosol	N		
C25.50	25.503	RCP	21	0.9		1.13	Sodosol	N		
C25.80	25.802	RCBC	24	2.4	0.9	1.03	Sodosol	N		
C25.87	25.874	RCBC	24	2.4	0.9	0.83	Sodosol	N		
C25.95	25.945	RCBC	1	3	0.5	1.80	Sodosol	N		
C25.97	25.965	RCBC	1	3	0.5	1.84	Sodosol	N		
C27.05	27.050	RCBC	15	1.5	1.2	1.09	Sodosol	N		
C27.15	27.146	RCBC	15	1.5	1.2	1.19	Sodosol	N		
C27.24	27.239	RCBC	25	1.5	1.2	2.21	Sodosol	Y	7.2	200
C27.33	27.331	RCBC	25	1.5	1.2	2.03	Sodosol	Y	7.2	200
C27.42	27.421	RCBC	20	1.5	1.2	1.36	Sodosol	N		

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – Culverts Span (m) - bridges	Height (m) – Culverts Soffit Level (m AHD) - bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream scour protection			
							Material	Scour protection required (Y)	Length	Rock d50
C27.53	27.527	RCBC	20	1.5	1.2	0.85	Sodosol	N		
C30.60	30.595	RCBC	7	3	1.2	2.76	Sodosol	Y	9.6	300
C31.40	31.397	RCP	3	1.2		1.80	Sodosol	N		
C31.44	31.438	RCP	5	1.35		2.43	Sodosol	Y	10.8	300
C32.03	32.030	RCP	10	0.9		2.57	Sodosol	Y	5.4	200
C32.80	32.800	RCP	4	0.9		2.34	Sodosol	Y	5.4	200
C33.66	33.664	RCP	9	1.5		1.86	Sodosol	N		
C35.18	35.175	RCP	12	2.1		2.25	Sodosol	Y	16.8	300
C41.20	41.195	RCBC	18	3	1.5	1.62	Kurosol	Y	9.0	200
C42.88	42.876	RCP	15	0.9		1.01	Kurosol	N		
C43.02	43.015	RCP	15	1.2		0.90	Kurosol	N		
C43.08	43.078	RCP	30	1.2		0.80	Kurosol	N		
C43.16	43.159	RCBC	9	3	1.5	0.72	Kurosol	N		
C43.34	43.341	RCP	45	1.2		0.51	Kurosol	N		
C43.56	43.555	RCP	10	1.2		0.60	Kurosol	N		
C43.66	43.661	RCP	15	1.2		0.25	Kurosol	N		
C43.77	43.767	RCP	15	1.2		0.39	Kurosol	N		
C43.86	43.864	RCP	15	1.2		0.54	Kurosol	N		
C43.97	43.970	RCP	15	1.2		0.77	Kurosol	N		
C44.32	44.324	RCP	15	1.2		0.80	Kurosol	N		
C44.67	44.672	RCP	15	1.2		1.13	Kurosol	N		
C44.88	44.877	RCP	30	0.9		1.12	Kurosol	N		
C44.99	44.993	RCP	35	0.9		1.22	Kurosol	N		
C45.24	45.236	RCP	35	0.9		1.29	Kurosol	N		
C45.30	45.300	RCP	35	0.9		1.31	Kurosol	N		

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – Culverts Span (m) - bridges	Height (m) – Culverts Soffit Level (m AHD) - bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream scour protection			
							Material	Scour protection required (Y)	Length	Rock d50
C45.39	45.392	RCP	40	0.9		1.36	Kurosol	N		
C45.46	45.455	RCP	40	0.9		0.87	Kurosol	N		
C45.53	45.530	RCP	40	0.9		1.11	Kurosol	N		
C45.67	45.668	RCP	7	0.9		1.26	Kurosol	N		
C46.46	46.464	RCP	10	1.2		2.21	Kurosol	Y	7.2	200
C48.41	48.406	RCP	21	2.4		3.47	Rudosol	Y	24.0	500
C49.83	49.825	RCBC	3	3	1.5	2.32	Rudosol	Y	12.0	300
C49.97	49.972	RCBC	8	2.1	0.9	1.83	Rudosol	Y	5.4	200
C51.50	51.495	RCP	7	1.35		2.97	Rudosol	Y	10.8	300
310-BR03	52.580	BRIDGE		208	282.4	1.65	Rudosol	N/A		
C53.20	53.201	RCP	7	1.2		1.67	Kurosol	Y	7.2	200
C53.62	53.618	RCBC	2	1.2	0.9	2.47	Kurosol	Y	5.4	200
C54.44	54.439	RCBC	2	0.9	0.9	2.25	Kurosol	Y	5.4	200
C55.06	55.056	RCP	7	1.05		2.49	Rudosol	Y	6.3	200
310-BR04	55.550	BRIDGE		207	287.2	1.49	Rudosol	N/A		
C60.18	60.175	RCP	1	0.9		2.35	Rudosol	Y	5.4	200
C60.49	60.490	RCP	11	2.4		2.60	Rudosol	Y	19.2	400
C61.60	61.600	RCP	5	2.4		3.23	Rudosol	Y	24.0	500
C61.90	61.900	RCP	5	2.4		3.47	Rudosol	Y	24.0	500
C62.52	62.524	RCBC	2	1.2	0.9	2.68	Rudosol	Y	5.4	200
C62.94	62.940	RCP	2	2.4		3.03	Rudosol	Y	24.0	500
C63.15	63.150	RCP	5	2.4		3.23	Rudosol	Y	24.0	500
C64.50	64.495	RCP	5	0.9		2.49	Rudosol	Y	5.4	200
C65.11	65.114	RCBC	13	2.1	1.2	1.32	Rudosol	N		
C66.23	66.234	RCBC	8	2.4	1.8	3.19	Sodosol	Y	14.4	400

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – Culverts Span (m) - bridges	Height (m) – Culverts Soffit Level (m AHD) - bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream scour protection			
							Material	Scour protection required (Y)	Length	Rock d50
C66.81	66.813	RCP	16	1.2		1.60	Rudosol	Y	7.2	200
310-BR05	67.320	BRIDGE		344	287.8	2.30	Rudosol	N/A		
C67.57	67.566	RCP	8	1.2		0.18	Rudosol	N		
C67.64	67.635	RCP	8	1.2		0.24	Rudosol	N		
C67.70	67.704	RCP	8	1.2		0.69	Rudosol	N		
C67.83	67.834	RCP	20	1.2		0.65	Rudosol	N		
C67.96	67.958	RCP	20	1.2		0.91	Rudosol	N		
C68.09	68.088	RCP	20	1.2		0.96	Rudosol	N		
C68.41	68.410	RCP	20	1.2		1.39	Rudosol	N		
C68.75	68.748	RCBC	40	2.1	2.1	0.96	Rudosol	N		
C68.89	68.889	RCP	2	1.2		1.07	Rudosol	N		
C69.02	69.020	RCP	2	1.2		1.51	Rudosol	Y	7.2	200
C69.10	69.097	RCP	2	1.2		1.60	Rudosol	Y	7.2	200
C69.14	69.142	RCP	2	1.5		1.54	Rudosol	Y	9.0	200
C69.21	69.214	RCP	2	1.8		1.57	Rudosol	Y	10.8	200
C69.28	69.280	RCP	2	1.8		1.50	Rudosol	Y	10.8	200
C69.41	69.410	RCP	5	1.8		1.37	Rudosol	N		
C69.54	69.540	RCP	5	1.8		1.40	Rudosol	N		
C69.67	69.670	RCP	5	1.8		1.45	Rudosol	N		
C69.80	69.800	RCP	5	1.8		1.39	Rudosol	N		
C70.50	70.500	RCP	2	1.2		2.74	Rudosol	Y	9.6	300
C71.51	71.510	RCBC	3	2.4	0.9	1.53	Rudosol	Y	5.4	200
C73.33	73.330	RCP	2	1.2		2.86	Sodosol	Y	9.6	300
C73.43	73.430	RCP	1	1.2		1.87	Sodosol	N		
C73.52	73.520	RCP	1	1.35		3.40	Sodosol	Y	8.1	200

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – Culverts Span (m) - bridges	Height (m) – Culverts Soffit Level (m AHD) - bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream scour protection			
							Material	Scour protection required (Y)	Length	Rock d50
C73.61	73.605	RCP	1	0.9		1.62	Sodosol	N		
C73.71	73.705	RCBC	3	3	1.5	3.20	Sodosol	Y	8.1	200
C74.97	74.970	RCP	3	1.2		2.75	Sodosol	Y	9.6	300
C76.57	76.570	RCBC	16	1.2	0.9	1.60	Sodosol	N		
C77.20	77.195	RCP	6	1.5		2.28	Sodosol	Y	9.0	200
C77.47	77.465	RCP	4	0.9		2.33	Sodosol	Y	5.4	200
C77.77	77.770	RCBC	8	1.2	1.2	1.92	Sodosol	N		
C78.28	78.280	RCBC	4	2.1	0.9	1.36	Sodosol	N		
C79.02	79.015	RCBC	5	2.4	1.5	2.51	Sodosol	Y	12.0	300
C79.53	79.525	RCP	7	1.35		1.78	Sodosol	N		
C79.98	79.980	RCBC	7	0.9	0.9	1.76	Sodosol	N		
C80.65	80.645	RCBC	6	1.8	1.5	1.70	Sodosol	N		
C81.19	81.185	RCBC	7	2.1	2.1	1.74	Sodosol	N		
C82.35	82.350	RCBC	18	2.1	2.1	1.98	Sodosol	N		
C83.51	83.505	RCBC	8	2.4	1.5	1.66	Sodosol	N		
C84.38	84.380	RCBC	35	2.4	2.4	1.88	Sodosol	N		
C87.19	87.185	RCP	6	2.1		0.73	Sodosol	N		
C87.37	87.365	RCBC	15	2.4	1.8	1.14	Sodosol	N		
C87.54	87.540	RCBC	3	1.8	1.8	1.75	Sodosol	N		
C88.11	88.110	RCP	2	1.2		1.75	Sodosol	N		
310-BR06	88.280	BRIDGE		159	329.9	2.21	Sodosol	N/A		
C90.96	90.960	RCBC	31	2.4	2.4	2.11	Sodosol	Y	19.2	300
C92.08	92.080	RCBC	16	2.4	1.2	2.42	Sodosol	Y	9.6	300
C92.94	92.940	RCP	17	1.5		2.01	Sodosol	Y	9.0	200
C93.61	93.610	RCBC	8	1.8	1.8	1.45	Sodosol	N		

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – Culverts Span (m) - bridges	Height (m) – Culverts Soffit Level (m AHD) - bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream scour protection			
							Material	Scour protection required (Y)	Length	Rock d50
310-BR07	93.930	BRIDGE		203	326.1	1.24	Sodosol	N/A		
C94.91	94.910	RCBC	5	2.1	0.9	1.73	Sodosol	N		
C95.07	95.065	RCBC	15	2.4	1.5	1.86	Sodosol	N		
C96.20	96.195	RCBC	8	2.4	1.2	2.23	Sodosol	Y	7.2	200
C97.29	97.290	RCP	2	0.9		1.50	Sodosol	N		
C97.38	97.380	RCP	2	0.9		1.59	Sodosol	N		
310-BR08	97.590	BRIDGE		320	332.9	1.93	Sodosol	N/A		
C97.83	97.825	RCP	10	2.1		0.88	Sodosol	N		
C98.36	98.360	RCP	10	0.9		1.74	Sodosol	N		
C98.87	98.865	RCP	1	1.5		3.18	Sodosol	Y	12.0	400
C99.38	99.375	RCP	17	0.9		1.22	Sodosol	N		
C99.77	99.765	RCP	1	1.5		2.14	Sodosol	Y	12.0	300
C99.84	99.840	RCP	14	0.9		1.48	Sodosol	N		
C100.00	100.000	RCP	8	1.5		1.26	Sodosol	N		
310-BR10	100.400	BRIDGE		636	334.6	2.16	Sodosol	N/A		
C101.49	101.485	RCBC	2	1.5	1.2	1.40	Vertosol	N		
C102.55	102.545	RCBC	2	1.5	0.9	2.24	Vertosol	Y	5.4	200
310-BR11	104.400	BRIDGE		110	355.2	2.61	Sodosol	N/A		
C104.94	104.939	RCP	18	0.9		1.10	Sodosol	N		
C105.09	105.094	RCP	6	0.9		1.25	Sodosol	N		
C105.11	105.110	RCP	6	0.9		1.29	Sodosol	N		
C105.13	105.126	RCP	6	0.9		1.16	Sodosol	N		
C106.54	106.543	RCP	5	1.2		2.44	Sodosol	Y	9.6	300
C107.22	107.222	RCBC	4	2.4	1.5	3.49	Sodosol	Y	12.0	400
C107.81	107.808	RCBC	2	0.9	0.9	1.94	Vertosol	Y	5.4	200

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – Culverts Span (m) - bridges	Height (m) – Culverts Soffit Level (m AHD) - bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream scour protection			
							Material	Scour protection required (Y)	Length	Rock d50
C107.97	107.965	RCP	4	1.2		2.56	Vertosol	Y	9.6	300
C108.46	108.455	RCP	4	1.2		2.77	Vertosol	Y	9.6	300
C109.43	109.430	RCBC	12	1.5	1.5	1.88	Vertosol	Y	9.0	200
C110.91	110.913	RCBC	3	1.2	1.2	2.33	Vertosol	Y	9.6	300
C111.17	111.165	RCBC	3	1.2	1.2	1.26	Vertosol	N		
C111.26	111.260	RCBC	4	1.2	0.9	1.68	Vertosol	Y	5.4	200
C112.33	112.325	RCBC	3	1.2	1.2	2.58	Vertosol	Y	9.6	300
C113.00	113.000	RCBC	4	1.2	1.2	3.45	Vertosol	Y	9.6	400
C113.28	113.280	RCP	7	1.2		2.42	Vertosol	Y	9.6	300
C114.27	114.270	RCP	11	0.9		1.21	Vertosol	N		
C114.36	114.360	RCBC	9	1.8	1.5	3.22	Vertosol	Y	12.0	300
C114.90	114.899	RCBC	3	1.2	1.2	2.92	Vertosol	Y	9.6	300
C115.00	115.003	RCP	3	0.9		1.98	Vertosol	Y	5.4	200
C115.33	115.329	RCP	3	0.9		2.10	Vertosol	Y	5.4	200
310-BR28	115.530	BRIDGE		94	459.2	2.05	Vertosol	N/A		
C117.39	117.385	RCBC	13	3	1.5	1.56	Vertosol	Y	9	200
C117.59	117.585	RCBC	6	1.8	1.8	2.02	Vertosol	Y	14.4	300
C117.69	117.693	RCBC	1	1.2	1.2	2.11	Vertosol	Y	7.2	200
C118.09	118.085	RCBC	10	2.1	1.2	1.30	Vertosol	N		
C118.42	118.415	RCBC	6	3	1.5	2.16	Vertosol	Y	9	200
C118.59	118.590	RCBC	17	1.2	1.2	0.76	Vertosol	N		
C118.89	118.890	RCP	10	0.9		1.47	Vertosol	N		
C119.02	119.023	RCBC	1	2.4	1.5	2.53	Vertosol	Y	12	300
C119.29	119.285	RCBC	15	1.2	1.2	0.86	Vertosol	N		
C119.37	119.365	RCBC	7	3	1.5	1.89	Vertosol	Y	9	200

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – Culverts Span (m) - bridges	Height (m) – Culverts Soffit Level (m AHD) - bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream scour protection			
							Material	Scour protection required (Y)	Length	Rock d50
C119.74	119.740	RCBC	2	2.4	1.2	1.68	Vertosol	Y	7.2	200
C119.86	119.860	RCBC	8	1.5	1.2	1.23	Vertosol	N		
C120.07	120.065	RCBC	3	1.5	0.9	0.99	Vertosol	N		
C120.24	120.240	RCBC	29	2.4	1.5	1.73	Vertosol	Y	9	200
C120.75	120.750	RCBC	11	2.4	1.5	3.01	Vertosol	Y	12	300
C124.44	124.435	RCBC	7	1.5	1.5	1.81	Vertosol	Y	9	200
C125.47	125.470	RCBC	6	2.4	2.1	2.27	Vertosol	Y	16.8	300
C125.82	125.820	RCP	1	1.8		2.64	Vertosol	Y	14.4	300
C126.76	126.760	RCP	12	0.9		0.60	Sodosol	N		
C126.80	126.800	RCP	12	0.9		1.08	Sodosol	N		
310-BR37	126.970	BRIDGE		189	419.8	2.95	Sodosol	N/A		
310-BR38	128.060	BRIDGE		249	408.5	2.28	Sodosol	N/A		
C128.88	128.880	RCP	23	1.35		1.60	Sodosol	N		
C129.63	129.625	RCP	5	1.2		2.74	Sodosol	Y	9.6	300
C131.39	131.385	RCP	18	2.1		2.10	Sodosol	Y	16.8	300
C133.53	133.530	RCBC	4	1.8	1.2	2.44	Sodosol	Y	9.6	300
C133.90	133.900	RCBC	1	1.5	1.2	2.42	Sodosol	Y	7.2	200
C134.37	134.370	RCBC	3	1.5	1.2	2.73	Sodosol	Y	9.6	300
C135.28	135.275	RCBC	4	1.2	0.9	2.37	Sodosol	Y	5.4	200
C135.82	135.815	RCBC	4	1.2	0.9	2.43	Sodosol	Y	5.4	200
C137.83	137.830	RCP	8	1.35		1.31	Sodosol	N		
C137.88	137.880	RCP	11	1.65		0.79	Sodosol	N		
C137.92	137.920	RCP	9	1.8		1.01	Sodosol	N		
310-BR21	138.180	BRIDGE		360	382.1	0.69	Sodosol	N/A		
310-BR22	138.880	BRIDGE		977	382.1	1.50	Sodosol	N/A		

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – Culverts Span (m) - bridges	Height (m) – Culverts Soffit Level (m AHD) - bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream scour protection			
							Material	Scour protection required (Y)	Length	Rock d50
C139.37	139.370	RCP	11	1.8		1.06	Vertosol	N		
C139.44	139.440	RCP	8	2.1		0.45	Vertosol	N		
C139.50	139.495	RCP	8	2.1		0.78	Vertosol	N		
C139.56	139.555	RCP	11	1.8		0.84	Vertosol	N		
C139.71	139.710	RCP	9	1.65		0.99	Vertosol	N		
C139.73	139.733	RCBC	4	2.4	1.8	1.04	Vertosol	N		
C139.78	139.775	RCP	10	2.1		1.59	Vertosol	Y	16.8	300
C140.09	140.085	RCP	7	1.8		1.73	Vertosol	Y	10.8	200
C140.11	140.110	RCP	7	1.8		1.74	Vertosol	Y	10.8	200
C140.17	140.170	RCP	6	2.1		0.93	Vertosol	N		
C140.21	140.205	RCP	6	2.1		1.20	Vertosol	N		
C140.23	140.230	RCP	6	2.1		1.14	Vertosol	N		
C140.25	140.250	RCP	6	2.1		1.13	Vertosol	N		
C140.27	140.270	RCP	6	2.1		1.48	Vertosol	N		
C140.32	140.315	RCP	9	1.8		1.24	Vertosol	N		
C140.38	140.375	RCP	6	2.1		1.02	Vertosol	N		
C140.40	140.395	RCP	6	2.1		1.00	Vertosol	N		
C140.43	140.430	RCP	7	1.8		0.85	Vertosol	N		
C140.46	140.460	RCP	5	2.1		0.82	Vertosol	N		
C140.49	140.485	RCP	6	2.1		0.76	Vertosol	N		
C140.51	140.510	RCP	6	2.1		0.83	Vertosol	N		
C140.55	140.550	RCP	5	2.1		1.11	Vertosol	N		
C140.59	140.590	RCP	5	2.1		1.16	Vertosol	N		
C140.64	140.635	RCP	6	2.1		1.47	Vertosol	N		
C140.67	140.670	RCP	5	2.1		1.58	Vertosol	Y	16.8	300

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – Culverts Span (m) - bridges	Height (m) – Culverts Soffit Level (m AHD) - bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream scour protection			
							Material	Scour protection required (Y)	Length	Rock d50
C140.78	140.780	RCP	6	2.1		1.83	Vertosol	Y	16.8	300
C140.83	140.825	RCP	6	2.1		1.84	Vertosol	Y	16.8	300
C140.87	140.870	RCP	4	2.1		1.91	Vertosol	Y	16.8	300
C140.91	140.910	RCP	6	2.1		1.88	Vertosol	Y	16.8	300
C140.98	140.975	RCP	6	2.1		1.80	Vertosol	Y	16.8	300
C141.03	141.030	RCP	4	2.1		1.81	Vertosol	Y	16.8	300
C141.07	141.070	RCP	6	2.1		1.84	Vertosol	Y	16.8	300
C141.11	141.110	RCP	6	2.1		1.86	Vertosol	Y	16.8	300
C141.20	141.200	RCP	6	2.1		1.70	Vertosol	Y	16.8	300
C141.24	141.240	RCP	6	2.1		1.78	Vertosol	Y	16.8	300
C141.29	141.285	RCP	6	2.1		1.56	Vertosol	Y	16.8	300
C141.32	141.320	RCP	4	2.1		1.73	Vertosol	Y	16.8	300
310-BR24	141.670	BRIDGE		674	382.1	0.77	Vertosol	N/A		
C142.02	142.015	RCP	6	2.1		1.25	Vertosol	N		
C142.04	142.040	RCP	6	2.1		1.40	Vertosol	N		
C142.08	142.080	RCP	6	2.1		1.45	Vertosol	N		
C142.13	142.125	RCP	6	2.1		1.37	Vertosol	N		
C142.15	142.153	RCP	6	2.1		1.30	Vertosol	N		
C142.19	142.185	RCP	6	2.1		1.38	Vertosol	N		
C142.22	142.220	RCP	6	2.1		1.26	Vertosol	N		
C142.25	142.245	RCP	6	2.1		1.34	Vertosol	N		
C142.28	142.283	RCP	5	2.1		1.58	Vertosol	Y	16.8	300
C142.36	142.360	RCP	6	2.1		1.48	Vertosol	N		
C142.41	142.410	RCP	6	2.1		1.36	Vertosol	N		
C142.44	142.443	RCP	6	2.1		1.38	Vertosol	N		

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – Culverts Span (m) - bridges	Height (m) – Culverts Soffit Level (m AHD) - bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream scour protection			
							Material	Scour protection required (Y)	Length	Rock d50
C142.48	142.475	RCP	6	2.1		1.32	Vertosol	N		
C142.50	142.500	RCP	5	2.1		1.39	Vertosol	N		
C142.54	142.540	RCP	4	2.1		1.49	Vertosol	N		
C142.58	142.577	RCP	5	2.1		1.38	Vertosol	N		
310-BR25	143.580	BRIDGE		1,941	382.1	2.21	Vertosol	N/A		
310-BR26	144.880	BRIDGE		623	382.1	0.59	Vertosol	N/A		
C145.16	145.160	RCBC	4	1.2	0.9	1.90	Vertosol	Y	5.4	200
C145.21	145.205	RCBC	4	1.2	0.9	1.58	Vertosol	Y	5.4	200
C145.25	145.245	RCBC	4	1.2	0.9	1.68	Vertosol	Y	5.4	200
C145.32	145.320	RCBC	2	1.2	0.9	1.81	Vertosol	Y	5.4	200
C145.40	145.395	RCBC	4	1.2	0.9	1.01	Vertosol	N		
C145.72	145.720	RCBC	10	1.5	0.9	1.59	Vertosol	Y	5.4	200
C145.83	145.825	RCBC	4	1.2	0.9	0.83	Vertosol	N		
C145.89	145.887	RCBC	10	1.5	0.9	1.79	Vertosol	Y	5.4	200
C145.92	145.920	RCBC	4	1.2	0.9	1.09	Vertosol	N		
C145.98	145.975	RCBC	4	1.2	0.9	1.03	Vertosol	N		
C146.03	146.025	RCBC	10	1.5	0.9	1.39	Vertosol	N		
C146.56	146.560	RCBC	6	1.2	0.6	0.70	Vertosol	N		
C146.62	146.622	RCBC	4	1.2	0.6	1.01	Vertosol	N		
C147.58	147.580	RCP	6	1.05		0.32	Vertosol	N		
C147.63	147.625	RCP	6	1.05		0.49	Vertosol	N		
C147.66	147.663	RCP	6	1.05		0.72	Vertosol	N		
C147.73	147.725	RCP	7	1.05		0.89	Vertosol	N		
310-BR27	148.700	BRIDGE		1584	383.8	0.67	Vertosol	N/A		
C149.39	149.385	RCP	10	1.35		0.57	Vertosol	N		

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – Culverts Span (m) - bridges	Height (m) – Culverts Soffit Level (m AHD) - bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream scour protection			
							Material	Scour protection required (Y)	Length	Rock d50
C149.42	149.415	RCP	12	1.2		0.55	Vertosol	N		
C149.45	149.450	RCP	3	1.35		0.83	Vertosol	N		
C149.76	149.763	RCP	8	1.2		0.59	Vertosol	N		
C149.80	149.798	RCP	8	1.2		0.63	Vertosol	N		
C149.83	149.825	RCP	8	1.2		0.62	Vertosol	N		
C149.87	149.872	RCP	6	1.35		0.20	Vertosol	N		
C149.91	149.914	RCP	6	1.35		0.58	Vertosol	N		
C149.96	149.956	RCP	8	1.2		0.24	Vertosol	N		
C150.01	150.006	RCP	8	1.05		0.63	Vertosol	N		
C151.11	151.108	RCBC	3	0.9	0.9	1.18	Vertosol	N		
C152.15	152.150	RCBC	2	0.9	0.9	1.45	Vertosol	N		
C153.22	153.222	RCBC	7	2.1	0.9	1.97	Vertosol	Y	5.4	200
C154.31	154.305	RCP	11	1.2		2.66	Vertosol	Y	9.6	300
C157.96	157.960	RCBC	3	2.4	1.2	2.02	Vertosol	Y	7.2	200
C159.13	159.130	RCP	5	1.8		2.81	Vertosol	Y	14.4	300
C159.87	159.865	RCP	5	1.5		1.86	Vertosol	Y	9	200
C161.02	161.015	RCP	2	1.5		2.06	Vertosol	Y	9	200
310-BR29	161.255	BRIDGE		90	431.3	0.49	Vertosol	N/A		
C161.53	161.530	RCP	3	1.8		2.90	Vertosol	Y	14.4	400
C163.01	163.010	RCP	5	1.8		2.60	Vertosol	Y	14.4	300
C163.09	163.085	RCP	11	2.1		2.56	Vertosol	Y	16.8	400
C163.79	163.785	RCP	3	1.8		3.18	Vertosol	Y	14.4	400
C164.83	164.825	RCBC	2	2.4	1.2	2.50	Vertosol	Y	9.6	300
C165.81	165.805	RCBC	2	1.2	1.2	2.44	Vertosol	Y	9.6	300
C167.32	167.322	RCBC	4	2.4	0.9	1.85	Vertosol	Y	5.4	200

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – Culverts Span (m) - bridges	Height (m) – Culverts Soffit Level (m AHD) - bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream scour protection			
							Material	Scour protection required (Y)	Length	Rock d50
C167.74	167.735	RCBC	2	1.2	1.2	2.38	Vertosol	Y	9.6	300
C168.59	168.585	RCP	3	2.4		3.48	Vertosol	Y	24	500
C169.27	169.265	RCBC	5	2.4	2.4	2.47	Vertosol	Y	19.2	400
C169.74	169.735	RCBC	4	1.8	1.5	1.98	Vertosol	Y	9	200
C170.62	170.620	RCP	9	2.4		2.77	Vertosol	Y	19.2	400
310-BR30	170.945	BRIDGE		141	514.2	0.90	Vertosol	N/A		
C172.27	172.270	RCP	3	1.35		1.49	Vertosol	N		
310-BR40	172.440	BRIDGE		95	530.7	0.89	Vertosol	N/A		
C175.61	175.605	RCP	4	1.8		2.95	Vertosol	Y	14.4	400
C176.36	176.355	RCP	1	2.1		3.49	Vertosol	Y	21	500
C176.74	176.735	RCP	5	2.1		2.60	Vertosol	Y	16.8	400
C177.35	177.350	RCP	7	1.8		2.45	Vertosol	Y	14.4	300
C179.93	179.925	RCP	9	2.1		3.02	Vertosol	Y	21	500
C180.50	180.500	RCP	4	1.8		2.74	Vertosol	Y	14.4	300
C181.71	181.705	RCBC	6	2.4	2.1	2.16	Vertosol	Y	16.8	300
C182.28	182.275	RCP	6	2.1		2.56	Vertosol	Y	16.8	400
310-BR42	183.630	BRIDGE		89	531.6	1.96	Vertosol	N/A		
C184.87	184.872	RCBC	10	2.4	1.8	2.25	Vertosol	Y	14.4	300
C185.91	185.910	RCP	14	1.8		3.03	Vertosol	Y	14.4	400
C186.88	186.875	RCBC	3	1.5	0.9	1.87	Vertosol	Y	5.4	200
C187.00	186.995	RCP	3	1.2		1.85	Vertosol	Y	7.2	200
C188.72	188.717	RCBC	11	1.8	1.2	3.10	Vertosol	Y	9.6	300
C190.81	190.807	RCP	5	1.5		3.34	Vertosol	Y	12	300
C191.83	191.825	RCP	5	2.7		2.60	Vertosol	Y	20.8	400
C193.38	193.380	RCBC	2	1.5	0.9	2.30	Vertosol	Y	5.4	200

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – Culverts Span (m) - bridges	Height (m) – Culverts Soffit Level (m AHD) - bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream scour protection			
							Material	Scour protection required (Y)	Length	Rock d50
C193.41	193.410	RCP	3	1.05		2.80	Vertosol	Y	6.3	200
C194.66	194.657	RCP	3	1.2		2.79	Vertosol	Y	9.6	300
C195.19	195.185	RCP	3	1.5		2.41	Vertosol	Y	12	300
C195.64	195.635	RCP	14	1.05		2.10	Vertosol	Y	6.3	200
C195.93	195.925	RCP	2	1.05		1.40	Vertosol	N		
C196.03	196.028	RCP	2	1.05		1.70	Vertosol	Y	6.3	200
310-BR20	196.115	BRIDGE		113	442.3	2.80	Vertosol	N/A		
310-BR31	197.260	BRIDGE		250	430.3	2.60	Vertosol	N/A		
C197.42	197.417	RCP	15	2.4		2.10	Vertosol	Y	19.2	300
C197.49	197.491	RCP	11	1.5		2.20	Vertosol	Y	9	200
C197.53	197.525	RCP	10	1.2		2.00	Vertosol	Y	7.2	200
C197.71	197.705	RCP	17	1.05		2.10	Vertosol	Y	6.3	200
310-BR32	197.960	BRIDGE		202	428.7	3.10	Vertosol	N/A		
C198.26	198.255	RCP	15	1.05		1.60	Vertosol	Y	6.3	200
310-BR33	198.730	BRIDGE		95	436.7	1.40	Vertosol	N/A		
C199.55	199.547	RCBC	6	0.9	0.9	1.93	Vertosol	Y	5.4	200
C199.96	199.955	RCBC	3	0.9	0.9	1.88	Vertosol	Y	5.4	200
C200.24	200.235	RCBC	3	0.9	0.9	1.65	Vertosol	Y	5.4	200
C200.70	200.695	RCP	2	1.8		3.00	Vertosol	Y	14.4	400
C201.25	201.246	RCP	5	1.5		1.97	Vertosol	Y	9	200
C201.52	201.524	RCBC	4	1.8	1.5	3.18	Vertosol	Y	12	300
310-BR34	203.060	BRIDGE		151	481.3	0.70	Vertosol	N/A		
C203.17	203.170	RCP	2	1.05		1.40	Vertosol	N		
310-BR35	204.460	BRIDGE		134	465.3	1.40	Vertosol	N/A		
C204.92	204.915	RCP	2	1.05		1.30	Vertosol	N		

Culvert ID	Chainage (km)	Type	Number	Diameter/width (m) – Culverts Span (m) - bridges	Height (m) – Culverts Soffit Level (m AHD) - bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream scour protection			
							Material	Scour protection required (Y)	Length	Rock d50
C205.09	205.090	RCP	12	1.05		1.60	Vertosol	Y	6.3	200
C205.14	205.137	RCP	2	1.05		1.20	Vertosol	N		
C205.30	205.296	RCP	4	1.05		1.50	Vertosol	Y	6.3	200
C205.37	205.370	RCP	15	1.05		1.60	Vertosol	Y	6.3	200
C205.47	205.467	RCP	5	1.05		1.20	Vertosol	N		
C205.60	205.600	RCP	2	1.05		1.80	Vertosol	Y	6.3	200
C205.87	205.865	RCP	7	1.05		2.00	Vertosol	Y	6.3	200
C206.43	206.427	RCBC	1	1.8	0.9	2.28	Vertosol	Y	5.4	200
C206.95	206.945	RCBC	16	2.4	1.2	1.40	Vertosol	N		

APPENDIX Q1

Hydrology and Flooding Technical Report—Volume I

Appendix E Local Drainage Structures and Impact Outcomes

INLAND RAIL—BORDER TO GOWRIE ENVIRONMENTAL IMPACT STATEMENT

IR_1534

Appendix E

Local drainage structures and impact outcomes

Culvert ID	Chainage (km)	Type	Number	Diameter/ Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Flow Through Structure (m^3/s)	1% AEP Upstream Water Level-Design (m AHD)	1% AEP Upstream Headwater Depth-Design (m)	1% AEP Freeboard to Formation (m)	Impacts at Rail Corridor		
										1% AEP Afflux (m)	Existing Time of Inundation (hrs)	Change in Time of Inundation (hrs)
C6.60	6.600	RCBC	3	1.5	1.2	7.6	234.41	1.16	0.36	0.07	6.52	5.21
C8.39	8.393	RCBC	8	1.2	1.2	14.4	234.73	1.04	0.94	0.15	6.47	2.50
C13.00	13.000	RCBC	13	2.4	1.2	40.0	237.97	1.00	1.26	0.20	6.63	1.33
C17.89	17.885	RCP	2	1.05		3.2	240.27	1.40	1.27	-0.11	6.77	-0.46
C18.51	18.506	RCP	2	1.2		4.1	241.83	1.47	0.65	0.19	6.64	2.00
C18.87	18.870	RCP	4	1.2		8.8	242.53	1.56	0.79	0.06	6.62	1.65
C20.00	19.995	RCBC	6	1.2	1.2	9.6	244.07	0.96	1.12	-0.04	6.72	2.12
C22.42	22.424	RCBC	8	1.2	1.2	12.5	244.65	0.97	1.02	0.16	6.82	1.71
C23.05	23.046	RCBC	6	1.2	1.2	8.8	244.59	0.93	1.21	0.06	6.87	0.10
C23.53	23.530	RCBC	7	1.2	1.2	10.2	245.02	0.89	0.59	0.11	6.86	0.20
C24.41	24.410	RCBC	3	1.2	1.2	4.7	244.95	0.92	0.82	0.03	6.83	0.97
C24.84	24.839	RCBC	1	1.2	1.2	3.2	245.33	1.73	0.30	0.01	5.64	0.40
C30.60	30.595	RCBC	7	3	1.2	45.4	250.62	1.38	0.96	0.07	6.47	2.20
C31.40	31.397	RCP	3	1.2		3.8	252.35	0.99	1.17	0.11	3.42	1.57
C31.44	31.438	RCP	5	1.35		12.0	252.67	1.46	0.89	0.12	6.58	1.86
C32.03	32.030	RCP	10	0.9		11.8	253.56	1.30	0.51	0.19	4.03	2.13
C32.80	32.800	RCP	4	0.9		4.2	258.50	1.15	0.69	0.31	3.36	0.62
C33.66	33.664	RCP	9	1.5		16.0	260.18	1.09	1.80	0.19	2.78	2.20
C35.18	35.175	RCP	12	2.1		51.9	257.58	1.60	1.60	0.28	2.57	1.13
C41.20	41.195	RCBC	18	3	1.5	46.6	265.92	0.71	2.21	0.36	2.34	1.13
C46.46	46.464	RCP	10	1.2		17.2	271.50	1.26	0.90	0.03	1.58	0.20
C48.41	48.406	RCP	21	2.4		91.5	273.91	0.60	14.22	0.30	2.74	0.65
C49.83	49.825	RCBC	3	3	1.5	15.1	295.25	1.11	0.75	0.25	1.78	0.18
C49.97	49.972	RCBC	8	2.1	0.9	16.7	294.95	0.79	0.32	0.04	1.51	0.15

Culvert ID	Chainage (km)	Type	Number	Diameter/ Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Flow Through Structure (m^3/s)	1% AEP Upstream Water Level- Design (m AHD)	1% AEP Upstream Headwater Depth- Design (m)	1% AEP Freeboard to Formation (m)	Impacts at Rail Corridor		
										1% AEP Afflux (m)	Existing Time of Inundation (hrs)	Change in Time of Inundation (hrs)
C51.50	51.495	RCP	7	1.35		21.4	282.81	1.84	4.89	0.03	1.83	0.13
C53.20	53.201	RCP	7	1.2		7.2	276.74	0.87	2.39	0.29	1.27	0.24
C53.62	53.618	RCBC	2	1.2	0.9	3.9	280.50	1.21	0.49	0.02	1.02	0.15
C54.44	54.439	RCBC	2	0.9	0.9	2.6	283.26	1.05	0.38	0.00	0.80	0.20
C55.06	55.056	RCP	7	1.05		10.7	274.54	1.38	9.46	0.06	1.96	0.18
C60.18	60.175	RCP	1	0.9		0.9	307.80	0.93	7.50	0.09	0.26	0.00
C60.49	60.490	RCP	11	2.4		42.2	295.47	1.11	17.63	0.37	2.51	0.47
C61.60	61.600	RCP	5	2.4		33.4	290.16	2.28	15.24	0.17	2.49	0.04
C61.90	61.900	RCP	5	2.4		34.4	291.05	2.14	12.15	0.06	2.65	0.11
C62.52	62.524	RCBC	2	1.2	0.9	3.3	299.49	1.01	2.28	0.00	0.47	0.00
C62.94	62.940	RCP	2	2.4		11.0	290.76	1.76	10.13	0.13	1.40	0.85
C63.15	63.150	RCP	5	2.4		27.1	286.24	1.46	14.22	0.25	2.45	0.25
C64.50	64.495	RCP	5	0.9		5.7	296.61	1.27	1.06	0.15	2.25	0.80
C65.11	65.114	RCBC	13	2.1	1.2	14.8	295.01	0.55	1.34	0.38	1.94	1.53
C66.81	66.813	RCP	16	1.2		13.7	285.14	0.77	2.16	0.40	2.46	1.65
C70.50	70.500	RCP	2	1.2		4.3	290.44	1.39	0.66	0.18	1.31	1.05
C71.51	71.510	RCBC	3	2.4	0.9	6.0	297.37	0.69	1.11	0.02	1.77	0.18
C73.33	73.330	RCP	2	1.2		4.5	297.47	1.61	1.38	0.00	1.17	0.00
C73.43	73.430	RCP	1	1.2		1.4	295.90	1.10	2.80	0.00	0.73	0.36
C73.52	73.520	RCP	1	1.35		2.9	294.73	1.72	3.77	0.02	1.00	0.12
C73.61	73.605	RCP	1	0.9		0.6	294.47	0.72	3.84	0.00	0.04	0.00
C73.71	73.705	RCBC	3	3	1.5	25.0	292.82	1.58	5.35	0.40	4.21	1.75
C74.97	74.970	RCP	3	1.2		5.9	305.41	1.41	2.22	0.03	1.29	0.09
C76.57	76.570	RCBC	16	1.2	0.9	15.4	308.03	0.68	1.47	0.36	1.76	0.45

Culvert ID	Chainage (km)	Type	Number	Diameter/ Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Flow Through Structure (m^3/s)	1% AEP Upstream Water Level-Design (m AHD)	1% AEP Upstream Headwater Depth-Design (m)	1% AEP Freeboard to Formation (m)	Impacts at Rail Corridor		
										1% AEP Afflux (m)	Existing Time of Inundation (hrs)	Change in Time of Inundation (hrs)
C77.20	77.195	RCP	6	1.5		12.9	307.39	1.19	2.12	0.37	4.78	0.40
C77.47	77.465	RCP	4	0.9		4.2	308.39	1.16	1.11	0.28	0.63	0.74
C77.77	77.770	RCBC	8	1.2	1.2	12.2	308.32	0.62	1.18	0.28	2.11	0.66
C78.28	78.280	RCBC	4	2.1	0.9	5.0	309.59	0.59	0.96	0.07	0.56	0.12
C79.02	79.015	RCBC	5	2.4	1.5	21.8	307.51	1.24	3.52	0.08	4.29	0.04
C79.53	79.525	RCP	7	1.35		9.5	307.85	0.96	3.53	0.17	3.78	0.57
C79.98	79.980	RCBC	7	0.9	0.9	6.2	311.35	0.78	0.33	0.08	0.84	0.50
C80.65	80.645	RCBC	6	1.8	1.5	12.8	311.73	0.67	1.52	0.28	1.54	0.15
C81.19	81.185	RCBC	7	2.1	2.1	18.5	310.30	0.47	4.21	0.37	4.67	0.90
C82.35	82.350	RCBC	18	2.1	2.1	49.2	314.12	0.93	3.15	0.38	2.60	0.62
C83.51	83.505	RCBC	8	2.4	1.5	17.4	319.01	0.74	0.99	0.35	3.95	0.44
C84.38	84.380	RCBC	35	2.4	2.4	85.5	317.25	0.92	4.80	0.40	4.11	1.00
C87.54	87.540	RCBC	3	1.8	1.8	5.1	324.30	1.05	4.76	0.00	0.37	0.01
C88.11	88.110	RCP	2	1.2		2.2	325.32	0.89	5.52	0.36	1.29	0.53
C90.96	90.960	RCBC	31	2.4	2.4	110.1	330.64	1.01	5.85	0.40	2.59	0.91
C92.08	92.080	RCBC	16	2.4	1.2	45.9	333.45	0.95	0.32	0.40	3.24	1.00
C92.94	92.940	RCP	17	1.5		25.1	328.66	0.94	1.79	0.37	3.57	2.06
C93.61	93.610	RCBC	8	1.8	1.8	9.0	325.41	0.60	3.05	0.28	3.77	0.47
C94.91	94.910	RCBC	5	2.1	0.9	9.1	324.09	0.71	0.81	0.23	1.24	0.36
C95.07	95.065	RCBC	15	2.4	1.5	45.9	323.25	0.95	1.25	0.38	6.21	1.50
C96.20	96.195	RCBC	8	2.4	1.2	23.5	326.30	0.94	0.57	0.08	1.22	0.30
C98.87	98.865	RCP	1	1.5		3.2	332.39	1.68	2.21	0.15	7.46	0.20
C101.49	101.485	RCBC	2	1.5	1.2	1.4	336.90	0.79	2.06	0.21	6.18	2.40
C102.55	102.545	RCBC	2	1.5	0.9	3.9	345.28	0.98	0.36	0.08	1.59	0.46

Culvert ID	Chainage (km)	Type	Number	Diameter/ Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Flow Through Structure (m^3/s)	1% AEP Upstream Water Level- Design (m AHD)	1% AEP Upstream Headwater Depth- Design (m)	1% AEP Freeboard to Formation (m)	Impacts at Rail Corridor		
										1% AEP Afflux (m)	Existing Time of Inundation (hrs)	Change in Time of Inundation (hrs)
C106.54	106.543	RCP	5	1.2		9.7	369.59	1.41	1.31	0.03	0.91	0.09
C107.22	107.222	RCBC	4	2.4	1.5	22.7	373.65	1.66	3.06	0.22	1.75	0.35
C107.81	107.808	RCBC	2	0.9	0.9	2.1	381.40	0.90	0.30	0.02	0.52	0.50
C107.97	107.965	RCP	4	1.2		8.2	381.97	1.46	1.73	0.23	1.22	0.30
C108.46	108.455	RCP	4	1.2		9.0	383.61	1.60	6.19	0.12	0.68	0.24
C109.43	109.430	RCBC	12	1.5	1.5	19.6	399.56	0.83	2.44	0.33	1.98	0.36
C110.91	110.913	RCBC	3	1.2	1.2	4.0	420.01	0.85	0.55	0.05	0.91	0.31
C111.17	111.165	RCBC	3	1.2	1.2	1.8	422.78	0.54	0.92	0.01	0.40	0.26
C111.26	111.260	RCBC	4	1.2	0.9	4.5	424.50	0.75	0.40	0.00	1.10	0.04
C112.33	112.325	RCBC	3	1.2	1.2	4.7	436.00	1.00	0.67	0.00	0.50	0.00
C113.00	113.000	RCBC	4	1.2	1.2	8.8	434.66	1.45	3.55	0.37	0.86	0.44
C113.28	113.280	RCP	7	1.2		14.2	436.32	1.39	2.53	0.01	1.33	0.03
C114.27	114.270	RCP	11	0.9		3.0	446.35	0.45	1.95	0.01	3.73	0.03
C114.36	114.360	RCBC	9	1.8	1.5	34.2	448.83	1.57	0.47	0.36	3.03	1.19
C114.90	114.899	RCBC	3	1.2	1.2	5.4	452.89	1.16	1.71	0.02	0.62	0.03
C115.00	115.003	RCP	3	0.9		2.8	453.04	0.98	2.56	0.01	0.55	0.01
C115.33	115.329	RCP	3	0.9		3.0	452.99	1.06	5.81	0.00	0.44	0.00
310-BR28	115.530	BRIDGE		94	459.2	17.7	452.00	N/A	2.79	0.03	0.12	0.10
C117.39	117.385	RCBC	13	3	1.5	30.5	458.07	0.67	2.79	0.27	2.94	1.11
C117.59	117.585	RCBC	6	1.8	1.8	14.5	458.25	0.95	1.78	0.02	2.57	0.49
C117.69	117.693	RCBC	1	1.2	1.2	1.2	458.73	0.49	0.81	0.06	1.34	0.67
C118.09	118.085	RCBC	10	2.1	1.2	10.6	457.07	0.52	1.02	0.18	1.64	1.04
C118.42	118.415	RCBC	6	3	1.5	27.1	456.16	1.06	0.69	0.07	2.67	0.54
C118.59	118.590	RCBC	17	1.2	1.2	1.6	454.66	0.14	1.52	0.16	1.73	1.83

Culvert ID	Chainage (km)	Type	Number	Diameter/ Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Flow Through Structure (m^3/s)	1% AEP Upstream Water Level- Design (m AHD)	1% AEP Upstream Headwater Depth- Design (m)	1% AEP Freeboard to Formation (m)	Impacts at Rail Corridor		
										1% AEP Afflux (m)	Existing Time of Inundation (hrs)	Change in Time of Inundation (hrs)
C118.89	118.890	RCP	10	0.9		5.5	454.05	0.70	1.04	0.06	0.71	0.60
C119.02	119.023	RCBC	1	2.4	1.5	4.1	454.73	1.17	0.45	0.02	2.56	0.27
C119.29	119.285	RCBC	15	1.2	1.2	2.1	453.97	0.19	1.43	0.12	2.36	1.69
C119.37	119.365	RCBC	7	3	1.5	20.9	454.96	0.78	1.14	0.00	2.87	0.18
C119.74	119.740	RCBC	2	2.4	1.2	4.2	457.89	0.73	1.10	0.03	2.00	0.85
C119.86	119.860	RCBC	8	1.5	1.2	5.1	458.26	0.46	1.25	0.13	1.17	1.11
C120.07	120.065	RCBC	3	1.5	0.9	0.8	458.38	0.25	1.26	0.08	0.14	0.55
C120.24	120.240	RCBC	29	2.4	1.5	71.8	458.68	0.5	1.03	0.36	3.79	1.52
C120.75	120.750	RCBC	11	2.4	1.5	51.4	459.01	1.39	1.00	0.09	1.59	0.33
C124.44	124.435	RCBC	7	1.5	1.5	7.6	436.85	0.62	5.62	0.03	1.85	0.01
C125.47	125.470	RCBC	6	2.4	2.1	27.2	424.79	0.9	6.75	0.29	3.69	1.57
C125.82	125.820	RCP	1	1.8		4.5	424.80	1.82	2.99	0.04	0.53	0.10
C128.88	128.880	RCP	23	1.35		22.7	406.95	0.80	2.67	0.40	3.42	1.26
C129.63	129.625	RCP	5	1.2		9.3	409.91	1.36	1.09	0.23	3.27	0.88
C131.39	131.385	RCP	18	2.1		33.6	400.95	0.85	5.09	0.39	3.54	2.35
C133.53	133.530	RCBC	4	1.8	1.2	14.4	394.57	1.30	0.30	0.21	2.59	1.18
C133.90	133.900	RCBC	1	1.5	1.2	3.0	392.94	1.29	0.62	0.28	1.52	1.63
C134.37	134.370	RCBC	3	1.5	1.2	9.7	391.57	1.40	0.35	0.29	3.73	1.87
C135.28	135.275	RCBC	4	1.2	0.9	7.4	388.39	1.13	0.36	0.28	1.63	1.84
C135.82	135.815	RCBC	4	1.2	0.9	7.7	386.56	1.18	0.33	0.24	3.83	0.48
C151.11	151.108	RCBC	3	0.9	0.9	1.0	386.93	0.40	1.60	0.00	2.60	0.16
C152.15	152.150	RCBC	2	0.9	0.9	1.1	389.06	0.23	0.64	0.19	5.47	1.98
C153.22	153.222	RCBC	7	2.1	0.9	17.4	390.42	0.91	2.04	0.18	5.57	2.25
C154.31	154.305	RCP	11	1.2		23.5	393.42	1.49	0.59	0.30	5.64	3.50

Culvert ID	Chainage (km)	Type	Number	Diameter/ Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Flow Through Structure (m^3/s)	1% AEP Upstream Water Level- Design (m AHD)	1% AEP Upstream Headwater Depth- Design (m)	1% AEP Freeboard to Formation (m)	Impacts at Rail Corridor		
										1% AEP Afflux (m)	Existing Time of Inundation (hrs)	Change in Time of Inundation (hrs)
C157.96	157.960	RCBC	3	2.4	1.2	8.7	403.10	0.89	0.61	0.03	5.66	2.50
C159.13	159.130	RCP	5	1.8		24.7	407.05	1.95	0.77	0.09	4.66	3.00
C159.87	159.865	RCP	5	1.5		7.8	411.59	0.96	5.00	0.20	1.62	1.25
C161.02	161.015	RCP	2	1.5		3.7	425.65	1.05	4.61	0.33	3.66	1.25
310-BR29	161.255	BRIDGE		90	431.3	12.3	426.00	N/A	7.29	0.00	0.00	0.00
C161.53	161.530	RCP	3	1.8		9.8	429.21	1.19	6.22	0.22	1.69	0.36
C163.01	163.010	RCP	5	1.8		9.3	436.36	1.12	14.96	0.34	1.79	0.35
C163.09	163.085	RCP	11	2.1		29.5	436.85	0.92	15.34	0.40	1.79	0.56
C163.79	163.785	RCP	3	1.8		12.1	450.76	1.27	9.80	0.06	0.83	0.19
C164.83	164.825	RCBC	2	2.4	1.2	9.9	472.60	1.32	0.31	0.01	0.37	-0.02
C165.81	165.805	RCBC	2	1.2	1.2	4.8	484.26	1.31	0.30	0.03	1.17	0.00
C167.32	167.322	RCBC	4	2.4	0.9	10.0	501.86	0.81	0.74	0.07	5.87	0.21
C167.74	167.735	RCBC	2	1.2	1.2	4.0	506.61	1.11	0.69	0.02	5.67	0.39
C168.59	168.585	RCP	3	2.4		25.7	497.81	2.60	7.81	0.17	1.44	0.16
C169.27	169.265	RCBC	5	2.4	2.4	37.8	498.78	1.35	5.29	0.25	3.75	0.79
C169.74	169.735	RCBC	4	1.8	1.5	11.1	497.19	0.69	5.86	0.00	4.18	0.11
C170.62	170.620	RCP	9	2.4		29.4	500.62	1.88	11.68	0.38	5.46	1.30
310-BR30	170.945	BRIDGE		141	514.2	14.1	506.75	N/A	9.44	0.00	5.84	0.01
C172.27	172.270	RCP	3	1.35		2.5	521.63	0.70	8.83	0.13	5.12	1.37
310-BR40	172.440	BRIDGE		95	530.7	13.7	520.20	N/A	12.49	0.09	5.72	0.25
C175.61	175.605	RCP	4	1.8		18.3	558.59	1.44	4.12	0.15	1.51	0.33
C176.36	176.355	RCP	1	2.1		8.2	564.77	2.50	1.67	0.21	1.84	0.11
C176.74	176.735	RCP	5	2.1		15.8	563.65	1.10	4.70	0.36	1.26	0.00
C177.35	177.350	RCP	7	1.8		17.8	569.70	1.15	1.73	0.26	5.66	0.38

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										1% AEP Afflux (m)	Existing Time of Inundation (hrs)	Change in Time of Inundation (hrs)
C179.93	179.925	RCP	9	2.1		32.5	547.51	1.06	13.44	0.35	2.78	0.55
C180.50	180.500	RCP	4	1.8		17.3	545.57	1.58	8.60	0.21	2.77	0.68
C181.71	181.705	RCBC	6	2.4	2.1	27.7	532.41	0.84	7.59	0.40	5.79	0.18
C182.28	182.275	RCP	6	2.1		14.6	533.93	0.94	5.20	0.34	4.50	0.34
310-BR42	183.630	BRIDGE		89	531.6	76.9	520.02	N/A	13.55	0.00	5.82	0.01
C184.87	184.872	RCBC	10	2.4	1.8	29.9	520.97	0.49	2.84	0.37	5.72	0.46
C185.91	185.910	RCP	14	1.8		35.1	518.29	0.62	9.58	0.38	5.72	0.32
C186.88	186.875	RCBC	3	1.5	0.9	4.3	531.64	0.76	0.64	0.09	0.98	0.61
C187.00	186.995	RCP	3	1.2		4.1	531.73	1.06	1.09	0.03	1.03	0.20
C190.81	190.807	RCP	5	1.5		17.4	489.87	1.77	7.30	0.07	2.00	0.39
C194.66	194.657	RCP	3	1.2		6.1	458.12	1.49	0.56	0.40	2.92	0.95
C195.19	195.185	RCP	3	1.5		6.7	439.90	1.21	13.49	0.09	4.96	0.70
C199.55	199.547	RCBC	6	0.9	0.9	6.0	442.25	0.86	0.53	0.02	1.16	0.76
C199.96	199.955	RCBC	3	0.9	0.9	3.0	446.77	0.86	0.81	0.08	2.41	0.00
C200.24	200.235	RCBC	3	0.9	0.9	2.1	450.27	0.67	0.61	0.02	0.26	0.00
C200.70	200.695	RCP	2	1.8		8.6	451.93	1.70	4.36	0.17	0.65	0.22
C201.25	201.246	RCP	5	1.5		9.9	457.41	1.13	5.34	0.13	0.78	0.64
C201.52	201.524	RCBC	4	1.8	1.5	17.7	460.85	1.45	5.19	0.24	2.42	0.67
C206.43	206.427	RCBC	1	1.8	0.9	2.2	455.04	0.94	0.37	0.32	0.60	0.598