APPENDIX M

Hydrology and Flooding

Part 1 of 2

GOWRIE TO HELIDON ENVIRONMENTAL IMPACT STATEMENT

INLAND RAIL

ARTC

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

IR_1536

Inland Rail Gowrie to Helidon EIS

Appendix M – Hydrology and Flooding Technical Report

Australian Rail Track Corporation

Reference: 3200

Document Number:
2-0001-320-EAP-10-RP-0212-3

Contents

Appendices

Figures

Tables

Glossary

The following terms and acronyms are used within this document.

Term or Acronym	Description
1D	One dimensional
2D	Two dimensional
AAToS	Annual Average Time of Submergence (hrs/yr)
AEP	Annual Exceedance Probability
AGRD	Austroads Guide to Road Design Part 5B: Drainage (Austroads 2013)
AHD	Australian Height Datum
ARI	Average Recurrence Interval
ARR 1987	Australian Rainfall and Runoff Guidelines – 1987 edition (Cth)
ARR 2016	Australian Rainfall and Runoff Guidelines – 2016 edition (Cth)
ARTC	Australian Rail Track Corporation
B2G	Border to Gowrie
Backwater	Upstream movement of water from a downstream catchment in flood
BoM	Bureau of Meteorology
BRCFS	Brisbane River Catchment Flood Study
Ch	Chainage
CL	Continuing loss rate (mm/hr)
Cth	Commonwealth
DEA	Design Event Approach
DEM	Digital Elevation Model
DES	Department of Environment and Science
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
DS	Downstream
Existing Case	Hydraulic modelling case pre-Project (i.e. existing conditions)
EY	Exceedances per year
FFA	Flood Frequency Analysis
FFJV	Future Freight Joint Venture
GIS	Geographic Information System
G2H	Gowrie to Helidon
H2C	Helidon to Calvert
hr	Hour
IFD	Intensity-Frequency-Duration
IL	Initial Loss
km	kilometres
km^2	Square kilometres
LGA	Local Government Area

Term or Acronym	Description
LiDAR	Light Detection and Ranging
LP3	Log Pearson III
LVRC	Lockyer Valley Regional Council
MCS	Monte Carlo Simulation
m	metres
m^3/s	Cubic metres per second
MDBA	Murray-Darling Basin Authority
ML	Megalitres
mm	millimetres
mm/hr	millimetres per hour
m AHD	metres above Australian Height Datum
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
QLD	Queensland
QR	Queensland Rail
RCBC	Reinforced concrete box culvert
RCP	Reinforced concrete pipe
Streamflow	Estimated flow recorded by a stream gauge (m^3/s)
The Project	The Gowrie to Helidon project
ToR	Terms of Reference
ToS	Time of Submergence (hrs)
TRC	Toowoomba Regional Council
TSRC	Toowoomba Second Range Crossing Project
US	Upstream

Executive summary

Inland Rail is a once-in-a-generation Program connecting regional Australia to domestic and international markets, transforming the way we move freight around the country. It will complete the 'spine' of the national freight network between Melbourne and Brisbane via regional Victoria, New South Wales and Queensland. This new 1,700 kilometre (km) line is the largest freight rail infrastructure project in Australia and is expected to commence operations in 2026.

The Inland Rail Gowrie to Helidon (G2H) Project (the 'Project') is a new dual gauge rail line connecting Gowrie (north-west of Toowoomba) with Helidon (east of Toowoomba). This section is approximately 28 km long and will include a new tunnel, approximately 6.24 km long, to create an efficient route through the steep terrain of the Toowoomba Range. The G2H Project provides a connection between the Border to Gowrie (B2G) project to the west and Helidon to Calvert (H2C) project to the east, along with connections into the existing Queensland Rail Western Line.

The Project is located within the Toowoomba and Lockyer Valley local government areas (LGAs).

There are four waterway catchments that the Project alignment crosses, with the main waterway being Gowrie Creek. The Project is sited within the Gowrie Creek floodplain between Charlton and Mount Kynoch, however the majority of the Project is within tunnel or is co-located with the existing Queensland Rail (QR) Western Line paralleling Gowrie Creek. However, flooding within the Gowrie Creek catchment affects the western tunnel portal location and potentially, the intermediate ventilation shaft at Cranley, along with the local road network including the realigned road Gowrie Junction Road.

The other waterways crossings include Oaky Creek, Six Mile Creek and the Upper Lockyer Creek. Six Mile Creek and Oaky Creek flow under the Project alignment where it is on viaduct structures and there is minimal impediment to the waterway. The Project alignment has a single bridge crossing over Lockyer Creek in its upper reaches upstream of the confluence of Rocky Creek with Lockyer Creek.

The 2011 flood event, which impacted these waterways and associated floodplains, resulted in isolation of properties and communities including Gowrie and Kingsthorpe, causing extensive damage to the existing rail and road networks, along with property damage.

The purpose of this investigation was to better understand and quantify the existing flooding characteristics of the each of the four waterways in the vicinity of the Project and to assess and mitigate any potential impacts from the Project on the existing flooding regimes. The key objectives of this report are to provide information on the data investigation, hydrologic and hydraulic calibration, impact assessment and mitigation and to provide comment on the performance of the Project design.

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

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

A Developed Case was prepared using the Existing Case models and incorporating the Project design. 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 (refer Table 1) that were developed for the Project. The flood impact objectives were initially developed based on a review of objectives used for other large infrastructure projects in rural and urban areas as well as consideration of industry practice and use of engineering judgement.

Table 1 Flood impact objectives

Parameter	Objectives				
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 and grazing land/forest areas and other non-agricultural land
	≤ 10 mm	≤ 50 mm	≤ 100 mm	≤ 100 mm	≤ 200 mm with localised areas up to 400 mm
	Changes in peak water levels are to be assessed against the above proposed limits. It is noted that changes in peak water levels can have varying impacts upon different infrastructure/land and flood impact objectives were developed to consider the flood sensitive receptors in the vicinity of the 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 duration of inundation[1]	Identify changes to time of inundation through determination of time of submergence (ToS). For roads, determine Annual Average 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 taking into account 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 by 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

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 significant portion of the Project alignment consists of tunnel or high-level viaduct structures and as such has little or no impact on existing flood conditions.

A comprehensive consultation exercise has been 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

- Continue to work with directly impacted landowners and stakeholders affected by the alignment throughout the detailed design, construction and operational phases of the Project
- Continue to work with Toowoomba Regional Council (TRC), Lockyer Valley Regional Council (LVRC) and State government departments throughout the detailed design, construction and operational phases of the Project.

1 Introduction

1.1 Inland Rail Program

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

This new 1,700 kilometre (km) line is the largest freight rail infrastructure project in Australia and is expected to commence operations in 2026.

1.2 Gowrie to Helidon Project

The Inland Rail section of Gowrie to Helidon (G2H) (the 'Project") is a new dual gauge rail line connecting the New South Wales/Queensland Border to Gowrie (B2G) project, at the western end near Gowrie, with the Helidon to Calvert (H2C) project at the eastern end, northwest of Helidon (as shown on Figure 1.1). This section is approximately 28 km long and includes a 6.24 km tunnel, to create an efficient route through the steep terrain of the Toowoomba Range. The Project provides connections to the existing Queensland Rail (QR) West Moreton System west of Gowrie, and east of Gowrie towards Toowoomba and at Helidon.

The Project was declared a 'coordinated project' requiring an environmental impact statement (EIS) under the *State Development and Public Works Organisation Act 1971* in March 2017. The Project was also declared a 'controlled action' under the *Environment Protection and Biodiversity Conservation Act 1999* in March 2017. As such the Project is to be assessed under bilateral agreement between the Commonwealth and Queensland governments in accordance with the Terms of reference for an environmental impact statement: Inland Rail – Gowrie to Helidon project (ToR).

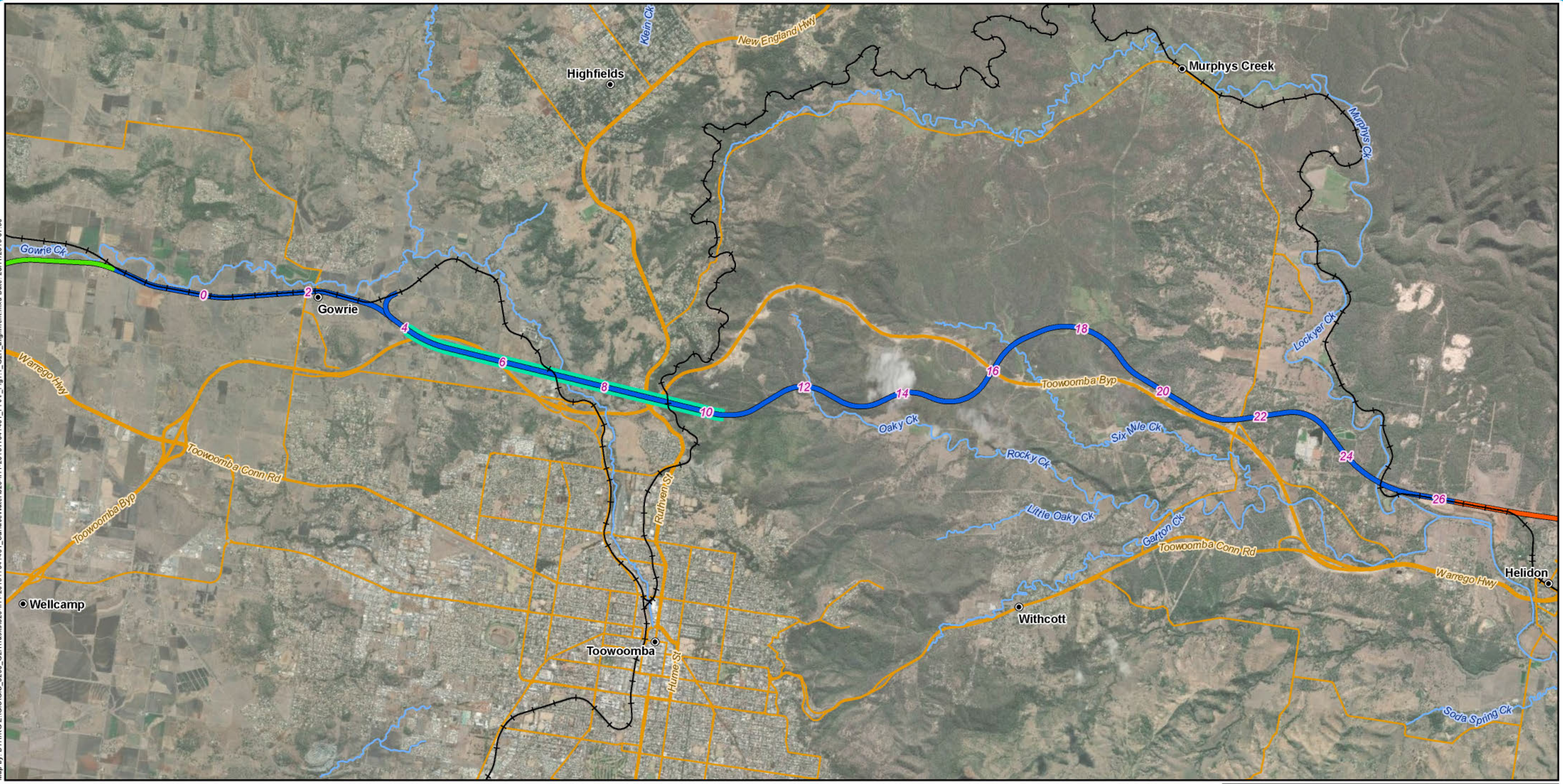
The Project is located within the local government areas (LGAs) of Toowoomba and Lockyer Valley. Key features of the Project include:

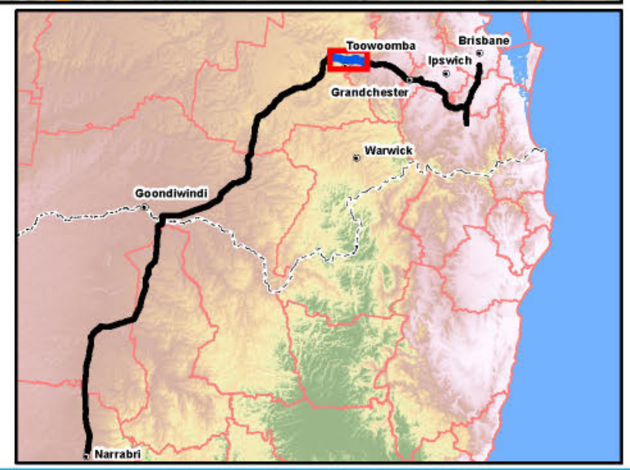
- 28 km of new single track dual gauge railway (bidirectional track); with approximately 4.8 km co-located with the existing Western Line rail corridor west of Gowrie and 0.8 km co-located with the Main Line rail corridor to east of Lockyer Creek
- A 6.24 km long tunnel to be constructed through the Toowoomba Range
- Connections into the existing West Moreton System to the west and east of Gowrie (Western Line) and northwest of Helidon (Main Line)
- Bridges and viaducts to accommodate topographical variation, crossings of waterways and other infrastructure
- Reinforced concrete pipe (RCP) culverts and reinforced concrete box culverts (RCBC)
- Grade-separated crossings of rail and road interfaces throughout the alignment route
- Changes to the local road network, including Gowrie Junction Bridge, a road over rail bridge at Gowrie.

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. Secondly a detailed quantitative assessment has been undertaken to better understand and quantify the existing flooding characteristics of each of the high-risk watercourses (Gowrie Creek, Oaky Creek, Six Mile Creek) in the vicinity of the Project and to assess and mitigate any potential impacts associated with the Project on the existing flooding regime of each waterway. The investigation also addresses the relevant requirements of the Project ToR.

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





Legend

- 5 Chainage (km)
- Localities
- Existing rail
- B2G project alignment
- G2H project alignment
- H2C project alignment
- Tunnel
- Major roads
- Minor roads
- Defined watercourses

A3 scale: 1:80,000

0 0.8 1.6 2.4 3.2 4km

Future Freight
Integrating Community, Environment and Engineering

Issue date: 03/02/2021 Version: 0
Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure 1.1: Gowrie to Helidon alignment

The key purpose of this report is to provide details of investigations undertaken including data collection and review; development and calibration of hydrologic and hydraulic models; design event modelling; impact assessment of the Project alignment; and 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 (Gowrie Creek, Oaky Creek, Six Mile Creek and Lockyer Creek) to establish the Base Case (or Existing Case) flood conditions for the range of flood events up to 1% Annual Exceedance Probability (AEP) as well as the 1 in 2,000 AEP, 1 in 10,000 AEP and Probable Maximum Flood (PMF) events. Low-risk and medium-risk watercourses have also been assessed either as part of the floodplain modelling or independently as local catchment drainage.
- Determine existing flood conditions including flood levels, flows and velocities
- Undertake hydrologic and hydraulic modelling of the Project design including the rail design, road 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 stream flow 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, i.e. regional flooding versus local catchment flooding
- Determination of high, medium and low risk watercourses that the alignment crosses qualitatively considering:
  - The catchment size, resulting flood flows and velocities
  - The land use in the vicinity of the rail alignment
  - The extent and depth of flood inundation
  - The duration of flood events and catchment response time
  - The proximity to and nature of flood sensitive receptors (eg houses, sheds, roads etc)
- Development of tailored hydrologic and hydraulic modelling for each major waterways or local drainage crossing
- 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 was adopted for this Project as ARR 2019 was not released when this investigation commenced.
- 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 design (i.e. rail and road), along with the drainage structures (Developed Case) in the hydraulic models and simulation of ARR 2016 design events including the 20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF events
- Assessment of impacts of the Project 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 Project design for factors including climate change and blockage risk.

The hydrologic 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 and the tunnel flood immunity

- 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 Catchment areas

The Project alignment begins approximately 3.7 km west of Gowrie, at Charlton where it connects with the eastern end of the B2G project. It then runs east, parallel to the existing QR Western Line rail corridor (southern side), for approximately 4.8 km before diverging from the Western Line rail corridor and passing into the proposed western tunnel portal within the vicinity of Boundary Street and the Toowoomba Bypass interchange at Gowrie Junction.

The Project alignment then continues through the Toowoomba Range via an approximately 6.24 km long tunnel under the localities of Cranley, Mount Kynoch and Ballard, with an intermediate ventilation shaft (and associated infrastructure) at Cranley. On the eastern side of the tunnel, the alignment exits the range through the eastern tunnel portal near Mt Kynoch and continues down the Toowoomba Range via a series of viaducts, embankments and cuttings, through Ballard, Mount Lofty, Withcott, Lockyer, Postmans Ridge and Helidon Spa. The Project crosses Lockyer Creek at Helidon and again runs parallel to the existing Main Line rail corridor (northern side) to connect with the H2C project, to the north-west of Helidon.

The Project traverses the catchments of the Condamine River and Lockyer Creek, with the boundary between these two catchments along the crest of the Toowoomba Range. The Condamine River drains west toward the Balonne-Condamine drainage basin and is part of the Drainage Division for the Murray Darling Basin. Lockyer Creek drains north-east toward the Brisbane River and is part of the Drainage Division for the North East Coast.

The Condamine River Basin is one of the largest catchments in the Murray Darling Basin (Murray-Darling Basin Authority (MDBA) 2018). The main rivers of the catchment, the Condamine and the Maranoa, rise in elevated country in Queensland. However, two-thirds of the catchment is flat floodplain country, with a complex system of rivers and creeks joining and breaking away from the Balonne River across the catchment. The catchment's extensive floodplains provide habitat for a diverse range of plants and endangered plant communities (MDBA 2018). Sub-catchments within the Condamine River Basin include: North-Western, South-Western, Kumbarilla Ridge, Central Condamine, Oakey Creek, Upper Condamine, Middle Condamine, Southern Condamine, Lower Condamine, South-Eastern, Emu Creek.

Lockyer Creek is a major tributary of the Brisbane River catchment, joining the Brisbane River approximately 3 km downstream of Wivenhoe Dam. The Lockyer Creek catchment extends down to O'Reilly's Weir, with a total catchment area of approximately 3,000 square kilometres (km²) (Department of Environment and Science (DES) 2015). The catchment features numerous tributaries, including Oaky Creek, Rocky Creek, Six Mile Creek, Fifteen Mile Creek, Murphys Creek and Alice Creek (upstream of Helidon), Flagstone Creek and Sandy Creek (upstream of Grantham), Ma Ma Creek and Tenthill Creek (upstream of Gatton), and Sandy Creek (adjacent to Forest Hill) and Laidley Creek and Buaraba Creek between Gatton and the Brisbane River. The Lockyer Creek catchment varies significantly, with steep headwater areas and wide flat floodplain in the lower reaches.

A notable feature of Lockyer Creek is that the main channel is perched (i.e. the elevation of the creek banks is higher than the surrounding floodplain). This feature is particularly dominant in the lower catchment. Flows in excess of the channel capacity break out of the main creek channel around the confluence of Lockyer Creek and Laidley Creek at Glenore Grove. Overbank flows have limited opportunity to interact with the channel flows and exhibit a longer travel time between Glenore Grove and the confluence with the Brisbane River.

3.2 Waterways

There are four waterway catchments that the Project crosses, being Gowrie Creek, Oaky Creek, Six Mile Creek and Lockyer Creek. Oaky Creek and Six Mile Creek are tributaries of Lockyer Creek. The Project traverses a portion of the Gowrie Creek floodplain area with flood inundation affecting the western tunnel portal location and the intermediate ventilation shaft location.

Lockyer Creek, Six Mile Creek and Oaky Creek all flow under the Project where it is on viaduct and there is minimal impediment to each waterway.

Flooding occurred in a number of these waterways during the 2011 flood event, flooding properties and communities including Gowrie Junction, Kingsthorpe, Murphys Creek and Grantham, leading to loss of lives and causing extensive damage to the existing rail and road networks, along with property damage.

Detailed hydrologic and hydraulic modelling has been undertaken due to the catchment size and substantial floodplain flows associated with each of these watercourses. Details on each of these catchments are outlined in the following sections.

3.2.1 Gowrie Creek

Gowrie Creek, located at the headwaters of the Condamine River catchment, is the largest of Toowoomba's urban creeks. The catchment drains northerly through the city of Toowoomba and includes East and West Creeks which converge to form Gowrie Creek near the city centre.

North of the urban extent of Toowoomba the Gowrie Creek floodplain traverses a primarily rural landscape, until Brinam (east) where the watercourse flows easterly through Gowrie and Kingsthorpe towards the township of Oakey. The Project is primarily located within the Gowrie Creek floodplain between Charlton and Mount Kynoch. Flows within Gowrie Creek are also influenced by releases from Wetalla wastewater treatment plant.

The Project runs parallel to Gowrie Creek, on the southern side of the existing Western Line from Charlton to east of Gowrie where the Project deviates from the existing rail corridor to the southeast. The Project intersects a number of drainage lines and tributaries of Gowrie Creek, draining northwards from Cotswold Hills and surrounds, in this area. The realignment of Gowrie Junction Road will require a new crossing of Gowrie Creek approximately 0.1 km downstream of the existing crossing.

The Project also crosses under Gowrie Creek at Cranley, between the Western Line and Goombungee Road. The intermediate ventilation shaft and associated infrastructure located in Cranley, intersects a small tributary of Gowrie Creek. Figures A1-A and B1-A present locality plans for Gowrie Creek and the intermediate ventilation shaft respectively. The Toowoomba Bypass is also a key feature of the landscape in this area.

3.2.2 Lockyer Creek

The Project is located in the northwest corner of the Lockyer Creek catchment, in the upper Lockyer Creek catchment. The catchment area is approximately 370 km^2, and includes Murphys Creek, Gatton Creek and Lockyer Creek sub-catchments. The catchment includes the steep slopes and foothills of the Great Dividing Range and as such many of the waterways can experience high flows despite the relatively low rainfall (DES 2015). The upper catchment remains mostly forested whereas the mid and lower catchment of Lockyer Creek catchment has been largely cleared.

The Project is crossed by Oaky Creek, Six Mile Creek and Lockyer Creek, which are ephemeral systems.

Oaky Creek drains southwards off the Great Diving Dividing Range at Ballard, with the Toowoomba Bypass influencing the hydrology in the upper sub-catchment. Oaky Creek is crossed by the Project alignment, Oaky Creek Viaduct, at Jones Road at Ballard. A tributary of Oaky Creek also passes under the proposed eastern tunnel portal at Ballard.

South of the confluence of the tributary and Oaky Creek, immediately downstream of the Project, the waterway forms Rocky Creek. The Project traverses side slopes and intersects drainage lines associated with Rocky Creek but does not impact the floodplain or channel of Rocky Creek and as such Rocky Creek is not discussed further in this report.

To the north of the Toowoomba Bypass rail bridge in Withcott, the Project traverses the Six Mile Creek catchment. Six Mile Creek catchment includes the foothills of the Great Dividing Range at Withcott. The catchment is predominately woodland with clearing associated with the main channel and the adjacent floodplain. The Project traverses Six Mile Creek on viaduct, near Gittins Roads at Withcott.

The Project crosses into upper Lockyer Creek catchment near Wards Hill and runs parallels to the Toowoomba Bypass to Murphys Creek Road. The Project than deviates to the north around Withcott Seedlings and traverses floodplain areas of Lockyer Creek, crossing the main channel of Lockyer Creek (i.e. Lockyer Creek Viaduct) and the Main Line at Helidon. To the east of Lockyer Creek the Project is located to the north of the existing Main Line.

Figures C1-A, D1-A and E1-A present locality plans for Oaky Creek, Six Mile Creek and Lockyer Creek respectively.

3.3 Floodplain infrastructure

Key existing infrastructure on floodplain areas in the proximity of the Project alignment includes:

- Gowrie Creek
  - Western Moreton System (Western Line)
  - East Paulsens Road
  - Paulsens Road
  - Gowrie Junction Road
  - Morris Road
  - McMahons Road
  - Old Homebush Road
  - Utilities and services managed by Toowoomba Regional Council (TRC) and other third parties
- Oaky Creek
  - Jones Road
- Six Mile Creek
  - Gittins Road
- Lockyer Creek
  - Western Moreton Rail System (Main Line)
  - Airforce Road and Cattos Road
  - Roma Brisbane Gas Pipeline (underground)
- State Controlled roads
  - Toowoomba Bypass (ID319A)
  - New England Highway (ID22A)
  - Murphys Creek Road (ID4104)

The existing Western Line rail corridor and Paulsens Road run parallel to Gowrie Creek (southern side). A number of Gowrie Creek tributaries flow northwards toward the main creek channel and cross under the existing rail line, Gowrie Junction Road, Paulsens Road, East Paulsens Road, Morris Road and McMahon Road. Old Homebush Road crosses the main channel of Gowrie Creek, while at the western extent of the Project the Western Line also crosses Gowrie Creek, though the Project alignment has already deviated to the southwest as part of the B2G project.

From the hydraulic modelling undertaken it has been identified that the majority of the Western Line is above the 1% AEP flood level with overtopping in localised places only (refer Section 8.2.2.3). The remaining local roads are low-level and modelling demonstrates they are inundated by Gowrie Creek flood events smaller than the 1% AEP event.

Jones Road is within the Oaky Creek catchment. This road is low level in parts and inundated by frequent events. Gittins Road is within the Six Mile Creek catchment and is also low level with a floodway crossing downstream of the Project alignment that is inundated by frequent events.

The existing Main Line which traverses over the proposed Toowoomba Range Tunnel at Ballard, meanders down the Toowoomba Range paralleling, to the north, Murphys Creek. To the east of the township of Murphys Creek, the Main Line parallels Lockyer Creek to Helidon. Airforce Road and Cattos Road run on the northern side of the existing Main Line. At this location the Main Line, the existing Cattos Road and Airforce Road are not impacted by the 1% AEP flood event.

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 has been undertaken to meet these design criteria with a series of iterations undertaken to incorporate design refinement and stakeholder and community feedback. The resulting design outcomes relative to these design criteria are detailed in Section 9.

Table 4.1 Project hydraulic design criteria

Performance criteria	Requirement
Flood immunity	Rail line – 1% AEP flood immunity with 300 millimetre (mm) freeboard to formation level. Tunnel portals – 1 in 10,000 AEP event flood immunity.
Hydraulic analysis and design	Hydrologic and hydraulic analysis and design to be undertaken based on 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 of tidal zone. ARR 2016 blockage assessment guidelines are to be applied.
Scour protection of structures	All bridges and culverts to 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 ToR and 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. The identification of acceptable impacts will take into account flood sensitive receptors (i.e. existing dwellings, sheds, farm buildings and infrastructure, crops, roads etc) and land use within the floodplain.

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 (currently in use)	Agricultural and grazing land/forest areas and other non-agricultural land
	≤ 10 mm	≤ 50 mm	≤ 100 mm	≤ 100 mm	≤ 200 mm with localised areas up to 400 mm

Parameter	Objectives
	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 duration of inundation[1]	Identify changes to time of inundation through determination of time of submergence (ToS). For roads, determine Annual Average 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 taking into account 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 by 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 in ARR 2016.

Accordingly, all design events are quoted in terms of AEP using percentage probability. An extract of Figure 1.2.1 from ARR 2016 Book 1 (refer Table 4.3) details the relationship between Average Recurrence Interval (ARI) and AEP for a range of design events.

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

Exceedances per year (EY)	AEP (%)	AEP (1 in x)	Average Recurrence Interval (years)
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
0.01	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.

4.4 Relevant standards and guidelines

The design standards and guidelines applicable for the hydrologic and hydraulic investigation are:

- AS7637:2014: Railway Infrastructure – Hydrology and Hydraulics (Australian standards 2014)
- 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), Fourth Edition, United States Department of Transport – Federal Highway Administration, Virginia, USA, Richardson, EV and Davis, SR: 2001 (United States Department of Transport 2001)
- Hydraulic Design of Energy Dissipaters for Culverts and Channels, Hydraulic Engineering Circular Number 14 (HEC-14), Third Edition United States Department of Transport – Federal Highway Administration, Virginia, USA, Thompson, PL & Kilgore, RT; 2006 (United States Department of Transport 2006)

5 Data collection and review

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

- Local authorities including TRC and Lockyer Valley Regional Council (LVRC)
- The Bureau of Meteorology (BoM) – rainfall and stream gauging data
- Department of Natural Resources, Mines and Energy (DNRME) – stream gauging data
- Queensland Rail – existing infrastructure details
- Queensland Reconstruction Authority – Brisbane River Catchment Flood Study
- Toowoomba Second Range Crossing (TSRC) Project – flood modelling reports.

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

5.1 Previous studies

A number of previous hydrologic and hydraulic studies were sourced as part of this study. A review of each study was undertaken to determine suitability for use on the Project as documented in the following sections. The data used from these studies and the modelling approach adopted for each catchment is summarised in Section 6.1.

5.1.1 Gowrie Creek

5.1.1.1 Toowoomba Regional Council, Gowrie Creek Flood Risk and Management Study Volume 1 (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 ARR 1987 and therefore design flood estimates are not consistent with ARR 2016 requirements.

5.1.1.2 Gowrie Creek Flood Risk and Management Peer Review (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.

5.1.1.3 Toowoomba Regional Council, Work Package 4, Historical study for Kingsthorpe and Gowrie Junction, Final Report, DHI/WRM (TRC 2014a)

This study focused on a small reach of Gowrie Creek between Kingsthorpe and Gowrie Junction and therefore does not cover the full extent of the proposed alignment. It also 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. The design flood hydrology was based upon ARR 1987 and therefore design flood estimates are not consistent with ARR 2016 requirements.

5.1.1.4 Toowoomba Regional Council, Work Package 8, 2D Flood study for Cotswold Hills (Gowrie Creek Catchment) Final Report, DHI/WRM 2014 (TRC 2014b)

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 (XP-RAFTS) and a coupled 1D/2D MIKE FLOOD hydraulic model. The design flood hydrology was based upon ARR 1987 and therefore design flood estimates are not consistent with ARR 2016 requirements.

5.1.1.5 Gowrie Creek – Flood Assessment Report (APB 2016)

This study covered the upper reaches of Gowrie Creek and was undertaken as part of the TSRC Project (now known as Toowoomba Bypass). A TUFLOW model was developed for the study. It used the (TRC 2014a) hydrologic model which was based upon ARR 1987 and therefore design flood estimates are not consistent with ARR 2016 requirements.

5.1.2 Oaky Creek

No previous study information was available for this waterway.

5.1.3 Six Mile Creek

5.1.3.1 Six Mile Creek – Flood Assessment Report (APB 2017)

This study of Six Mile Creek was undertaken as part of the TSRC Project. A RAFTS hydrologic model of the catchment was developed as per ARR 1987 guidelines and therefore design flood estimates are not consistent with ARR 2016 requirements. A TUFLOW model was developed for the TSRC study area covering an area downstream of the Project alignment crossing of Six Mile Creek and therefore the hydraulic model could not be used for this current investigation.

5.1.4 Lockyer Creek

5.1.4.1 Brisbane River Catchment Flood Study Hydrology Phase Final Report (Aurecon 2015)

The study area for the Brisbane River Catchment Flood Study (BRCFS) encompassed the entire Brisbane River Catchment. More specifically the modelling includes Lockyer Creek and its tributary catchments (including Oaky and Six Mile creeks). Hydrologic models were developed and calibrated against a range of historical flood events and these models were used to determine design flood estimates. Key aspects of the hydrologic component of the study included:

- Review and update of stream gauge flow ratings
- Recalibration of the Brisbane River hydrologic models developed by Seqwater in 2013
- Estimation of stream flows and volumes using hydrologic/rainfall-based methods (Design Event approach in accordance with ARR 1987 and Monte Carlo Simulation (MCS))
- Flood frequency analysis at key stream gauge locations throughout the catchment
- Reconciliation of flows predicted by the different methods to produce design flow estimates to be adopted for the Brisbane River catchment.

Key review findings were:

- The BRCFS hydrologic model has been well calibrated against a range of recent flood events including the 1974, 1996, 1999, 2011 and 2013 flood events
- The BRCFS hydrologic model needed to be modified to produce flow estimates at the location of the proposed Project alignment
- The resulting BRCFS hydrologic model needed to be updated to be compliant with the hydraulic design requirements.

5.1.4.2 Lockyer Valley Flood Model Update Stage 2 (Jacobs 2016)

The Stage 2 Lockyer Valley Flood Model Study incorporated amendments to the original Lockyer Valley flood model which was originally developed for LVRC for the purposes of development control and assessment of flood mitigation options.

Key review findings were:

- The hydrologic model was well calibrated against a range of recent flood events and the hydraulic model was also calibrated to the January 2011 and January 2013 flood events. It should be noted that this is not the BRCFS (Aurecon 2015) URBS hydrologic model.
- The resulting hydrologic model was considered to be non-compliant with the Project hydraulic design criteria as it did not fully follow ARR 2016 guidelines. The key aspect of the application of the design flood hydrology in the Jacobs (2016) study that could be questioned is the use of temporal patterns derived from ARR 1987. Two additional events (20% and 5% AEP) would need to be simulated for this model to satisfy the hydraulic design requirements.

5.1.4.3 The Big Flood: Will It Happen Again?, Final Report (The Big Flood Study team 2016)

The Big Flood study aimed to enhance historical flood records with non-stream gauge data sources (e.g. paleoflood data) while developing understanding of channel and floodplain geomorphic flood risks throughout Lockyer Valley to better manage and predict future floods and associated impacts.

Key review findings were:

- Information on hydrologic models for Lockyer Creek or their calibration against recorded events including the recent 2011 and 2013 floods was not detailed.
- Information on hydraulic models for Lockyer Creek or their calibration against recorded events including the recent 2011 and 2013 floods was not detailed.

5.2 Survey data

Using Geographic Information Systems (GIS) software, a Digital Elevation Model (DEM) was generated with a 1 metre (m) grid resolution for use on the Project based on the 2015 dataset. Australian Rail Track Corporation (ARTC) provided LiDAR data from 2015 as 1 m grid DEM tiles. This dataset was used for modelling within the Project disturbance footprint (comprising the permanent operational disturbance footprint and the temporary construction 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 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, with preference given to the most recent data available.

Future Freight Joint Venture (FFJV) undertook an inspection to understand the veracity of the LiDAR against existing field survey near the Project alignment. It was determined that the LiDAR accuracy was appropriate for use.

An additional survey of culvert arrangements along the QR West Moreton System (Western and Main Lines) was undertaken in November 2018 and has been incorporated into the hydraulic modelling and therefore no As-Built drainage structure drawings were sourced from QR. It is proposed that in the next stage of design this information will be reviewed to ensure current existing structures are modelled.

For the current assessment it was assumed that the Toowoomba Second Range Crossing (TSRC) works included drainage structures to maintain the existing flow paths and flows that drain towards Gowrie Creek, the Intermediate Tunnel Shaft, Oaky Creek and Lockyer Creek. This assumption will be reviewed in the next stage of design. The proposed crossing of Six Mile Creek is located upstream of the TSRC on a steeply grading catchment and therefore would not be affected by the TSRC works.

5.3 Aerial imagery

Aerial imagery was provided by ARTC and has been used to identify and confirm topographic and vegetative characteristics of catchment areas. Aerial imagery captured in 2015 was provided. Additional imagery for areas not covered in the provided aerial imagery was sourced from QGIS imagery in an open source format.

5.4 Existing drainage structure data

Existing drainage structure geometry was obtained from the following sources:

- Previous studies (refer Section 5.1)
- Site inspection(s)
- QR As-Built Drawings, and/or
- Field survey.

Details of existing drainage structures and sources are outlined in Section 6.3.1.2 for Gowrie Creek and Section 6.3.5.2 for Lockyer Creek.

5.5 Stream gauge data

Stream gauges are used to provide a record of observed stream levels. These were originally manually recorded 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 stream flow. A rating curve is required to convert recorded levels into an equivalent stream discharge. The most reliable source of data for deriving a rating curve are actual 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. Stream 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.

5.5.1 Gowrie Creek

There are two stream gauges within Gowrie Creek, being Oakey and Cranley. The location of these stream gauges is presented in Figure A1-B with the gauge details outlined in Table 5.1. The use of these two stream gauges in calibration of the Gowrie Creek hydrologic model is detailed in Section 7.2.

Table 5.1 Stream gauges within Gowrie Creek catchment

Station name	Station number	Ownership	Number of records (years)	Record commenced	Comment
Oakey	422332B	DNRME	27	1992	Approximately 6 km downstream of hydraulic model boundary
Cranley	422332A	DNRME	49	1969	Approximately 3 km upstream of hydraulic model boundary

5.5.2 Oaky Creek

No stream gauge data is available within this Lockyer Creek sub-catchment.

5.5.3 Six Mile Creek

No stream gauge data is available within this Lockyer Creek sub-catchment.

5.5.4 Lockyer Creek

Although there are several stream gauges located throughout the Lockyer Creek catchment, including long-term records at Gatton (143904) and Helidon (143203C) at the downstream end of the Project, the majority of these sites are not considered to be particularly reliable. The primary gauge location used in the BRCFS was at Glenore Grove (143807). This is not an ideal gauge site, being located near the confluence of Lockyer Creek and Laidley Creek, however it is the most downstream location where a relatively consistent relationship between water level and flow can be obtained. Downstream of Glenore Grove the perched banks of the Lockyer Creek main channel enable the channel and floodplain to have different and independent flood levels.

The Glenore Grove rating curve was derived during the BRCFS using a hydraulic model of the confluence area, calibrated against recorded levels and in-stream flow measurements recorded downstream at Lyons Bridge which is located at Lowood. Flow distribution issues affecting the gauge site are highlighted in Figure 5.1. Laidley Creek bifurcates at the confluence with Lockyer Creek, with flows able to combine both upstream and downstream of the Glenore Grove gauge site. Gauge levels however are dependent primarily on water levels generated by the combined flows in the channel downstream of the confluence, and sensitivity testing using different flow splits between Lockyer Creek and Laidley Creek confirmed that the gauge is relatively independent of the source of the flows. During larger events, flow breaks out of both Lockyer Creek and Laidley Creek, including areas upstream of the gauge site, and spills into the lower Lockyer floodplain. However, the breakout is a function of the capacity of the creek channel in the vicinity of the gauge. Thus, although only a proportion of the flow actually passes the gauge site, the gauge level still exhibits a response that can be related to the total creek flow. The rating curve is therefore considered to provide a reasonable estimate of the combined Lockyer Creek and Laidley Creek flow, but it is very sensitive to changes in level at high flows; small changes (or errors) in water level potentially represent large changes in flow.

Figure 5.1 Flow patterns around the Glenore Grove gauge site for low and high flows

Three stream gauges are located downstream of Glenore Grove; at Lyons Bridge; Rifle Range Road; and O'Reilly's Weir. O'Reilly's Weir is near the confluence with the Brisbane River and is strongly influenced by backwater during Brisbane River floods. The BRCFS did not investigate this site in any detail (since it was interested primarily in Brisbane River flood events). The other two gauges have reliable ratings based on numerous instream flow measurements, but due to the perched nature of the lower Lockyer Creek channel can only reliably record in-stream flows. Significant flows can bypass the gauge locations at Lyons Bridge and Rifle Range Road without being registered by the stream gauges.

The most reliable rating in the Lockyer Creek catchment in terms of flow measurement is located on Laidley Creek at the Warrego Highway (143229a). This site has stream flow measurements up to 985 cubic metres per second (m^3/s), which is over 70% of the highest recorded flow (this is a very high ratio for most stream gauges). Unfortunately, Laidley Creek represents only 16% of the overall Lockyer Creek catchment and the stream gauge site is potentially affected by backwater from Lockyer Creek.

Two stream gauges are located in relatively close proximity in the Gatton area. The flood warning gauge operated by the BoM at Gatton has isolated flood peak records dating as far back as 1893. Although it appears to be in a reasonable location, with large flows well contained within the main channel, the site has no official rating and no at-site flow measurements. A rating for the gauge was derived from hydraulic modelling conducted by SKM in 2013 as part of the 'Lockyer Creek Flood Risk Management Study', however the flows used to calibrate this model (and hence derive the rating) are not necessarily consistent with the BRCFS. Since 2000, Seqwater has operated a gauge further upstream at Gatton Weir, although there is limited information available at this site due to the short period of operation.

DNRME has historically operated three separate stream gauges in the upper Lockyer Creek catchment at Helidon, but with some period of overlap. Although the combined records extend back to 1926, review of the data identified issues with the gauge data availability and consistency:

- Helidon No.1 (1926-1971) has only minor flow gauging and exhibits a number of minor drifts in datum
- Helidon No.2 has the highest flow gauging but both level record and flow measurements indicate that a significant datum shift occurred in 1976
- Helidon No.3 (1987-) has limited flow gauging (up to 3.4 m and 110 m^3/s).

The Helidon stream gauge is noteworthy for its record of the 2011 flood event. The gauge record identifies a peak level of 14 m gauge height, nearly double the highest level recorded in the previous 86 years of records. This corresponds to a flow of over 3,000 m^3/s based on extrapolation of the rating curve, 3.4 times larger than the next largest flood (1974). Since the gauge failed during the 2011 flood with the last reliable level of ~11 m, and the projected peak water level is so far above the level to which the rating curve can be confidently be extrapolated from the flow measurement data, the exact magnitude and probability of the flood is subject to significant uncertainty.

5.6 Rainfall data

5.6.1 Gowrie Creek catchment

Twenty-seven daily rainfall and 14 pluviograph rainfall gauging stations exist within 30 km of the centre of Gowrie Creek catchment. Figure A1-B: Hydrology setup shows the location of daily rainfall gauging and pluviograph stations. Details of the rainfall data used for calibration purposes is detailed in Section 7.2.

5.6.2 Lockyer Creek catchment

Rainfall data for all historical events modelled was embedded within the previous BRCFS hydrologic models. This data was adopted for the current historical event investigation. Rainfall data adopted for the Existing Case and Developed Case modelling is outlined in Section 8.1.2.

5.7 Anecdotal flood data

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

- Previous studies as detailed in Section 5.1
- Local councils (TRC and LVRC)
- Landholders and stakeholders.

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 the performance of the hydraulic model to replicate historical flood conditions.

5.8 Site inspection

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

6 Development of models

6.1 Summary

A summary of the modelling approach for each catchment is outlined in Table 6.1. All hydrologic and hydraulic modelling was undertaken with the guidance of a qualified engineer. Validation with historical data was undertaken where available and sensitivity checks were undertaken to test assumptions. The development of these models is outlined in the following sections.

Table 6.1 Hydrologic and hydraulic modelling approach summary

Catchment	Hydrologic modelling approach	Hydraulic modelling approach	Locality plan
Gowrie Creek – the Project runs parallel to Gowrie Creek for 4.8 km and intersects a number of drainage lines/tributaries associated with Gowrie Creek. The greenfield section of the Project crosses a tributary of Gowrie Creek east of Gowrie. The Project also requires the realignment of a local road over Gowrie Creek.	Existing RAFTS model only covers the upstream catchment. A new RAFTS model was developed using parameters from previous studies.	Existing hydraulic models did not cover the entire alignment. A new TUFLOW model was developed and validated against past studies.	Figure A1-A
Gowrie Creek – Intermediate ventilation shaft – Gowrie Creek and its tributary pass over the tunnel alignment near the proposed intermediate ventilation shaft at Cranley.	The Gowrie Creek RAFTS model was used to estimate flow hydrographs.	A new TUFLOW model was developed and validated against past studies.	Figure B1-A
Oaky Creek – passes under a viaduct.	An URBS model was developed and validated against past studies.	A new TUFLOW model was developed and validated against past studies.	Figure C1-A
Six Mile Creek – passes under a viaduct.	An URBS model was developed and flows validated against past studies.	A new TUFLOW model was developed and validated against past studies.	Figure D1-A
Upper Lockyer Creek – passes under a viaduct.	Adopted H2C Project URBS model of Lockyer Creek catchment.	Adopted H2C Project TUFLOW model of Lockyer Creek.	Figure E1-A

Figures relating to each of the catchments/locations are included in the following appendices:

- Appendix A – Gowrie Creek
- Appendix B – Intermediate ventilation shaft
- Appendix C – Oaky Creek
- Appendix D – Six Mile Creek
- Appendix E – Lockyer Creek.

6.2 Hydrologic models

6.2.1 Gowrie Creek

6.2.1.1 Model development

A RAFTS hydrologic model for the Gowrie Creek catchment was developed using catchment details and parameters from previous studies. A review of the two existing hydrologic models of Gowrie Creek is outlined in Table 6.2.

The Gowrie Creek hydrologic model provides inflows for both the hydraulic model covering the alignment from the western tunnel portal westwards to the connection with the B2G project and for the localised hydraulic model around the intermediate ventilation shaft.

Table 6.2 Gowrie Creek hydrologic modelling

Study	Software used	Area of interest	Calibration and validation information	Information used for Project assessment
Toowoomba Regional Council, Gowrie Creek Flood Risk and Management Study Volume 1 (TRC 2013)	RAFTS	This model covers the upstream extents of Gowrie Creek catchment within Toowoomba	The model is validated for 17 December 2010, 27 December 2010 and 10 January 2011 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 (TRC 2014b)	RAFTS	This model covers Cotswold Hills catchment. The focus of the Cotswold Hills study was local flooding in the tributary catchments of Gowrie Creek. This catchment is located south of the Project alignment. This model does not incorporate Gowrie Creek itself.	The model was validated with the Rational Method and is calibrated to the 10 January 2011 event	▪ Delineated catchments and hydrologic model parameters

6.2.1.2 Sub-catchments

The delineation of sub-catchments for the upstream catchment of Gowrie Creek and for 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-catchments, is presented in Figure A1-B.

6.2.1.3 Fraction impervious and roughness

The catchment roughness (PERN) and percentage impervious were based on land use layer from Queensland Globe database and aerial imagery. The Gowrie Creek catchment can be broadly divided into two areas (Area 1 and Area 2) based on land use as shown in Figure A1-B. Area 1 is a mostly urbanised area located in the upstream portion of the catchment and Area 2 is mostly rural and floodplains located in the middle and downstream reaches of the catchment.

Fraction imperviousness of all sub-catchments within Area 1 was defined based on the TRC (2013a) model. Within Area 2, fraction impervious values for the Cotswold Hills sub-catchments were based on the TRC (2013b) model and for the rest of sub-catchments in Area 2, fraction imperviousness was estimated based on land use data downloaded from Queensland Globe (https://qldglobe.information.qld.gov.au/) in April 2018 and GIS data provided by TRC.

6.2.1.4 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 Case TUFLOW model.

6.2.2 Oaky Creek

6.2.2.1 Model development

An URBS hydrological model was developed of the Oaky Creek catchment. The catchment map is presented in Figure C1- B .The URBS parameters for the Oaky Creek model are listed in Table 6.3.

Table 6.3 Oaky Creek URBS model parameters

Parameter	Value
Alpha	0.1
Beta	1

6.2.2.2 Fraction impervious

The URBS modelling was based on the current land use (i.e. primarily forest, bushland and pasture as evidenced from aerial imagery) which is in line with GIS data provided by LRVC. A fraction impervious of zero was therefore used in the modelling.

6.2.3 Six Mile Creek

6.2.3.1 Model development

An URBS hydrological model was developed of Six Mile Creek catchment. The catchment map is presented in Figure D1- B. The URBS parameters for Six Mile Creek model are listed in Table 6.4.

Table 6.4 Six Mile Creek URBS model parameters

Parameter	Value
Alpha	0.1
Beta	0.5

6.2.3.2 Fraction impervious

The URBS modelling was based on the current land use (i.e. primarily forest, bushland and pasture as evidenced from aerial imagery) which is in line with GIS data provided by LRVC. A fraction impervious of zero was therefore used in the modelling. This approach is consistent with the previous Six Mile Creek Flood Assessment Report Study (APB 2017).

6.2.4 Lockyer Creek

For Lockyer Creek the hydrologic modelling from the BRCFS (Aurecon 2015) has been adopted. This modelling was considered to be the most robust and up-to-date and had been recently accepted by LVRC.

The BRCFS undertook a detailed hydrologic assessment of the Brisbane River catchment, followed by hydraulic modelling of the Brisbane River (downstream of Wivenhoe Dam) and lower tributaries. Hydrologic modelling for the BRCFS was undertaken using the URBS software package. The hydrologic models were originally developed by Seqwater but were reviewed and revised as part of the BRCFS in response to changes to the stream gauge ratings and (preliminary) hydraulic modelling of the lower Brisbane River undertaken by Brisbane City Council. Initial development of the models is reported in '*Brisbane River Flood Hydrology Models*' (Seqwater, 2013).

The Brisbane River hydrologic model configuration separates the catchment into seven separate sub-models – the Upper Brisbane (upstream of Wivenhoe), its major tributary Stanley River (upstream of Somerset Dam), the Lower Brisbane, Lockyer Creek, the Bremer River and two of its tributaries, Warrill Creek and Purga Creek, which join upstream of Ipswich. The Lockyer Creek hydrologic sub-model has been used for the current investigation with the model layout in Figure E1-B. Minor modifications were made to the hydrologic model in order to produce flow estimates at locations of interest along the Project alignment.

6.3 Hydraulic models

6.3.1 Gowrie Creek hydraulic model

6.3.1.1 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 A1-C.The 2D model topography was modified to adequately represent the drainage flowpaths and the existing road/rail crest levels.

6.3.1.2 Hydraulic structures

Structure geometry information contained within the previous hydraulic models was used in this assessment. (TRC 2013 and 2014b). Two culverts at Stankes Road and Burkes Road (Gowrie Junction) 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 were identified within the hydraulic model domain with the structures summarised in Table 6.5 and Table 6.6.

Table 6.5 Gowrie Creek – Identified existing structures within the hydraulic model extent

Structure modelling ID	Type	Infrastructure	Upstream invert (m AHD)	Downstream invert (m AHD)	Diameter/ width (m)	Height (m)	Number of cells
COT-02	RCP	Roderick Drive	542.46	542.06	1.2	-	3
KIN-GOJ-25	RCBC	QR Western Line	467.40	467.02	2.8	3.0	3
KIN-GOJ-26	RCBC	QR Western Line	472.60	472.20	3.0	2.1	2
KIN-GOJ-32	RCBC	QR Western Line	481.40	481.05	1.8	1.8	6
KIN-GOJ-23	RCP	QR Western Line	465.90	465.10	0.5	-	1
KIN-GOJ-24	RCP	QR Western Line	469.00	466.70	1.2	-	1
KIN-GOJ-29	RCBC	Gowrie Junction Road Overpass	487.20	486.80	1.2	0.3	4
KIN-35a	RCBC	QR Western Line	502.18	501.40	1.2	0.7	4
KIN-35b	RCP	QR Western Line	501.85	501.40	0.6	-	2
KIN-GOJ-36	RCBC	QR Western Line	504.90	504.70	1.5	1.5	1
KIN-35c	RCBC	East Paulsens Road	500.70	500.40	1.2	1.2	1
KIN_26e	RCP	Paulsens Road	472.75	472.50	0.5	-	1

Structure modelling ID	Type	Infrastructure	Upstream invert (m AHD)	Downstream invert (m AHD)	Diameter/ width (m)	Height (m)	Number of cells
KIN-GOJ-27	RCBC	QR main line	477.50	477.30	1.2	0.6	2
COT-01	RCP	Boundary Street	564.41	562.60	1.8	-	1
KIN22-Draper	RCP	QR Western Line	458.70	458.30	1.0	-	6
KIN-GOJ-21	RCBC	QR Western Line	455.00	454.80	3.0	2.1	5
KIN-GOJ-19	RCBC	Leesons Road	447.33	447.08	2.1	1.1	3
KIN-GOJ-18	RCBC	QR Western Line	448.40	448.10	2.1	1.2	2
KIN-GOJ-3	RCBC	QR Western Line	443.30	443.10	1.2	0.8	2
KIN-GOJ-2	RCBC	QR Western Line	442.80	442.60	1.2	0.9	5
KIN-GOJ-34	RCBC	QR Western Line	497.50	497.10	1.2	0.3	1
KIN-GOJ-28	RCBC	Krenkes Road	481.50	481.20	1.2	0.7	1
Stankes_Road	RCBC	Stankes Road	491.50	491.20	0.5	0.5	1
Burkes_Road	RCBC	Burkes Road	497.70	497.40	0.5	0.5	1

Table 6.6 Gowrie Creek – Identified existing bridges within the hydraulic model extent

Bridge modelling ID	Road name	Bridge length (m)	Obvert level (m AHD)	Deck depth (m)
KIN-GOJ-33	East Paulsens Road	22	492.60	0.60
KIN-GOJ-31-old Homebush	Old Homebush Road	38	478.50	0.70
KIN_GOJ_20	QR Western Line	64	454.25	1.75
KIN_GOJ_1	Kingsthorpe Haden Road	43	435.40	1.10

Details of existing QR structures were obtained from QR via an RFI process and augmented by a local survey. This information will be reviewed in the next stage of design and the flood modelling will be updated as required. The adopted existing structures are detailed in Table 6.7.

Table 6.7 Gowrie Creek – Existing Queensland Rail structures

Approximate Inland Rail Chainage (km)	Existing nearby or downstream QR culverts		
	Number of cells	Diameter or Span (m)	Height (m)
-1.76	5	3	2.1
-1.42	6	0.9	-
-0.25	1	1.6	-
0.11	1	1.2	1.2
0.21	3	3	2.7
1.03	2	3	2.1
1.46	2	1.2	0.6
2.41	6	1.8	1.8

6.3.1.3 Roughness

The hydraulic roughness generally reflects the type of development and ground cover that exists within the hydraulic model extents. The distribution of roughness categories adopted was based on the information supplied in the hydraulic model developed for the Historical study for Kingsthorpe and Gowrie Junction (TRC 2014a), the hydraulic model developed for the 2D Flood study for Cotswold Hills (TRC 2014b), the land use layer from Queensland Globe database, TRC GIS data and aerial imagery.

Specific roughness values applied to the model are detailed in Table 6.8. Figure A1-C shows the spatial discretisation of land use in the 2D model domain.

Table 6.8 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

6.3.1.4 Boundary conditions

The Gowrie Creek RAFTS 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 catchment areas within the hydraulic model extents were applied throughout the TUFLOW model.

Internal inflow boundaries were applied as SA polygons with the flow applied to the lowest point of each SA polygon. The proposed western tunnel portal is located within sub-catchments C7 and C17. The western 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 A1-C).

A normal depth boundary condition was applied at the downstream boundary. Since the downstream boundary is not a well-defined water level, a stage-discharge relationship was used in TUFLOW to define the boundary condition.

6.3.2 Intermediate ventilation shaft hydraulic model

The intermediate ventilation shaft is located near a tributary of Gowrie Creek at Cranley. The site location of the intermediate tunnel shaft is shown in Figure B1-A. The Gowrie Creek hydrologic model setup upstream of the intermediate tunnel shaft, including extent and sub-catchments, is presented in Figure B1-B.

6.3.2.1 Model setup

The intermediate tunnel shaft hydraulic model was set up on a 2 m grid spacing and developed in TUFLOW HPC. The TUFLOW model set up is presented in Figure B1-C.

6.3.2.2 Hydraulic structures

Two culverts are located under QR’s Western Line rail corridor within the vicinity of the intermediate ventilation shaft at Cranley, being:

- One rail culvert (1/3.0 x 2.7 RCBC) located at the tributary of Gowrie Creek
- A second rail culvert is located near Wetalla Wastewater Treatment Plant. The outflow from the second culvert is conveyed via a culvert under Wetalla Wastewater Treatment Plant into Gowrie Creek. The dimension of this culvert was assumed as 1.2 m x 0.9 m RCBC.

In addition to these structures, a road culvert (3/3.6 m x 3.6 m RCBC) is located at the Gowrie Creek crossing under the Wetalla Wastewater Treatment Plant access road. The dimensions of this culvert were taken from TRC’s Gowrie Creek hydraulic model (TRC 2013a).

6.3.2.3 Roughness

The hydraulic roughness generally reflects the types of development and ground cover that exists within the hydraulic model extents. The distribution of roughness categories adopted for this study was based on the information supplied in the (TRC 2014a) model, (TRC 2014b) model, land use layer from Queensland Globe database, TRC GIS data and aerial imagery. Specific roughness values applied to the model are detailed in Table 6.9. Figure B1-D shows the spatial discretisation of land use in the 2D model domain.

Table 6.9 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

6.3.2.4 Boundary conditions

The Gowrie Creek RAFTS 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 catchment areas within hydraulic model extents were applied throughout the TUFLOW model.

A normal depth boundary condition was applied at the downstream boundary. Since the downstream boundary is not a well-defined water level, a stage-discharge relationship was used in TUFLOW to define the boundary condition.

6.3.3 Oaky Creek hydraulic model

6.3.3.1 Model setup

A two-dimensional hydraulic model was developed using the TUFLOW software to simulate the flood behaviour. The Oaky Creek hydraulic model was set up on a 2 m grid and run in TUFLOW Classic. The TUFLOW model set up is presented in Figure C1- C.

6.3.3.2 Hydraulic structures

No existing local structure information was incorporated into the hydraulic model.

6.3.3.3 Roughness

The hydraulic roughness generally reflects the types of development and ground cover that exists within the hydraulic model extent. As no previous study information or calibration data was available the roughness parameters selected were consistent with the Six Mile Creek catchment. Table 6.10 provides the Manning's n values used in the modelling, while Figure C1- C shows the spatial discretisation of land use in the 2D model domain.

Table 6.10 Manning's n values

Land use	Manning's n
Dense vegetation	0.120
Road and road corridor	0.030
Creek	0.080
Pasture	0.045

6.3.3.4 Boundary conditions

The Oaky Creek URBS model outputs were applied as inflows into the TUFLOW model. A total inflow was used at the upstream end of the model. A normal depth boundary condition was applied at the downstream boundary. Since the downstream boundary is not a well-defined water level, a stage-discharge relationship was used in TUFLOW to define the boundary condition.

6.3.4 Six Mile Creek hydraulic model

6.3.4.1 Model setup

A two-dimensional hydraulic model was developed using the TUFLOW software to simulate the flood behaviour. The Six Mile Creek hydraulic model was set up on a 2 m grid and run in TUFLOW HPC. The TUFLOW model set up is presented in Figure D1- C.

6.3.4.2 Hydraulic structures

No existing local structure information was incorporated into the hydraulic model.

6.3.4.3 Roughness

The hydraulic roughness reflects the types of development and ground cover that exists within the hydraulic model extent. The roughness parameters selected were consistent with the Six Mile Creek – Flood Assessment Report (APB 2016).

Table 6.11 provides the Manning's n values used in the modelling, while Figure D1- C shows the spatial discretisation of land use in the 2D model domain.

Table 6.11 Manning's n values

Land use	Manning's n
Dense vegetation	0.120
Road and road corridor	0.030
Creek	0.080
Pasture	0.045

6.3.4.4 Boundary conditions

The Six Mile Creek URBS model outputs were applied as inflows into the TUFLOW model. A total inflow was used at the upstream end of the model. A normal depth boundary condition was applied at the downstream boundary. Since the downstream boundary is not a well-defined water level, a stage-discharge relationship was used in TUFLOW to define the boundary condition.

6.3.5 Lockyer Creek

6.3.5.1 Model setup and resolution

The LVRC hydraulic model previously updated by Jacobs (2016) and provided to Aurecon was a TUFLOW nested-grid model. The nested-grid model contained eight separate sub-model areas with varying degrees of terrain resolution; ranging from 40 m to 5 m. This model was converted into a single-area model and a comparison between the terrain resolutions is presented in Table 6.12.

Table 6.12 Lockyer Creek hydraulic model areas and terrain resolutions

Previous Model Area	LVRC model resolution (m)	Adopted model resolution (m)
Upper Lockyer Creek: Withcott to Gatton	20	10
Upper Lockyer Creek: Gatton to Lake Clarendon	20	10
Laidley Creek and Sandy Creek surrounding Forest Hill	20	10
Laidley Creek North	10	10
Laidley Creek South	20	10
Lower Lockyer Creek	40	10
Forest Hill Township	5	10

Along with the consistent model resolution, the hydraulic model was changed to run with the TUFLOW HPC software. The hydraulic model has been reviewed for stability. The cumulative mass error is recorded as 0% from the model log, indicating the model is not gaining or losing water through the simulation. The water levels and flows have been plotted for culverts (one dimensional structures) to check for any peak instabilities that may affect the results. The hydraulic model was determined appropriate for use. The TUFLOW HPC model and the TUFLOW nested-grid model and were compared around the Project footprint and determined to be sufficiently consistent. The adopted hydraulic model layout is presented in Figure E1- C.

6.3.5.2 Hydraulic structures

The majority of hydraulic structures were maintained from the LVRC base hydraulic model (Jacobs 2016). One existing culvert within the H2C portion of the hydraulic model around Ch 49.56 km which was confirmed to be a bridge and details were sourced from ground survey. Additional existing structures were added to the hydraulic model where identified in the ground survey. Hydraulic structures were modelled as outlined in Table 6.13.

Table 6.13 Model representation of hydraulic structures – Lockyer Creek

Hydraulic structure	Model representation
Culvert	1-Dimensional structure
Bridges	2-Dimensional layered flow constriction
Longitudinal drainage	2-Dimensional channels

Existing QR Main Line structure details near Lockyer Creek were sourced from a field survey. For the G2H portion of the Lockyer Creek hydraulic model this included one drainage structure as detailed in Table 6.14

Table 6.14 Lockyer Creek – Existing Queensland Rail structures

Approximate Inland Rail Chainage (km)	Existing nearby or downstream QR culverts		
	No of cells	Diameter or Span (m)	Height (m)
25.94	1	1.35	-

6.3.5.3 Boundary conditions

The BRCFS URBS model outputs were applied as inflows into the TUFLOW 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 TUFLOW hydraulic model covers a significant proportion of the middle of the Lockyer Creek catchment. It uses inflows taken from the URBS hydrologic model as both total channel flows at creek inflows at the hydraulic model boundary and local sub-catchment flows at points within the model boundary. Initial comparisons of the URBS hydrologic routing and TUFLOW hydraulic routing identified that the TUFLOW flows tended to lag the URBS flows. This trend was also identified in the Jacobs (2016) study. The sub-catchment hydrographs that are input into TUFLOW include attenuation and lag due to local catchment storage routing from URBS. Because a real catchment does not have a distinct interface between sub-catchment and main-stream routing, this carries the risk of double-counting storage in the lower sub-catchment tributaries. It was found that by reducing the sub-catchment lag parameter, β, improved the match between the two models. Note that this modification is applied to the inflows within the TUFLOW model domain, not the calibrated URBS model. Table 6.15 shows the adopted lag parameters for hydraulic model inflows.

Table 6.15 Adopted catchment routing lag parameters for the Lockyer Creek hydraulic model

Hydraulic model inflow location	Adopted sub-catchment lag parameter (β)
Hydraulic model boundary	3.1
Sub-catchments within hydraulic model	1.5

A normal depth boundary condition was applied at the downstream boundary. It was confirmed the downstream boundary is sufficiently downstream as to not influence the hydraulic model results in the vicinity of the Project alignment.

7 Joint calibration

7.1 Introduction

The hydraulic models developed generally cover the mid to lower portion of the hydrologic models. Routing and attenuation of the hydrologic model is therefore partially replicated within the hydraulic model. The hydraulic model inflows therefore consist of total reach flows where the hydraulic model boundary intersects any major tributary (more than one upstream catchment) and local sub-catchment flows where the catchment centroid lies inside the hydraulic model boundary.

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

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

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

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

The historical events were selected to represent a range of magnitudes and durations. A summary of the calibration process on Gowrie Creek and Lockyer Creek (tributaries include Oaky Creek and Six Mile Creek) is provided in the following sections.

7.2 Gowrie Creek

7.2.1 Hydrologic model calibration

The RAFTS hydrologic model developed for the Gowrie Creek catchment was calibrated against the following historical events:

- 27 December 2010
- 10 January 2011.

Rainfall data in the upper catchment showed that the 2011 event was between a 1% and a 1 in 500 AEP event magnitude. The 2010 event was estimated to be approximately a 5% AEP magnitude. In the vicinity of the Project, the 2011 event is the largest event on record.

Daily rainfall and pluviograph data were available for a number of rainfall stations for these two historical events as shown in Table 7.1 and Table 7.2. Observed stream flow gauge data at two stations (Oakey and Cranley stream gauges) was available for the 27 December 2010 event. However, for the 10 January 2011 event, observed flow records at Oakey stream gauge were not reliable near the peak of the flood event.

Table 7.1 Gowrie Creek – 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
Mount 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
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
Mount 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 7.2 Gowrie Creek – 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)

Station name	Station number	Easting	Northing	Owner
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)

7.2.1.1 December 2010 calibration event

Adopted IL values for the pervious area were minimal as the catchment received substantial rain in the two weeks preceding the event that saturated the catchment. The loss parameters that were used in calibration are outlined in Table 7.3. Note that the modelling parameters were spilt into two areas to be consistent with past modelling as outlined in Section 8.1.4.1.

Table 7.3 Rainfall loss model used for 27 December 2010 event calibration

Location	Area type	IL (mm)	CL (mm/hr)
Area 1	Old urban Impervious	8.0	4.0
	New urban Impervious	1.5	0.0
	Pervious Upstream (US)	20.0	4.0
Area 2	New urban Impervious	1.5	0.0
	Pervious Downstream (DS)	15.0	2.5

Figure 7.1 and Figure 7.2 present plots of observed and simulated flow hydrographs for the 2010 event at Cranley and Oakey stream gauges respectively. There is a good fit between observed and simulated discharge hydrographs at both stations. The total runoff volume and timing of the peak correspond reasonably well with observed peaks. The difference in peak discharge is summarised in Table 7.4. There is a good match between observed and simulated results with the difference between peak flows being less than 6%.

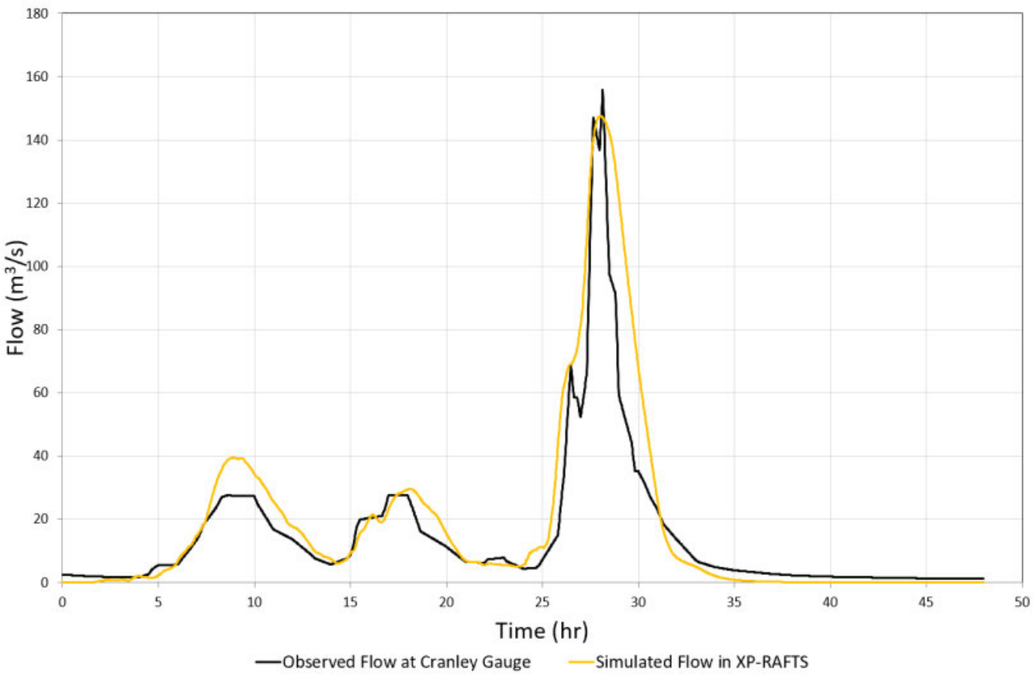


Figure 7.1 **Comparison of gauged and modelled hydrographs at Cranley gauge – 2010 event**

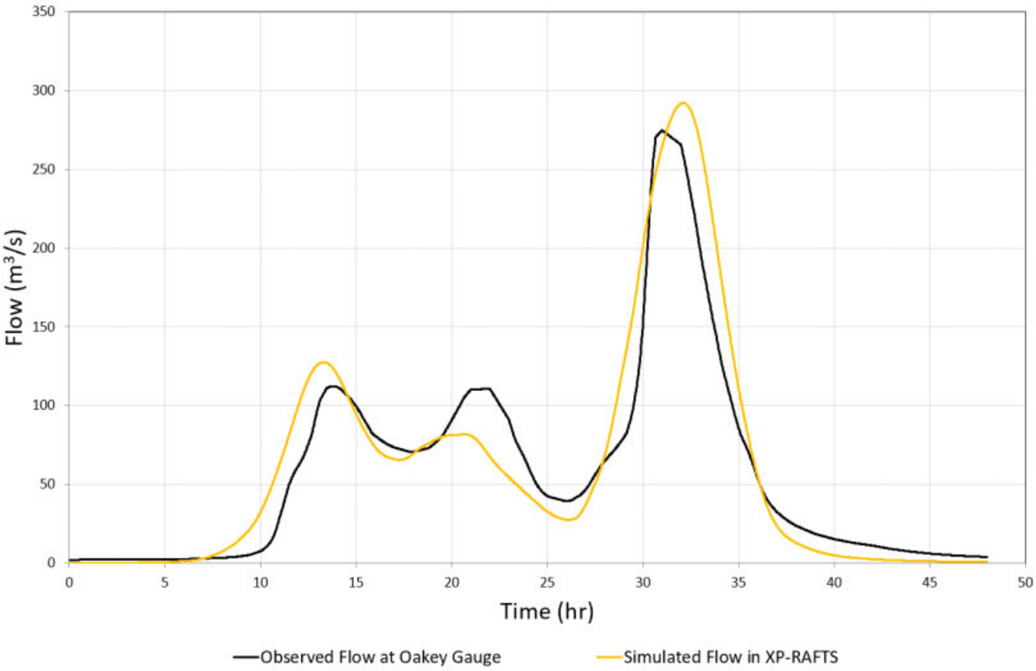


Figure 7.2 **Comparison of gauged and modelled hydrographs at Oakey gauge – 2010 event**

Table 7.4 **Comparison of observed and simulated peak flows at stream gauges for 2010 calibration event**

Stream gauge	Observed Peak flow (m^3/s)	RAFTS Peak flow (m^3/s)	% Difference between observed and simulated
Oakey	275	292	+5.8%
Cranley	156	148	-5.4%

7.2.1.2 January 2011 calibration event

The loss parameters that were used in the 2011 calibration are outlined in Table 7.5. A larger IL was used in this model in comparison with previous studies as in this current investigation, the January 2011 event is simulated as a three-day event while in previous studies it was simulated as a shorter event.

Routing parameter (K) was modified to match the peak flow values and the continuing losses (CL) 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 (IL) values for the pervious area are small as the catchment received significant rain in the two weeks preceding the event.

Table 7.5 Rainfall loss model used for January 2011 calibration event

Location	Area type	IL (mm)	CL (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 7.3 shows the plot of simulated hydrograph against observed flow at the Cranley gauge for the 2011 event. The three peaks match relatively well with regards to timing. The difference between simulated and observed major peak flow is less than 1%. This indicates the model has predicted the peak and timing of observed event very well.

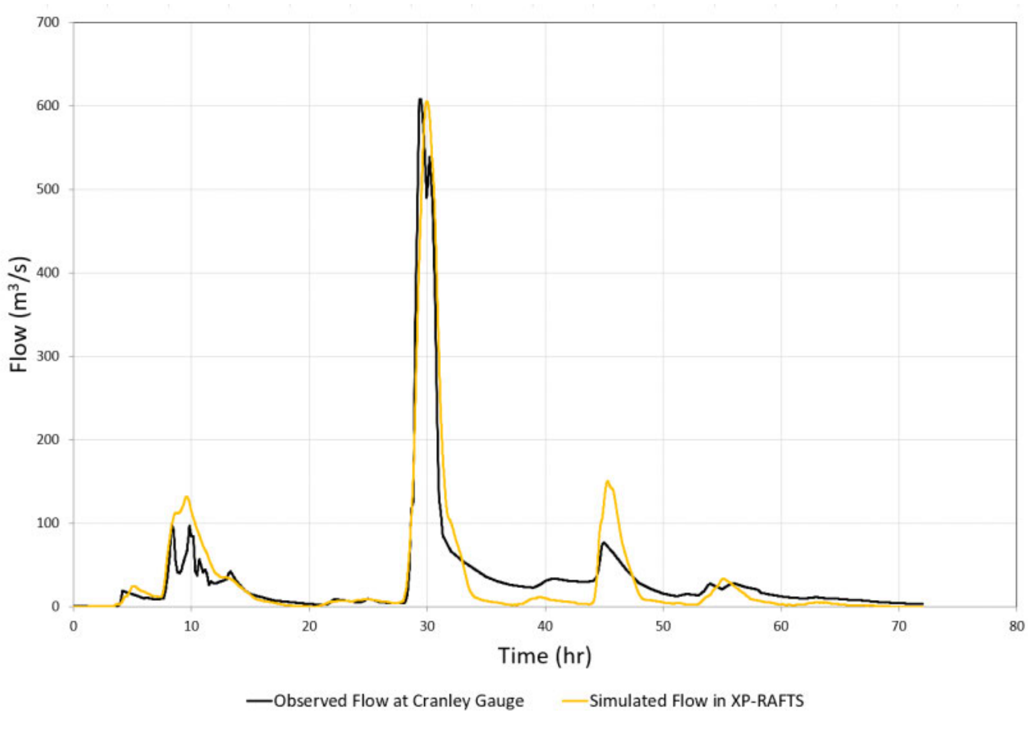


Figure 7.3 Comparison of gauged and modelled hydrographs at Cranley gauge – 2011 event

Figure 7.4 presents a plot of the simulated hydrograph against observed flow at the Oakey stream gauge for the 2011 event. The two smaller peaks have been simulated well in terms of timing and this indicated that the routing parameters used in the model are reasonable. The rising limb of the major peak and timing is simulated well against the observed data. However, the simulated major peak flow is significantly larger than the observed peak flow. The recession of simulated hydrograph also matches well with the observed data.

From review of the recorded data it is evident that there was a problem with the stream gauge at the peak of the flood event. The agreement with the smaller peaks suggests that it is a high stage rating issue. The previous study (TRC 2014a) also identified uncertainties in gauging rating curve.

The hydrologic model calibration gives reasonable confidence that the hydrologic model can be used for the design event simulations and refinement of the Project design.

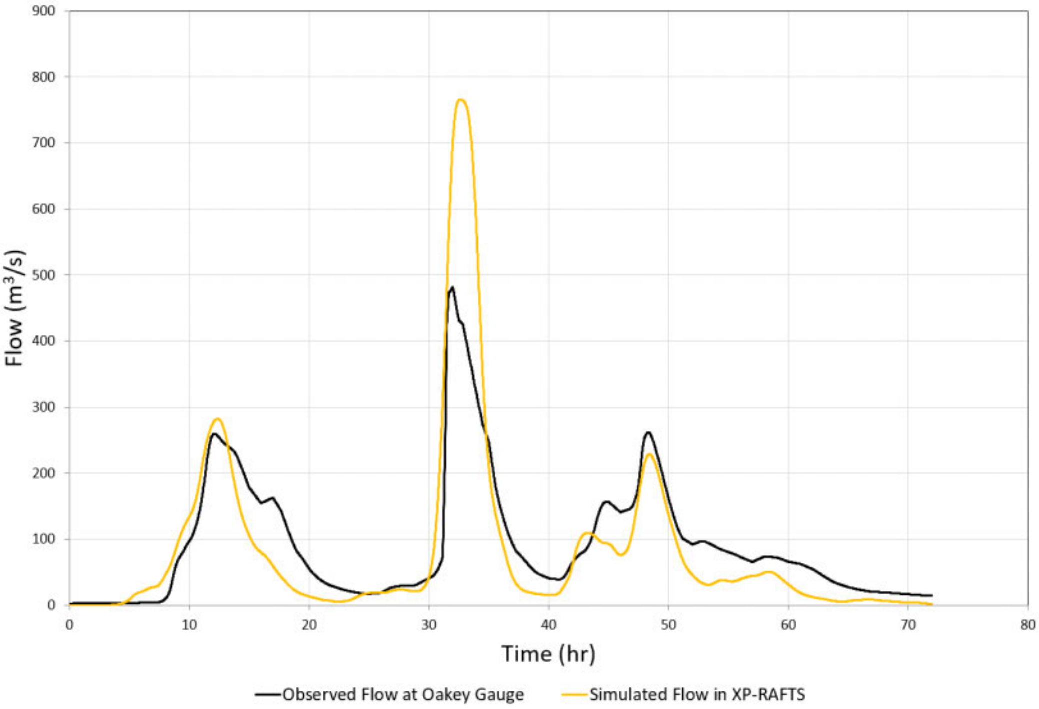


Figure 7.4 **Comparison of gauged and modelled hydrographs at Oakey gauge – 2011 event**

Table 7.6 **Comparison of observed and simulated peak flows at Cranley and Oakey stream gauges for 2011 calibration event**

Stream gauge*	Observed Peak flow (m^3/s)	Simulated Peak flow (m^3/s)	% Difference between observed and simulated
Cranley	609	605	-0.7%
Oakey	-*	750	-

Table note:

* Issue with Oakey stream gauge at peak in 2011 event

7.2.2 Hydraulic model calibration

7.2.2.1 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 7.2.1.1.

Observed flood markers were surveyed after the January 2011 event by TRC as presented in TRC (2014a and 2014b) studies. A total of 11 flood markers were available within the extent of the hydraulic model as shown in Figure A2-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 stream flow 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 7.7 provides a comparison of the modelled flood levels and observed levels for debris marks which were sourced as detailed in Section 5.7.

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 7.7 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*
3	480.17	480.33	+0.15
4	482.61	482.91	+0.30
5	482.80	482.54	-0.25*
6	492.91	492.97	+0.06
7	492.53	492.65	+0.12
8	519.02	518.84	-0.18*
9	518.75	518.73	-0.02*
10	450.17	450.16	-0.01*
11	455.88	455.58	-0.30*

Table note:

* Nearest wet location reported

7.3 Lockyer Creek

7.3.1 Hydrologic model calibration

Detailed calibration of the Lockyer Creek URBS hydrologic model (including Oaky and Six Mile creeks) was undertaken for the BRCFS. This hydrologic model was adopted for the current investigation with minimal changes. No additional calibration of the hydrologic model was undertaken however a review of the previous calibration was undertaken as detailed in the following sections.

7.3.2 Review of BRCFS hydrologic investigation

The hydrologic models developed and calibrated by Seqwater were revised and recalibrated as part of the BRCFS. The recalibration process focussed initially on five flood events: January 1974, May 1996, February 1999, January 2011 and January 2013. These events were selected as they represent moderate to major floods and they also contain the best recent records in terms of spatial and temporal rainfall and stream flow information. The calibration parameters were then validated against a further 43 historical flood events (33 events from between 1955 and 2013 and 10 older events dating back to 1887). Events prior to 1955 have limited pluivograph data and so the temporal representation of these events is generally less reliable.

Parameters derived from the calibration/validation process are listed in Table 7.8. Model results using these parameters were compared across the full range of verification events, generally showing a good correlation between calculated and rated peak flow rates and event volumes with no obvious flow rate related bias at all the examined flow gauges.

Table 7.8 Tributary sub-model adopted parameters

Sub-catchment	Alpha	Beta	m	n
Lockyer Creek	0.49	3.1	0.8	0.85
Bremer River	0.79	2.8	0.8	0.85
Warrill Creek	0.79	2.5	0.8	0.85
Purga Creek	0.93	3.8	0.8	0.85

For each of the tributary hydrologic sub-models, the calibration process focussed on achieving a good match of the flow hydrograph at the primary calibration gauge site (refer Section 5.5.4), typically at or near the downstream end of the catchment.

For calibration events, losses can act to make up for inaccuracies in the rainfall data. The calibration rainfall data are recorded at isolated stream gauge sites and then interpolated across the catchment. If the rainfall was concentrated around the stream gauge site, therefore leading to an overestimate of the actual rainfall across the catchment, this can be compensated for by increasing losses, and vice versa. Forty-eight historical rainfall/flood events were simulated during the BRCFS to calibrate/validate the hydrologic models. The median initial and continuing losses are shown in Figure 7.5 and Figure 7.6 respectively. The 25th and 75th percentile losses are shown to give an indication of variability.

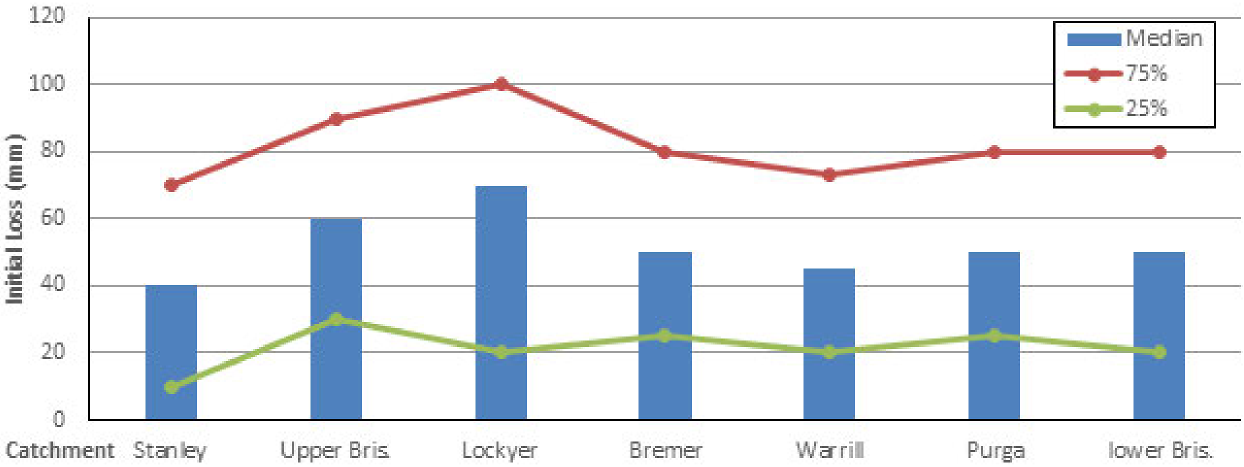


Figure 7.5 BRCFS calibration events median initial losses

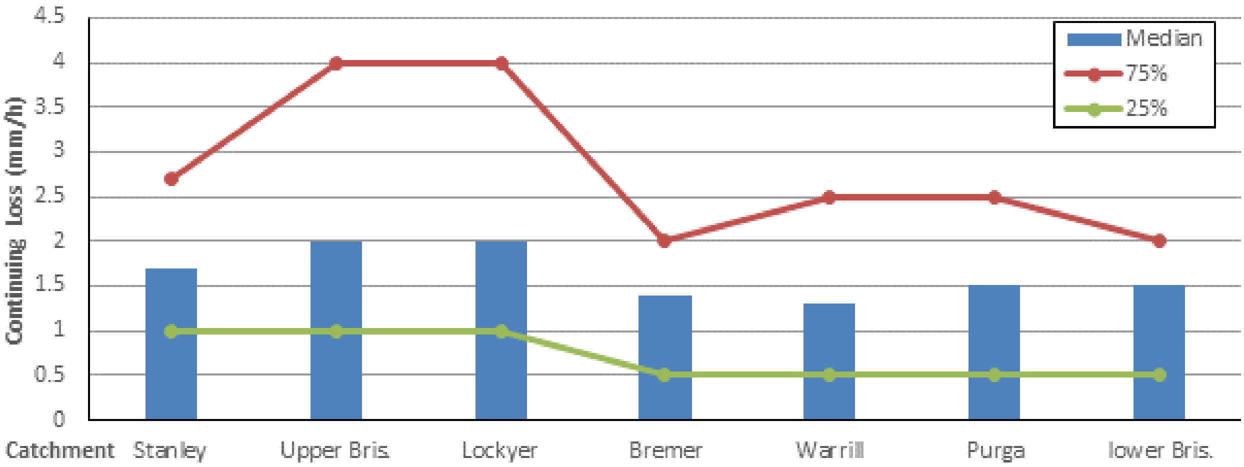


Figure 7.6 BRCFS calibration events median continuing losses

7.3.3 Lockyer Creek joint calibration

The Lockyer Creek hydraulic model encompasses a large portion of the catchment from upstream of Helidon to downstream of the Warrego Highway. The complexity of the creek system meant that calibration had to be looked at holistically across the catchment rather than just in localised areas and this process is what has been documented.

The primary calibration parameter for the hydraulic model is the hydraulic roughness, represented in TUFLOW hydraulic model as a Manning's roughness coefficient, *n*. Calibration of the Lockyer Creek hydraulic model involved:

- Comparison of the TUFLOW hydraulic model prediction of the relationship between level and flow with stream gauge ratings. A detailed review of the stream gauge ratings was undertaken for a number of key stream gauges in the Lockyer Creek, which provided a relationship between observed flows and levels that were consistent.
- Comparison of TUFLOW hydraulic model level and flow hydrographs for the calibration events to confirm that they match both the shape and timing of observed flow
- Comparison of TUFLOW hydraulic model levels with anecdotal flood level data from relevant local councils and stakeholders.

Initial estimates for roughness were based on the previous nested LVRC TUFLOW hydraulic model (Jacobs 2016). These values were then refined within the hydraulic model to achieve the desired relationship between flow and level through the model calibration process. Refined roughness values fall within the ranges outlined in Table 7.9 and are indicative of the conditions present in each waterway. It should be noted that, as with the ratings, these values are understood to be indicative of typical creek/catchment conditions and may be different during any individual flood event.

Table 7.9 Lockyer Creek Manning's roughness coefficients adopted for the TUFLOW models

Land use	Manning's n
Roads and paved areas	0.025 to 0.030
Waterbodies/farm dams etc	0.025 to 0.045
Channels and low vegetated creeks	0.045 to 0.060
Low-medium vegetation	0.045 to 0.070
Medium vegetated creeks	0.060 to 0.080
Riparian and dense vegetation	0.080 to 0.110
Demolished buildings	0.030 to 1.000
Farmland, pasture and crops	0.050 to 1.000
Urbanised areas	0.090 to 0.500
Fences	1.200
Buildings	4.000

It is difficult to define a specific hydraulic roughness for Lockyer Creek with certainty. The creek has a large main channel, varying in depth from approximately 14 m at Glenore Grove to over 18 m at Gatton. Due to the intermittent nature of flows and floods in the creek, the channel appears to have a relatively rough invert in terms of both elevation and vegetation. Aerial and local photography shows areas with ponding and clear of vegetation, while other areas are covered in trees and bushes. Similarly, the channel banks vary between grassed and heavy vegetation, often within a short distance. These areas are to some degree influenced by a series of low-level recharge weirs along the creek. Catchment conditions change with time and this may influence flood behaviour. For the purposes of consistency, the same Manning's roughness coefficient has been used for design and historical event modelling, i.e. conditions present in the channel during the calibration events should be representative of the conditions during future floods and therefore are appropriate for the design event modelling.

The roughness and stream conditions are also likely to vary historically. The area around Gatton where in particular, very high flows are fully contained within the creek channel, was subject to significant vegetation loss and scour during the major floods in 2011 and 2013, as shown in Figure 7.7. It should be noted that, even without the vegetation, the channel is not hydraulically 'smooth', as small to mid-scale irregularities in the channel section and the significant large-scale meandering of the channel are accounted for in the Manning's roughness parameters.

Figure 7.7 **Lockyer Creek channel condition at Gatton (a) 2009 and (b) 2014 (Google StreetView)**

Roughness parameters for the Lockyer Creek hydraulic model were determined by comparing the model level-depth relationships with the ratings at the stream gauge sites within the model limits, while also ensuring that flow velocities are reflective of travel speed through the catchment, as demonstrated by validation to historical floods.

Figure 7.8 shows the relationship between level and flow at Glenore Grove for the 2011 flood event. Glenore Grove was the primary stream gauge site used to calibrate the Lockyer Creek URBS hydrologic model. The rating curve is generally considered to be reliable up to bank-full flow, which is approximately 1,000 m³/s. Larger flows overflow away from the channel, resulting in a very flat rating curve sensitive to changes in water level. Despite being located at a complex junction of Lockyer Creek and Laidley Creek, the hydraulic model shows good agreement with the BRCFS rating relationship.

Similar rating curves for Gatton and Gatton Weir are provided in Figure 7.9. Both rating curves are based primarily on a best-fit of observed peak level and hydrologic model estimates of the peak flow for several historical flood events. Neither rating curve is considered to be particularly reliable, but nevertheless should give an indication of the expected flood levels corresponding to a modelled flow. The hydraulic model appears to underestimate low level floods, (in the range of 90 to 95 m AHD) at the Gatton stream gauge. It has been identified that the current hydraulic model does not include the low-level Smithfield Road crossing, located approximately 400 m downstream of the Gatton stream gauge location, which may contribute to this discrepancy. Otherwise, the match is considered to be reasonable given the uncertainty in the stream gauge ratings.

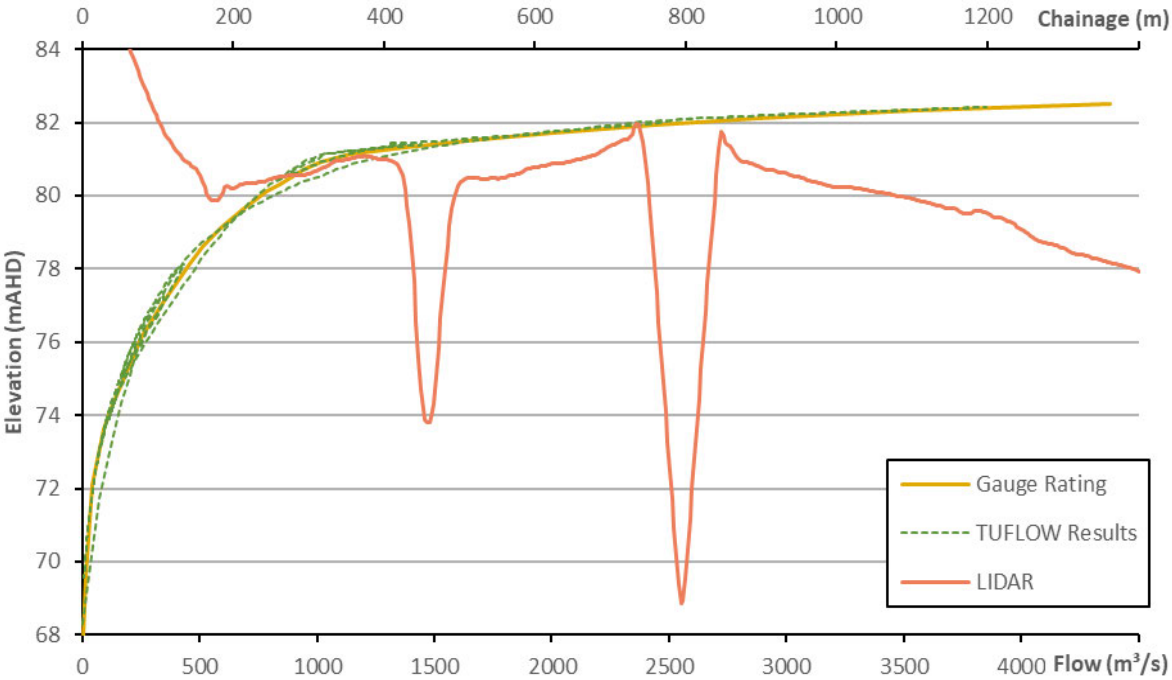
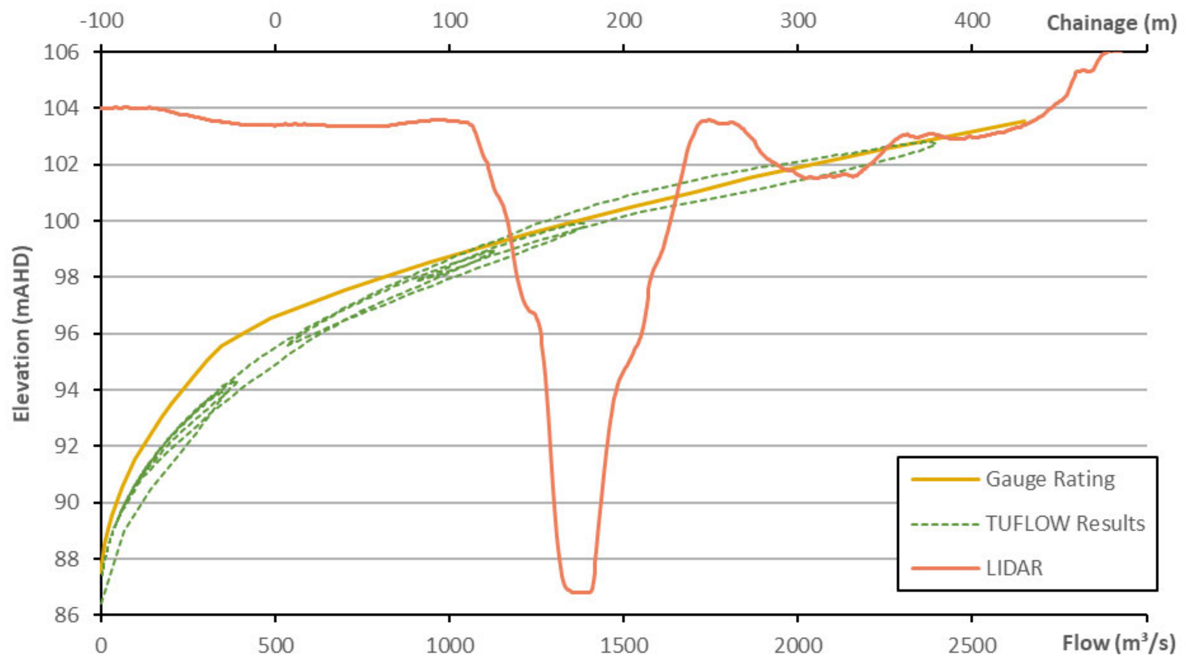


Figure 7.8 **Comparison of hydraulic model level-depth relationship with Glenore Grove stream gauge rating**



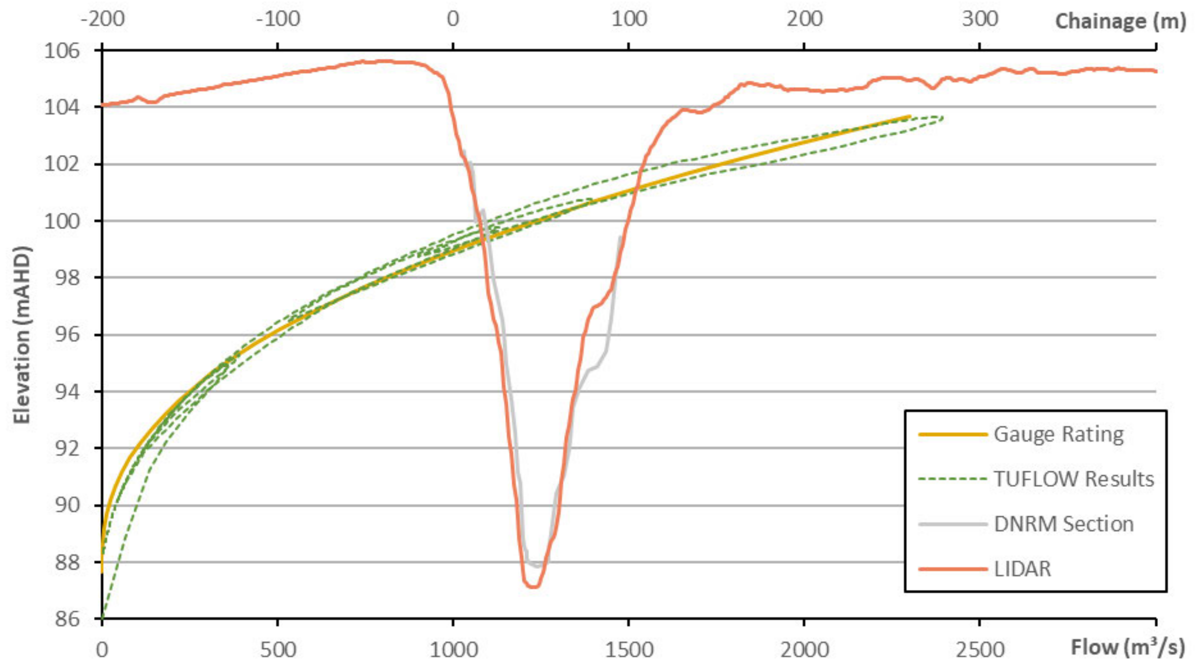


Figure 7.9 Comparison of hydraulic model with Gatton (Top) and Gatton Weir (Bottom) stream gauge ratings

The Warrego Highway stream gauge provides information on flows in Laidley Creek, the major tributary that joins Lockyer Creek at Glenore Grove. The gauge rating is theoretically reliable, being based on measured flow data up to nearly 1,000 m³/s, however no additional review/improvement was undertaken during the BRCFS. The relationship between level and flow at Warrego Highway for the 2011 flood hydrograph, shown in Figure 7.10, identifies significant hysteresis (floodplain storage effects that result in the level and flow having different relationships on the rising and falling limbs of the flood) for higher flows. While there is potentially some backwater effect from Lockyer Creek, which is less than 5 km downstream, these effects are also likely the product of a wide floodplain constrained by the Warrego Highway. Therefore, although the hydraulic model generally shows good agreement with the stream flow measurements, it suggests that there may be some uncertainty in the stream gauged flows. Notably, the hydraulic model rating curve deviates from the stream gauge rating curve above ~1,000 m³/s.

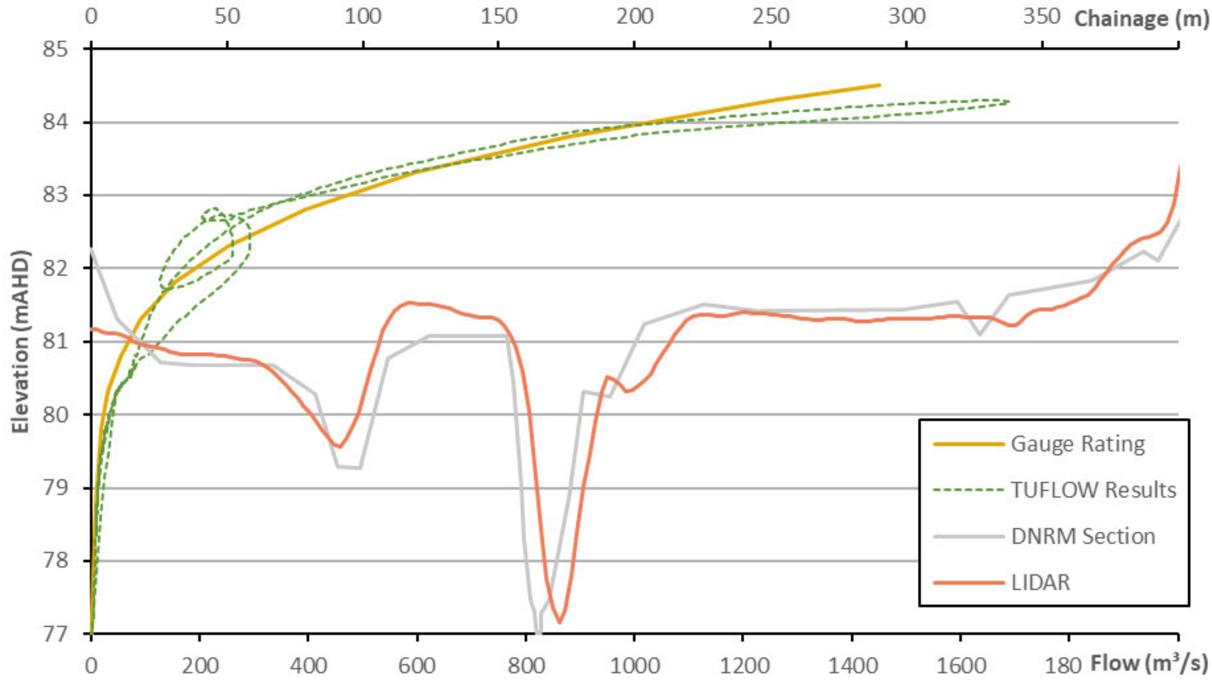


Figure 7.10 **Comparison of hydraulic model level-depth relationship with Warrego Highway stream gauge rating**

The Helidon stream gauge is located near the upstream boundary of the hydraulic model. The hydraulic model results, shown in Figure 7.11, match very closely the BRCFS rating. Several other stream gauges are located in the catchment but are located on minor streams or have limited or unreliable calibration data and have not been subjected to detailed assessment.

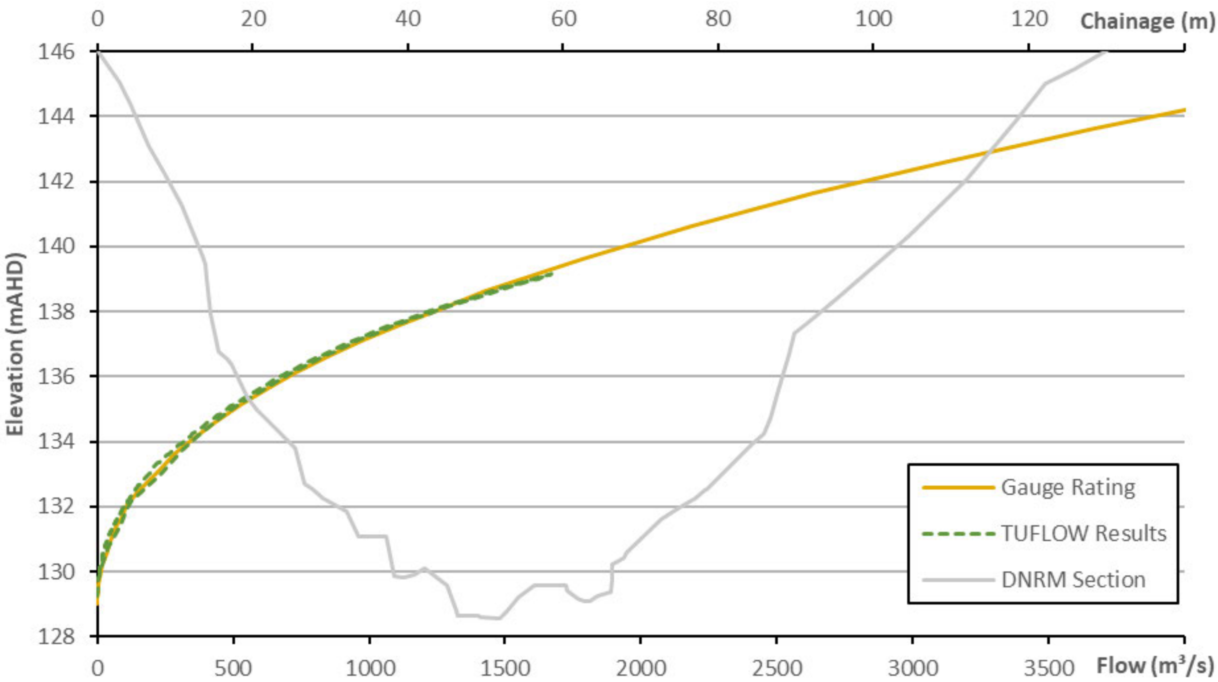


Figure 7.11 **Comparison of hydraulic model level-depth relationship with the Helidon stream gauge rating**

7.3.3.1 January 1974 flood event

The January 1974 flood was a major flood event affecting much of the Brisbane River catchment. It was (and for much of the catchment still is) the largest flood since 1893. Unfortunately, limited historical information is available for the Lockyer Creek catchment for both stream gauge and rainfall data. Significant variation in rainfall depth was recorded across the catchment with depths in excess of 600 mm recorded in the Laidley Creek catchment upstream of Mulgowie but less than 250 mm registered across much of the central catchment around Tenthill. Only 24-hour rainfall totals are available across most of the catchment.

Comparisons of the flow hydrographs for the 1974 flood event produced by the URBS hydrologic model and TUFLOW hydraulic model at Gatton and Glenore Grove are provided in Figure 7.12 and Figure 7.13 respectively. A good match of the hydrograph shape and timing is achieved at Gatton, however, the match at Glenore Grove was predominantly focused on the rising limb as the gauge failed on 27 January 1974.

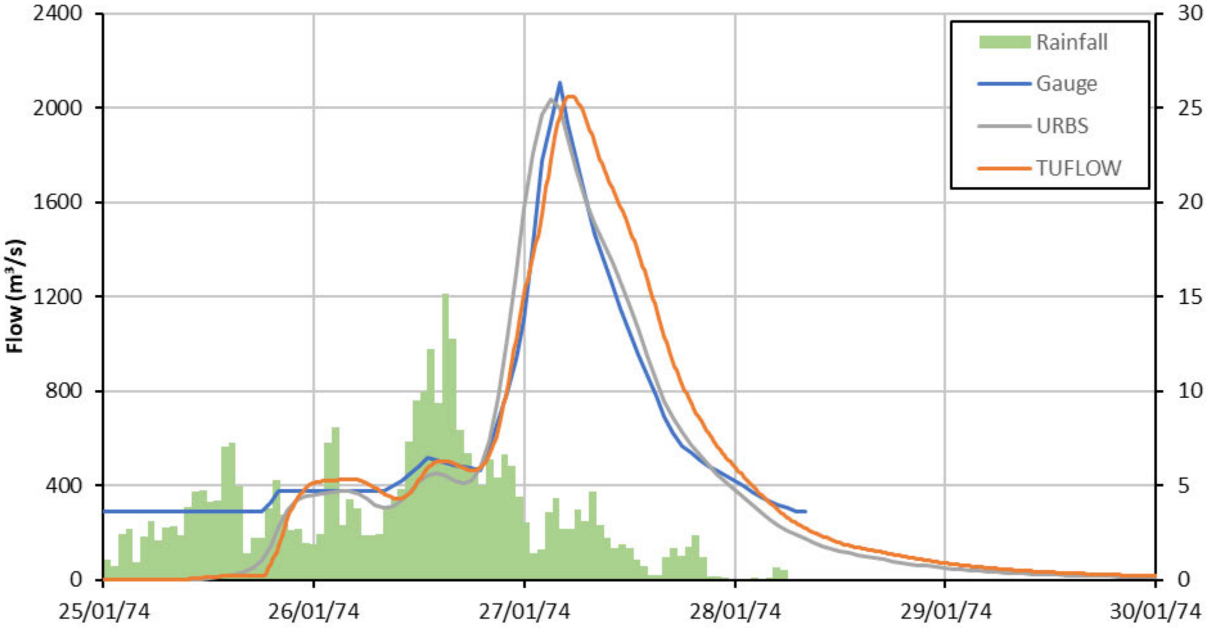


Figure 7.12 Modelled and rated flow hydrographs at Gatton for the January 1974 flood

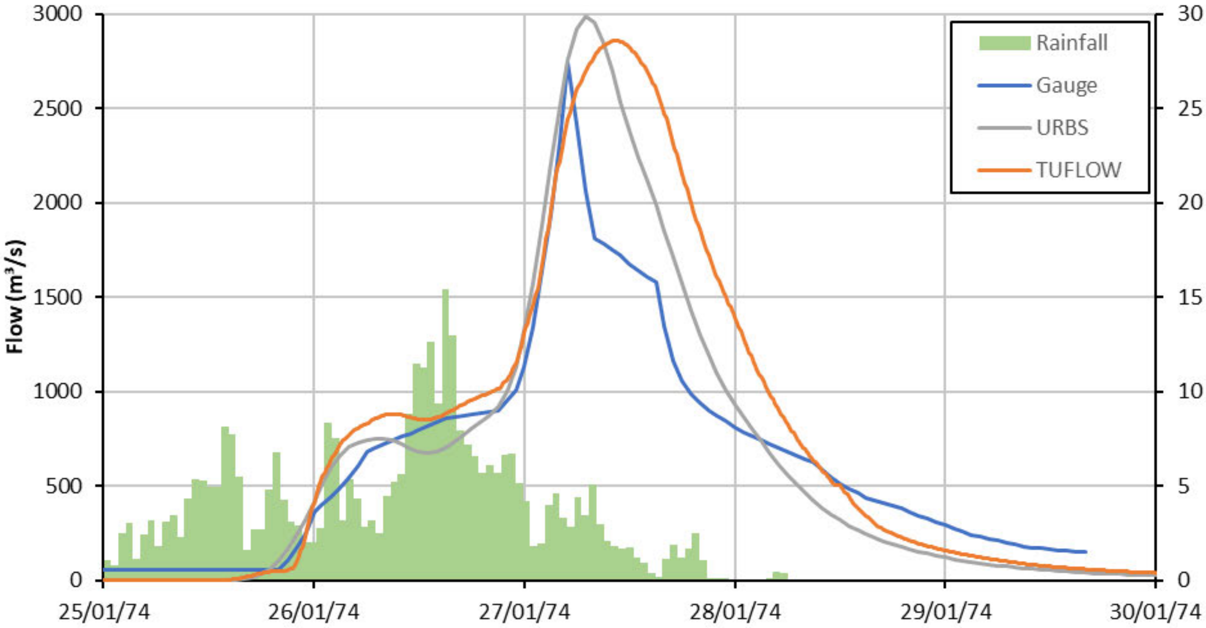


Figure 7.13 Modelled and rated flow hydrographs at Glenore Grove for the January 1974 flood

The modelled hydrographs at Helidon, shown in Figure 7.14, tend to underestimate the peak flow (this could potentially be improved by modifying the rainfall losses, which were selected based on the major stream gauges downstream), but the overall shape of the hydrograph is relatively well matched considering the lack of detail in the rainfall data.

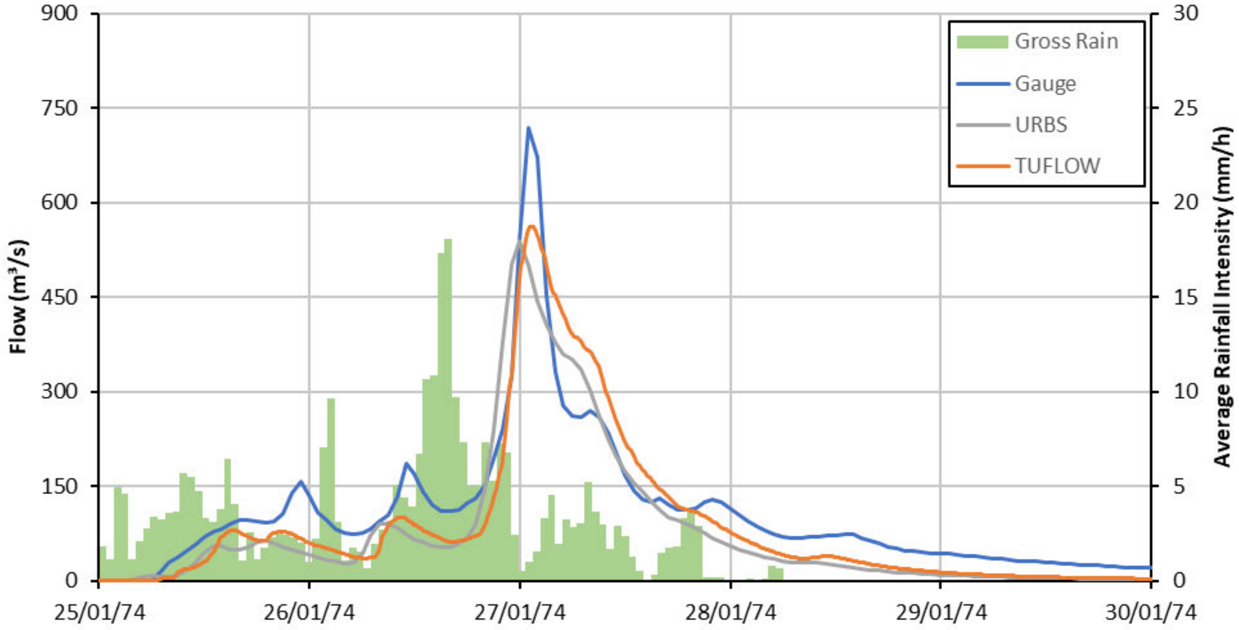


Figure 7.14 Modelled and rated flow hydrographs at Helidon for the January 1974 flood

There is little useful historical information to confirm calibration of Laidley Creek for the 1974 flood as the Warrego Highway gauge on Laidley Creek was not open. The stream gauge in the upper Laidley Creek catchment at Mulgowie matches the rising limb very well up to ~280 m^3/s but then appears to have failed. Comparing the hydraulically routed flows from the hydraulic model with the hydrologically routed flows at the Warrego Highway, shown in Figure 7.15 identifies two issues:

- Although the overall shape of the hydrograph appears similar, the hydraulic model flows lag the hydrologic model flows
- During large events, flows break out of Lockyer Creek and flow southward into Laidley Creek upstream of the Warrego Highway. The flows extracted from the hydraulic model include this additional flow, whereas the hydrologic model does not currently include this bypass flow and reports only the flows arriving from the Laidley Creek catchment.

Effects of this additional overflow and delay downstream at Glenore Grove appear to be minimal and the hydrographs match relatively well. It should be noted that due to the complex flow patterns around Glenore Grove, which include breakouts from Lockyer Creek to the north and eastwards from Laidley Creek during high flows, it is difficult to ensure an exactly consistent comparison between the extracted hydraulic model flows and the hydrologic model flows (which for more practical purposes should be considered as 'flows arriving within the Glenore Grove region').

The flood inundation extents for the 1974 calibration event are presented in Figure E2-A.

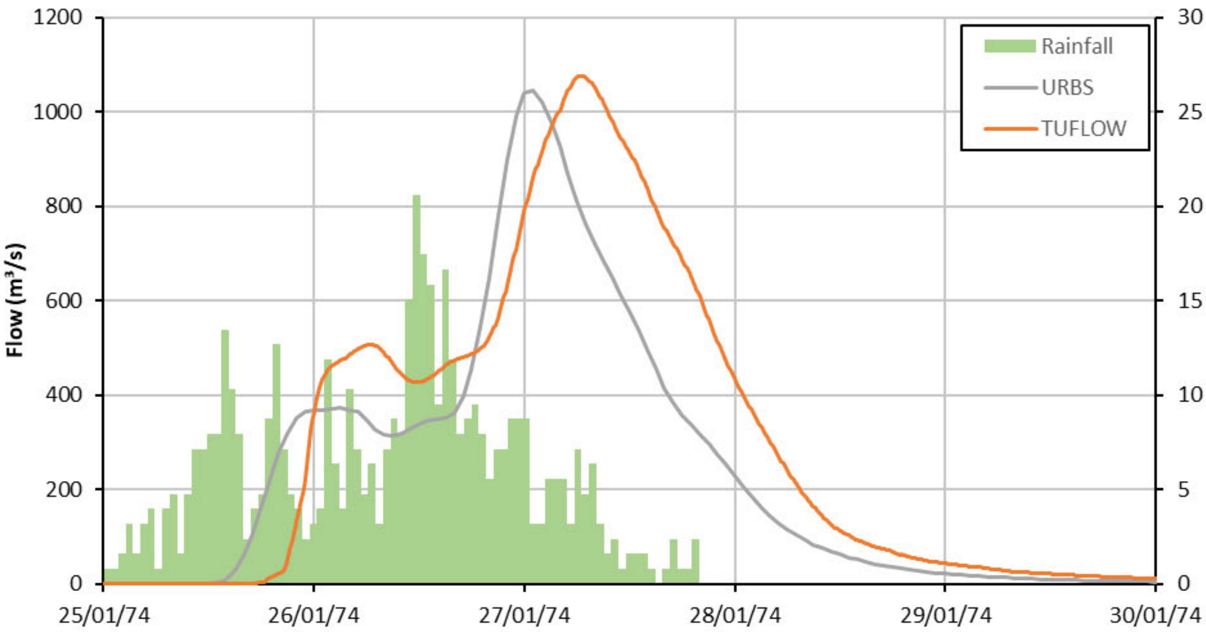


Figure 7.15 Modelled flow hydrographs at Warrego Highway for the January 1974 flood

7.3.3.2 May 1996 flood event

Similar to other events affecting Lockyer Creek (e.g. January 2011), the May 1996 flood appears to be the combination of several different rainfall storm cells affecting different tributaries and resulting in a number of distinct peaks over several days. Total rainfall depths across the catchment were typically of the order of 300 mm to 400 mm, however isolated gauges recorded depths in excess of 550 mm to 600 mm. This rainfall distribution makes it difficult to select rainfall losses or other model parameters that can calibrate the model across multiple gauges, or even for different flood peaks at the same gauge. Some of these issues are illustrated in flow hydrographs at Helidon shown in Figure 7.16, where the models achieve a reasonable match of the shape of the largest peak on 3 May (which could be improved by slightly increasing the losses), but the gauge records a second peak not reflected in the rainfall record. This has flow-on effects throughout the system, as the ‘missing’ peak would help fill in the distinct trough in the modelled flow hydrographs at Gatton and Glenore Grove. Overall, a reasonable match can be achieved at most of the smaller gauges, including the Warrego Highway (refer Figure 7.19), Mulgowie etc. albeit using different losses at each location, however the way the flows combine at Glenore Grove is not particularly well matched. This may be attributed to the limited number of continuous rainfall records (and hence temporal patterns) available in the catchment, which will tend to reinforce each recorded burst when interpolated across a wider area.

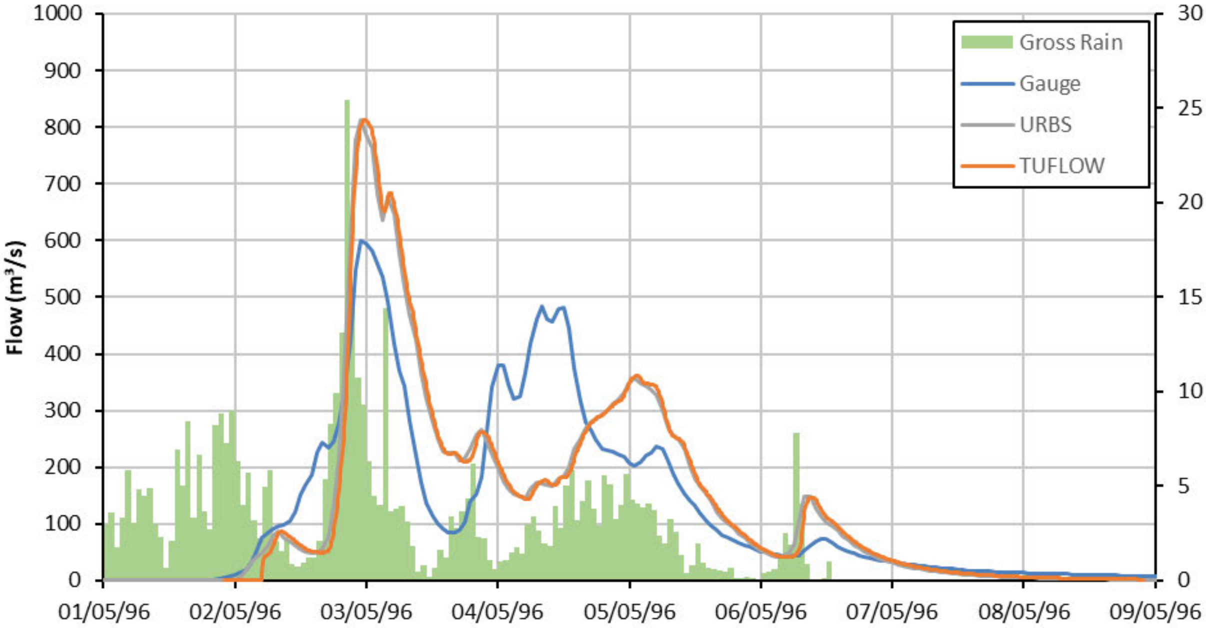


Figure 7.16 **Modelled and rated flow hydrographs at Helidon for the May 1996 flood**

Although the May 1996 flood event is not an ideal historical event for calibration of the Lockyer Creek model, it does provide an example of a mid-size multi-peak flood event to compare the hydrologic and hydraulic routing. Comparisons of the flow hydrographs for the May 1996 flood event produced by the hydrologic model and hydraulic model at Gatton Weir, Glenore Grove and Warrego Highway are provided in Figure 7.17, Figure 7.18 and Figure 7.19 respectively. These show very good agreement between the routed flows at Gatton, though it is noted that the gauge was manually read during the flood event (hence the three points on Figure 7.17 rather than continuous gauge data). For Laidley Creek at the Warrego Highway, the shape of the hydrographs is similar, however the hydraulic model flows tend to lag the hydrologic flows. Notably, the timing of the hydraulic model appears to provide a better match to the recorded historical flows/levels. Although this may suggest that it may better represent the hydraulic hydrograph travel speeds in Laidley Creek over the hydrologic model, this does not seem to produce consistency as in the January 2011 flood the hydrologic model shows good agreement with the recorded timing while the hydraulic model lags behind.

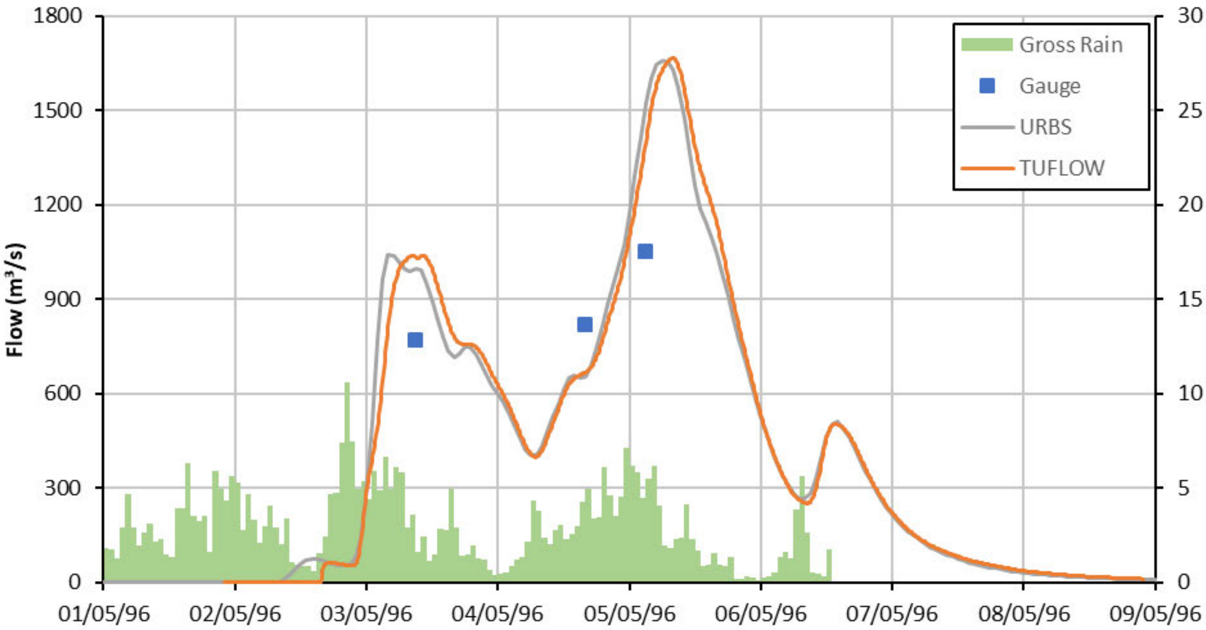


Figure 7.17 **Modelled and rated flow hydrographs at Gatton for the May 1996 flood**

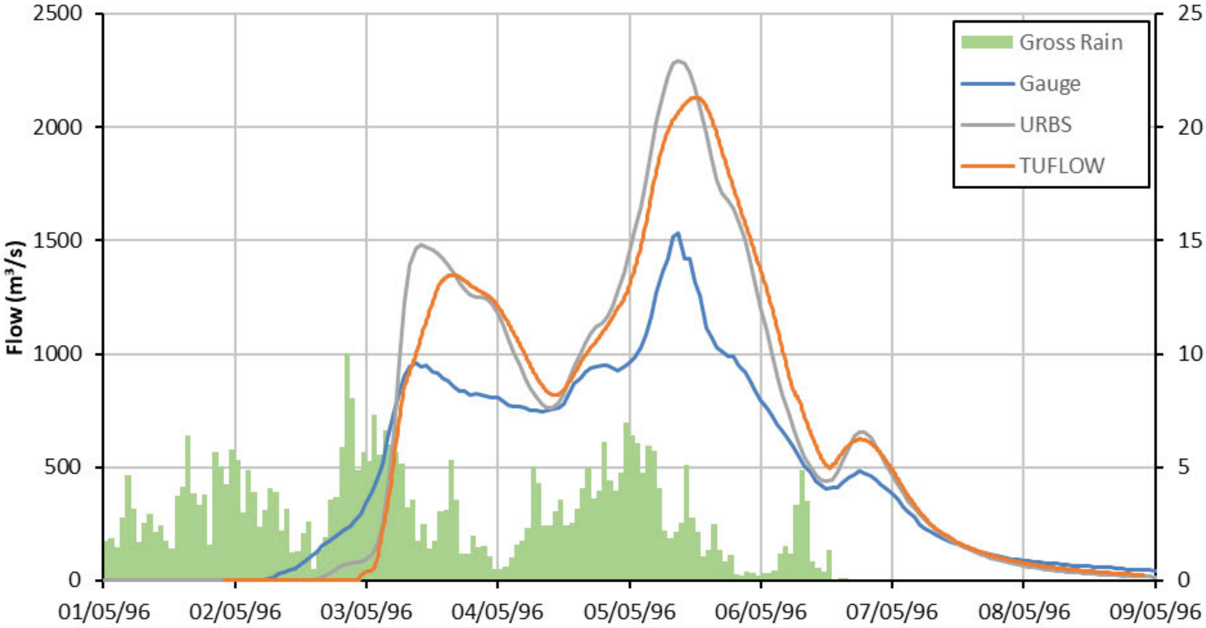


Figure 7.18 **Modelled and rated flow hydrographs at Glenore Grove for the May 1996 flood**

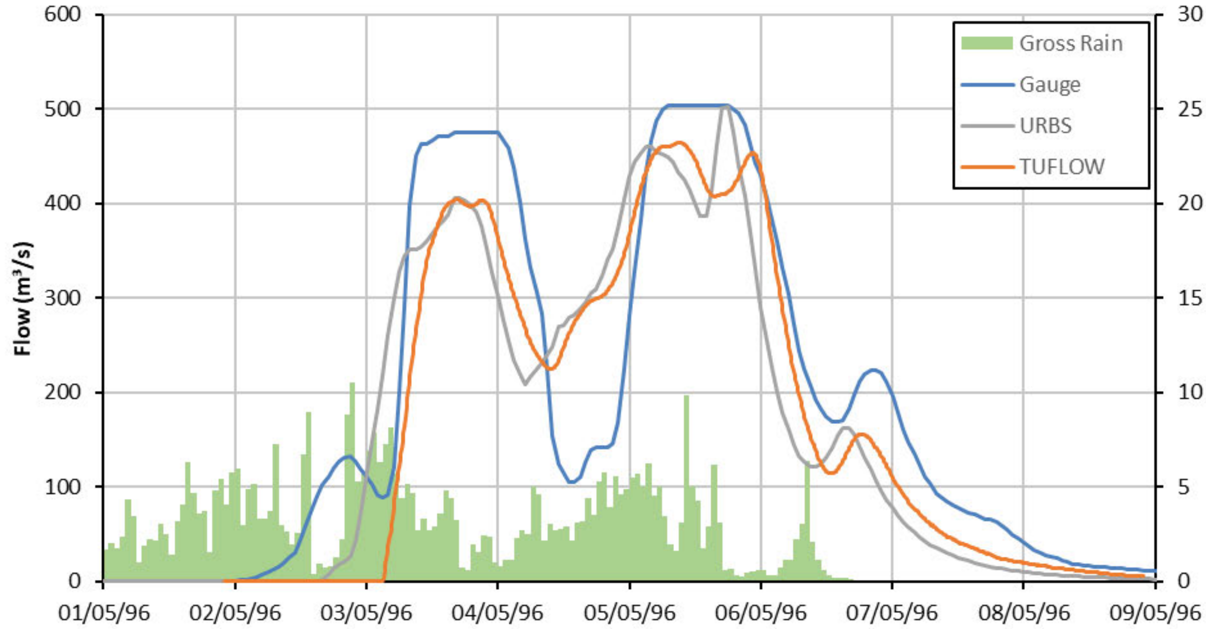


Figure 7.19 **Modelled and rated flow hydrographs at Warrego Highway for the May 1996 flood**

7.3.3.3 February 1999 flood event

Despite being a relatively large flood in the upper Brisbane River upstream of Wivenhoe, the February 1999 flood event was only a minor flood in the Lockyer Creek catchment, with rainfall more heavily concentrated over the upper Lockyer catchment (note the disparity in rainfall depths between Warrego Highway and Helidon shown in Figure 7.23 and Figure 7.20) . The catchment was relatively dry at the commencement of the 1999 event, evidenced by nearly two days of rain before any significant flow is recorded at any of the stream gauges, and large initial losses are required to match the observed runoff volumes. A good match of the recorded flows was achieved at the major stream gauges at Gatton and Glenore Grove in Figure 7.21 and Figure 7.22 respectively. Due to the relatively minor rainfall volumes over the Lockyer catchment, adopted rainfall losses have a significant influence on the resulting flow hydrographs. Similarly, the 'missing' first peak in the hydrograph at Helidon in Figure 7.20 is highly dependent on the adopted initial loss, and more notably the loss used to calibrate to Glenore Grove almost completely removes the Laidley Creek rainfall and flows from the hydrologic model (and consequently in the hydraulic model), as shown in Figure 7.23. A greatly improved match of the Laidley Creek records at Mulgowie and Warrego Highway can be achieved by adopting lower rainfall losses in the hydrologic model, however this results in too much flow in Lockyer Creek.

The minor inflows from Laidley Creek during the February 1999 flood demonstrate that the timing and routing of low flows in Lockyer Creek through to Glenore Grove are well represented in both the hydrologic and hydraulic models.

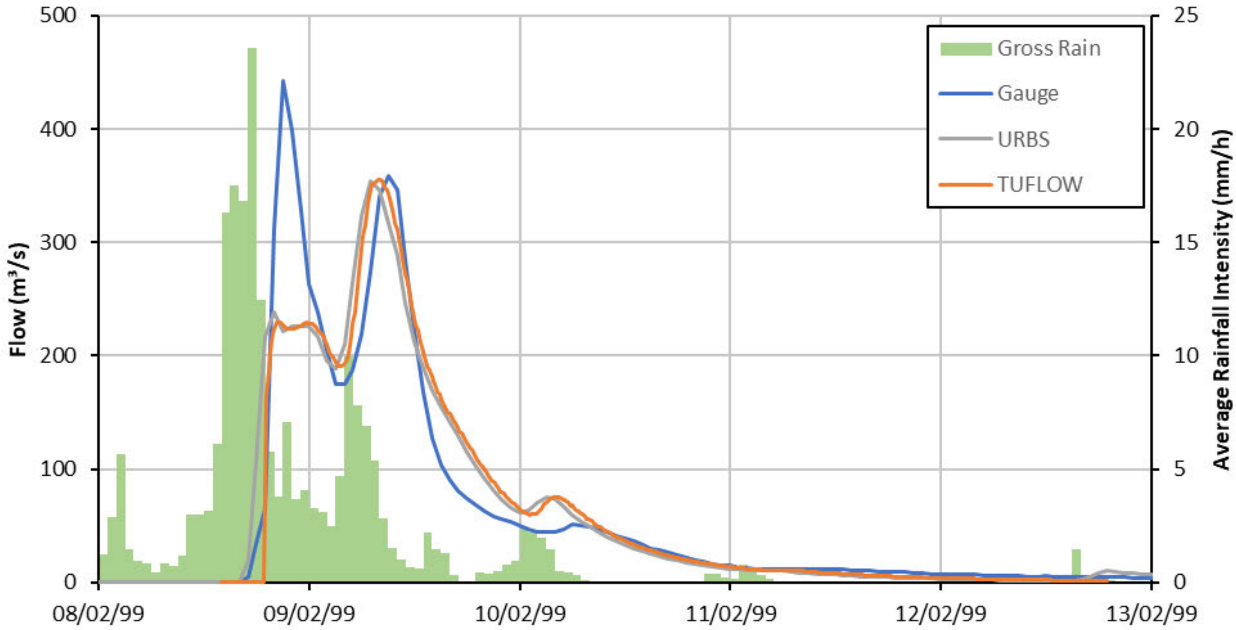


Figure 7.20 **Modelled and rated flow hydrographs at Helidon for the February 1999 flood**

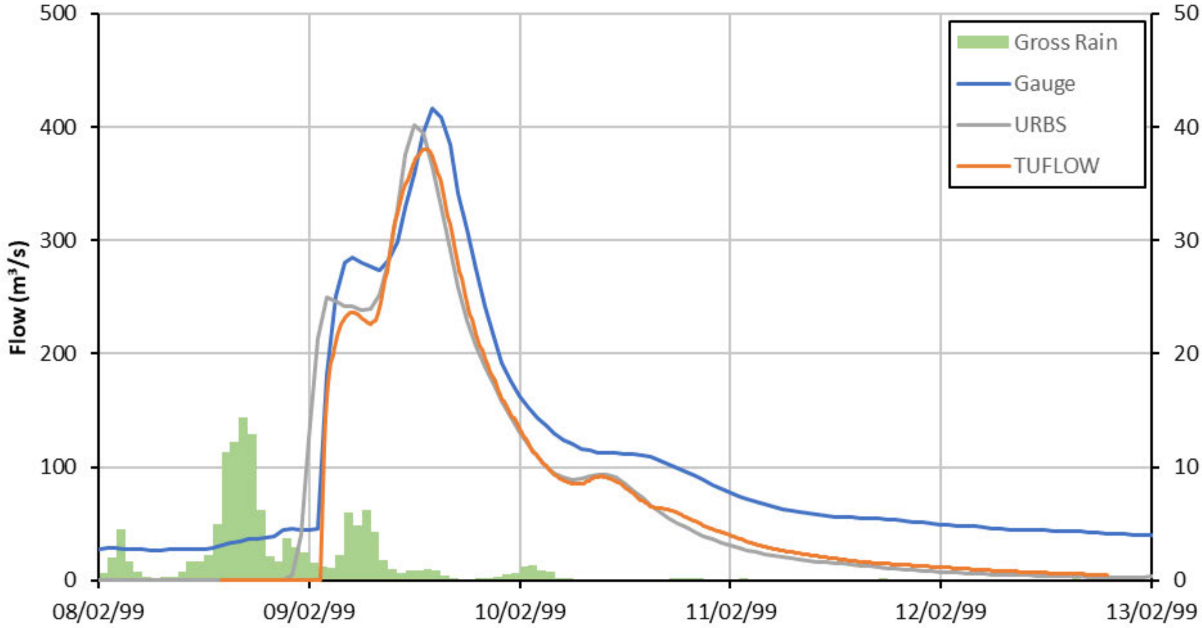


Figure 7.21 **Modelled and rated flow hydrographs at Gatton for the February 1999 flood**

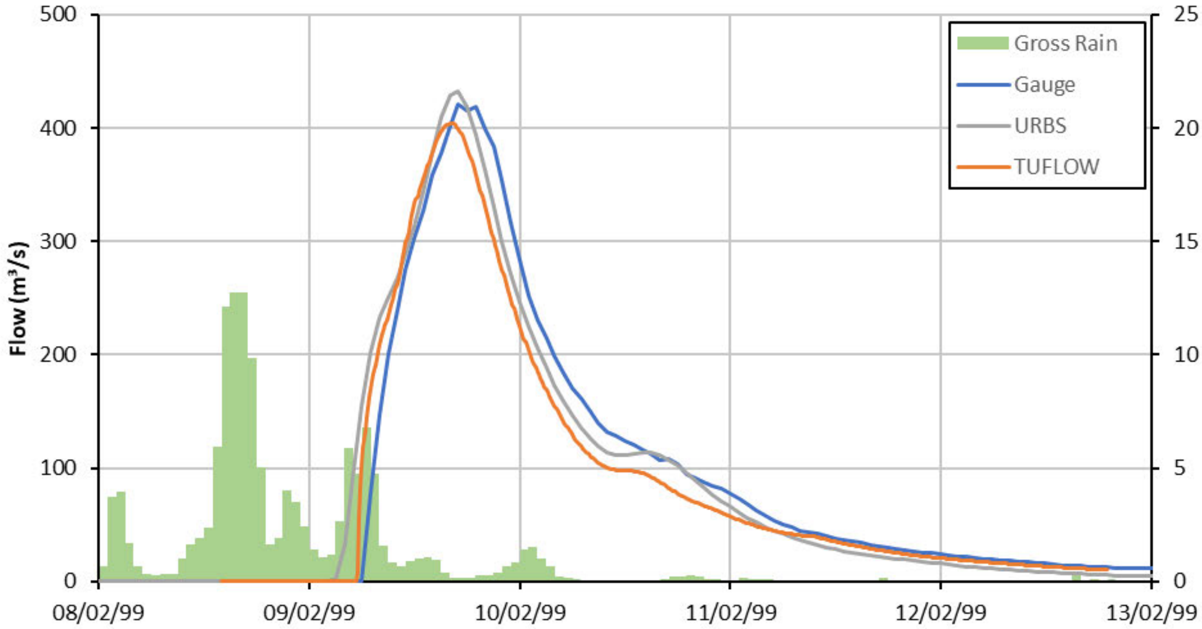


Figure 7.22 **Modelled and rated flow hydrographs at Glenore Grove for the February 1999 flood**

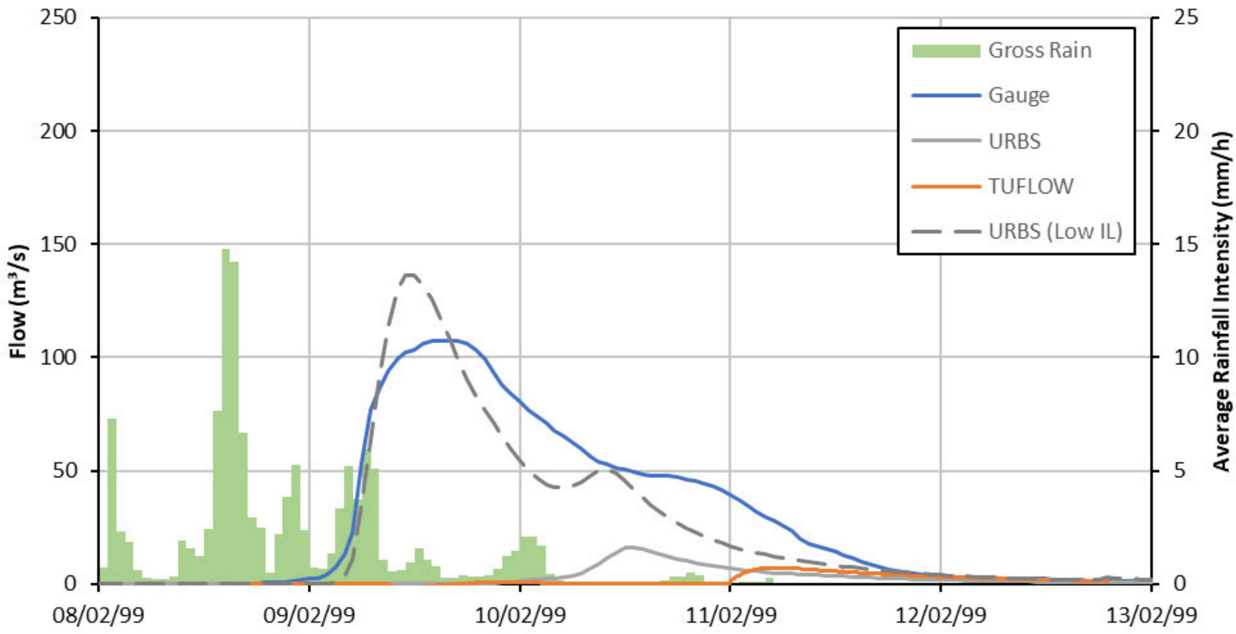


Figure 7.23 Modelled and rated flow hydrographs at Warrego Highway for the February 1999 flood

7.3.3.4 January 2011

The January 2011 flood is the largest recorded flood in much of the Lockyer Creek catchment. The overall event is actually the combination of several distinct rainfall bursts originating at different times in different parts of the catchment. Flash flooding in the upper Lockyer Creek catchment on 10 January 2011 caused significant loss of life and property damage at Murphys Creek, Helidon and Grantham and was followed by more widespread flooding on 11 January 2011 that resulted in larger flows at Gatton and across the southern catchment. These are respectively the second last and last of five distinct peaks observed at Gatton Weir shown in Figure 7.25.

Significant attention has been given to examining the flash flood that struck Grantham. Unfortunately, relatively little data is available for reliably estimating the peak flow. As discussed in Section 5.5.4, the Helidon stream gauge failed prior to the peak. A peak level of just under 14 m has been estimated, corresponding to a flow of around 3,000 m³/s using the BRCFS derived rating curve. Other attempts to generate the flow hydrograph (e.g. WRM 2015) have estimated a peak flow as high as 4,600 m³/s. These estimates do not appear to have been reconciled with the expected flood recurrence, and when compared to both a Flood Frequency Analysis (FFA) and rainfall-based methods (MCS and Design Event modelling), would appear to correspond to events in excess of a 1 in 100,000 AEP event. They also do not address the fact that flows of this magnitude at Helidon would cause the second-last flood peak to exceed the following larger peak observed at Gatton. Another complicating issue for a flow of this magnitude is that it would necessitate significant attenuation of the flood peak to have occurred between Helidon and Gatton, where the 10 January flood peaked over 1.2 m lower than the peak on the following day (estimated at around 1,800 m³/s and 2,200 m³/s respectively). This would only be possible for a very sharp flood peak. There is no reliable data in this regard due to the failure of the gauge.

Some reports (e.g. Gearing 2015) suggest that the January 2011 flood resulted in flood levels at Grantham similar to those caused by the January 1974 flood, with the significant difference between the two events being the rate of rise and lack of warning in the January 2011 flood. The January 1974 flood produced a rated flow of 840 m³/s at Helidon, with other tributary flows bringing the estimated peak at Grantham to around 2,000 m³/s. Assuming that the 10 January 2011 flow was of similar magnitude, mostly originating from the upper Lockyer Creek, this would correspond to a peak of around 1 in 2,000 AEP at Helidon. Although this appears to be a more statistically probable estimate of the magnitude of the event, it is far from conclusive evidence, noting that other reports suggest that the January 2011 flood event was larger than the January 1974 flood event at Grantham and large changes in flow may not correspond to large differences in water level once flow breaks out onto the floodplain around Grantham.

Review of radar data conducted by the BoM (Report to Queensland Floods Commission of Inquiry, March 2011), identified that the limited rainfall gauges in the upper Lockyer catchment did not capture the highest rainfall bursts that occurred. Conversely, ground truthing of the radar data (comparison with recorded rainfall) indicated that the storm complex had relatively low radar intensity returns for a storm in South East Queensland with such high rainfall amounts. This appears to be consistent with the Jacobs assessment (Jacobs 2016), which noted that runoff from rainfall patterns developed from radar data produced too much total flow volume. This demonstrates that there is a high degree of uncertainty regarding the rainfall that occurred during the January 2011 flood.

A key characteristic of the 10 January 2011 flash flood was its extreme rate of rise. It is described by witnesses as a flood wave more reminiscent of a dam break, spawning the colloquial description as an ‘inland tsunami’, carrying a significant debris load. The stream channel of Lockyer Creek was heavily vegetated prior to the flood and experienced significant scour and removal of vegetation throughout the flood event as shown in Figure 7.7. This could potentially have had significant influence on the shape and magnitude of the flood wave. The forefront of the flood would have to push through heavy vegetation while the tail of the flood travelled faster through cleared channel, causing concentration of the flow peak. (Note that this would also produce higher flood levels than would be estimated using a smoother post-flood channel roughness). The debris picked up and carried by the flow could also act to retard the front of the flood-wave. High debris flows in steep channels are often characterised by a very steep front as flow builds up behind a ‘moving dam’ of debris. Both phenomena would contribute to a concentrated peak flow well in excess of a rainfall-generated flood. The simplified routing parameters of a standard hydrologic model would struggle to represent these complex phenomena, and indeed they are difficult to represent even in a 2D hydraulic model. A time or depth-dependent roughness could be used to represent the higher roughness experienced by the front of the flood wave, but still may not truly represent changes to the fluid properties caused by high-debris concentration.

The above discussion is provided to highlight the significant uncertainties regarding the calibration of the models to the 10 January 2011 flash flood encompassing all calibration process (unreliable input rainfall data, unreliable peak and unknown duration of the target hydrograph, uncertain and variable condition of the stream during the event). The BRCFS URBS model results, shown in Figure 7.24, shows a reasonable match of a number of the minor flood peaks, particularly the longer duration burst commencing 9 January 2011, but significantly underestimates the magnitude of the 10 January 2011 flood peak. This was not considered a serious issue for the BRCFS, which was focussed on the lower Lockyer and Brisbane River catchments for which the subsequent peak was more important. Although the 10 January 2011 flood is recognised as being a very significant event in the upper Lockyer catchment, both in terms of its magnitude and consequence, placing undue weight on attempting to replicate the characteristics of a flash flood may be to the detriment of the overall model calibration, given the significant uncertainties regarding the event. The study has therefore not attempted to replicate the characteristics of the 10 January 2011 flood. It is nevertheless noted that:

- Modelled flood levels in areas upstream of Gatton where the 10 January 2011 flash flood peaked higher than the 11 January 2011 peak will not be represented correctly
- Design event modelling (particularly the hydrologic assessment) may not correctly assess the severity of flash floods that could potentially occur in the upper catchment. Flood frequency analysis suggests that significant events are rare (> 1 in 100 AEP), but they may occur more frequently than estimated by standard analysis techniques. Assessment of such events will not impact on the Project design and assessment is outside the scope of the current investigation.

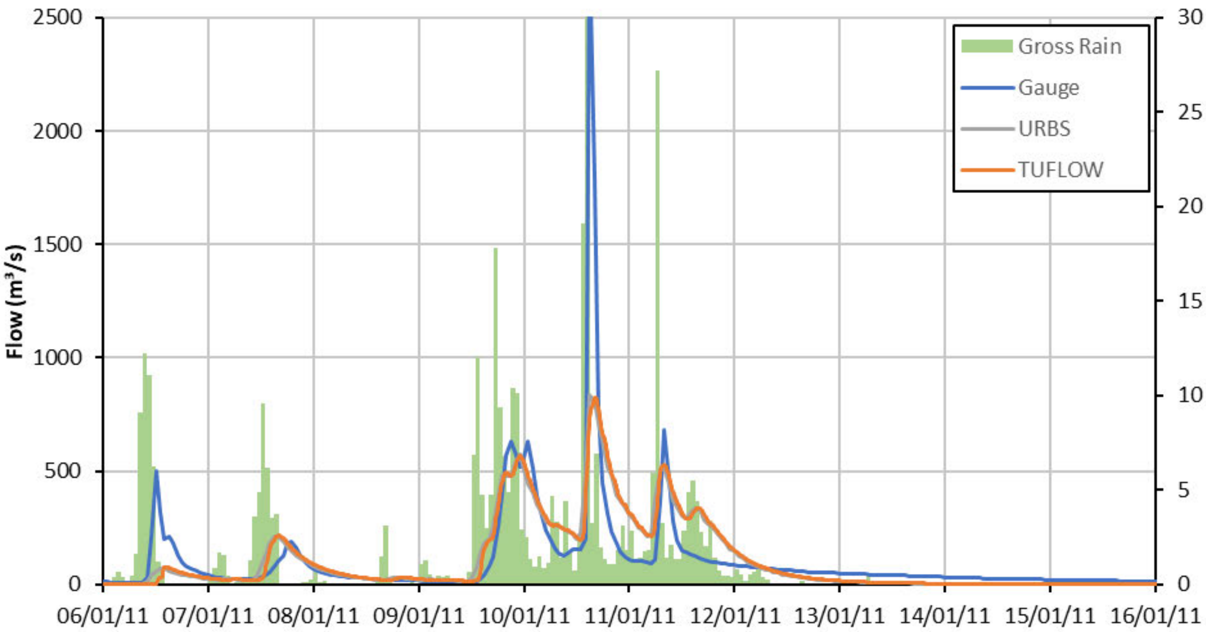


Figure 7.24 Modelled and rated flow hydrographs at Helidon for the January 2011 flood

Comparisons of the flow hydrographs for the January 2011 flood event produced by the hydrologic model and hydraulic model at Gatton Weir, Glenore Grove and Warrego Highway are provided in Figure 7.25, Figure 7.26 and Figure 7.27 respectively. These show that both the hydrologic and hydraulic models achieve a good match of the hydrograph shape and timing at Gatton Weir (note that the rating is not particularly reliable at high flows).

In Laidley Creek at the Warrego Highway, the hydraulic hydrograph again lags behind the hydrologic hydrograph by approximately 2 hours. Unlike the May 1996 and January 2013 flood events, for the January 2011 flood the hydrologic hydrograph appears to better match the timing of the observed flood. As with the January 1974 flood, flows from Lockyer Creek overflow southward into Laidley Creek upstream of the Warrego Highway gauge. At the flood peak, the combined flows at the gauge are therefore higher than are predicted by the hydrologic model. This suggests the good match between the hydrologic and historical peaks is somewhat of a coincidence. As shown in Figure 7.10, the current DNRME rating and the hydraulic level-depth relationship begin to deviate above 84 m AHD (~1,000 m^3/s), which is coincidentally also the level to which flow measurements provide good confidence in the rating curve. The DNRME rating curve was reviewed during the BRCFS and was adopted without change due to the (apparent) good match between the hydrologic peak and the rated flow. The TUFLOW relationship would suggest that the observed levels should correspond to higher flows than are predicted, which is consistent with the inclusion of additional overflow from Lockyer Creek.

At Glenore Grove, the hydraulic and hydrologic hydrographs show good agreement early in the event when flows are primarily contained within the main Lockyer Creek channel, (where the major floods are coming from the upper Lockyer region). Later in the event, when Laidley Creek provides a more significant contribution, the hydraulic hydrograph tends to lag behind the hydrologic during the flood peak. The mismatch in timing between the Lockyer Creek and Laidley Creek flows appears to be the greatest contributor to the difference in the hydrograph at Glenore Grove. Adopting a lower β value for the entire hydrologic model, (for other events β was only modified for the sub-catchments inflows within the hydraulic model boundary), was found to slightly improve the timing issue.

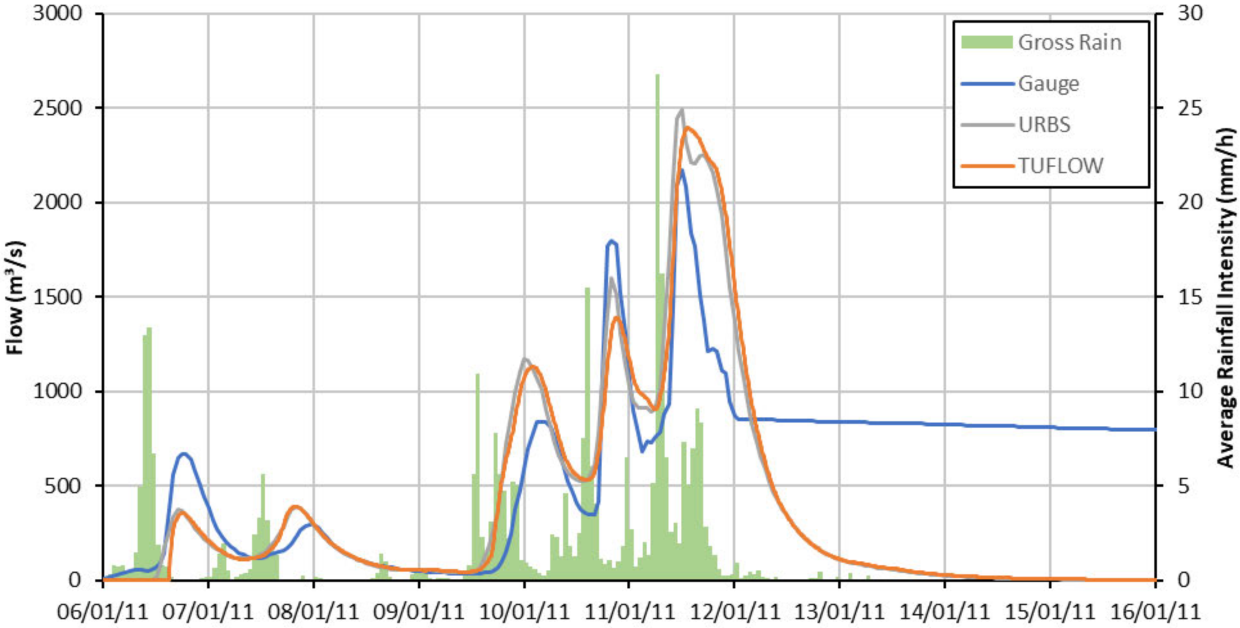


Figure 7.25 Modelled and rated flow hydrographs at Gatton Weir for the January 2011 flood

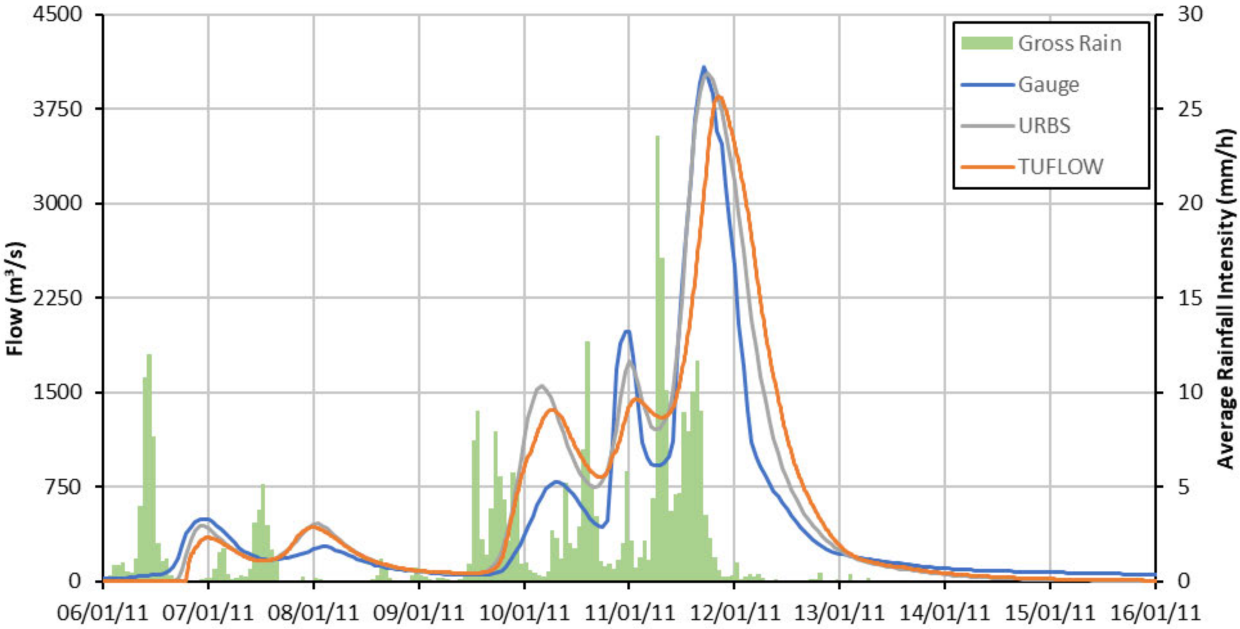


Figure 7.26 Modelled and rated flow hydrographs at Glenore Grove for the January 2011 flood

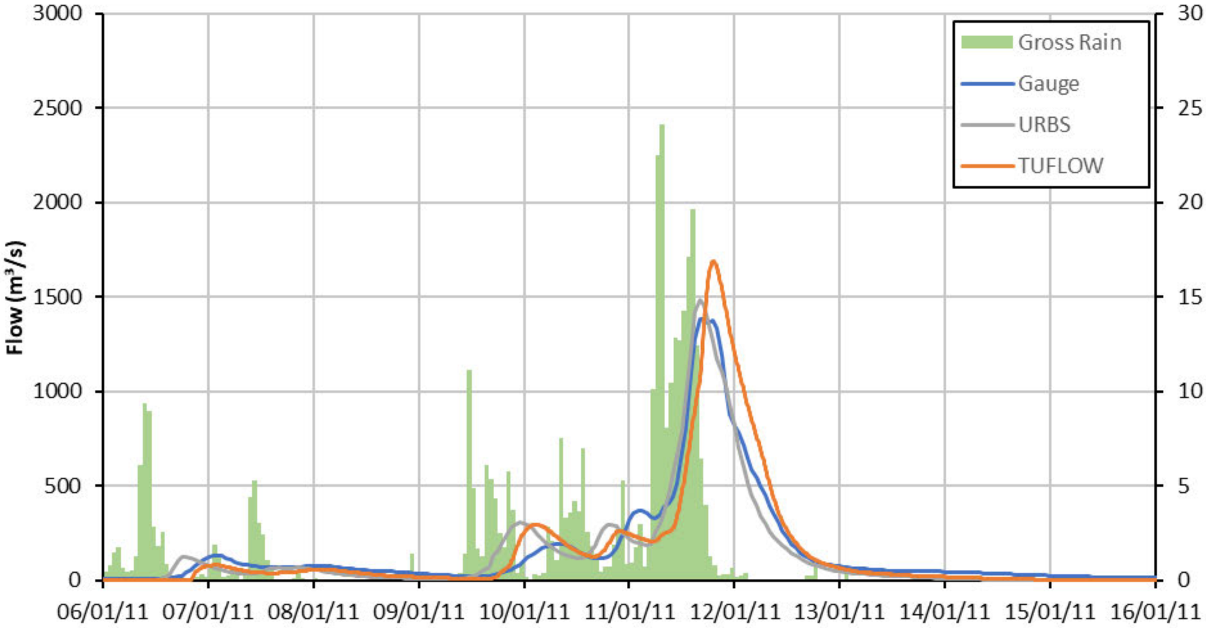


Figure 7.27 Modelled and rated flow hydrographs at Warrego Highway for the January 2011 flood

LVRC provided a number of flood markers in the Lockyer Catchment for the January 2011 event. These recorded levels have a range of accuracies based on their source. Of these markers 162 were used to confirm the Lockyer Creek model calibration. The flood markers in the vicinity of the Project alignment are presented in Figure E2-B.

As outlined above the hydrologic model does not replicate the magnitude of the 10 January 2011 flash flood, and consequently flood levels in the area between Helidon and Grantham are consistently underestimated. Excluding this region, 75% of the flood marker points are within 300 mm of the hydraulic model results and 92% of the flood markers are within 500 mm of the hydraulic model results. Importantly, the hydraulic model does not consistently under- or over-estimate the flood levels. The distribution of these calibration points is outlined in Figure 7.28 with no flood markers being located above Helidon. The model calibration performance is similar to the previous calibration undertaken in the SKM *Lockyer Creek Flood Risk Management Study* (SKM 2013).

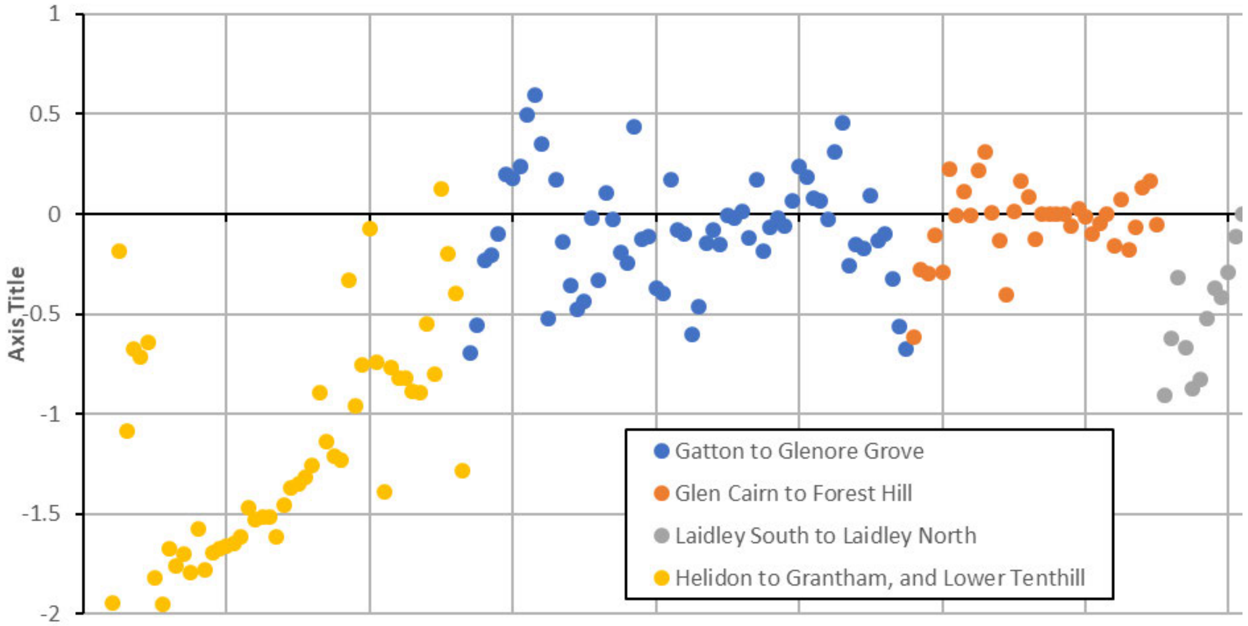


Figure 7.28 Lockyer Creek 2011 – Flood marker difference

7.3.3.5 January 2013

Unlike the short bursts of the January 2011 flood, the January 2013 flood was caused by prolonged, wide-spread rainfall producing a single flood peak. Comparisons of the flow hydrographs for the 2013 flood event produced by the hydrologic model and hydraulic model at Gatton Weir, Glenore Grove and Warrego Highway are provided in Figure 7.29, Figure 7.30 and Figure 7.31 respectively.

A good match of the hydrograph shape and timing is achieved at Gatton Weir (note that the rating is not particularly reliable at high flows). The hydrograph shape is also reasonable at Warrego Highway (note that the gauge did not capture the flood peak). As with the other calibration events, the timing of the hydraulic hydrograph lags behind the hydrologic hydrograph by a few hours. In this case the hydraulic model appears to better match the rising limb of the recorded flood, but the receding limb is closer to the hydrologic model (as are flows at Glenore Grove). The effect of this delay carries downstream to Glenore Grove where there is some lag and minor attenuation of the hydrograph but otherwise a reasonable match of the general shape.

Notably, due to the more consistent rainfall (and potentially aided by the installation of more rainfall gauges within the catchment), the hydrologic calibration could also achieve a good match of the recorded flood hydrographs at most of the minor stream gauges throughout the catchment, including Helidon, Tenthill, Sandy Creek, Mulgowie, Showground and Forest Hill (noting that the reliability of some of these gauge ratings has not been confirmed and some gauges are located at sites where the channel is perched and can only record flows up to bank full) using consistent rainfall losses across the entire catchment (e.g. the Helidon record shown in Figure 7.32 requires only a 5% increase in initial loss to match the recorded peak).

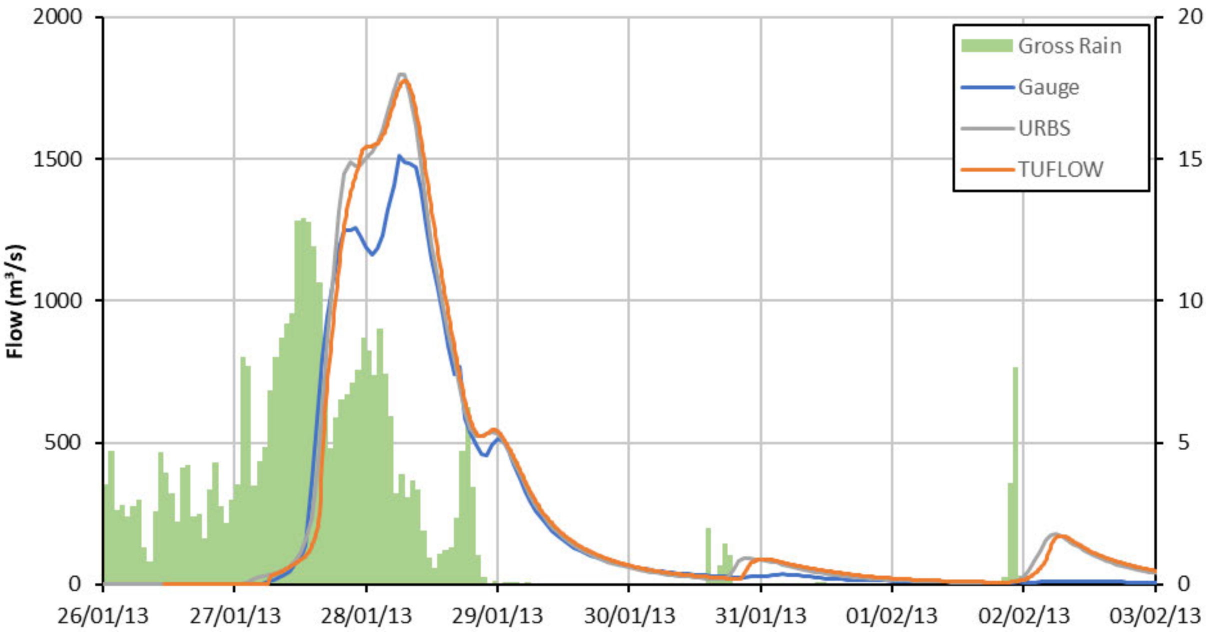


Figure 7.29 Modelled and rated flow hydrographs at Gatton Weir for the January 2013 flood

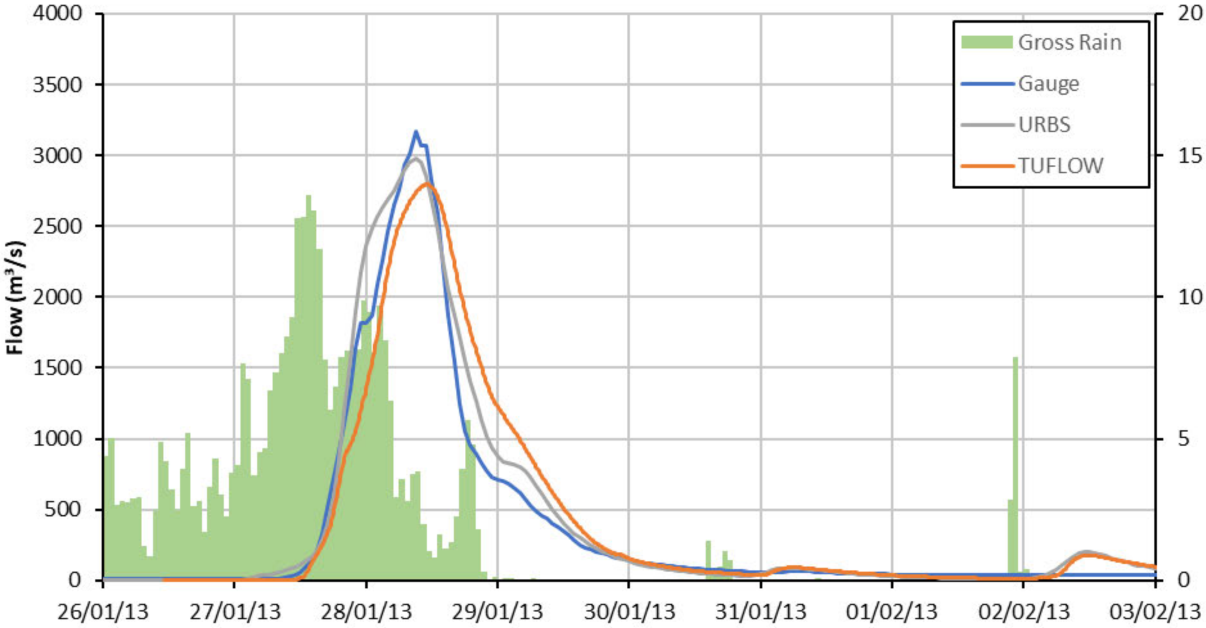


Figure 7.30 **Modelled and rated flow hydrographs at Glenore Grove for the January 2013 flood**

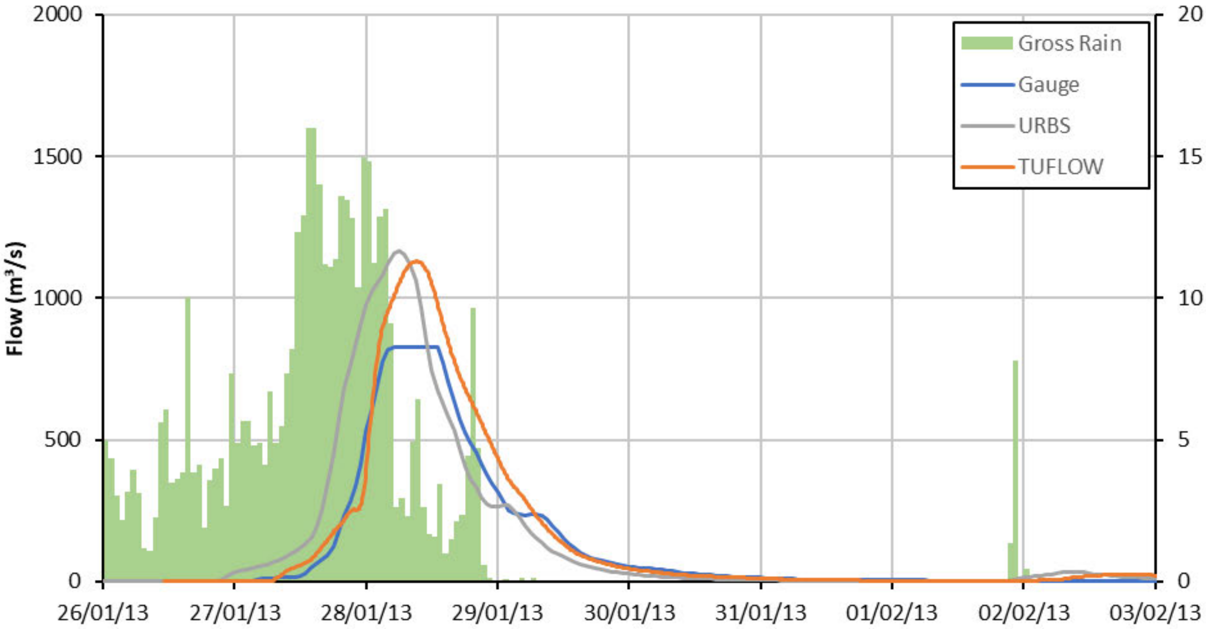


Figure 7.31 **Modelled and rated flow hydrographs at Warrego Highway for the January 2013 flood**

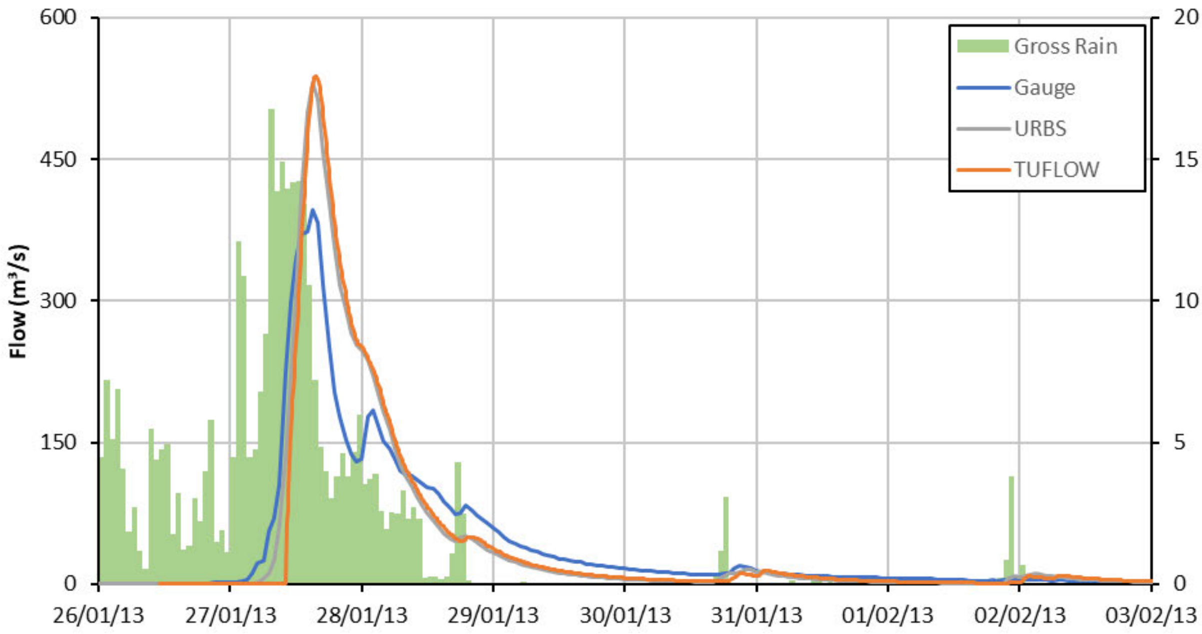


Figure 7.32 Modelled and rated flow hydrographs at Helidon for the January 2013 flood

LVRC provided a number of flood markers in the Lockyer Catchment for the 2013 event. These recorded levels have a range of accuracies based on their source. Of these markers 168 were used to confirm the Lockyer Creek model calibration.

In general, 71% of the flood markers are within 300 mm of the hydraulic results. Further to this 86% of the flood markers are within 500 mm of the hydraulic results. The hydraulic model does not consistently under- or over-estimate the flood levels. The distribution of these calibration points is outlined in Figure 7.33.

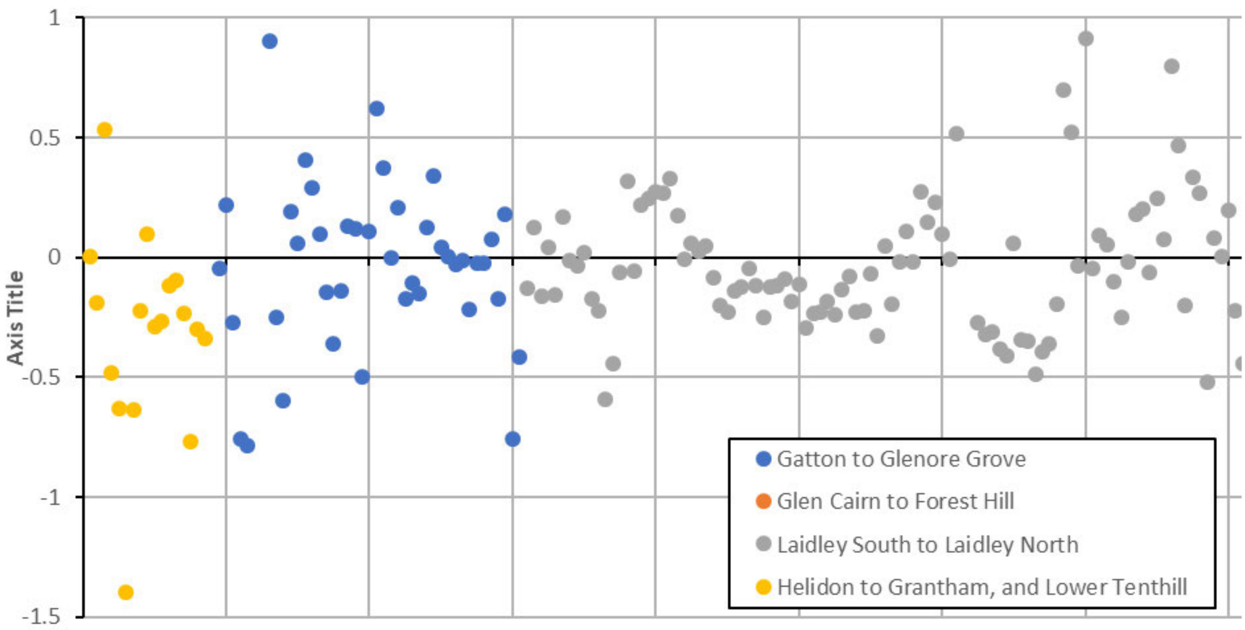


Figure 7.33 Lockyer Creek 2013 – Flood marker difference

8 Existing case modelling

8.1 Hydrology

8.1.1 Approach

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

The general procedure for conducting the design event assessment has been:

- Obtain rainfall Intensity-Frequency-Duration (IFD) relationships, temporal patterns, losses and other parameters pertinent to each catchment
- Simulate the ensemble of design events for a range of durations for each AEP
- Determine the design flows for each AEP. The median peak flow of the critical storm duration (the duration that causes the highest median peak flow) has been adopted. Since an ensemble of ten patterns is tested, the median value technically lies between the fifth and sixth ranked values, so the current practice is to conservatively take the sixth.
- Compare the resulting 2016 design event flow estimates with a FFA, where available, and modify the design parameters where necessary to achieve consistency (see discussion for key catchments in Section 8.1.5)
- Extract design hydrograph(s) for use in the hydraulic models.

8.1.2 Rainfall data

Rainfall IFD relationships for each sub-catchment within each hydrologic model were obtained from the online ARR Data Hub (https://data.arr-software.org/). Table 8.1 shows the change in catchment average 24-hour 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 8.1 Change in 24-hour rainfall depth from ARR 2013 to ARR 2016 IFD tables

Catchment	Rainfall depths (mm)/Change in depth (%)		
	50% AEP event	10% AEP event	1% AEP event
Gowrie	90.5 → 71.6 (-20.9%)	124.8 → 114.0 (-8.7%)	187.7 → 176.0 (-6.2%)
Six Mile Creek	97.2 → 75.9 (-22%)	142.3 → 120.0 (-16%)	225.8 → 187.0 (-17%)
Oaky Creek	100.8 → 77.1 (-24%)	147.6 → 122.0 (-17%)	234.5 → 190.0 (-19%)
Lockyer Creek to Glenore Grove	69.9 → 79.4 (14%)	112.9 → 119.0 (5%)	173.7 → 183.2 (5%)

Comparison with the 2013 IFD data, indicates that there is typically a slight increase in rainfall intensity across the lower Lockyer Creek catchment with the 2016 IFD. For upper Lockyer Creek and Gowrie Creek there is a slight decrease between the 2013 and 2016 IFD data.

8.1.3 Extreme rainfall

Extreme rainfall events have been quantified during this Project; namely the 1 in 10,000 AEP and Probable Maximum Precipitation (PMP). PMP estimates derived using generalised methods outlined in the Generalised Tropical Storm Method Revised and Generalised Short Duration Method (BoM, 2003) were adopted. Procedures referenced in Book VIII of ARR 2016 have been used to interpolate design rainfall estimates between 1 in 2,000 AEP (i.e. credible limit of extrapolation) and the PMP.

8.1.4 Design rainfall losses

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

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

Although the initial and continuing losses can be attributed to physical properties of the catchment (respectively unfilled storages and infiltration for example), losses can serve other less physically based purposes in both calibration and design event modelling. Design event methodology is based on the assumption that the process for transforming design rainfall to design flood estimates is AEP neutral; that is, rainfall AEP can be directly correlated to flow AEP and there is no introduced bias that would result in the design flood estimates having a different frequency to that of the original design rainfall. Although there is almost certainly some correlation, other factors such as losses and temporal patterns can influence the relationship. It is therefore implicit in the assumption that the adopted losses are 'AEP neutral'. Modification of the losses provides a mechanism for reconciling the flow produced by rainfall-based design event methods with that determined by alternative independent methods (e.g. FFA).

8.1.4.1 Gowrie Creek design rainfall losses

In this assessment, an initial and continuing loss model was applied. Rainfall loss parameters used in the previous studies were reviewed. In TRC (2013a), impervious areas were divided into Old urban and New urban with different loss values. In TRC (2014b), 40% of the urban area (low-density urban area) is considered as impervious. As described in Section 6.2.1.3, Gowrie Creek catchment was divided into two areas as shown in Figure A1-B and imperviousness was defined as follow:

- Area 1:
  - Old urban Impervious: Refers to areas of the catchment that have been 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.
  - New urban Impervious: Refers to all other impervious areas than the Old urban impervious. Initial and continuous rainfall losses for this area type were defined based on TRC 2013a.

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

- Pervious US: Refers to all pervious area within Area 1. 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 TRC 2014b.
  - Pervious DS: Refers to all pervious area within the Area 2. Initial and continuous rainfall losses for this area type were defined based on TRC 2014b and ARR hub data where applicable.
  - A range of applicable losses were used as specified Table 8.2. It should be noted that these losses were used with ARR 1987 design rainfall.

Table 8.2 Range of rainfall losses used in previous studies for 5% AEP to 1% AEP events

Location	Area type	IL range (mm)	CL range (mm/hr)
Area 1	Old urban Impervious	8.0	4.0
	New urban Impervious	1.5	0.0
	Pervious US	37.0	2.5 to 6.0
Area 2	New urban Impervious	0.0 to 1.5	0.0
	Pervious DS	15 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 stream flow gauging stations, Oakey and Cranley. The adopted losses are presented in Table 8.3.

Table 8.3 Adopted design rainfall losses for Gowrie Creek

Location	Area type	IL (mm)	CL (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

8.1.4.2 Oaky Creek design rainfall losses

As no calibration data was available for Oaky Creek the rainfall losses were selected to be consistent with the Six Mile Creek model. Losses are outlined in Table 8.4.

Table 8.4 Adopted design rainfall losses for Oaky Creek

Catchment	ARR Data Hub		Adopted	
	IL (mm)	CL (mm/hr)	IL (mm)	CL (mm/hr)
Oaky Creek	40	1.1	10	1.1

8.1.4.3 Six Mile Creek design rainfall losses

As no calibration data was available for Six Mile Creek the rainfall losses were selected through model validation with the previous Six Mile Creek study (APB 2016b) as outlined in Section 5.1.3. Losses are outlined in Table 8.5.

Table 8.5 Adopted design rainfall losses for Six Mile Creek

Catchment	ARR Data Hub		Adopted	
	IL (mm)	CL (mm/hr)	IL (mm)	CL (mm/hr)
Six Mile Creek	40	1.1	10	1.1

8.1.4.4 Lockyer Creek design rainfall losses

In this assessment, an initial and continuing loss model was applied with ARR and adopted losses for the Lockyer Creek model listed in Table 8.6. It is noted that ARR Data Hub (https://data.arr-software.org/) values (in particular losses) are based on generalised regression of catchment characteristics and are intended to provide typical values for use where local catchment specific data is unavailable. Forty-eight historical rainfall/flood events were simulated during the BRCFS to calibrate/validate the hydrologic models. Although significant variability of the losses is observed, at least partially due to discrepancies in the recorded rainfall distribution, the median losses give a reasonable indication of the typical catchment characteristics assuming equal probability that the rainfall is over- or under-estimated.

Table 8.6 ARR 2016 catchment losses

Catchment	ARR Data Hub		Adopted	
	IL (mm)	CL (mm/hr)	IL (mm)	CL (mm/hr)
Lockyer Creek	31	1.3	31 (≥1% AEP) 56 (2% AEP) 110 (<2% AEP)	2.0

8.1.5 Flood Frequency Analyses

8.1.5.1 Gowrie Creek

A flood frequency analysis was undertaken for the two stream gauges on Gowrie Creek, Cranley (422332A) and Oakey (422332B), using FLIKE software. The FFA was based on the maximum historical instantaneous flow discharge for each year of available record, referred to as the annual series. The annual series of each gauge was fitted against different probability models to find the distribution model that best fit to the records which was Log Pearson 3 (LP3). As presented in Table 5.1, Cranley and Oakey stream gauges have 49 and 27 years of recorded flow respectively. The following sections provide further details regarding the FFA process and results.

Cranley stream gauge

The Cranley stream gauge location has not changed since installation in 1969. Generally, the Generalised Extreme Value and LP3 distributions are recommended for stream flow 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 8.1 shows that the LP3 distribution fits reasonably well to the Cranley stream 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 flood event record.

The January 2011 event was a record-breaking flood, being of the order of three times the size of the second highest flood (1981). TRC (2013a) reported that the Cranley stream gauge had malfunctioned during the 10 January 2011 event after an approximate flow of 330 m^3/s at 2.00 pm was recorded. TRC (2013a) developed a hydrologic model for the January 2011 event that simulated 645 m^3/s of peak flow at the Cranley stream gauge. A review study undertaken in 2013 (TRC 2013a) estimated a peak flow of 560 m^3/s at the Cranley stream gauge and provided a range of 293 m^3/s to 399 m^3/s peak flow for a 1% AEP event.

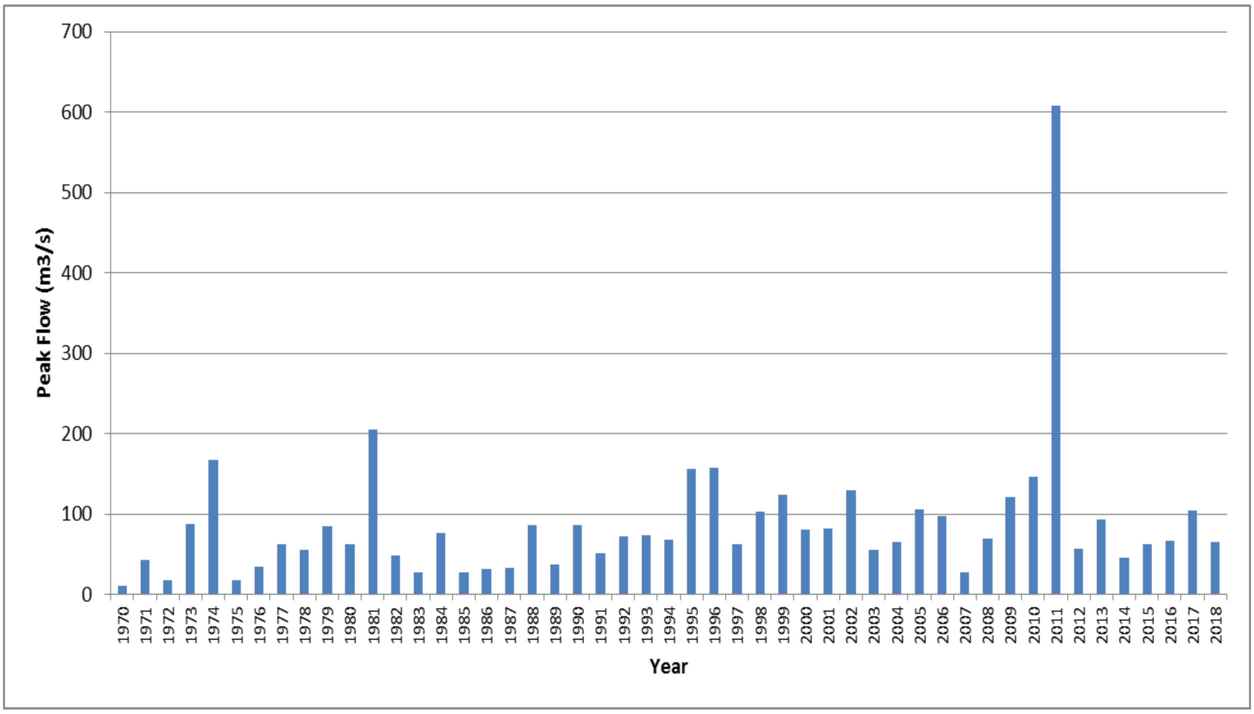


Figure 8.1 The annual series used in FFA for Cranley stream gauge

Based on data from DNRME's Water Monitoring Information Portal (https://water-monitoring.information.qld.gov.au/), the reported peak flow for the January 2011 event at the Cranley stream 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 8.2.

The estimation of probability quantiles limits for the LP3 model for a 0.5 Exceedance Year to a 1% AEP event is presented in Table 8.7. The estimated 1% AEP peak flow is 370 m^3/s, which is within the range as specified in the TRC (2013a) study.

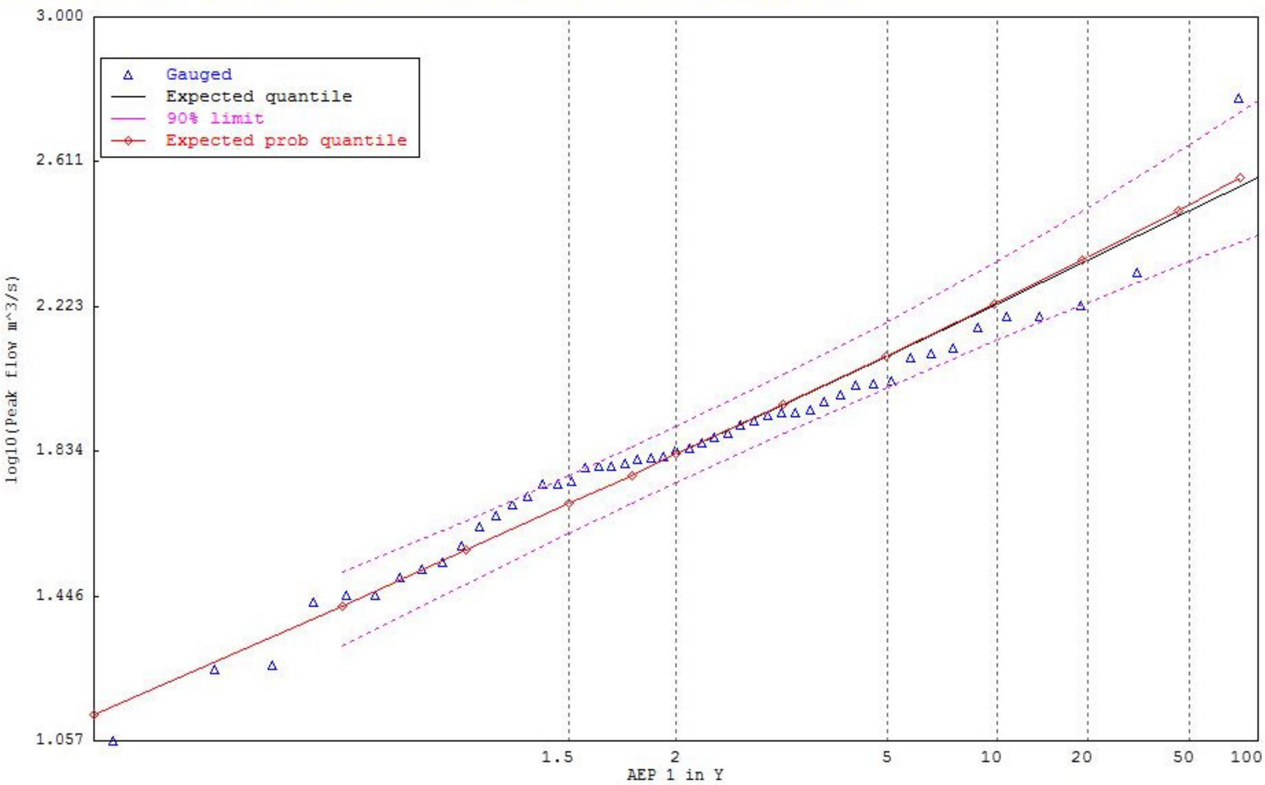


Figure 8.2 Probability model distribution – LP3 model – Cranley stream gauge – Plot scale log-normal

Table 8.7 Flood Frequency Analysis results for Cranley stream gauge based on LP3 model

AEP	Expected probability quantile (m^3/s)	90% probability limit (m^3/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

Oakey stream gauge

Original gauged data

The annual series at Oakey stream 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 8.3 was used in the LP3 model and Figure 8.4 shows that the LP3 distribution fits reasonably well with the Oakey stream 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 Figure 8.4.

According to TRC (2014c), DNRME had confirmed that there were issues with rating curves at the Oakey stream gauge. A technical report to the Queensland Floods Commission of Inquiry (BMT WBM, 2011) noted that Oakey stream gauge data for the January 2011 event is unvalidated. Comparing the reported Oakey stream gauge peak flow (482 m^3/s) for the January 2011 event with Cranley (610 m^3/s) suggests that revision of gauge flow might be required as detailed in the next section.

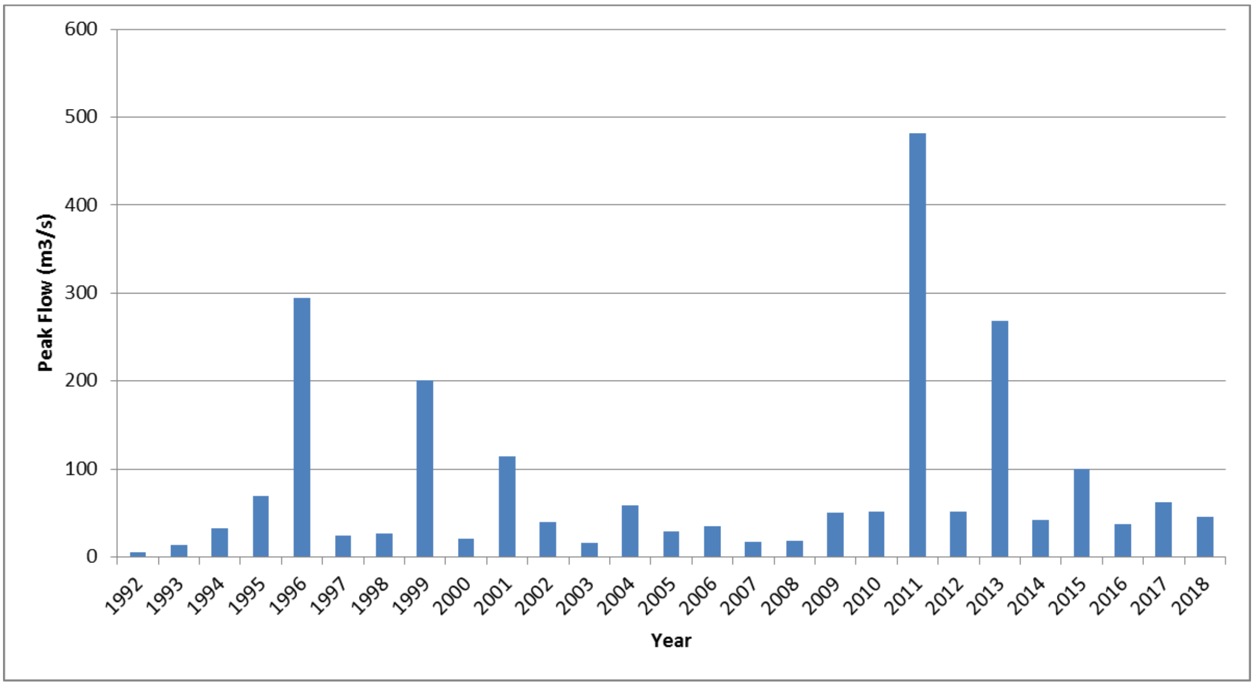


Figure 8.3 The annual series used in FFA for Oakey stream gauge

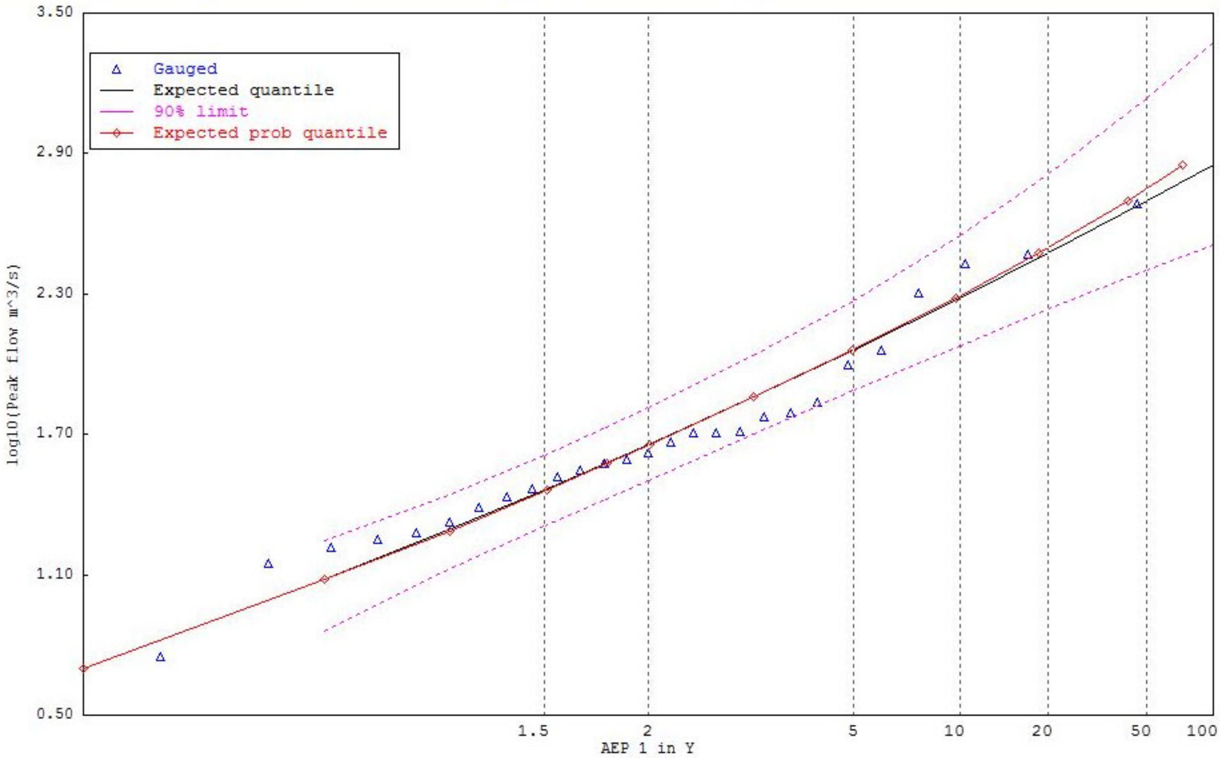


Figure 8.4 Probability model distribution – LP3 model – Oakey stream gauge – plot scale log-normal

Table 8.8 Flood Frequency Analysis results for Oakey stream 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% AEP	115	77	186
10%	192	120	353
5%	298	172	644
2%	497	253	1,375
1%	705	323	2,382

Revised stream gauge data

As mentioned previously, there was an issue with the estimation of peak flow during the January 2011 event at Oakey stream gauge. The January 2011 event had three peak flows. The volume simulated in the hydrologic model for these three peaks was compared with the volume observed at the Oakey stream gauge for the three peaks, as summarised in Table 8.9.

The simulated 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 simulated 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 8.9 Comparison between simulated and observed volume at Oakey stream gauge for three peaks observed during January 2011 event

Peak flow time (hr)	Simulated volume in hydrologic model (ML)	Recorded volume at Oakey stream gauge (ML)	Difference between observed and simulated 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%

Table note:
ML = megalitres

It appears that the Oakey stream gauge did not accurately record the peak flow hydrograph during the January 2011 flood event. Therefore, the peak historical 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.

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 stream gauge. Figure 8.5 presents the FFA results for the revised flow at the Oakey stream gauge.

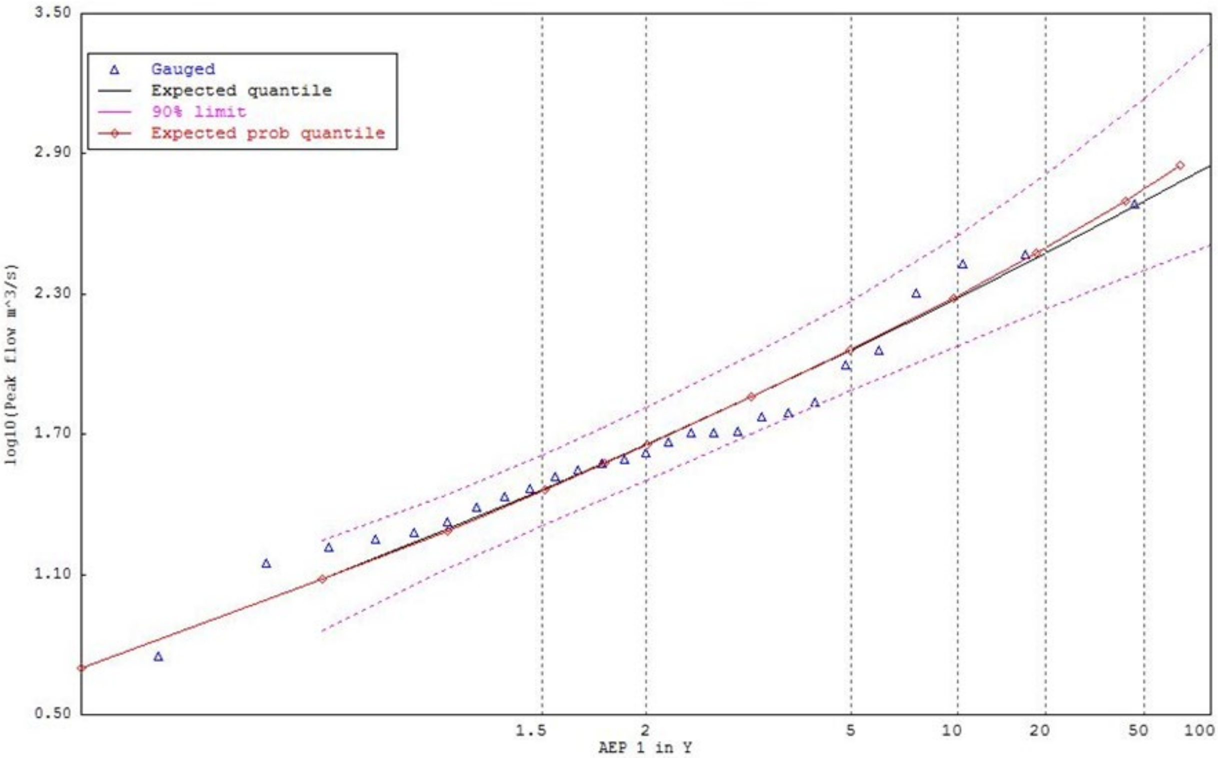


Figure 8.5 Revised Probability model distribution – LP3 model – Oakey stream gauge – plot scale log-normal

Table 8.10 Revised Flood Frequency Analysis results for Oakey stream gauge based on LP3 model

AEP in Y	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

Summary

The FFA results for both stream gauges were compared to the estimated design event results. Both stream gauges showed a good correlation with peak flows within 10% of the FFA as presented in Table 8.11.

Table 8.11 Comparison between estimated peak flows from FFA analysis with simulated flow in RAFTS at Cranley and Oakey stream gauges

Stream gauge	FFA estimated flow for 1% AEP (m^3/s)	RAFTS estimated flow for 1% AEP (m^3/s)	Difference%
Cranley	370	406	+9.7%
Oakey	780	712	-8.7%

8.1.5.2 Lockyer Creek

Flow estimates for Lockyer Creek at Glenore Grove determined using the ARR 2016 design event methodology were compared with the results of the BRCFS MCS and flood frequency analyses as presented in Figure 8.6. An initial assessment was undertaken using the recommended ARR loss parameters (initial loss = 31 mm, continuing loss = 1.3 mm/hr). These showed a good agreement with the BRCFS assessment for rare events (≥ 1% AEP), however significantly overestimated the flows for frequent events. This phenomenon was also observed during the BRCFS when using constant losses, and significantly higher losses were applied to the frequent events to reconcile the MCS and ARR 1987 design event flows with the observed flood record.

Examination of the Lockyer Creek catchment identifies several reservoirs (Atkinson, Clarendon and Dyer) that are not explicitly represented in the hydrologic model, as well as a large number of farm dams and minor-stream storages, as typified in Figure 8.7. These are likely to significantly increase the amount of water storage available within the catchment, particularly during drier seasons when water levels are low. Local farmers are also known to pump directly from Lockyer Creek alluvium when the creek is flowing. A significant proportion of the catchment is also cultivated for agriculture, which potentially leads to higher infiltration rates as compared to untilled natural catchment areas. These characteristics are consistent with initial and continuing loss trends observed in the calibration events, where losses in the Lockyer Creek catchment are higher than in the other Brisbane River sub-catchments. They have also been confirmed through discussions with LVRC and Seqwater, who have observed that flows from Lockyer Creek can often be significantly lower than would have been expected from the amount of rainfall that fell on the catchment.

To reconcile the ARR 2016 design event peak flows with observed flood frequency records, the design event modelling was revised to adopt a continuing loss of 2 mm/hr based on the model calibration losses, consistent across all AEP, while initial losses were increased for the frequent events (refer Table 8.6 for the adopted AEP varying initial losses). The joint probability of rainfall, catchment losses, dam levels etc, has been accounted for by selecting variable 'AEP neutral' losses to reconcile the Design flows to the FFA. The reconciled values are shown in Table 8.12 and Figure 8.6.

Table 8.12 Comparison of 2016 FFJV Design Event Approach with BRCFS results at Glenore Grove

AEP (%)	BRCFS Lower 90% Confidence Interval (m³/s)	BRCFS FFA (m³/s)	BRCFS Upper 90% Confidence Interval (m³/s)	BRCFS MCS (m³/s)	FFJV DEA (m³/s)
50	80	120	200	99	140
20	420	620	940	570	630
10	840	1,240	1,900	1,200	1,300
5	1,380	2,050	3,200	2,000	2,170
2	2,200	3,340	5,460	3,200	3,280
1	2,870	4,450	7,560	4,000	4,250

Table notes:

MCS = Monte Carlo Simulation, DEA = Design Event Approach

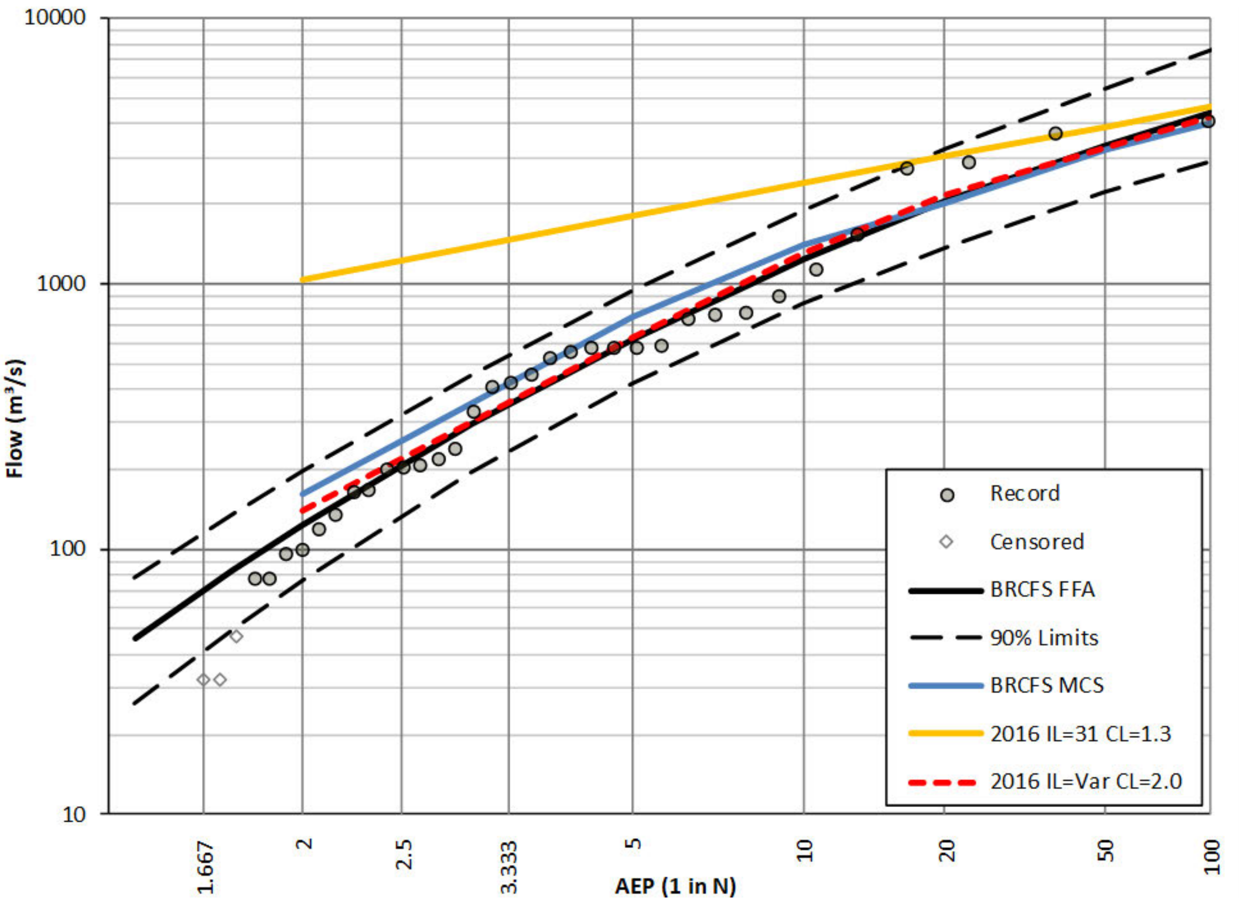


Figure 8.6 Comparison of 2016 FFJV DEA with BRCFS results at Glenore Grove

Figure 8.7 **Example of water storages scattered throughout the Lockyer Creek catchment**

8.2 Existing Case results

8.2.1 Hydrologic model peak flow assessment

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

- The models were simulated for a range of AEP events: 20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF
  - Each AEP was simulated for a range of durations from 30 minutes to 168 hours
  - Each duration was simulated for each of the ten associated temporal patterns
- Peak flood levels were mapped for each storm duration
- A critical duration assessment was undertaken to determine which duration produced the highest median flow of the ten temporal patterns for each event.

The critical durations for each AEP are outlined in Table 8.13 to Table 8.16 for each of the waterways.

Table 8.13 **Critical duration assessment for Gowrie Creek hydraulic model**

Location	Event	Duration (minutes)	Peak flow (m^3/s)
Old Homebush Road	20% AEP	720	191
Western tunnel portal		720	11
Old Homebush Road	10% AEP	360	257
Western tunnel portal		360	14

Location	Event	Duration (minutes)	Peak flow (m³/s)
Old Homebush Road	5% AEP	180	336
Western tunnel portal		90	21
Old Homebush Road	2% AEP	120	446
Western tunnel portal		120	37
Old Homebush Road	1% AEP	120	534
Western tunnel portal		90	45
Old Homebush Road	1 in 2,000 AEP	90	980
Western tunnel portal		90	82
Old Homebush Road	1 in 10,000 AEP	120	1724
Western tunnel portal		60	112

Table 8.14 Critical duration assessment for Oaky Creek hydraulic model

Event	Duration (minutes)	Peak flow (m³/s)
20% AEP	360	41
10% AEP	120	50
5% AEP	60	60
2% AEP	90	77
1% AEP	90	90
1 in 2,000 AEP	60	167
1 in 10,000 AEP	60	234

Table 8.15 Critical duration assessment for Six Mile Creek hydraulic model

Event	Duration (minutes)	Peak flow (m³/s)
20% AEP	60	75
10% AEP	60	84
5% AEP	60	102
2% AEP	30	125
1% AEP	30	145
1 in 2,000 AEP	30	280
1 in 10,000 AEP	30	330

Table 8.16 Critical duration assessment for Lockyer Creek hydraulic model

Event	Duration (hrs)	Peak flow (m³/s)
20% AEP	18	644
10% AEP	18	869
5% AEP	18	1,107
2% AEP	18	1,438
1% AEP	24	1,702
1 in 2,000 AEP	24	3,156
1 in 10,000 AEP	12	3,764

8.2.2 Gowrie Creek

8.2.2.1 Flood maps

Flood maps illustrating indicative flood extents and peak water levels were prepared and are presented in Appendix A on the following figures:

- 20% AEP: Figure A3-A
- 10% AEP: Figure A4-A
- 5% AEP: Figure A5-A
- 2% AEP: Figure A6-A
- 1% AEP: Figure A7-A
- 1 in 2,000 AEP: Figure A8-A
- 1 in 10,000 AEP: Figure A9-A
- PMF: Figure A10-A.

8.2.2.2 Flood inundation extent and flood levels

Figure A7-A presents the 1% AEP peak water levels and flood inundation extent for the Gowrie Creek floodplain for the Existing Case. Under the 1% AEP event the peak depth of water in the Gowrie Creek channel varies between 6 m and 8 m with flow spreading out onto the local floodplain area on either side of the creek and on the tributaries.

The existing QR Western Line runs parallel to the Project 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 Western Line rail formation in several sections.

8.2.2.3 Flood immunity of existing infrastructure

The existing Western Line rail corridor and Paulsens Road both run parallel to Gowrie Creek. The majority of the Western Line rail corridor is above the 1% AEP flood level, but it is inundated in localised places. Paulsens Road is low-level and is inundated by Gowrie Creek during frequent flood events.

Table 8.17 presents a summary of overtopping depths for the existing Western Line rail corridor and key roads near the Project alignment under a range of design events. The overtopping depths for the Western Line rail corridor are estimated levels above the rail formation level.

Table 8.17 Gowrie Creek – Existing Case – Overtopping depths of key infrastructure

Infrastructure	Approximate Project chainage (km)/location	Approximate overtopping depth (m)				
		1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Paulsens Road	Ch 0.10	0.75	0.74	0.39	0.41	0.14
Paulsens Road	Ch 0.20	0.77	0.69	0.56	0.53	0.48
Western Line	Ch 0.70	0.26	0.14	Dry	Dry	Dry
Paulsens Road	Ch 0.70	0.21	0.18	Dry	Dry	Dry
Western Line	Ch 0.97	0.10	0.05	Dry	Dry	Dry
Paulsens Road	Ch 0.97	0.53	0.52	0.44	0.42	0.33
Paulsens Road	Ch 1.04	0.61	0.60	0.51	0.43	0.38
Old Homebush Road	Bridge over Gowrie Creek	0.87	0.51	Dry	Dry	Dry

Infrastructure	Approximate Project chainage (km)/location	Approximate overtopping depth (m)				
		1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
McMahons Road	At connection to Gowrie Junction Road	0.06	0.05	Dry	Dry	Dry
East Paulsen Road	At low level crossing	1.61	1.60	1.33	1.13	1.00
Western Line	Upstream of East Paulsens Road low-level crossing	Dry	Dry	Dry	Dry	Dry
Morris Road	Upstream of East Paulsens Road low-level crossing	2.28	2.09	1.39	1.09	0.95
Ganzer Road	Ch 3.45	3.26	3.07	2.36	1.99	1.83

It should be noted that there are a number of utilities and services associated with the road reserves in this area, including a TRC rising sewer main and two recycled water pipelines.

8.2.2.4 Existing Case velocities

Peak Existing Case velocities on the floodplain areas are generally low, in the order of 0.5 to 1.0 metres per second (m/s) as shown in Figure A7-B. Existing velocities in the creek channels near the Project alignment for the 1% AEP event are shown in Table 8.18.

Table 8.18 Gowrie Creek – Existing Case – 1% AEP event peak velocities

Waterway	1% AEP Existing Case peak velocities (m/s)
Gowrie Creek channel	4.0 to 6.0
Tributary near western tunnel portal (Ch 3.40 km)	2.0 to 4.0

8.2.3 Intermediate ventilation shaft

8.2.3.1 Flood maps

Flood maps illustrating indicative flood extents and peak water levels within the vicinity of the intermediate ventilation shaft and associated infrastructure at Cranley were prepared and are presented in Appendix B on the following figures:

- 10% AEP: Figure B3-A
- 1% AEP: Figure B4-A
- 1 in 10,000 AEP: Figure B4-A.

8.2.3.2 Flood inundation extent and flood levels

Figure B3-A presents the 1% AEP peak water levels and flood inundation extent for the Gowrie Creek tributary near the intermediate ventilation shaft for the Existing Case. The peak depth in the channel of Gowrie Creek tributary is estimated to be up to 3.1 m and the 1% AEP extents vary between 20 m and 50 m approximately.

8.2.3.3 Flood immunity of existing infrastructure

Table 8.19 presents a summary of overtopping depths for the existing QR Western Line near the intermediate ventilation shaft under a range of design events. The overtopping depths for the QR Western Lineare estimated levels above the rail formation level.

Table 8.19 Intermediate Tunnel Shaft – Existing Case – Overtopping depths of key infrastructure

Infrastructure	Location	Maximum overtopping depth (m)			
		PMF	1 in 10,000 AEP	1% AEP	10% AEP
Existing QR Western Line	On Gowrie Creek tributary	0.73	0.32	Dry	Dry
Existing QR Western Line	Adjacent Wetalla wastewater treatment plant	0.27	0.18	0.09	Dry

8.2.3.4 Existing Case velocities

Peak Existing Case velocities for the 1% AEP event in the Gowrie Creek tributary are in the order of 2 to 5 m/s. On the floodplain velocities are generally in the order of 1 to 2.4 m/s as shown in Figure B6-B.

8.2.4 Oaky Creek

8.2.4.1 Flood maps

Flood maps illustrating indicative flood extents and peak water levels were prepared for Oaky Creek and are presented in Appendix C on the following figures:

- 20% AEP: Figure C2-A
- 10% AEP: Figure C3-A
- 5% AEP: Figure C4-A
- 2% AEP: Figure C5-A
- 1% AEP: Figure C6-A
- 1 in 2,000 AEP: Figure C7-A
- 1 in 10,000 AEP: Figure C8-A
- PMF: Figure C9-A.

8.2.4.2 Flood inundation extent and flood levels

Figure B7-A presents the 1% AEP peak water levels and flood inundation extent for Oaky Creek for the Existing Case. Under the 1% AEP event flood waters are 2.6 m deep in the creek channel and 1.0 m deep in the tributary in the vicinity of the Project alignment.

Due to the steepness of the terrain, flood waters under the 1% AEP event are generally contained within the main creek channel and the tributary channel with flood widths of approximately 20 m and 50 m respectively.

8.2.4.3 Flood immunity of existing infrastructure

Jones Road is a local roadway within the Oaky Creek catchment and runs along the western side of the creek. This road is at-grade and is inundated by frequent events. Table 8.20 presents a summary of overtopping depths under a range of design events.

Table 8.20 Oaky Creek – Existing case – Overtopping depths of key infrastructure

Infrastructure	Location	Approximate overtopping depth (m)				
		1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Jones Road	Approximately 80 m upstream of Project alignment	0.20	0.15	0.10	0.06	0.03

8.2.4.4 Existing Case velocities

Existing velocities in the Oaky Creek channel and in the western tributary channel near the Project for the 1% AEP event are shown in Table 8.21 and in Figure C6-B.

Table 8.21 Oaky Creek – Existing Case – 1% AEP event peak velocities

Waterway	1% AEP existing case peak velocities (m/s)
Oaky Creek – main creek crossing	2.0 to 3.5 m/s
Oaky Creek – western tributary	1.5 to 2.5 m/s

8.2.5 Six Mile Creek

8.2.5.1 Flood maps

Flood maps illustrating indicative flood extents and peak water levels for Six Mile Creek were prepared and are presented in Appendix D on the following figures:

- 20% AEP: Figure D2-A
- 10% AEP: Figure D3-A
- 5% AEP: Figure D4-A
- 2% AEP: Figure D5-A
- 1% AEP: Figure D6-A
- 1 in 2,000 AEP: Figure D7-A
- 1 in 10,000 AEP: Figure D8-A
- PMF: Figure D9-A.

8.2.5.2 Flood inundation extent and flood levels

Under the 1% AEP event flood waters are 4.5 m deep in the creek channel and 2.6 m deep in the tributary in the vicinity of the Project.

Due to the steepness of the terrain, flood waters under the 1% AEP event are generally contained to the creek channel and the tributary channel with flood widths of approximately 20 m and 40 m respectively as shown in Figure D6-A.

8.2.5.3 Flood immunity of existing infrastructure

Gittins Road is a local roadway within the Six Mile Creek catchment and crosses the creek approximately 150 m downstream of the Project alignment. This road is generally at natural surface level with a low-level floodway and culvert crossing over Six Mile Creek. This floodway is activated under frequent events and larger events with the flood depth and extent at the floodway increasing as the event size increases. Table 8.22 presents a summary of overtopping depths of Gittins Road under a range of design events.

Table 8.22 Six Mile Creek – Existing case – Overtopping depths of key infrastructure

Infrastructure	Location	Approximate overtopping depth (m)				
		1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Gittins Road	Floodway approximately 150 m downstream of Project alignment	2.50	2.30	2.10	1.85	1.70

8.2.5.4 Existing Case velocities

Existing velocities in the creek channel and in the western tributary channel near Project alignment for the 1% AEP event are shown in Table 8.23 and in Figure D6-B.

Table 8.23 Six Mile Creek – Existing Case – 1% AEP event peak velocities

Waterway	1% AEP existing case peak velocities (m/s)
Six Mile Creek – main creek crossing	2.0 to 3.0 m/s
Six Mile Creek – eastern tributary	1.5 to 2.5 m/s

8.2.6 Lockyer Creek

8.2.6.1 Flood maps

Flood maps illustrating indicative flood extents and peak water levels associated with Lockyer Creek were prepared and are presented in Appendix E on the following figures:

- 20% AEP: Figure E3-A
- 10% AEP: Figure E4-A
- 5% AEP: Figure E5-A
- 2% AEP: Figure E6-A
- 1% AEP: Figure E7-A
- 1 in 2,000 AEP: Figure E8-A
- 1 in 10,000 AEP: Figure E9-A
- PMF: Figure E10-A.

8.2.6.2 Flood inundation extent and flood levels

Under the 1% AEP event flood waters are approximately 6 m deep in the creek channel in the vicinity of the Project alignment.

Flood waters under the 1% AEP event, are generally contained to the Lockyer Creek channel with flood widths of approximately 60 m where the Project alignment crosses the creek, as shown in Figure E7-A.

8.2.6.3 Flood immunity of existing infrastructure

The QR Main Line runs adjacent to the bank of Lockyer Creek with Airforce Road and Cattos Road running on the northern side of the QR Main Line. None of this infrastructure is inundated by the 1% AEP event in the vicinity of the Project alignment. The Roma Brisbane Gas Pipeline is located underground including below the creek bed in this area.

8.2.6.4 Existing Case velocities

Existing velocities in the creek channel near the Project alignment for the 1% AEP event are shown in Table 8.24 and in Figure E7-B.

Table 8.24 Lockyer Creek – Existing Case – 1% AEP event peak velocities

Waterway	1% AEP existing case peak velocities (m/s)
Lockyer Creek	2.0 to 3.0 m/s

9 Developed case modelling

The Developed Case incorporates the Project design (i.e. rail and road) into the Existing Case hydraulic models. The Developed Case models have been run for the nominated design events and assessed against the hydraulic design criteria and flood impact objectives. Mitigation measures that have been incorporated into the Project design include:

- The Project has been designed to achieve the hydraulic design criteria (refer Section 4.1), and key design criteria including:
  - 50 year design life for formation and embankment performance
  - Track drainage ensures that the performance of the formation and track is not affected by water
  - Earthworks designed to ensure that the rail formation is not overtopped during a 1% AEP event
  - Embankment cross section can sustain flood levels up to the 1% AEP event
  - Tunnel design is immune to flood levels up to the 1 in 10,000 AEP event
- Bridges and viaducts are designed to withstand flood events up to and including the 1 in 2,000 AEP event
- Where possible, the Project is co-located with existing rail corridors to avoid introducing a new linear infrastructure corridor across floodplains
- The Project incorporates bridge and culvert structures to maintain existing flow paths and flood flow distributions
- Bridge and culvert structures have been located and sized to avoid increases in peak water levels, velocities and/or duration of inundation, and changes flow distribution in accordance with the flood impact objectives
- Progressive refinement of bridge extents and culvert banks (number of barrels and dimensions) has been undertaken as the Project design has evolved. This refinement process has considered engineering requirements as well as progressive feedback from stakeholders to achieve acceptable outcomes that address the flood impact objectives.
- Scour and erosion protection measures have been incorporated into the design in areas determined to be at risk, such as around culvert headwalls, drainage discharge pathways and bridge abutments
- A climate change assessment has been incorporated into the design of cross drainage structures for the Project in accordance with the ARR 2016 for the 1% AEP event to determine the sensitivity of the design, and associated impacts, to the potential increase in rainfall intensity
- Identification of flood sensitive receptors and engagement with stakeholders to determine acceptable design outcomes.

The following sections outline how the Project design addresses the hydraulic design criteria and flood impact objectives on each floodplain. For the hydraulic modelling, the adjacent B2G and H2C project alignments have been included in the Developed Case to quantify cumulative impacts.

Details of drainage structures for local drainage catchments that cross the Project alignment are provided in Section 9.6.

9.1 Gowrie Creek

The Project alignment is located to the south of the existing Western Line and runs parallel to the existing rail line and Gowrie Creek from Charlton (east of the Gowrie Creek bridge), before deviating to the southeast from the existing Western Line east of Gowrie. A number of Gowrie Creek tributaries flow northwards toward the main creek channel and cross under the existing Western Line and the Project alignment as well as a number of local roads.

The Project alignment does not intersect Gowrie Creek, but crosses a tributary of Gowrie Creek at Ch 3.45 km, near the proposed western tunnel portal as shown in Figure A1-A. As part of the works it is proposed to realign Gowrie Junction Road, resulting in a new crossing approximately 100 m downstream of the Old Homebush Road bridge over Gowrie Creek.

The Developed Case has also considered the permanent stockpile proposed at the western tunnel portal.

9.1.1 Drainage structures

The hydraulic design of the flood drainage structures was undertaken using the TUFLOW model (1D and 2D approach). On the Gowrie Creek floodplain, the Project design includes:

- One rail bridge (tributary of Gowrie Creek)
- Two rail reinforced concrete pipe culvert banks
- Seven rail reinforced concrete box culvert banks
- One road bridge (Gowrie Junction Road overpass over Gowrie Creek)
- One road reinforced concrete pipe culvert bank
- Five road reinforced concrete box culvert banks.

Details of the rail and road structures are outlined in Table 9.1 and Table 9.2 respectively. Figure A1-D present the locations of the proposed drainage structures.

Bridges have been modelled as a Layered Flow Constriction (LFC) in the TUFLOW model. Form loss coefficients have been calculated using Austroads (2018) and applied using the portion method. Each bridge has had a flow constriction coefficient applied to represent obstruction of waterway area due to the piers. The deck (layer 2) of the LFC has been 100% blocked. Where obverts vary across the structure, this is represented through a separate LFC points layer.

Table 9.1 Gowrie Creek – rail structure locations and details

Approximate Project chainage (km)	Structure name	Structure type	No of cells	Diameter or span (m)	Height (m)	Soffit level (m AHD)	Bridge length (m)
Ch -1.76	C-1.76	RCBC	16	2.4	1.2	-	-
Ch -1.42	C-1.42[1]	RCP	6	0.9	-	-	-
Ch -0.25	C-0.25[1,2]	RCP	4	1.65	-	-	-
Ch 0.11	C0.11[1]	RCBC	4	0.9	0.9	-	-
Ch 0.21	C0.21[1,2]	RCBC	6	3	2.7	-	-
Ch 1.03	C1.03[1,2]	RCBC	3	3	2.1	-	-
Ch 1.46	C1.46[1]	RCBC	2	1.2	0.6	-	-
Ch 2.41	C2.41[1]	RCBC	6	1.8	1.8	-	-
Ch 3.45	320-BR02	Bridge	-	-	-	494.4 to 495.3	70
Ch 3.54	C3.54[3]	RCBC	9	1.2	0.9		

Table notes:

1 The Developed Case alignment runs parallel to the QR Western Line rail corridor at this location. The existing culvert is proposed to be extended and matched through the proposed rail embankment.
2 The Developed Case alignment runs parallel to the QR Western Line rail corridor at this location. A new culvert(s) is proposed to be inserted through the QR rail embankment and the proposed rail embankment.
3 Located on a connection line to Western Line rail corridor.

Table 9.2 Gowrie Creek – road structure locations and details

Approximate Project chainage (km)	Road name	Structure type	No of cells	Diameter or span (m)	Height (m)	Soffit level (m AHD)	Bridge length (m)
Ch 1.85	Krienkes Road	RCBC	4	1.8	1.2	-	-
Ch 1.90	Gowrie Junction Road	RCP	3	1.8	-	-	-
Ch 1.90	McMahons Road	RCBC	5	1.8	0.6	-	-
Ch 1.90	Gowrie Junction Overpass (320-BR01)	Bridge	-	-	-	480.8 to 490.5	300
Ch 2.40	East Paulsens Road[1]	RCBC	6	1.8	1.8	-	-
Ch 3.30	East Paulsens Road[2]	RCBC	16	1.8	1.8	-	-

Table notes:

1 East Paulsens Road culverts located 30 m east of the existing Paulsens Road level crossing, near Project alignment Ch 2.40 km.
2 East Paulsens Road culverts located at the existing causeway on East Paulsens Road, near Project alignment connection to Western Line.

9.1.2 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 Project design outcomes relative to the hydraulic design criteria (refer Table 4.1) are presented in the following sections.

9.1.2.1 Flood immunity and overtopping risk

The Project has a 1% AEP immunity to formation level. The formation level of the Project 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 a minimum freeboard of 0.9 m.

The risk of overtopping of the Project alignment has been assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF Events). Table 9.3 presents the depth over formation and over top of rail during the 1 in 2,000 AEP, 1 in 10,000 AEP, and PMF events.

Table 9.3 Gowrie Creek – Overtopping of proposed rail formation and top of rail in extreme events

Approximate Project 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
Ch -1.76 to Ch -0.73	Dry	0.06	0.83	Dry	Dry	0.13
Ch -0.35 to Ch -0.23	Dry	Dry	0.62	Dry	Dry	Dry
Ch 0.67 to Ch 1.14	0.26	0.38	1.13	Dry	Dry	0.43
Ch 1.14 to Ch 1.45	0.06	0.26	0.89	Dry	Dry	0.19
Ch 1.45 to Ch 2.20	Dry	Dry	1.45	Dry	Dry	0.75
Ch 2.20 to Ch 2.50	Dry	0.14	1.26	Dry	Dry	0.56
Ch 2.50 to Ch 2.77	0.03	0.03	0.91	Dry	Dry	0.21
Ch 2.77 to Ch 3.23	0.18	0.27	1.09	Dry	Dry	0.39
Ch 3.55 to Ch 3.66	Dry	Dry	0.22	Dry	Dry	Dry
Ch 3.73 to Ch 3.92	Dry	Dry	0.42	Dry	Dry	Dry

9.1.2.2 Tunnel portal flood immunity

Figure A9-A presents the 1 in 10,000 AEP peak water levels near the proposed western tunnel portal. There is a tributary of Gowrie Creek that runs northwards close to the portal location with the closest peak water level being approximately 505 m AHD. The creek flood inundation extents do not reach the western tunnel portal. Diversion drains are proposed on either side of the tunnel portal to collect local runoff and prevent the runoff from entering the tunnel. In addition, a bund is proposed at the inlet of tunnel longitudinal drain to avoid local runoff entering the tunnel during extreme events (e.g. 1 in 10,000 AEP).

It is therefore considered that the western tunnel portal achieves the required 1 in 10,000 AEP event flood immunity.

9.1.2.3 Structures results

Table 9.4 presents hydraulic model results at each structure for the 1% AEP event. The hydraulic results at structures for flows, velocities and water surface levels for all events are presented in Appendix F.

Table 9.4 Gowrie Creek – 1% AEP event structure results

Approximate Project chainage (km)	Structure name	Structure type	Upstream peak water level (m AHD)	Freeboard to formation level (m)	Outlet velocity (m/s)	Peak discharge (m^3/s)
Ch 1.76	C-1.76	RCBC	458.3	2.1	1.6	55
Ch -1.42	C-1.42	RCP	460.8	0.9	1.0	2
Ch -0.25	C-0.25	RCP	466.0	3.2	1.3	2.4
Ch 0.11	C0.11[1]	RCBC	-	-	0	0
Ch 0.21	C0.21	RCBC	469.2	2.7	2.8	29
Ch 1.03	C1.0	RCBC	475.7	1.1	3.1	58
Ch 1.46	C1.46[1]	RCBC	478.1	1.9	0.7	0.5
Ch 2.41	C2.41[1]	RCBC	481.5	3.7	0	0
Ch 3.45	320-BR02	Bridge	494.6	1.0	3.6	125
Ch 3.54	C3.54	RCBC	495.4	2.7	2.1	4.1

Table note:

1 Culvert is required for extreme event flows and does not convey much flow in 1% AEP event.

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

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

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

There was insufficient information available at this stage to provide a meaningful scour assessment at each bridge site. A conservative scour estimation based on the 1 in 2,000 AEP event has been undertaken for pier substructure designs at each bridge site based on available information and will be refined during detailed design.

9.1.3 Flood impact objectives outcomes

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

9.1.3.1 Change in peak water levels

The impact of the Developed Case has been assessed through inclusion of the drainage structures and comparison of model results against the Existing Case results. Resulting changes in peak water levels (afflux) have been mapped and are presented in Appendix A on the following figures:

- 20% AEP: Figure A3-B
- 10% AEP: Figure A4-B
- 5% AEP: Figure A5-B
- 2% AEP: Figure A6-B
- 1% AEP: Figure A7-C
- 1 in 2,000 AEP: Figure A8-B
- 1 in 10,000 AEP: Figure A9-B
- PMF: Figure A10-B.

Locations of flood sensitive receptors are shown on all figures and are labelled on Figure A1-E. Details of flood sensitive receptors are presented in Appendix G with afflux for the full range of modelled events presented in Appendix H.

Under the 1% AEP event there are a few isolated occurrences of afflux on land predicted to be greater than 200 mm. These are:

- Around Ch -1.70 km, at the western end of the Project alignment, there is a localised area in which afflux is up to 470 mm. This dissipates to less than 200 mm within 25 m of the Project disturbance footprint. The majority of the impact is within the Project disturbance footprint and no flood sensitive receptors are affected.
- Around Ch 0.70 to 0.85 km, there is a localised area in which afflux is up to 390 mm. This dissipates to less than 200 mm within the Project disturbance footprint. This impact is caused by eliminating existing overtopping of the Western Line and Paulsens Road. The impact is within the Project disturbance footprint and no flood sensitive receptors are affected.
- On East Paulsens Road there is a proposed upgrade of the existing low-level crossing including new drainage structures. There is afflux of approximately 650 mm immediately upstream of the crossing which dissipates to 155 mm at the Western Line bridge and less than 100 mm at Morris Road.

There are no changes in peak water levels on any state-controlled roads for all events up to and including the PMF event.

Under the 1% AEP event there are two occurrences of afflux greater than 100mm in the vicinity of the QR Western Line. These are:

- Ch -1.76 km (at the G2H/B2G interface) – At this location the proposed alignment starts to deviate from the QR Western Line. The existing culverts under the QR Western Line are 5/3 x 2.1 RCBCs and there are no changes proposed to these structures. New culverts are included under the proposed alignment being 16/2.4 x 1.2 RCBCs (Ch-1.76). Under the 1% AEP existing case the Western Line overtops for a length of approximately 250 m. There are no impacts on the Gowrie Creek rail bridge.

Under the 1% AEP developed case event flood waters are retained behind the new rail embankment essentially throttling the flow reaching the QR Western Line. For the QR Western Line there is a very localised increase in peak water levels by up to 200 mm which occurs over a length of approximately 20 m however this section of the QR Western Line remains free from overtopping and flood levels are below the existing formation level. The rest of existing Western Line has improved immunity with the 250 m of rail that was previously overtopped now dry. This is because some local catchment flow is diverted by the new rail line to the west to another culvert in the B2G section. This has no impact on any other flood sensitive receptors.

- Ch 1.03 km – In this location the proposed alignment runs parallel (southern side) to the QR Western Line. Existing culverts under the QR Western Line are 2/3 x 2.1 RCBCs and it is proposed to extend these culverts under the Inland Rail alignment and add an additional 1/3 x 2.1 RCBC under both the QR Western Line and the proposed alignment. In the existing case at the C1.03 culvert crossing location, the 1% AEP flood level is below the top of rail of the Western Line by approximately 300mm and hence above existing formation level.

  In the developed case peak water levels have increased upstream of the proposed alignment with overtopping of the QR Western Line eliminated for extents around Ch 0.70 km and 0.97km. Instead of overtopping, flow ponds upstream of the proposed alignment before being conveyed through the upgraded culverts. The proposed alignment, and upgraded culverts, removes the existing overtopping and increases 1% AEP flood level on the downstream side of the QR Western Line by approximately 100mm near Ch 1.03 km. There is also an increase in the 1% AEP flood level of up to 180mm on Paulsens Road.

As part of detailed design further discussion with QR will be undertaken regarding the proposed alignment design and associated drainage structures.

9.1.3.2 Average annual time of submergence and time of submergence

The change in the ToS for the Gowrie Creek floodplain is presented in Figure A7-H. Table 9.5 presents details of the 1 % AEP event ToS for local infrastructure and locations which experienced an afflux level that slightly exceeds the flood impact objectives as discussed in Section 9.1.3.1. As can been seen the ToS is of short duration (generally < 2 to 5 hrs) for all locations due to the quick response of the catchment.

Increases in the ToS occur in a limited number of locations and do not affect flood sensitive receptors. There are a number of locations where the proposed works reduce the existing flood impact, including on the Western Line, Paulsens Road, McMahons Road and East Paulsens Road.

Table 9.5 ToS Comparison for Existing and Developed Case at key receptors – 1 % AEP event

Description	Approximate Project chainage (km)/location	Existing Case ToS (hrs)	Developed Case ToS (hrs)	Difference (hours)
Agricultural land	Ch -1.70	3	4.4	+1.4
Paulsens Road	Ch 0.10	2.1	1.8	-0.3
Paulsens Road	Ch 0.20 km	0.2	0	-0.2
Agricultural land	Ch 0.70 to Ch 0.85	1.3	0.6	-0.7
Western Line	Ch 0.70	3	0	-3
Paulsens Road	Ch 0.70	1.1	0	-1.1
Western Line	Ch 0.97	4.2	0	-4.2
Paulsens Road	Ch 0.97	0.2	1	+0.8
Paulsens Road	Ch 1.04	0.9	1.4	+0.5
Old Homebush Road	Bridge over Gowrie Creek	1.7	1.8	+0.1
McMahon Road	At connection to Gowrie Junction Road	0.6	0	-0.6
East Paulsen Road	At low level crossing	5.3	1.3	-4.0

Description	Approximate Project chainage (km)/location	Existing Case ToS (hrs)	Developed Case ToS (hrs)	Difference (hours)
Western Line	Upstream East Paulsens Road low-level crossing	0	0	0
Morris Road	Upstream East Paulsens Road low-level crossing	5.3	5.3	0
Ganzer Road	Ch 3.45	5.5	5.5	0

The AAToS for the 1% AEP event has been determined for local roads and the Western Line where overtopping occurs and is detailed in Table 9.6. The Project works do not result in a significant change to AAToS with a number of locations experiencing a reduction in AAToS. With the introduction of the Project alignment, and associated drainage, both the Western Line and Paulsens Road experience a reduction in overtopping with water held back by the Project alignment. The low-level crossing on East Paulsen Road is upgraded and therefore experiences a reduction in AAToS.

Table 9.6 AAToS Comparison for local roads and the Western Line

Description	Approximate Project chainage (km)/location	Existing Case AAToS (hrs/yr)	Developed Case AAToS (hrs/yr)	Difference (hrs/yr)
Paulsens Road	Ch 0.10	0.17	0.16	-0.01
Paulsens Road	Ch 0.20	0.01	0	0
Western Line	Ch 0.70	0.11	0.02	-0.09
Paulsens Road	Ch 0.70	0.04	0.01	-0.03
Western Line	Ch 0.97	0.06	0.02	-0.04
Paulsens Road	Ch 0.97	0.01	0.03	+0.03
Paulsens Road	Ch 1.04	0.03	0.06	+0.02
Old Homebush Road	Bridge over Gowrie Creek	0.10	0.11	+0.01
McMahon Road	At connection to Gowrie Junction Road	0.05	0	-0.05
East Paulsen Road	At low level crossing	2.22	0.10	-2.12
Western Line	Upstream East Paulsens Road low-level crossing	0.01	0.01	0
Morris Road	Upstream East Paulsens Road low-level crossing	5.93	5.93	0
Ganzer Road	Ch 3.45	7.39	7.36	-0.03

Table note:

hrs/yr = hours per year

9.1.3.3 Change in velocities

Figure A7-G presents the change in peak velocities, associated with the Project, under the 1% AEP event. In general, the changes are minor, with most changes in velocities experienced immediately adjacent to the Project disturbance footprint. Velocity changes within the Gowrie Creek main channel are minor (<0.03 m/s). The main area in which velocities change is around Ch 1.00 km which occurs as flows are redirected to the drainage culverts under the proposed and existing rail alignments at Ch 1.00 km preventing overtopping of the Western Line and Paulsens Road near Ch 0.70 km. This also prevents local overland flow through an existing Paulsen Road property near Ch 0.70 km. There is an increase in velocities of approximately 0.5 m/s upstream of Ch 1.00 km with an increase of approximately 0.3 m/s in the downstream overland flow path.

Further discussion will be undertaken with QR regarding the proposed alignment design and associated drainage structures.

9.1.3.4 Flood flow distribution

A key landowner concern was the potential changes to flow distributions. To assess potential changes to the flow distribution due to the Project, flows have been extracted from the hydraulic model at a number of locations across the floodplain as shown in Figure A7-I for the Existing and Developed cases for the 1% AEP event. The difference between the Existing Case and Developed Case was determined and is detailed in Table 9.7.

Table 9.7 Gowrie Creek – 1% AEP events – Flow comparison

Flow comparison location (refer Figure A7-I)	1% AEP event		
	Existing Case flow (m^3/s)	Developed Case flow (m^3/s)	% Change
L1	36	36	0
L2	65	65	0
L3	485	485	0
L4	532	533	+0.19
L5	69	68	-1.45
L6	600	605	+0.83
L7	28	28	0
L8	629	631	+0.32
L9	57	57	0
L10	20	20	0
L11	694	696	+0.29
L12	709	716	+0.99
L13	8	8	0
L14	10	10	0
L15	747	749	+0.27

Overall, the Project has minimal impacts on flood flows and floodplain conveyance/storage with significant floodplain structures, as detailed in Section 9.1.1, included and designed to maintain the existing flood regime.

9.1.3.5 Extreme event risk management

For the Existing Case, during extreme events there is widespread floodplain inundation with high flood depths as shown in Figures A8-A, A9-A and A10-A for the 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events respectively.

Impacts on the flooding regime as a result of the Project (i.e. Developed Case), are presented on Figures A-8B, A-9B and A-10B for the 1 in 2,000 AEP, 10,000 AEP and PMF events respectively. These impacts have been considered in relation to the Existing Case flood depths at flood sensitive receptors. Given the depth of flood waters that occur during extreme events in the Existing Case, particularly under the PMF event, the change in peak water levels associated with the Project would be unlikely to exacerbate flood conditions during extreme events.

Under these rare events, the bridge structures and culverts allow adequate passage of flow during the flood events and “damming” effects are therefore not expected to occur.

9.1.4 Sensitivity analysis

9.1.4.1 Blockage

The hydraulic design has included an assessment regarding the blockage of culverts. A significant community concern is the potential impact 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 roads/access roads and other infrastructure. Blockage potential has been assessed in accordance with the guidelines in ARR 2016. The blockage assessment resulted in no blockage factor being applied to bridges (as per the ARR 2016 guidelines, refer below) and a blockage factor of 25% being applied to culverts.

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

A 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 of culverts.

There is little change to the predicted impact on peak water levels as a result of reducing the applied culvert blockage allowance to 0%. As a result of increasing the blockage factor to 50%, increased afflux is experienced in localised areas upstream of the alignment, particularly around Ch 1.00 km and the East Paulsen Road low-level crossing upgrade. No flood sensitive receptors are adversely affected by 50% blockage scenario.

The impacts of the two blockage scenarios are shown in Figure A7-E (0%) and Figure A7-F (50%).

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

9.1.4.2 Climate change assessment

The impacts of climate change were assessed for the 1% AEP event to determine the sensitivity of the Project design to the potential increase in rainfall intensity. The assessment was undertaken in accordance with ARR 2016 guidelines.

The selected representative concentration pathway for the climate change analysis was 8.5. The climate change analysis was undertaken by increasing rainfall intensities in the IFDs for the contributing catchments. For the Project, 8.5 corresponds to an increase in temperature of 3.7 degrees Celsius in 2090 and an increase in rainfall intensity of 21% which was obtained from the ARR Data Hub (https://data.arr-software.org/).

The afflux levels under the climate change scenario are presented in Figure A7-D. Climate change leads to localised increases in afflux, in particular upstream of Ch 1.00 km. There are no flood sensitive receptors affected by the increase in peak water levels associated with climate change. Table 9.8 presents the climate change assessment outcomes with a reduction in minimum freeboard to 0.8 m.

Table 9.8 Gowrie Creek – Developed Case – 1% AEP event – Climate Change Assessment

Approximate Project chainage (km)	Structure name	1% AEP peak water levels (m AHD)	1% AEP +climate change in peak water levels (m AHD)	Difference in peak water level (m)	Freeboard to rail formation level with climate change (m)	Freeboard to rail formation level without climate change(m)
Ch -1.76	C-1.76	458.3	458.5	+0.2	1.9	2.1
Ch -1.42	C-1.42	460.8	460.9	+0.1	0.8	0.9

Approximate Project chainage (km)	Structure name	1% AEP peak water levels (m AHD)	1% AEP +climate change in peak water levels (m AHD)	Difference in peak water level (m)	Freeboard to rail formation level with climate change (m)	Freeboard to rail formation level without climate change(m)
Ch -0.25	C-0.25	466.8	466.9	+0.1	3.1	3.2
Ch 0.11	C0.11[1]	-	-	-	-	-
Ch 0.21	C0.21	469.2	470.1	+0.9	1.8	2.7
Ch 1.03	C1.0	475.7	475.9	+0.2	0.9	1.1
Ch 1.46	C1.46[1]	478.0	478.1	+0.1	1.8	1.9
Ch 2.41	C2.41[1]	481.5	481.7	+0.2	3.5	3.7
Ch 3.45	BR-002	494.6	494.7	+0.1	0.5	1
Ch 3.54	C3.54	495.4	495.5	+0.1	2.6	2.7

Table note:

1 Culvert is required for extreme event flows and does not convey much flow in 1% AEP event.

9.2 Intermediate ventilation shaft

9.2.1 Drainage structures

The Project runs through a tunnel for approximately 6.24 km, between Ch 4.10 km and Ch 10.40 km. A tunnel ventilation shaft is located at Cranley, near Ch 6.80 km, which is close to a tributary of Gowrie Creek.

At this location it is proposed to construct a ventilation building and other associated infrastructure (e.g. access road, car park and substation) with the required building pad encroaching into the channel of the waterway. To address impacts the design includes a mitigation measure consisting of a 120 m-long diversion channel around the pad with a 12 m base width with 1 in 2 batter slopes. Drainage structure locations are shown in Figure B1-C.

9.2.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case with the drainage structures and ventilation shaft pad embankment area included. Modelling of a range of events was undertaken (10%, 1%, 1 in 10,000 AEP and PMF events). The Project design outcomes relative to the hydraulic design criteria (refer Table 4.1) are presented in the following sections.

9.2.2.1 Flood immunity

Tunnel portals, including the intermediate ventilation shaft, require a 1 in 10,000 AEP event flood immunity. Table 9.9 presents the design outcomes and demonstrates that the intermediate ventilation shaft has more than the required level of flood immunity protection.

Table 9.9 Intermediate ventilation shaft – Flood immunity outcomes

Intermediate ventilation shaft pad level (m AHD)	1% AEP event		1 in 10,000 AEP event	
	Developed Case peak water level (m AHD)	Freeboard (m)	Developed Case peak water level (m AHD)	Freeboard (m)
543.50	540.20	3.30	540.57	2.93

9.2.2.2 Structures results

Table 9.10 presents hydraulic model results for the diversion channel for the 1% AEP event. Velocities within the proposed diversion channel exceed the recommended scour thresholds for unlined channels specified in Table 2.6 of AGRD. Scour protection has been designed in accordance with AGRD Section 2.9.2 and the velocities predicted from the hydraulic modelling.

Table 9.10 Gowrie Creek tributary – 1% AEP event – Diversion channel results

Local diversion channel chainage (m)	Developed Case water level (m AHD)	Channel grade (%)	Developed case channel velocity (m/s)	Scour protection – d_{50} (mm)/thickness (mm)
0	540.12	2.1%	2.3	150/225
20	539.57	2.1%	2.6	150/225
40	539.16	2.1%	2.6	150/225
60	538.69	2.1%	2.5	150/225
80	538.37	3.1%	2.4	150/225
100	538.37	3.1%	1.3	100/150
120	538.37	1.0%	0.8	Not required

9.2.3 Flood impact objectives outcomes

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

9.2.3.1 Change in peak water levels

The afflux due to the proposed words associated with the intermediate ventilation shaft pad has been assessed for the 1% and 1 in 10,000 AEP events. Peak water levels have been extracted in the local creek adjacent to the proposed intermediate ventilation shaft. The change in peak water levels at the intermediate ventilation shaft pad is presented in Table 9.11 for the 1% AEP and 1 in 10,000 AEP events. The locations of flood sensitive receptors are presented in Figure B1-D.

Table 9.11 Intermediate ventilation shaft – Change in peak water levels

Location	Tunnel Pad Level (m AHD)	1% AEP event			1 in 10,000 AEP event		
		Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Change in peak water levels (mm)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Change in peak water levels (mm)
Upstream of intermediate ventilation shaft pad	543.50	540.94	540.92	-20	541.34	541.33	-10
Adjacent intermediate ventilation shaft pad	543.50	540.03	540.20	+170	540.38	540.56	+180
Downstream of intermediate ventilation shaft pad	543.50	538.38	538.37	-10	539.45	539.44	-10

The extra flood storage that the proposed diversion channel creates lowers water levels upstream and downstream of the intermediate ventilation shaft pad. Changes in peak water levels extend approximately 45 m upstream and 40 m downstream of the proposed intermediate ventilation shaft pad under the 1% AEP event as shown in Figure B3-C. Changes in water levels extend approximately 50 m upstream and 70 m downstream of the proposed intermediate ventilation shaft under the 1 in 10,000 AEP event as shown in Figure B4-B.

9.2.3.2 Average annual time of submergence and time of submergence

As shown in Figure B3-F, under the 1% AEP event there are minor changes in ToS immediately downstream of the intermediate ventilation shaft pad, however no flood sensitive receptors are affected.

There are no roads located within the area where the changes in peak water levels occur under the 1% AEP event. Therefore, no AAToS calculations have been undertaken.

9.2.3.3 Change in velocities

Figure B3-E presents the change in peak velocities under the 1% AEP event associated with the Project. Changes in velocities are experienced 50 m upstream and 50 m downstream of the proposed intermediate ventilation shaft pad and relate to the introduction of the diversion channel around the intermediate ventilation shaft pad. These localised impacts do not affect any flood sensitive receptors.

Peak velocities up to 2.6 m/s are expected within the proposed diversion channel and appropriate scour protection has been designed as detailed in Table 9.11.

9.2.3.4 Flood flow distribution

Overall, the works associated with the intermediate ventilation shaft have negligible impacts on flood flows and floodplain conveyance/storage. Flood flows are contained to the proposed diversion channel.

9.2.3.5 Extreme event risk management

Figure B5-A presents the Existing Case flood inundation extents under the 1 in 10,000 AEP event. Figure B5-B presents the afflux associated with the proposed intermediate ventilation shaft pad and diversion channel under the 1 in 10,000 AEP event. The impacts under this extreme event are localised and do not affect any flood sensitive receptors.

9.2.4 Sensitivity analysis

9.2.4.1 Blockage

There are no bridges or culverts proposed at this location and therefore blockage assessment was not required.

9.2.4.2 Climate change assessment

The impacts of climate change were assessed for the 1% AEP event to determine the sensitivity of the Project design to the potential increase in rainfall intensity. The assessment was undertaken in accordance with ARR 2016 guidelines.

The selected representative concentration pathway for the climate change analysis was 8.5. The climate change analysis was undertaken by increasing rainfall intensities in the IFDs for the contributing catchments. For the Project, a representative concentration pathway of 8.5 corresponds to an increase in temperature of 3.7 degrees Celsius in 2090 and an increase in rainfall intensity of 21% which was obtained from the ARR Data Hub (https://data.arr-software.org/).

With an increase in rainfall intensity of 21% across the local catchment area, the intermediate ventilation shaft pad is predicted not to overtop under 1% AEP event with the climate change scenario.

The resulting peak water levels are presented in Table 9.12. Climate change results in increased peak water levels of up to 500 mm at structure locations for the 1% AEP event. The intermediate ventilation shaft pad is significantly higher than the 1% AEP climate change peak water levels at these locations. There are no flood sensitive receptors affected by changes in peak water levels due to climate change.

Table 9.12 Gowrie Creek tributary – Developed Case – 1% AEP event – Climate Change Assessment

Location	1% AEP peak water levels (m AHD)	1% AEP + climate change peak water levels (m AHD)	Difference in peak water level due to climate change (mm)	Freeboard to intermediate ventilation shaft pad level with climate change (m)
Upstream of intermediate ventilation shaft pad	540.92	541.04	+120	2.46
Proposed diversion channel (channel chainage 60 m)	538.69	538.88	+19	4.62
Adjacent intermediate ventilation shaft pad	540.20	540.29	+90	3.21
Downstream of intermediate ventilation shaft pad	538.37	538.87	+50	4.63

9.3 Oaky Creek

9.3.1 Drainage structures

The hydraulic design of the flood drainage structures was undertaken using the TUFLOW model (1d and 2d approach). Over Oaky Creek the Project design includes:

- Oaky Creek Viaduct.

Details of this structure are outlined in Table 9.13 and are shown in Figure C1-D. The bridge is high level and located at approximately Ch 12.00 km and has an overall length of 736 m and completely extends across the floodplain of Oaky Creek. There are no footings proposed within the channel of Oaky Creek and associated tributaries.

Bridges have been modelled as a Layered Flow Constriction (LFC) in the TUFLOW model. Form loss coefficients have been calculated using Austroads (2018) and applied using the portion method. Each bridge has had a flow constriction coefficient applied to represent obstruction of waterway area due to the piers. The deck (layer 2) of the LFC has been 100% blocked. Where obverts vary across the structure, this is represented through a separate LFC points layer.

Table 9.13 Oaky Creek – flood structure locations and details

Chainage (km)	Structure name	Structure type	Soffit level (m AHD)	Bridge length (m)
12.00	320-BR04	Bridge	360.20	736.0

9.3.2 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 Project design outcomes relative to the hydraulic design criteria (refer Table 4.1) are presented in the following sections.

9.3.2.1 Flood immunity and overtopping risk

The Project requires a 1% AEP immunity to formation level. In this area, the formation level of the Project alignment is driven by meeting geometric requirements and the Project alignment is well above flood levels for all events up to the Probable Maximum Flood with no overtopping occurring. The freeboard achieved under the 1% AEP event is in excess of 27 m.

9.3.2.2 Structures results

Table 9.14 presents hydraulic model results at the Oaky Creek Viaduct for all modelled events.

Table 9.14 Oaky Creek – Design event structure results at Oaky Creek Viaduct (320-BR04)

AEP	Peak water level (m AHD)	Rail formation level (m AHD)	Freeboard to formation level (m)	Velocity (m/s)	Peak discharge (m^3/s)
20%	334.69	362.20	27.51	2.2	29
10%	334.89	362.20	27.31	2.4	36
5%	334.95	362.20	27.25	2.6	42
2%	334.98	362.20	27.22	2.8	55
1%	335.05	362.20	27.15	2.9	64
1 in 2,000	335.42	362.20	26.78	3.6	118
1 in 10,000	335.63	362.20	26.57	4.1	165
PMF	336.26	362.20	25.94	5.4	495

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

9.3.3 Flood impact objectives outcomes

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

9.3.3.1 Change in peak water levels

The impact of the Developed Case has been assessed through inclusion of the bridge structure and comparison of model results against the Existing Case results. Resulting changes in peak water levels (afflux) have been mapped and are presented in Appendix C on the following figures:

- 20% AEP: Figure C2-B
- 10% AEP: Figure C3-B
- 5% AEP: Figure C4-B
- 2% AEP: Figure C5-B
- 1% AEP: Figure C6-C
- 1 in 2,000 AEP: Figure C7-B
- 1 in 10,000 AEP: Figure C8-B
- PMF: Figure C9-B.

Locations of flood sensitive receptors are shown on all figures and are labelled on Figure C1-E. Details of flood sensitive receptors are presented in Appendix G with afflux for the full range of modelled events presented in Appendix H.

Afflux at the Oaky Creek Viaduct is presented in Table 9.11 for the 1% AEP event.

Table 9.15 Oaky Creek – 1% AEP event – Change in peak water levels at Oaky Creek Viaduct (320-BR04)

Chainage (km)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Difference (mm)
12.00	335.05	335.05	0

There are no locations where changes in peak water levels lie outside the flood impact objectives. This is due to the viaduct nature of the bridge structure including large spans and high deck levels.

9.3.3.2 Average annual time of submergence and time of submergence

The change in the ToS is presented in Figure C6-D and no change in ToS occurs. Jones Road is a local road that passes under the Project alignment next to Oaky Creek. This road is low level in parts and with these locations inundated by shallow flow under the 20% AEP event and larger events.

Under the 1% AEP event there is no increase in the ToS and no flood sensitive receptors are affected. There is also no change to the AAToS at this location.

Table 9.16 outlines the AAToS for the 1% AEP Existing and Developed cases for Jones Road.

Table 9.16 AAToS comparison at Jones Road

Location	AAToS Existing Case (hrs/yr)	AAToS Developed Case (hrs/yr)	Difference (hrs/yr)
Jones Road	0.073	0.073	0

9.3.3.3 Change in velocities

Figure C6-E presents the change in peak velocities under the 1% AEP event associated with the Project. The changes in peak velocities are very minor and limited in extent.

9.3.3.4 Flood flow distribution

Overall, the Project has minimal impacts on flood flows and floodplain conveyance/storage with a significant viaduct structure included that maintains the existing flood regime.

9.3.3.5 Extreme event risk management

The flood inundation extents under the extreme events are presented in Figure C7-A, C8-A and C9-A. The Project alignment across Oaky Creek consists of a high-level viaduct structure and as such all flood events pass under the structure without any impacts or any overtopping under the modelled extreme events.

9.3.4 Sensitivity analysis

9.3.4.1 Blockage

Blockage potential has been assessed in accordance with the guidelines in ARR 2016. The blockage assessment resulted in no blockage factor being applied to the Oaky Creek Viaduct. ARR 2016 guidelines are focused on blockage of small bridges and culverts. The proposed Oaky Creek Viaduct is a high-level multi-span large bridge and ARR 2016 notes that there are limited instances of multiple span bridges being observed with blockages similar to those seen at single span bridges or culverts.

9.3.4.2 Climate change assessment

The impacts of climate change were assessed for the 1% AEP event to determine the sensitivity of the Project design to the potential increase in rainfall intensity. The assessment was undertaken in accordance with ARR 2016 guidelines.

The selected representative concentration pathway for the climate change analysis was 8.5. The climate change analysis was undertaken by increasing rainfall intensities in the IFDs for the contributing catchments. For the Project, 8.5 corresponds to an increase in temperature of 3.7 degrees Celsius in 2090 and an increase in rainfall intensity of 21% which was obtained from the ARR Data Hub (https://data.arr-software.org/).

Table 9.17 and Figure C6-F present the change in peak water levels associated with the Project for the 1% AEP event with climate change. Climate change leads to an increase in peak water levels upstream of the Project alignment of approximately 210 mm. This does not impact on any flood sensitive receptors and due to the high-level crossing structure, the high freeboard is maintained.

Table 9.17 Oaky Creek – 1% AEP event – Climate Change Assessment

Chainage (km)	Structure type	Existing Case 1% AEP peak water levels (m AHD)	Developed Case 1% AEP + Climate change peak water levels (m AHD)	Difference in peak water level (m)	Freeboard to rail formation level with climate change (m)
12.00	Bridge	335.05	335.26	+0.21	26.94

9.4 Six Mile Creek

9.4.1 Drainage structures

The hydraulic design of the flood drainage structures was undertaken using the TUFLOW model (1d and 2d approach). Over Six Mile Creek the Project design includes:

- TSCR and Six Mile Creek Viaduct

Details of this structure is outlined in Table 9.18 and is shown in Figure D1-D. The bridge is high level and located at approximately Ch 16.00 km and has an overall length of 966 m and completely extends across the floodplain of Six Mile Creek. There are no footings proposed within the channel of Six Mile Creek and associated tributaries.

Bridges have been modelled as a Layered Flow Constriction (LFC) in the TUFLOW model. Form loss coefficients have been calculated using Austroads (2018) and applied using the portion method. Each bridge has had a flow constriction coefficient applied to represent obstruction of waterway area due to the piers. The deck (layer 2) of the LFC has been 100% blocked. Where obverts vary across the structure, this is represented through a separate LFC points layer.

Table 9.18 Six Mile Creek – flood structure locations and details

Chainage (km)	Structure name	Structure type	Soffit level (m AHD)	Bridge length (m)
16.00	320-BR10	Bridge	292.78	966

9.4.2 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 Project design outcomes relative to the hydraulic design criteria (refer Table 4.1) are presented in the following sections.

9.4.2.1 Flood immunity and overtopping risk

The Project requires a 1% AEP immunity to formation level. In this area, the formation level of the Project alignment is driven by meeting geometric requirements and the Project alignment is well above flood levels for all events up to the PMF with no overtopping occurring. The freeboard achieved under the 1% AEP event is in excess of 40 m.

9.4.2.2 Structures results

Table 9.19 presents hydraulic model results at the Toowoomba Bypass and Six Mile Creek Viaduct for all modelled events.

Table 9.19 Six Mile Creek – Design event structure results at Toowoomba Bypass and Six Mile Creek Viaduct (320-BR10)

AEP	Peak water level (m AHD)	Rail formation level (m AHD)	Freeboard to formation level (m)	Velocity (m/s)	Peak discharge (m^3/s)
20%	248.73	292.78	44.05	2.1	73
10%	248.87	292.78	43.91	2.1	84
5%	249.02	292.78	43.76	2.3	101
2%	249.27	292.78	43.51	2.5	125
1%	248.43	292.78	44.35	2.6	145
1 in 2,000	250.30	292.78	42.48	3.5	275
1 in 10,000	251.15	292.78	41.63	4.0	375
PMF	254.20	292.78	38.58	6.8	973

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

9.4.3 Flood impact objectives outcomes

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

9.4.3.1 Change in peak water levels

The impact of the Developed Case has been assessed through inclusion of the bridge structure and comparison of model results against the Existing Case results. Resulting changes in peak water levels (afflux) have been mapped and are presented in Appendix D on the following figures:

- 20% AEP: Figure D2-B
- 10% AEP: Figure D3-B
- 5% AEP: Figure D4-B
- 2% AEP: Figure D5-B
- 1% AEP: Figure D6-C
- 1 in 2,000 AEP: Figure D7-B
- 1 in 10,000 AEP: Figure D8-B
- PMF: Figure D9-B.

Locations of flood sensitive receptors are shown on all figures and are labelled on Figure D1-E. Details of flood sensitive receptors are presented in Appendix G with afflux for the full range of modelled events presented in Appendix H.

Afflux at the bridge is presented in Table 9.20 for the 1% AEP event.

Table 9.20 Six Mile Creek – 1% AEP event – Change in peak water levels at Bridge 320-BR10

Chainage (km)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Difference (mm)
16.00	249.42	251.43	+60

There are no locations where changes in peak water levels lie outside the flood impact objectives. This is due to the viaduct nature of the bridge structure including large spans and high deck levels.

9.4.3.2 Average annual time of submergence and time of submergence

The change in the Time of Submergence (ToS) is presented in Figure D6-D and no significant change in ToS occurs. Gittens Road is a local access road that passes under the Project alignment next to Six Mile Creek. This road includes a low-level causeway that is inundated by frequent events.

Under the 1% AEP event there is no increase in the ToS and no flood sensitive receptors are affected. There is also no change to the AAToS.

Table 9.21 outlines the AAToS for the 1% AEP Existing and Developed Cases for Gittens Road.

Table 9.21 AAToS comparison at Gittens Road

Location	AAToS Existing Case (hrs/yr)	AAToS Developed Case (hrs/yr)	Difference (hrs/yr)
Gittens Road	1.152	1.152	0

9.4.3.3 Change in velocities

Figure D6-E presents the change in peak velocities under the 1% AEP event associated with the Project alignment. The change in peak velocities are very minor and very limited in extent.

9.4.3.4 Flood flow distribution

Overall, the Project has minimal impacts on flood flows and floodplain conveyance/storage with a significant bridge structure included that maintains the existing flood regime.

9.4.3.5 Extreme event risk management

The flood inundation extents under the extreme events are presented in Figures D7-A, D8-A and D9-A. The Project alignment across Six Mile Creek consists of a high-level viaduct structure and as such all flood events pass under the structure without any impacts or any overtopping under the modelled extreme events.

9.4.4 Sensitivity analysis

9.4.4.1 Blockage

Blockage potential has been assessed in accordance with the guidelines in ARR 2016. The blockage assessment resulted in no blockage factor being applied to the Toowoomba Bypass and Six Mile Creek Viaduct. ARR 2016 guidelines are focused on blockage of small bridges and culverts. The proposed Toowoomba Bypass and Six Mile Creek Viaduct is a high-level multi-span large bridge and ARR 2016 notes that there are limited instances of multiple span bridges being observed with blockages similar to those seen at single span bridges or culverts.

9.4.4.2 Climate change assessment

The impacts of climate change were assessed for the 1% AEP event to determine the sensitivity of the Project design to the potential increase in rainfall intensity. The assessment was undertaken in accordance with ARR 2016 guidelines.

The selected representative concentration pathway for the climate change analysis was 8.5. The climate change analysis was undertaken by increasing rainfall intensities in the IFDs for the contributing catchments. For the Project, 8.5 corresponds to an increase in temperature of 3.7 degrees Celsius in 2090 and an increase in rainfall intensity of 21% which was obtained from the ARR Data Hub (https://data.arr-software.org/).

Table 9.22 and Figure D6-F present the change in peak water levels associated with the Project for the 1% AEP event with climate change. Climate change leads to an increase in peak water levels upstream of the Project alignment of approximately 270 mm. This does not impact on any flood sensitive receptors and due to the high-level crossing structure, the high freeboard is maintained.

Table 9.22 Six Mile Creek – 1% AEP event – Climate Change Assessment

Chainage (km)	Structure type	1% AEP peak water levels (m AHD)	1% AEP + climate change peak water levels (m AHD)	Difference in peak water level (m)	Freeboard to rail formation level with climate change (m)
16.00	Bridge	249.42	249.69	+0.27	43.09

9.5 Lockyer Creek

9.5.1 Drainage structures

The hydraulic design of the flood drainage structures was undertaken using the TUFLOW model (1d and 2d approach). Over Lockyer Creek the Project design includes:

- Lockyer Creek Viaduct.

Details of this structure are outlined in Table 9.23 and are shown in Figure E1-D. The bridge is high level and located at approximately Ch 24.50 km and has an overall length of 506 m and extends across the floodplain of Lockyer Creek.

Bridges have been modelled as a Layered Flow Constriction (LFC) in the TUFLOW model. Form loss coefficients have been calculated using Austroads (2018) and applied using the portion method. Each bridge has had a flow constriction coefficient applied to represent obstruction of waterway area due to the piers. The deck (layer 2) of the LFC has been 100% blocked. Where obverts vary across the structure, this is represented through a separate LFC points layer.

Table 9.23 Lockyer Creek – flood structure locations and details

Project chainage (km)	Structure name	Structure type	Soffit level (m AHD)	Bridge length (m)
Ch 24.50	320-BR14	Bridge	167.45	506.00

9.5.2 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 Project design outcomes relative to the hydraulic design criteria (refer Table 4.1) are presented in the following sections.

9.5.2.1 Flood immunity and overtopping risk

The Project requires a 1% AEP immunity to formation level. In this area, the formation level of the Project alignment is driven by meeting geometric requirements and the Project alignment is well above flood levels for all events up to the PMF with no overtopping occurring. The freeboard achieved under the 1% AEP event is in excess of 20 m.

9.5.2.2 Structures results

Table 9.24 presents hydraulic model results at the Lockyer Creek Viaduct for all modelled events.

Table 9.24 Lockyer Creek – Design event structure results at Lockyer Creek Viaduct (320-BR14)

AEP	Peak water level (m AHD)	Rail formation level (m AHD)	Freeboard to formation level (m)	Velocity (m/s)	Peak discharge (m^3/s)
20%	143.72	167.45	23.73	1.4	96
10%	144.92	167.45	22.53	2.0	215
5%	145.57	167.45	21.88	2.2	295
2%	146.56	167.45	20.89	2.7	473
1%	147.02	167.45	20.23	3.0	612
1 in 2,000	149.05	167.45	18.40	3.7	1,089
1 in 10,000	149.65	167.45	17.80	3.9	1,294
PMF	154.70	167.45	12.75	6.3	3,995

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

9.5.3 Flood impact objectives outcomes

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

9.5.3.1 Change in peak water levels

The impact of the Developed Case has been assessed through inclusion of the Project design and comparison of model results against the Existing Case results for Lockyer Creek. Resulting changes in peak water levels have been mapped and are presented in Appendix E on the following figures:

- 20% AEP: Figure D3-B
- 10% AEP: Figure D4-B
- 5% AEP: Figure D5-B
- 2% AEP: Figure D6-B
- 1% AEP: Figure D7-C
- 1 in 2,000 AEP: Figure D8-B
- 1 in 10,000 AEP: Figure D9-B
- PMF: Figure D10-B.

Locations of flood sensitive receptors are shown on all figures and are labelled on Figure E1-E. Details of flood sensitive receptors are presented in Appendix G with afflux for the full range of modelled events presented in Appendix H.

Afflux at the bridge is presented in Table 9.25 for the 1% AEP event.

Table 9.25 Lockyer Creek – 1% AEP event – Change in peak water levels at Lockyer Creek Viaduct (320-BR14)

Project chainage (km)	Existing Case peak water level (m AHD)	Developed Case peak water level (m AHD)	Difference (mm)
Ch 24.50	146.97	147.02	+50

There are no locations where changes in peak water levels lie outside the flood impact objectives. This is due to the viaduct nature of the bridge structure including large spans and high deck levels.

There are no changes in peak water levels on any state controlled roads for all events up to and including the PMF event.

9.5.3.2 Average annual time of submergence and time of submergence

The change in the ToS is presented in Figure E7-D and no significant change in ToS occurs. The QR Main Line and the realigned Cattos Road pass under the Lockyer Creek Viaduct east of Lockyer Creek.

Under the 1% AEP event there is no increase in the ToS and no flood sensitive receptors are affected. There is also no change to the AAToS.

Table 9.26 outlines the AAToS for the 1% AEP Existing and Developed cases for Cattos Road and the QR Main Line.

Table 9.26 AAToS comparison at Cattos Road and QR Main Line

Location	AAToS Existing Case (hrs/yr)	AAToS Developed Case (hrs/yr)	Difference (hrs/yr)
Cattos Road	0.90	0.90	0
QR Rail Line	0.90	0.90	0

9.5.3.3 Change in velocities

Figure E7-E presents the change in peak velocities under the 1% AEP event associated with the Project alignment. The change in peak velocities are very minor and very limited in extent.

9.5.3.4 Flood flow distribution

Overall, the Project has minimal impacts on flood flows and floodplain conveyance/storage with a significant bridge structure included that maintains the existing flood regime.

9.5.3.5 Extreme event risk management

The flood inundation extents under the extreme events is presented in Figures E8-A, E9-A and E10-A. The Project alignment across Lockyer Creek consists of a high-level viaduct structure and as such all flood events pass under the structure without any impacts or any overtopping under the modelled extreme events.

9.5.4 Sensitivity analysis

9.5.4.1 Blockage

Blockage potential has been assessed in accordance with the guidelines in ARR 2016. The blockage assessment resulted in no blockage factor being applied to the Lockyer Creek Viaduct. ARR 2016 guidelines are focused on blockage of small bridges and culverts. The proposed Lockyer Creek Viaduct is a high-level multi-span large bridge and ARR 2016 notes that there are limited instances of multiple span bridges being observed with blockages similar to those seen at single span bridges or culverts.

9.5.4.2 Climate change assessment

The impacts of climate change were assessed for the 1% AEP event to determine the sensitivity of the Project design to the potential increase in rainfall intensity. The assessment was undertaken in accordance with ARR 2016 guidelines.

The selected representative concentration pathway for the climate change analysis was 8.5. The climate change analysis was undertaken by increasing rainfall intensities in the IFDs for the contributing catchments. For the Project, 8.5 corresponds to an increase in temperature of 3.7 degrees Celsius in 2090 and an increase in rainfall intensity of 21% which was obtained from the ARR Data Hub (https://data.arr-software.org/).

Table 9.27 and Figure E7-F present the change in peak water levels associated with the Project for the 1% AEP event with climate change for Lockyer Creek. Climate change leads to an increase in peak water levels upstream of the Project alignment of approximately 680 mm. This does not impact on any flood sensitive receptors and due to the high-level crossing structure the high freeboard is maintained.

Table 9.27 Lockyer Creek – Developed Case – 1% AEP event – Climate Change Assessment

Project chainage (km)	Structure type	Existing Case 1% AEP peak water levels (m AHD)	Developed Case 1% AEP + Climate change peak water levels (m AHD)	Difference in peak water level (m)	Freeboard to rail formation level with climate change (m)
Ch 24.50	Bridge	146.97	147.65	+0.68	23.55

9.6 Local catchment drainage

The following section details the hydraulic assessment that has been undertaken for cross drainage for the local catchments along the rail alignment which are outside the regional floodplain extents.

9.6.1 Hydrology

9.6.1.1 Drainage catchment classification

The Project alignment crosses a number of existing flowpaths of varying contributing catchment areas that contribute flows to the cross drainage structures. The existing catchments were categorised based on the contributing catchment areas to determine the appropriate hydrologic methods for the local drainage design. Table 9.28 shows the drainage catchment classification criteria and number of catchments relating to each classification.

Table 9.28 Drainage catchment classification

Catchment size	Drainage catchment classification	Number of catchments
Less than or equal to 10 km^2	Minor	38
Greater than 10 km^2 and less than or equal to 100 km^2	Moderate	0
Greater than 100 km^2	Major	2

The major floodplains (Gowrie Creek, Oaky Creek, Six Mile Creek and Lockyer Creek) are addressed in Sections 9.1 to 9.5.

9.6.1.2 Minor catchments

The 1% and 1 in 2,000 AEP catchment flows for the minor catchments were generated in accordance with ARR 2016 using the ILSAX hydrologic model 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 alignment.

As no calibration data was available to compare against the local catchment flows, the 1% AEP flows generated from ILSAX were compared against the traditional Rational Method. The flows generated using ILSAX compared closely to the flows generated from the traditional Rational Method and were within a tolerance of -8 to 9%.

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.

9.6.2 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 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.

9.6.2.1 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 9.6.2.2
- 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 I.

9.6.2.2 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 local catchments has been classified as heavily timbered/agricultural grazing/pastoral land based on aerial imagery. Sensitive agricultural land may be identified during further consultation with landholders which will need to be considered in the design at the next stage. The hydraulic impacts in the local catchments are considered 'localised' in comparison to regional flood impacts due to the shorter time of inundations and smaller flood extents. Therefore, afflux up to 400 mm was considered acceptable at the rail corridor in the local drainage catchments.

A maximum afflux of 400 mm has been achieved at the rail corridor in the local drainage catchments which meets the adopted criteria. The afflux and change in time of inundation is documented in Appendix I. The predicted impacts all comply with the flood impact objectives.

9.6.3 Sensitivity analysis

9.6.3.1 Blockage

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

The Project alignment intercepts steep catchments with dense vegetation in Lockyer Valley. At these locations, the design blockage factor was calculated to be between 50 to 100% for the 1% AEP (as per ARR 2016 Book 6 Chapter 6) which results in a high number of culverts to achieve the required flood impact criteria and design immunity. To mitigate the blockage potential at these culverts, debris deflector walls have been specified at the inlets of the culverts which decreases the blockage factor to 25% to account for sediment blockage.

A 25% blockage was adopted during feasibility design for all structures along the alignment and debris deflector walls have been indicated in the register where required.

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.

A minimum diameter or height of 900 mm was applied where possible for proposed culverts along the alignment to reduce the risk of blockage and maintenance requirements.

9.6.3.2 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 for the climate change analysis was 8.5 which represents a high emissions scenario. For the Project, a representative concentration pathway of 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 Datahub.

The climate change analysis was undertaken by increasing rainfall intensities within the IFDs for the local catchments. The climate change factor increases the 1% AEP local drainage water levels by a maximum of 0.66 m along the alignment. However, the flood immunity of the rail formation is not adversely affected by climate change within the local catchments with the minimum freeboard along the alignment being 0.21 m.

10 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.

11 Conclusions

The key objectives of the Hydrology and Flooding Technical Report are to provide information on the data investigation, hydrologic 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 four waterway catchments that the Project alignment crosses, with the main waterway being Gowrie Creek. Gowrie Creek flooding affects the western tunnel portal and intermediate tunnel shaft locations and the Project alignment traverses a significant portion of the Gowrie Creek floodplain area. The other waterways crossings include Oaky Creek, Six Mile Creek and the Upper Lockyer Creek. Six Mile Creek and Oaky Creek flow under the Project alignment where it is on viaduct and there is minimal impediment to the waterway. The Project alignment has a single bridge crossing over Lockyer Creek in its upper reaches before the confluence of Rocky Creek with Lockyer Creek and therefore the Project alignment does not cross Rocky Creek.

The Project runs on the southern side of the existing QR Western Line rail corridor at Charlton and Gowrie Junction, on the northern side of the existing QR Main Line at Helidon, and crosses both the Toowoomba and Lockyer Valley LGAs.

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 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 a Flood Frequency Analysis (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 (refer 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 11.1.

Table 11.1 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. Tunnel portals and intermediate tunnel shaft – 1 in 10,000 AEP event flood immunity to Gowrie Creek flood events has been achieved.

Performance criteria	Design outcomes
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: ▪ Gowrie Creek – One rail bridge – Nine rail culvert banks – One road bridge – Six road culvert banks ▪ Oaky Creek, Six Mile Creek and Lockyer Creek – one high-level rail bridge at each location. In addition, drainage structures are included for local catchment crossings.
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 specified in Table 3.1 of AGRD. Required lengths of scour protection have been determined and are predicted to fit within the Project disturbance footprint. A conservative scour estimation has been undertaken at each bridge site based on available information and will be refined during detailed design.
Structural design	1 in 2,000 AEP event has been modelled with details used for bridge design purposes.
Extreme events	Overtopping of the Project alignment under extreme events occurs at limited locations being: ▪ Gowrie Creek (as detailed in Table 9.3) – Above formation level at between Ch 0.67 km and Ch 1.45 km and between Ch 2.50 and 3.23 km under the 1 in 2,000 AEP event. – Above formation level at between Ch-1.76 km and Ch 0.73 km, between Ch 0.67 km and Ch 1.45 km and between Ch 2.20 and 3.23 km under the 1 in 10,000 AEP event – Below top of rail level for 1 in 2,000 and 1 in 10,000 AEP events – Above formation level and top of rail for whole alignment under PMF event ▪ Oaky Creek, Six Mile Creek and Lockyer Creek – no overtopping.
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, where applicable 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 11.2 summarises how the Project design performs against each of the flood impact objectives.

Table 11.2 **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 and grazing land/forest areas and other non-agricultural land
	≤ 10 mm	≤ 50 mm	≤ 100 mm	≤ 100 mm	≤ 200 mm with localised areas up to 400 mm
	Objective: Changes in peak water levels are to be assessed against the above proposed limits. **Outcome:** Generally, the Project design meets the above limits with a number of localised areas along the Project alignment near Gowrie Creek where these limits are slightly exceeded. These areas are generally agricultural land and the area upstream of East Paulsens Road where the road is being raised as part of the Project design. There are no changes in peak water levels on any state controlled roads for all events up to and including the PMF event. There are two locations on the QR Western Line where increases in peak water levels exceed 100mm. In future stages further discussions with QR will be undertaken regarding the proposed alignment design and associated drainage structures. No flood sensitive receptors are impacted by the changes in peak water levels under the 1% AEP event.				
Change in duration of inundation	**Objective:** Identify changes to time of inundation through determination of time of submergence (ToS). For roads, determine AAToS (if applicable) and consider impacts on accessibility during flood events. **Outcome:** Minor increases in the ToS occur in a limited number of locations and do not affect flood sensitive receptors. There are a number of locations where the proposed works reduce the existing ToS, including on the Western Line rail corridor, Paulsens Road, McMahons Road and East Paulsens Road. The Project design does not result in a significant change to AAToS with a number of locations experiencing a reduction in AAToS. With the introduction of the Project alignment, and associated drainage, both the Western Line rail corridor and Paulsens Road experience a reduction in overtopping with water held back by the Project alignment. The low-level crossing on East Paulsen Road is upgraded and therefore experiences a reduction in AAToS.				
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:** On the Gowrie Creek floodplain, 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, the assessment shows that the changes in peak water levels would be unlikely to exacerbate flood conditions during extreme events. On Oaky Creek, Six Mile Creek and Lockyer Creek the high-level viaduct/bridge crossings do not impact on peak water levels under the extreme events.				

Parameter	Objectives and Outcomes
Sensitivity testing	**Objective:** Consider risks posed by climate change and blockage in accordance with Australian Rainfall and Runoff 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 giving an increase in rainfall intensity of 21% across the catchment areas. The impacts resulting from changes in peak water levels under the 1% AEP event with climate change are generally similar to those assessed under the 1% AEP event. 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% being applied to culverts. Two blockage sensitivity scenarios were tested with both 0% and 50% blockage of all culverts assessed. The resulting changes in peak water levels associated with the Project alignment are still localised and do not impact on any flood sensitive receptors.

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 M

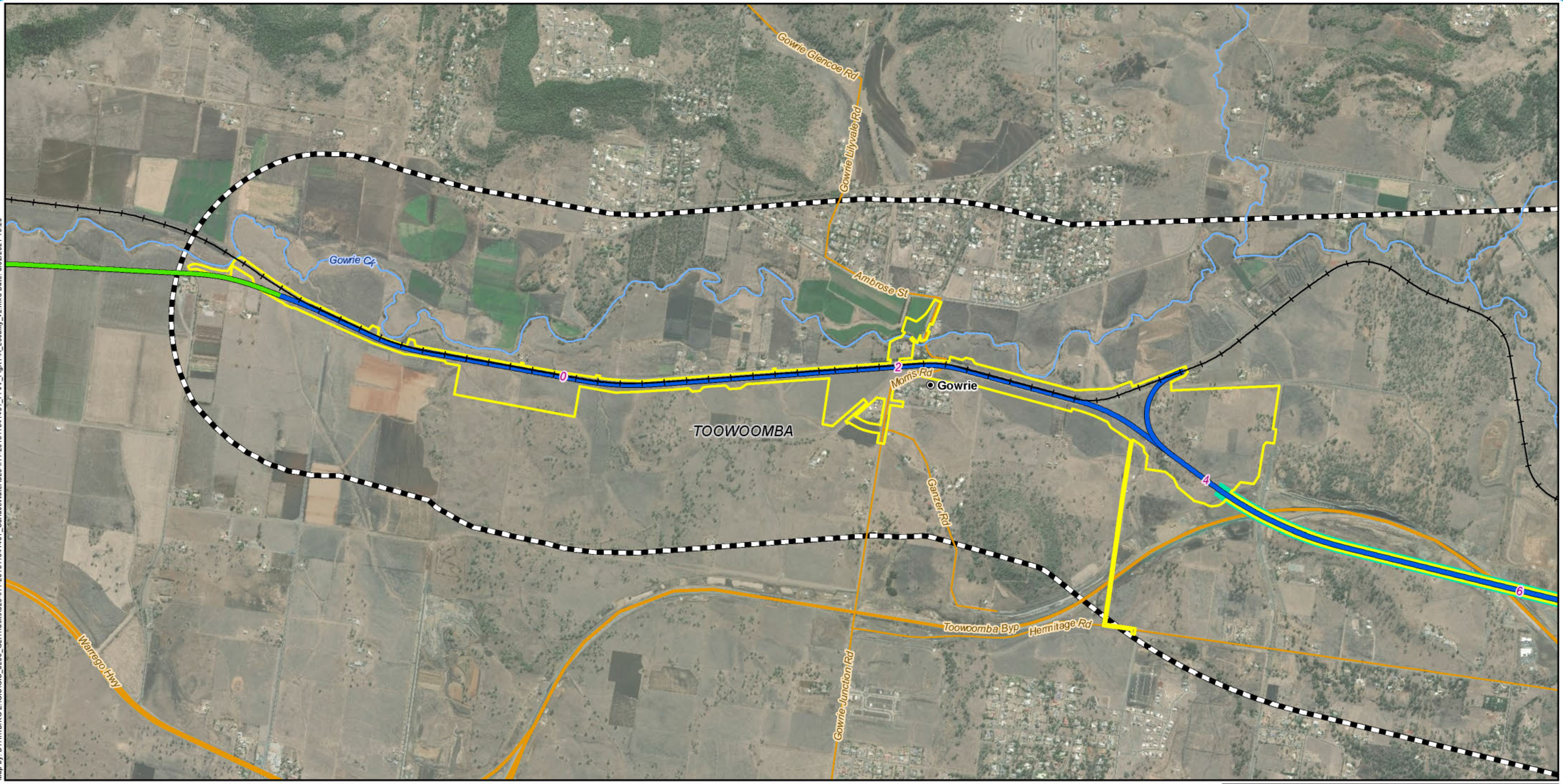
Hydrology and Flooding

Appendix A Gowrie Creek figures

GOWRIE TO HELIDON ENVIRONMENTAL IMPACT STATEMENT

Appendix A
Gowrie Creek Figures

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community



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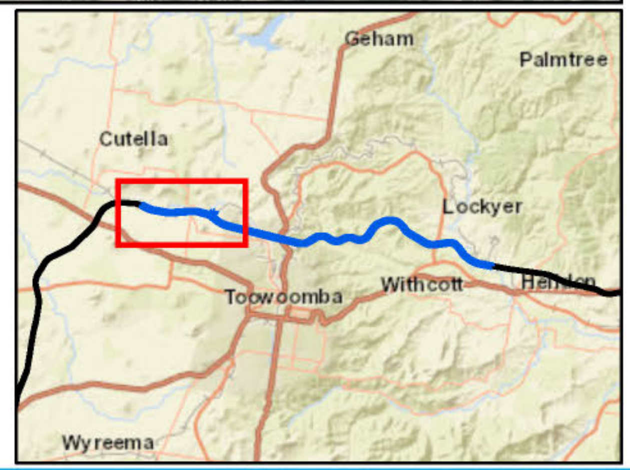

Legend

- 5 Chainage (km)
- Localities
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Watercourses
- EIS disturbance footprint
- EIS investigation corridor
- Local Government Areas

A3 scale: 1:25,000

0 0.2 0.4 0.6 0.8 1km

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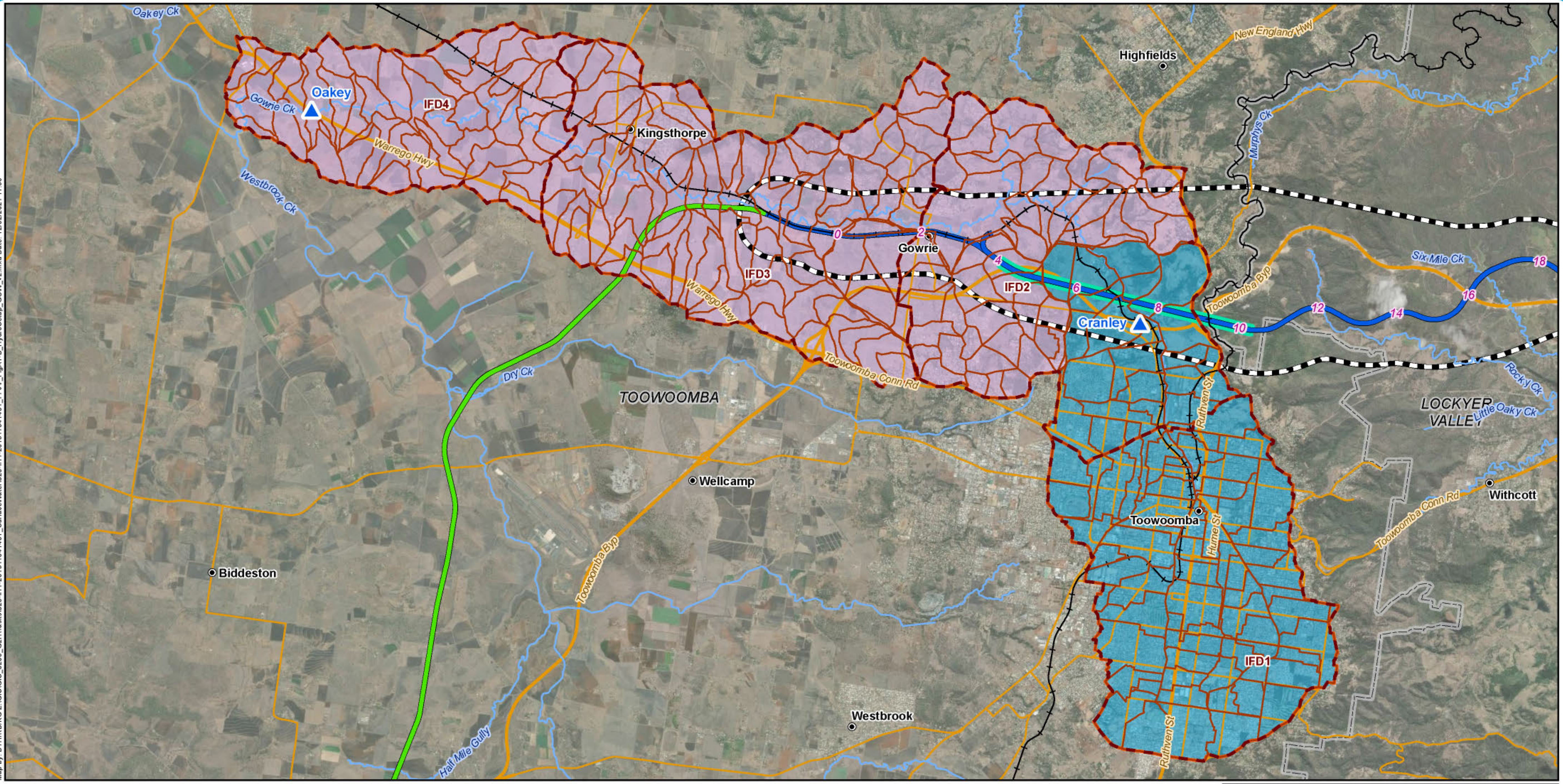
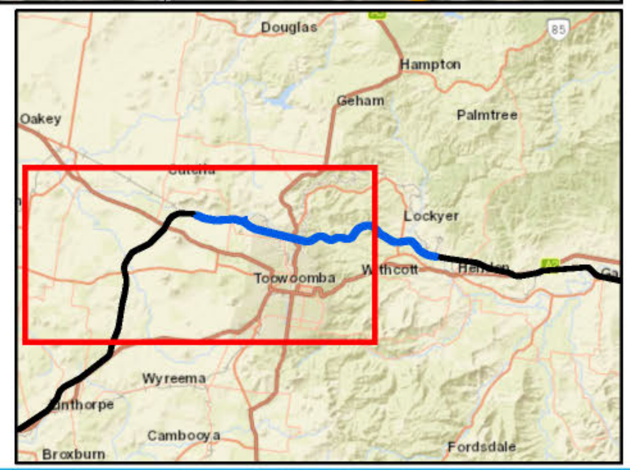
Gowrie to Helidon

Figure A1-A: Locality - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community





Legend

- 5 Chainage (km)
- Localities
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Watercourses
- EIS investigation corridor
- Local Government Areas
- Stream gauge locations
- Gowrie sub-catchments
- Gowrie model extent
- IFD areas

Gowrie loss model

- Area 1
- Area 2

A3 scale: 1:100,000

0 1 2 3 4 5km

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Gowrie to Helidon

Figure A1-B: Hydrology setup - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community

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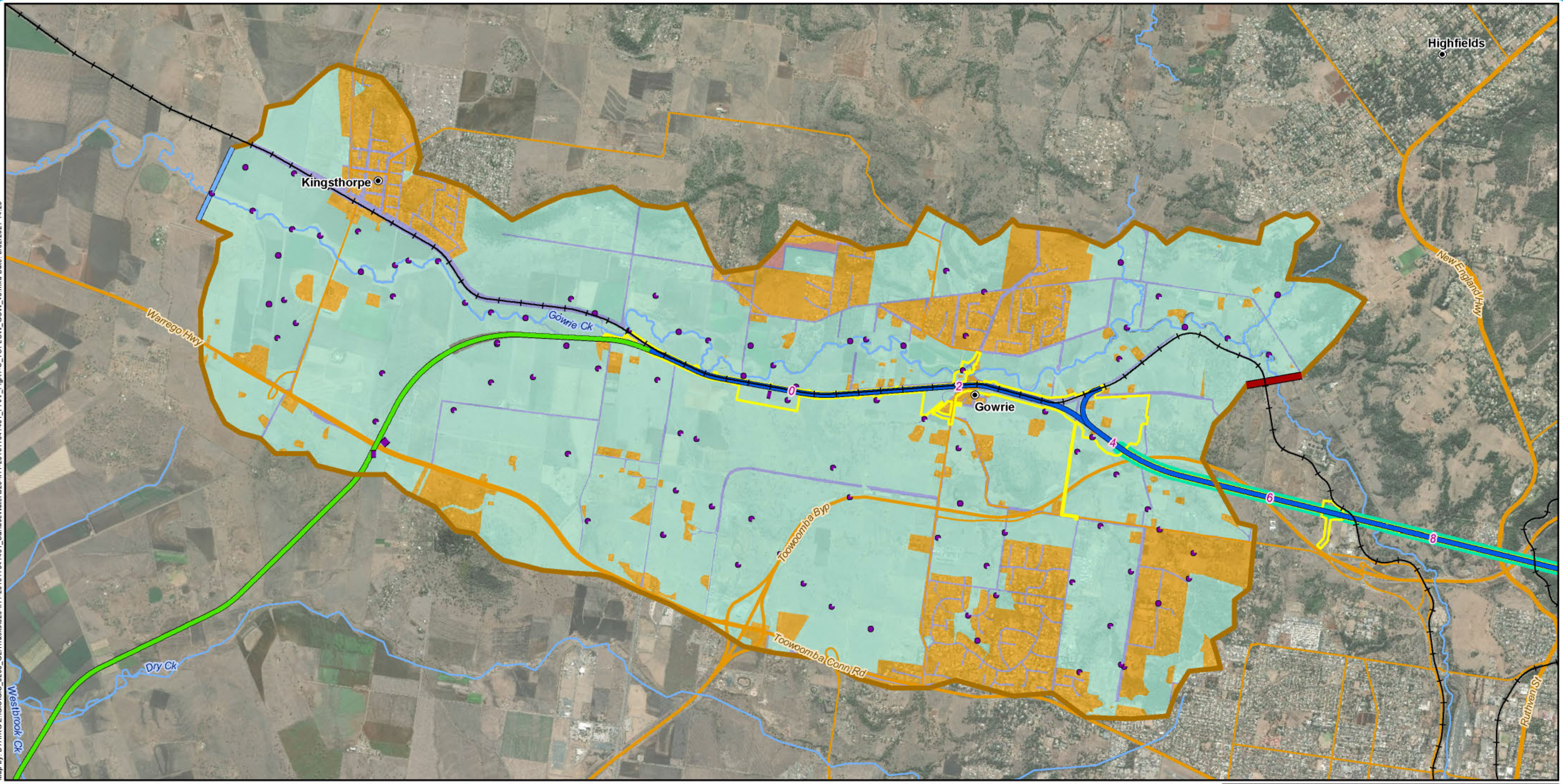

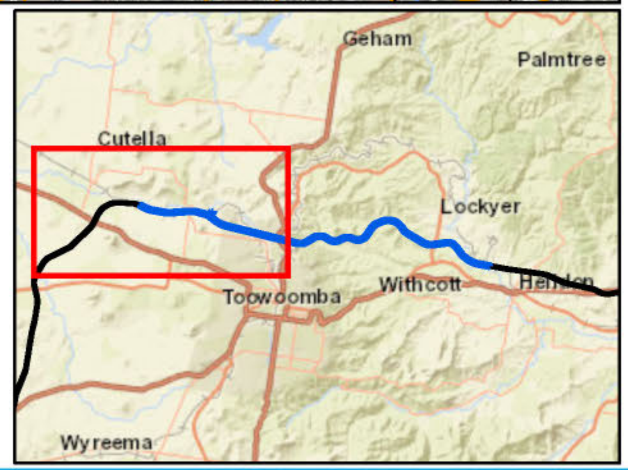

Legend

- 5 Chainage (km)
- Localities
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- EIS disturbance footprint
- Code boundary
- Downstream boundary
- Upstream boundary
- Internal boundary
- Watercourses

Materials

- Dense vegetation
- Creek overbank
- Developed area
- Floodplain
- Waterway
- Road

A3 scale: 1:50,000

0 0.4 0.8 1.2 1.6 2km

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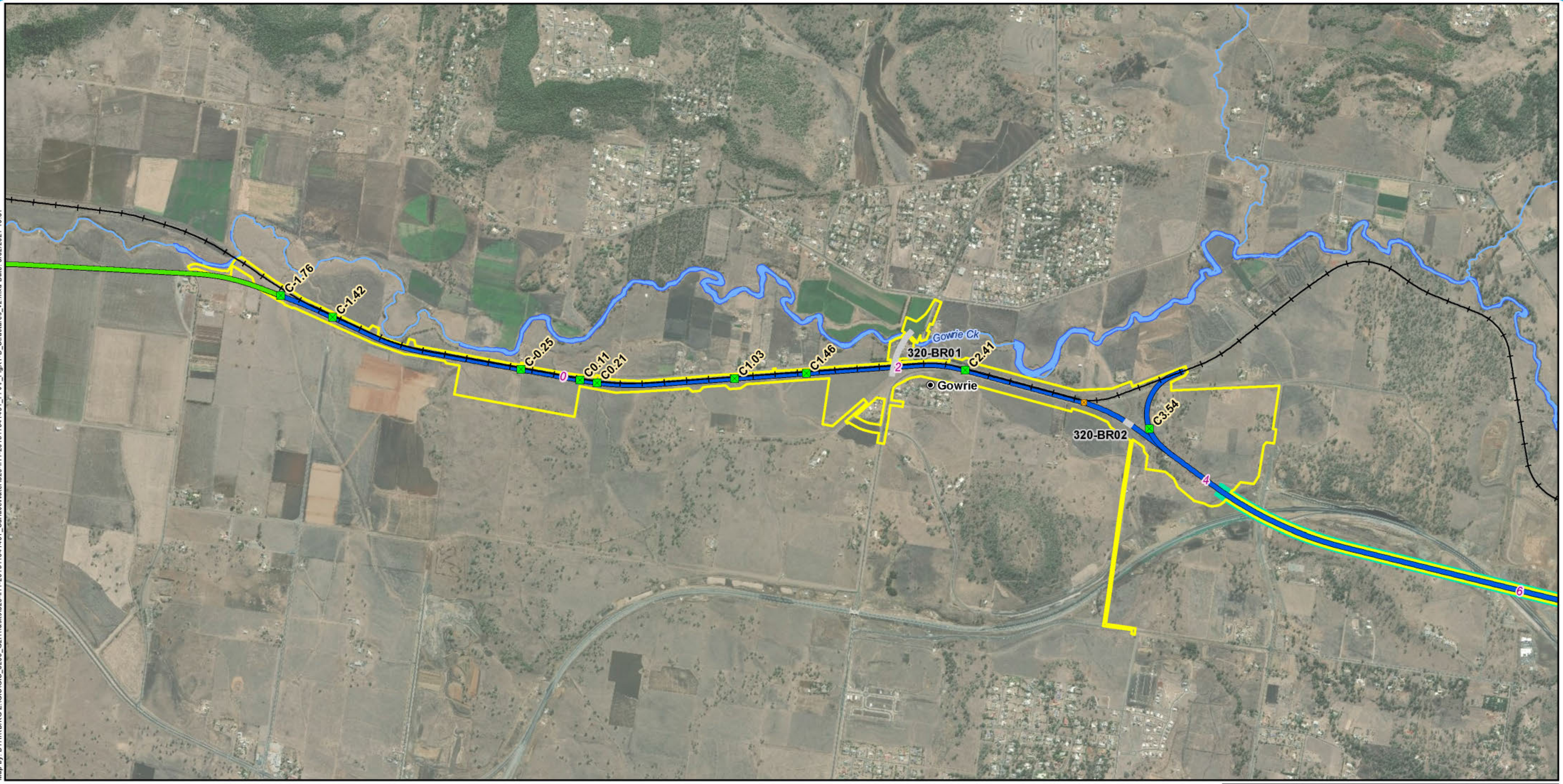
Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure A1-C: TUFLOW model setup - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community



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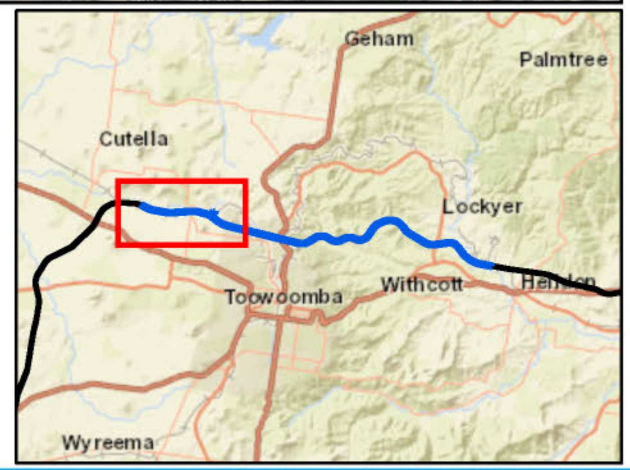
Legend

- 5 Chainage (km)
- Localities
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- EIS disturbance footprint
- Defined watercourses
- Watercourses
- Bridges

Drainage Structures

- Floodplain culvert
- Local drainage culvert



A3 scale: 1:25,000

0 0.2 0.4 0.6 0.8 1 km

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Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure A1-D: Design structures - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community

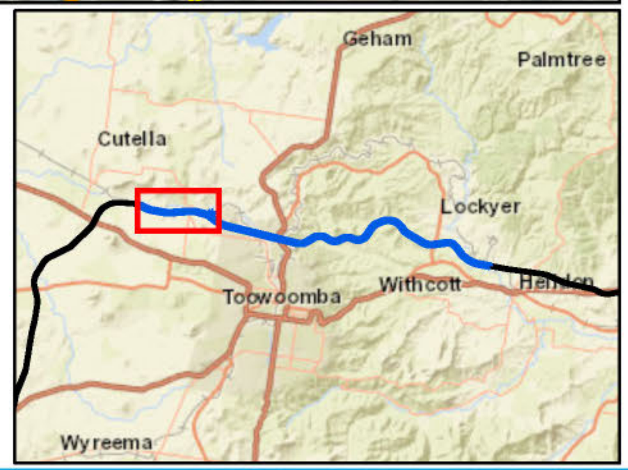
Map by: DTH/RB/KG Z:\GIS\GIS_3200_G2H\Tasks\320-IHY-201911041401_SurfaceWater\320-IHY-201911041401_FFJV_FigX1-E_FloodSensitiveReceptors_v2.mxd Date: 3/02/2021 16:18

Legend

- 5 Chainage (km)
- Localities
- Flood sensitive receptors
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- EIS disturbance footprint
- Local Government Areas

A3 scale: 1:16,000

0 0.1 0.2 0.3 0.4 0.5km



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Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure A1-E: Flood Sensitive Receptors - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community

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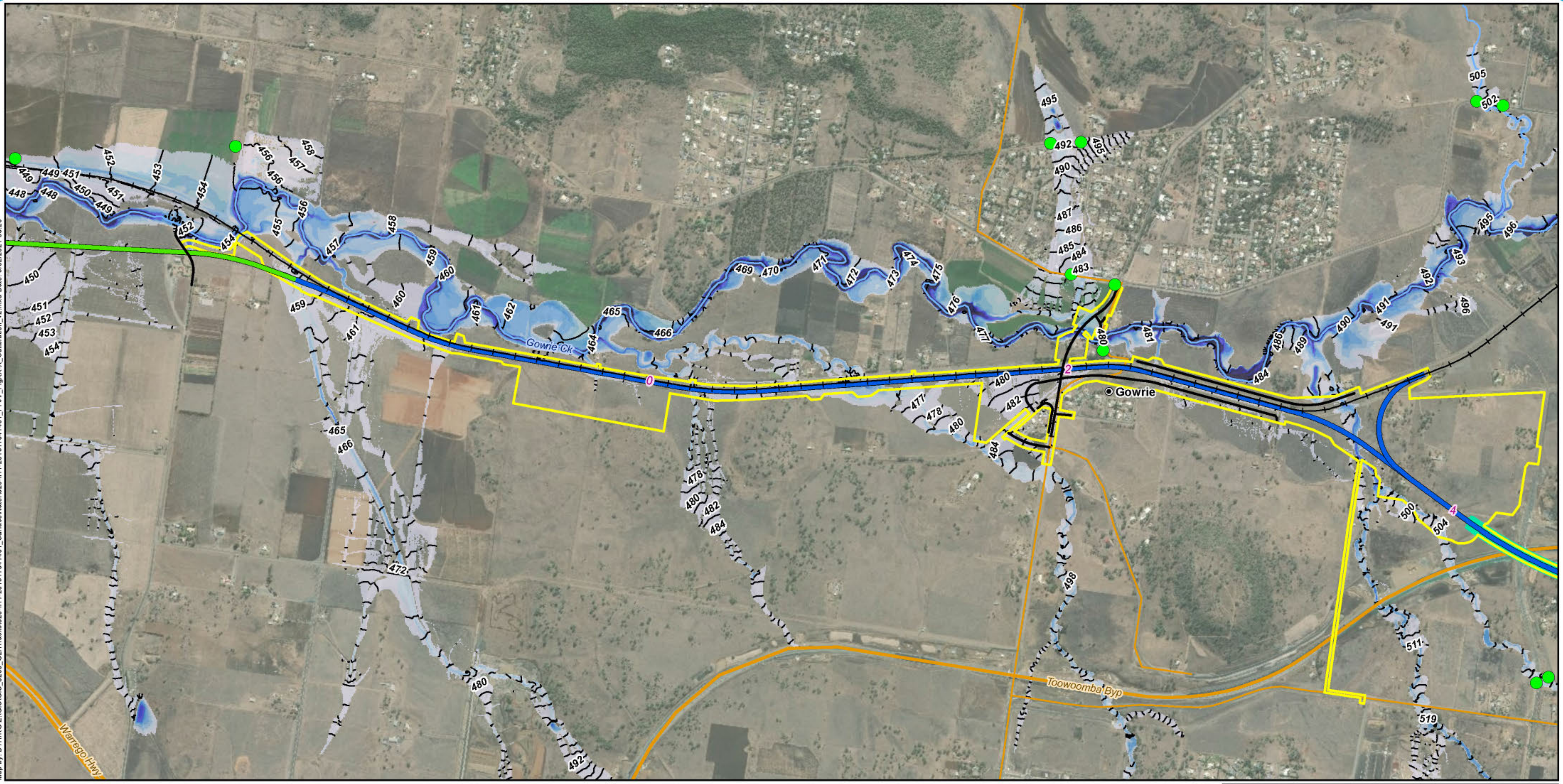
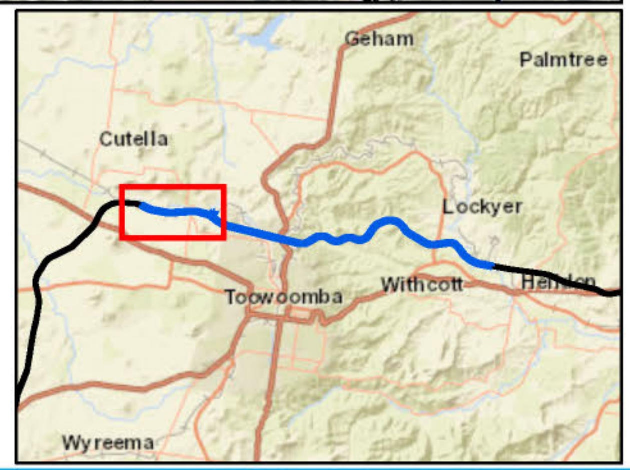


Legend

- 5 Chainage (km)
- Localities
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Road design
- EIS disturbance footprint
- Watercourses
- 0.5m contour mAHD
- Flood marker (2011)

Depth (m)

- 0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 2.5
- 2.5 - 3.0
- 3.0 - 3.5
- 3.5 - 4.0
- 4.0 - 4.5
- 4.5 - 5.0
- > 5.0



A3 scale: 1:20,000

0 0.15 0.3 0.45 0.6 0.75 km

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Issue date: 05/02/2021 Version: 2
Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure A2-A: 2011 Calibration event - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community

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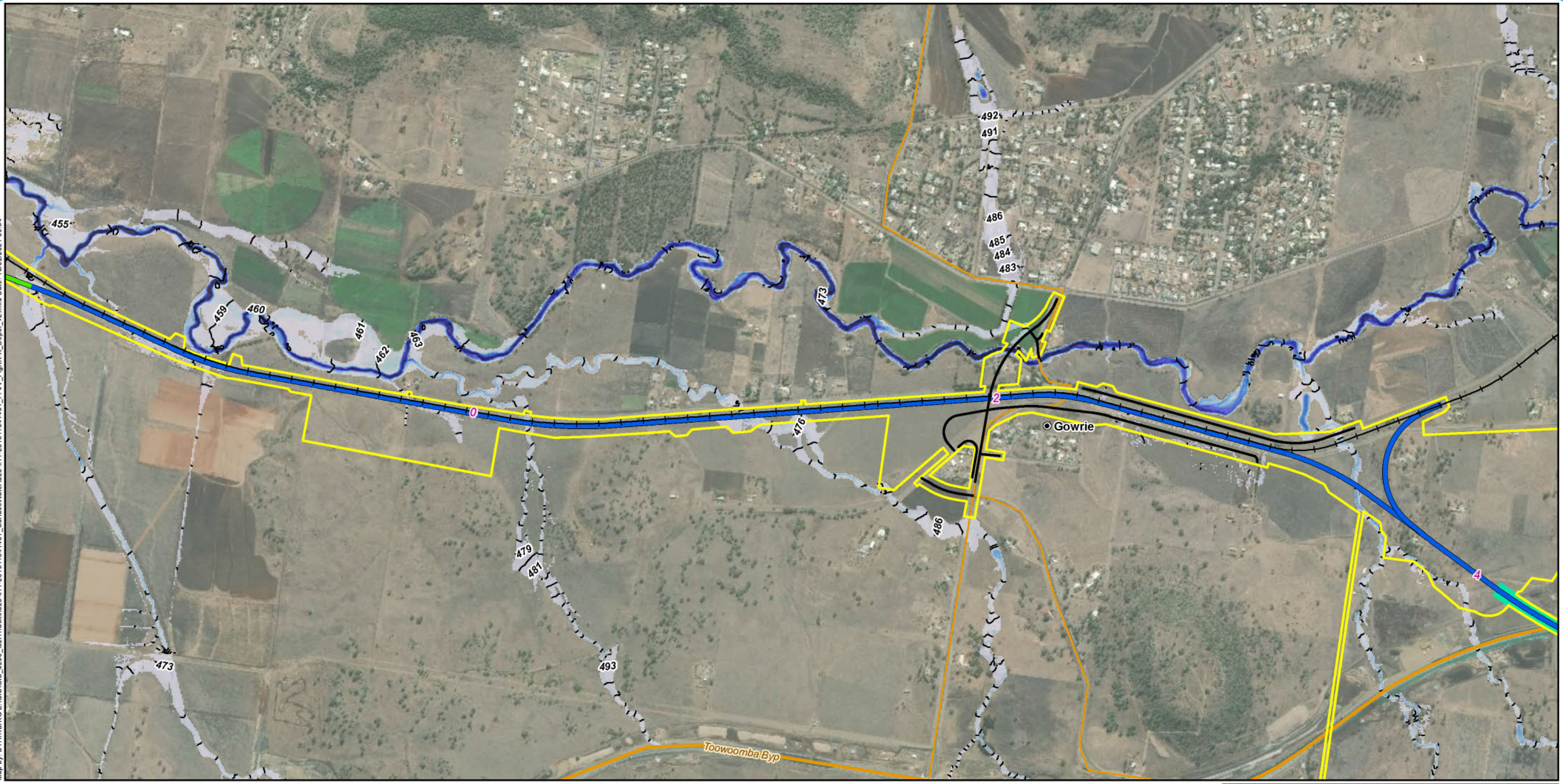
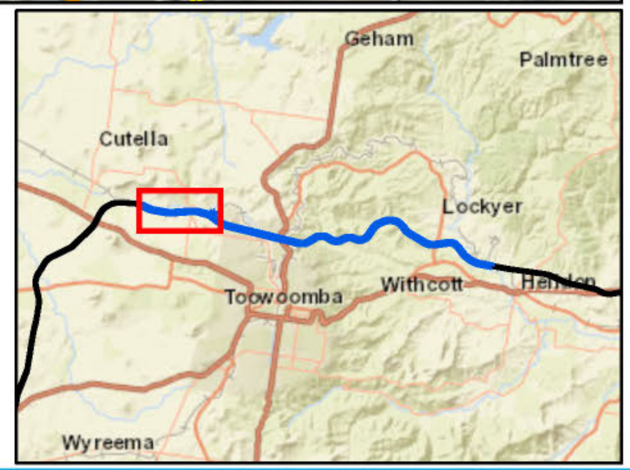


Legend

- 5 Chainage (km)
- Localities
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Road design
- EIS disturbance footprint
- 1.0m contour mAHD

Depth (m)

- 0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 2.5
- 2.5 - 3.0
- 3.0 - 3.5
- 3.5 - 4.0
- 4.0 - 4.5
- 4.5 - 5.0
- > 5.0



A3 scale: 1:16,000

0 0.1 0.2 0.3 0.4 0.5km

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Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure A3-A: Existing Case - Inundation Extent - 20% AEP event - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community

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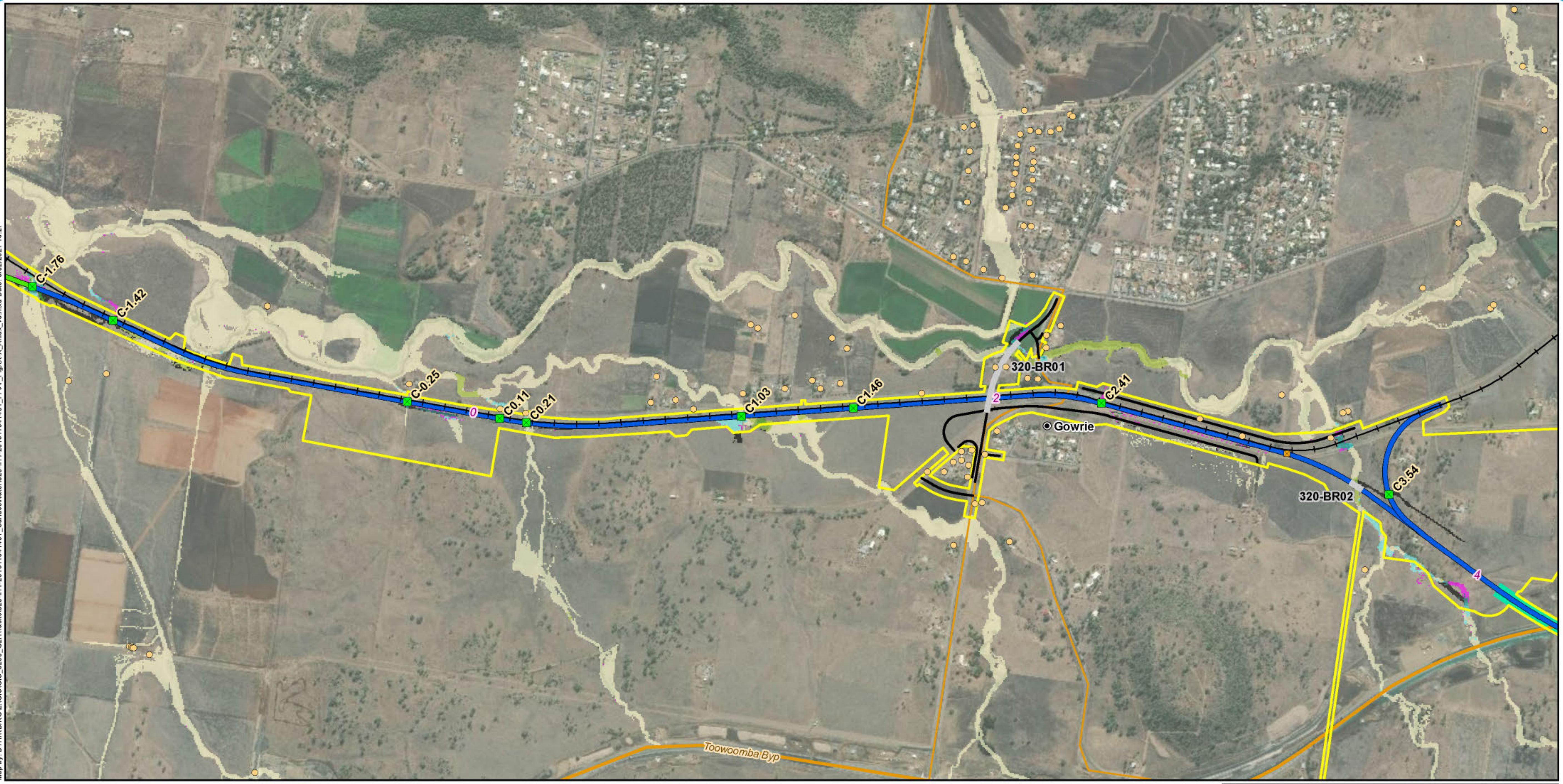
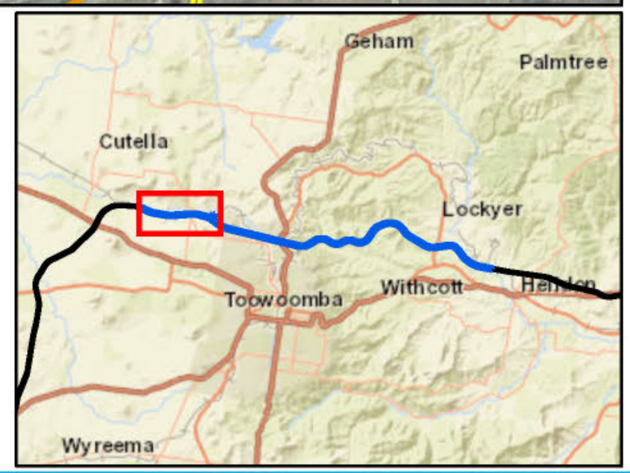




Legend

- 5 Chainage (km)
- Localities
- Flood sensitive receptors
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Road design
- EIS disturbance footprint
- Was Wet Now Dry
- Was Dry Now Wet

Drainage Structures

- Floodplain culvert
- Local drainage culvert
- Bridges

Change in peak water levels (m)

- < -0.5
- -0.5 to -0.2
- -0.2 to -0.1
- -0.1 to -0.05
- -0.05 to -0.01
- -0.01 to 0.01
- 0.01 to 0.05
- 0.05 to 0.1
- 0.1 to 0.2
- 0.2 to 0.5
- > 0.5

A3 scale: 1:16,000

0 0.1 0.2 0.3 0.4 0.5km

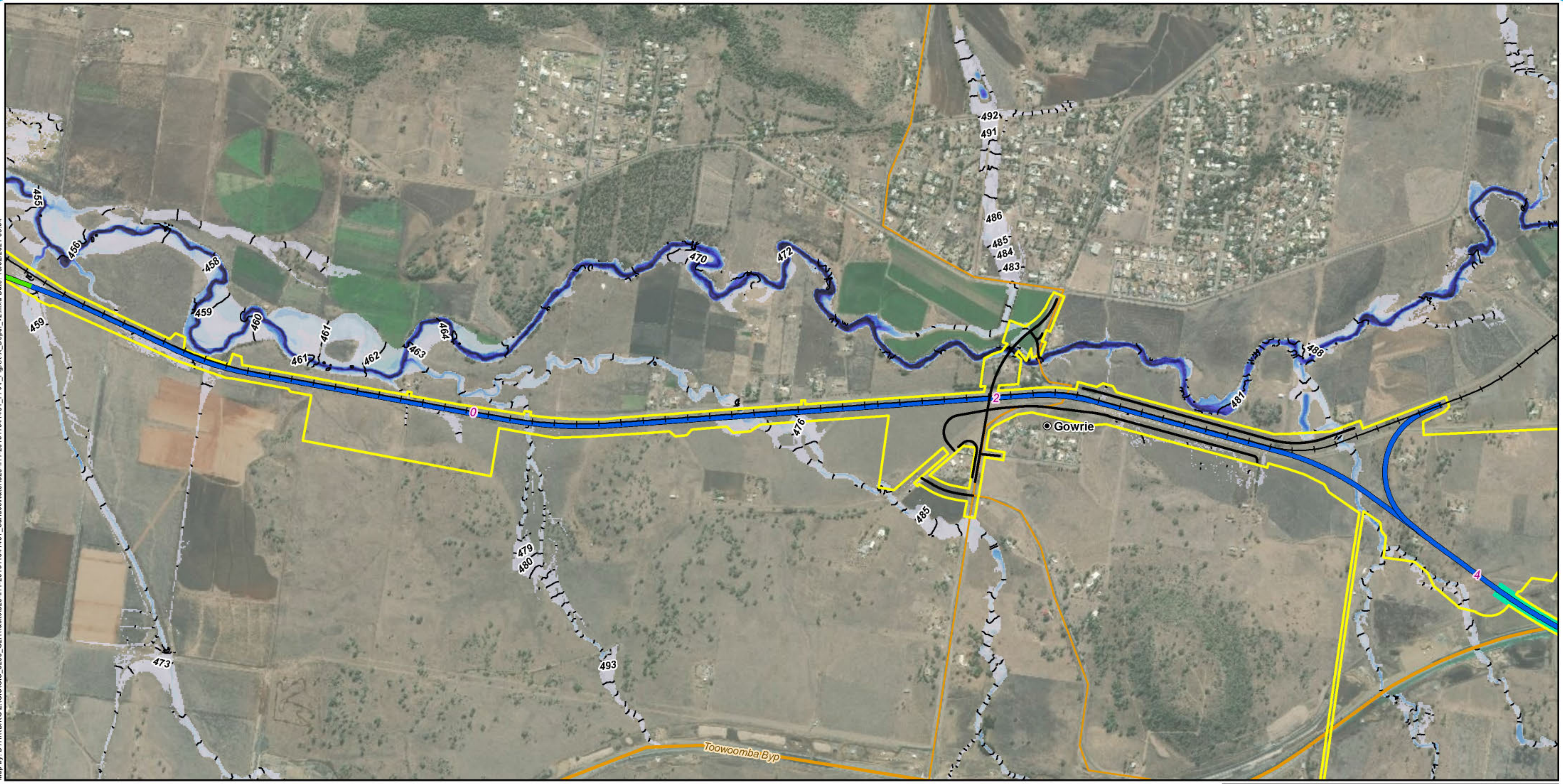
Future Freight
Integrating Community, Environment and Engineering

Issue date: 03/02/2021 Version: 3
Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure A3-B: Developed Case - Afflux - 20% AEP event - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community



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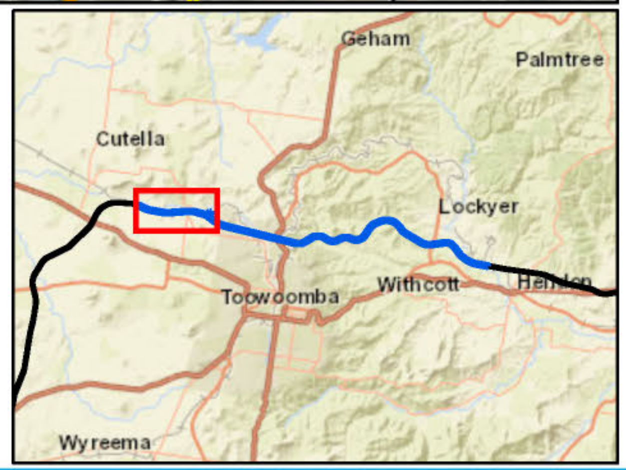
Legend

- 5 Chainage (km)
- Localities
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Road design
- EIS disturbance footprint
- 1.0m contour mAHD

Depth (m)

- 0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 2.5
- 2.5 - 3.0
- 3.0 - 3.5
- 3.5 - 4.0
- 4.0 - 4.5
- 4.5 - 5.0
- > 5.0



A3 scale: 1:16,000

0 0.1 0.2 0.3 0.4 0.5km

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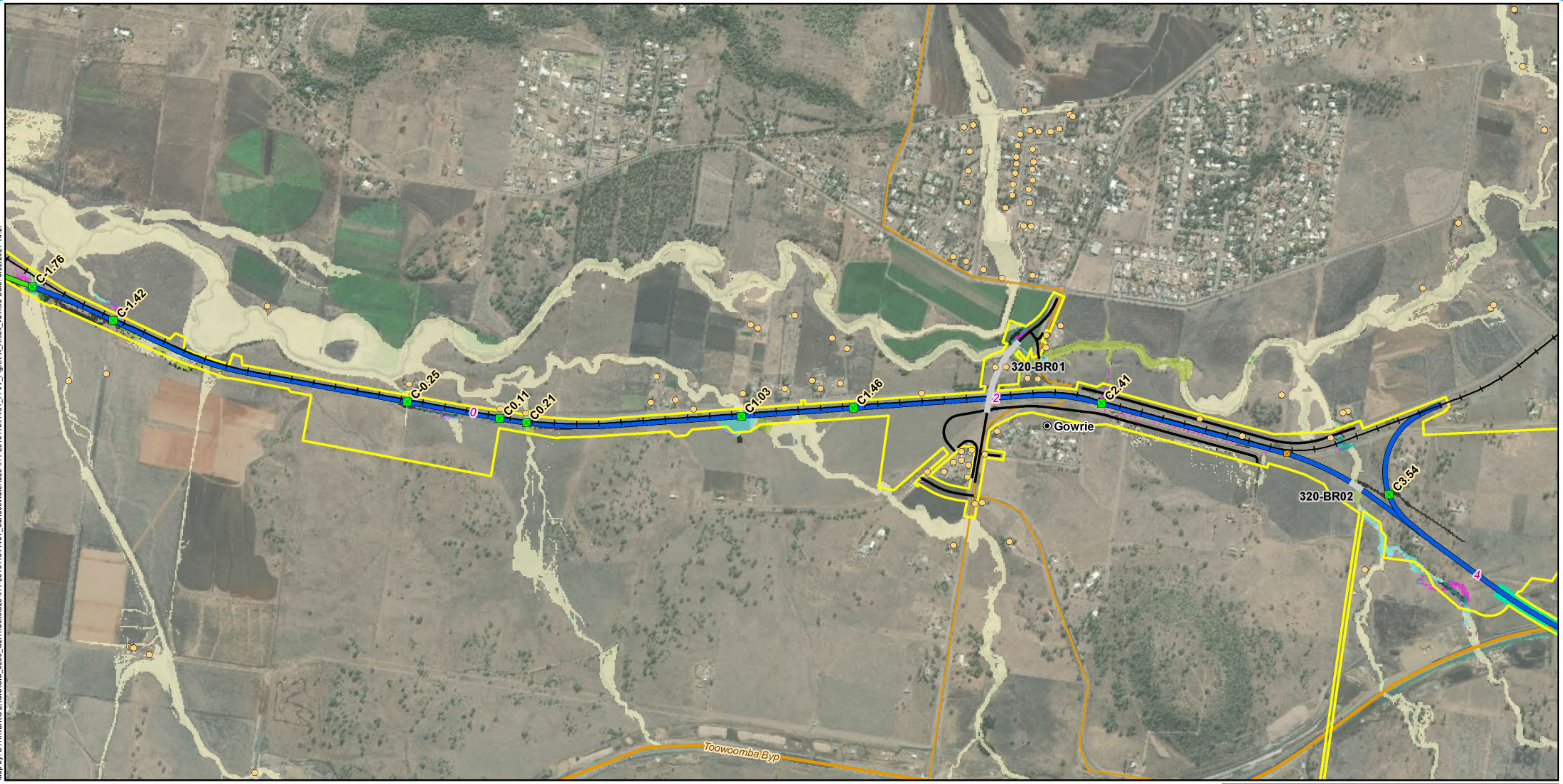
Issue date: 10/02/2021 Version: 2

Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure A4-A: Existing Case - Inundation Extent - 10% AEP event - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community



Map by: DTH/RB/KG Z:\GIS\GIS_3200_G2H\Tasks\320-IHY-201911041401_SurfaceWater\320-IHY-201911041401_FFJV_FigXX-X_Afflux_v3.mxd Date: 3/02/2021 16:27

Legend

- 5 Chainage (km)
- Localities
- Flood sensitive receptors
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Road design
- EIS disturbance footprint
- Was Wet Now Dry
- Was Dry Now Wet

Drainage Structures

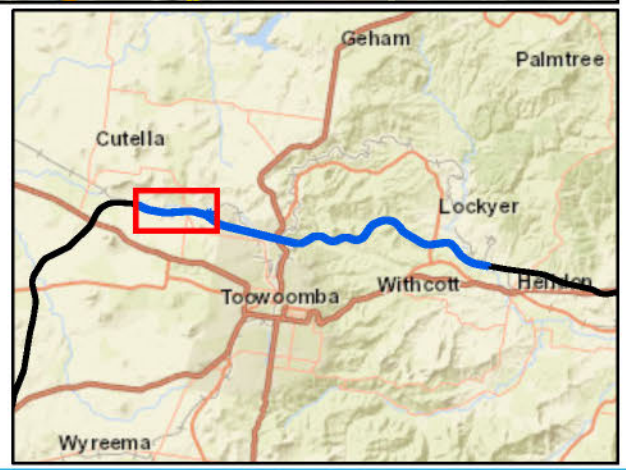
- Floodplain culvert
- Local drainage culvert
- Bridges

Change in peak water levels (m)

- < -0.5
- -0.5 to -0.2
- -0.2 to -0.1
- -0.1 to -0.05
- -0.05 to -0.01
- -0.01 to 0.01
- 0.01 to 0.05
- 0.05 to 0.1
- 0.1 to 0.2
- 0.2 to 0.5
- > 0.5

A3 scale: 1:16,000

0 0.1 0.2 0.3 0.4 0.5km



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Integrating Community, Environment and Engineering

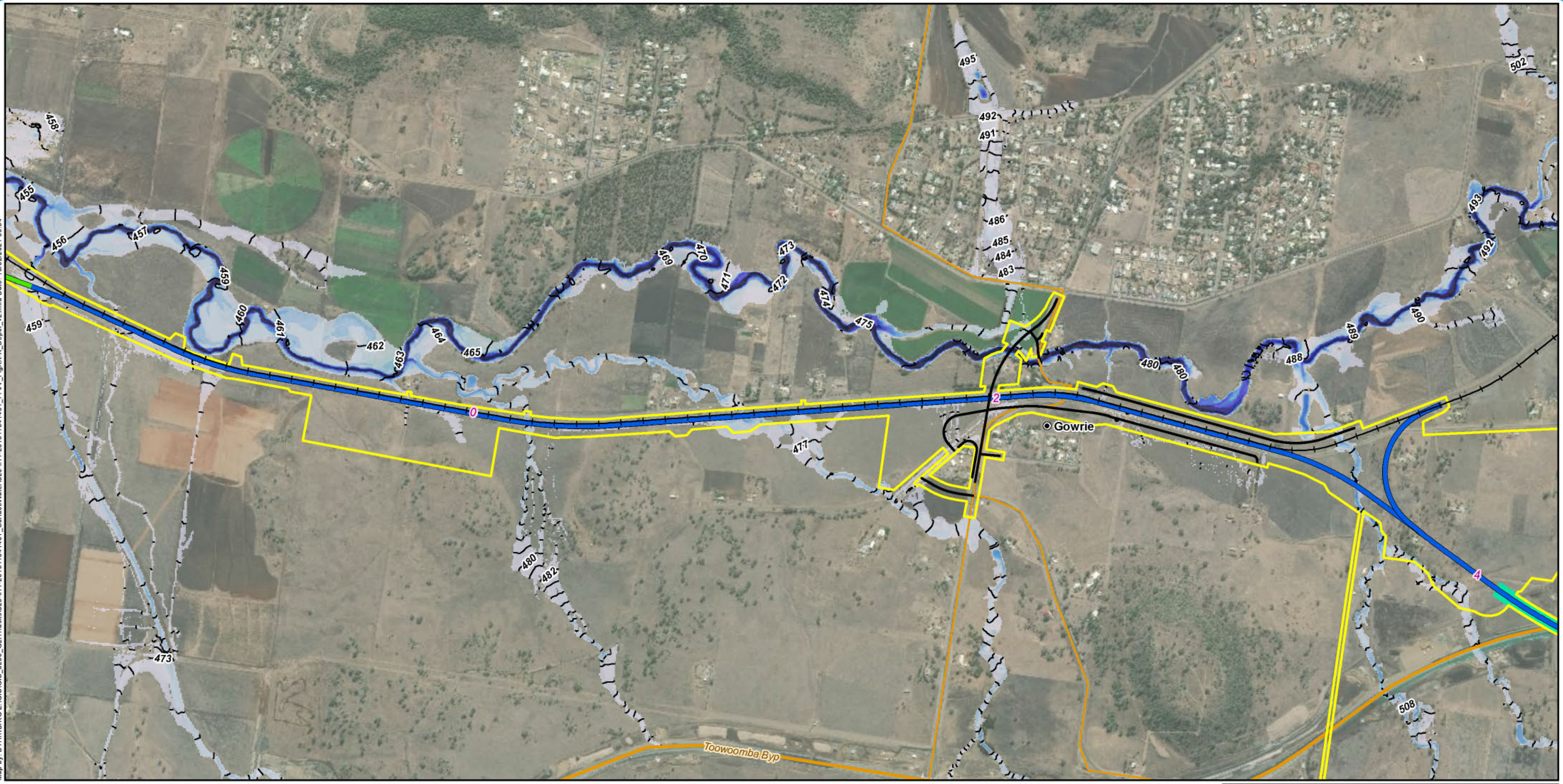
Issue date: 03/02/2021 Version: 3
Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure A4-B: Developed Case - Afflux - 10% AEP event - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community

Map by: DTH/RB/KG Z:\GIS\GIS_3200_G2H\Tasks\320-IHY-201911041401_SurfaceWater\320-IHY-201911041401_FFJV_FigXX-X_Depth_v2.mxd Date: 10/02/2021 09:54

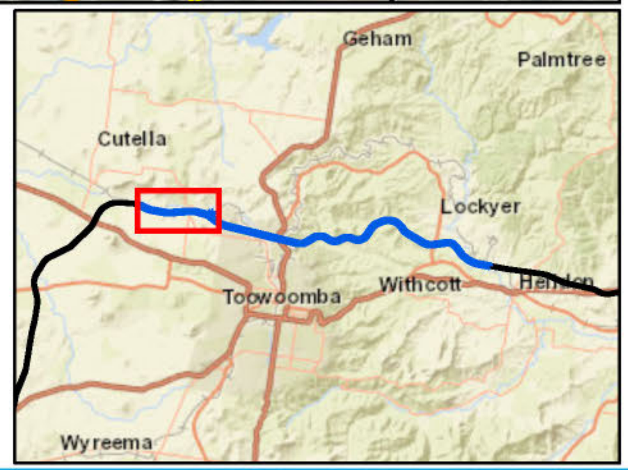


Legend

- 5 Chainage (km)
- Localities
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Road design
- EIS disturbance footprint
- 1.0m contour mAHD

Depth (m)

- 0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 2.5
- 2.5 - 3.0
- 3.0 - 3.5
- 3.5 - 4.0
- 4.0 - 4.5
- 4.5 - 5.0
- > 5.0



A3 scale: 1:16,000

0 0.1 0.2 0.3 0.4 0.5km

Future Freight
Integrating Community, Environment and Engineering

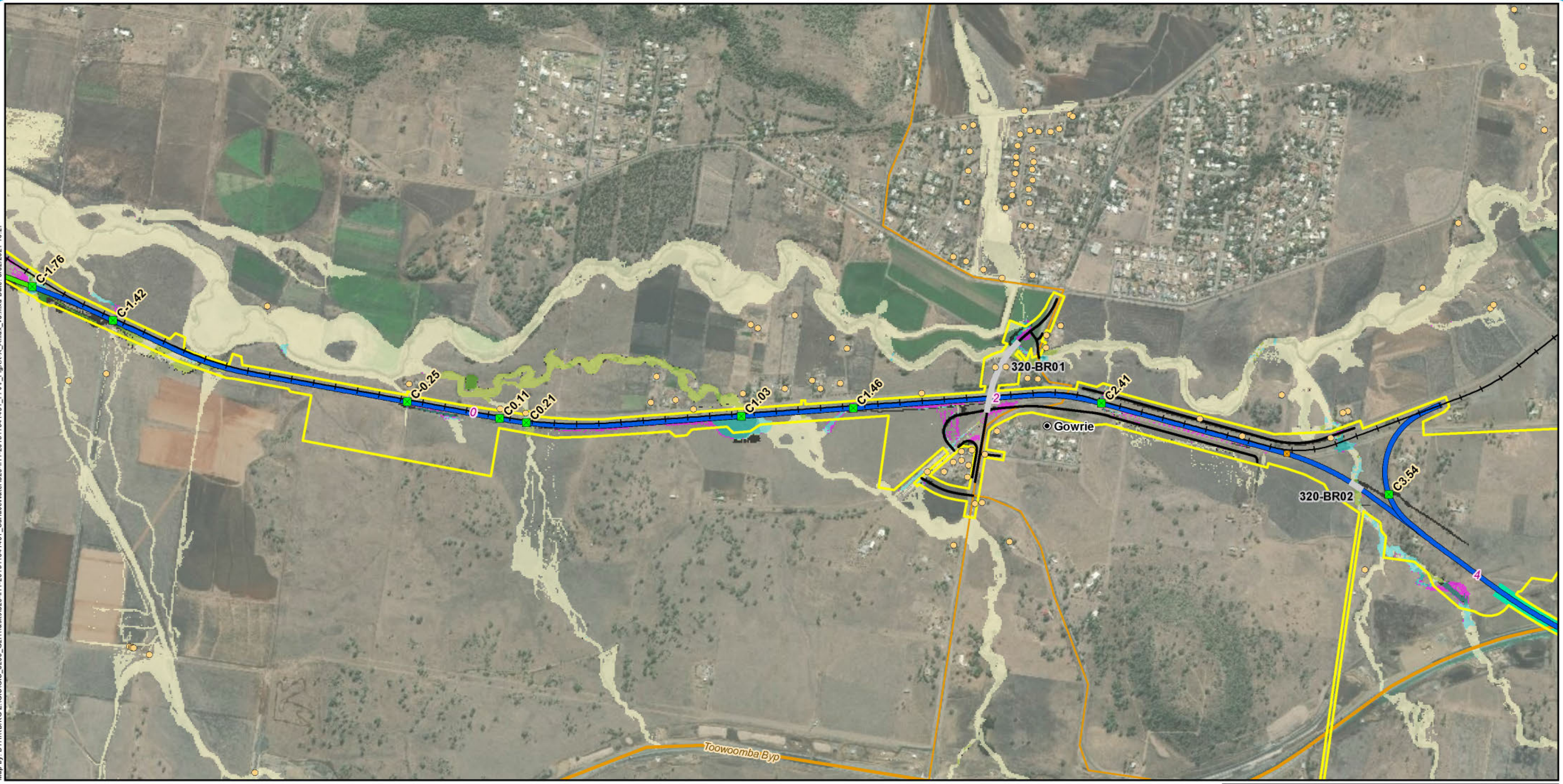
Issue date: 10/02/2021 Version: 2
Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure A5-A: Existing Case - Inundation Extent - 5% AEP event - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community

Map by: DTH/RB/KG Z:\GIS\GIS_3200_G2H\Tasks\320-IHY-201911041401_SurfaceWater\320-IHY-201911041401_FFJV_FigXX-X_Afflux_v3.mxd Date: 3/02/2021 16:27



Legend

5 Chainage (km)
Localities
Flood sensitive receptors
Existing rail
B2G project alignment
G2H project alignment
Tunnel
Major roads
Minor roads
Road design

EIS disturbance footprint
Was Wet Now Dry
Was Dry Now Wet

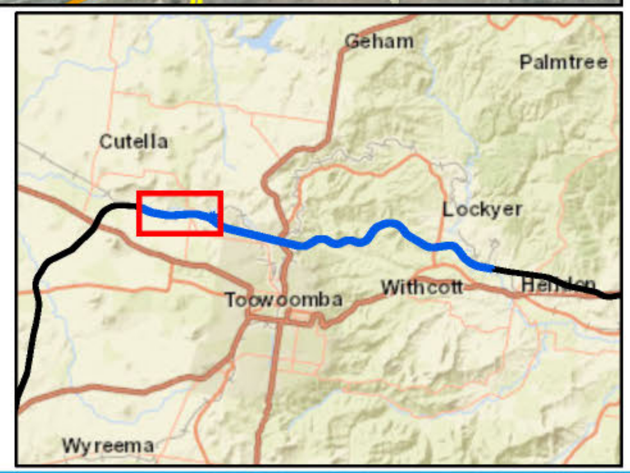
Drainage Structures

Floodplain culvert
Local drainage culvert
Bridges

Change in peak water levels (m)

< -0.5
-0.5 to -0.2
-0.2 to -0.1
-0.1 to -0.05
-0.05 to -0.01
-0.01 to 0.01
0.01 to 0.05
0.05 to 0.1
0.1 to 0.2
0.2 to 0.5
> 0.5

A3 scale: 1:16,000
0 0.1 0.2 0.3 0.4 0.5km



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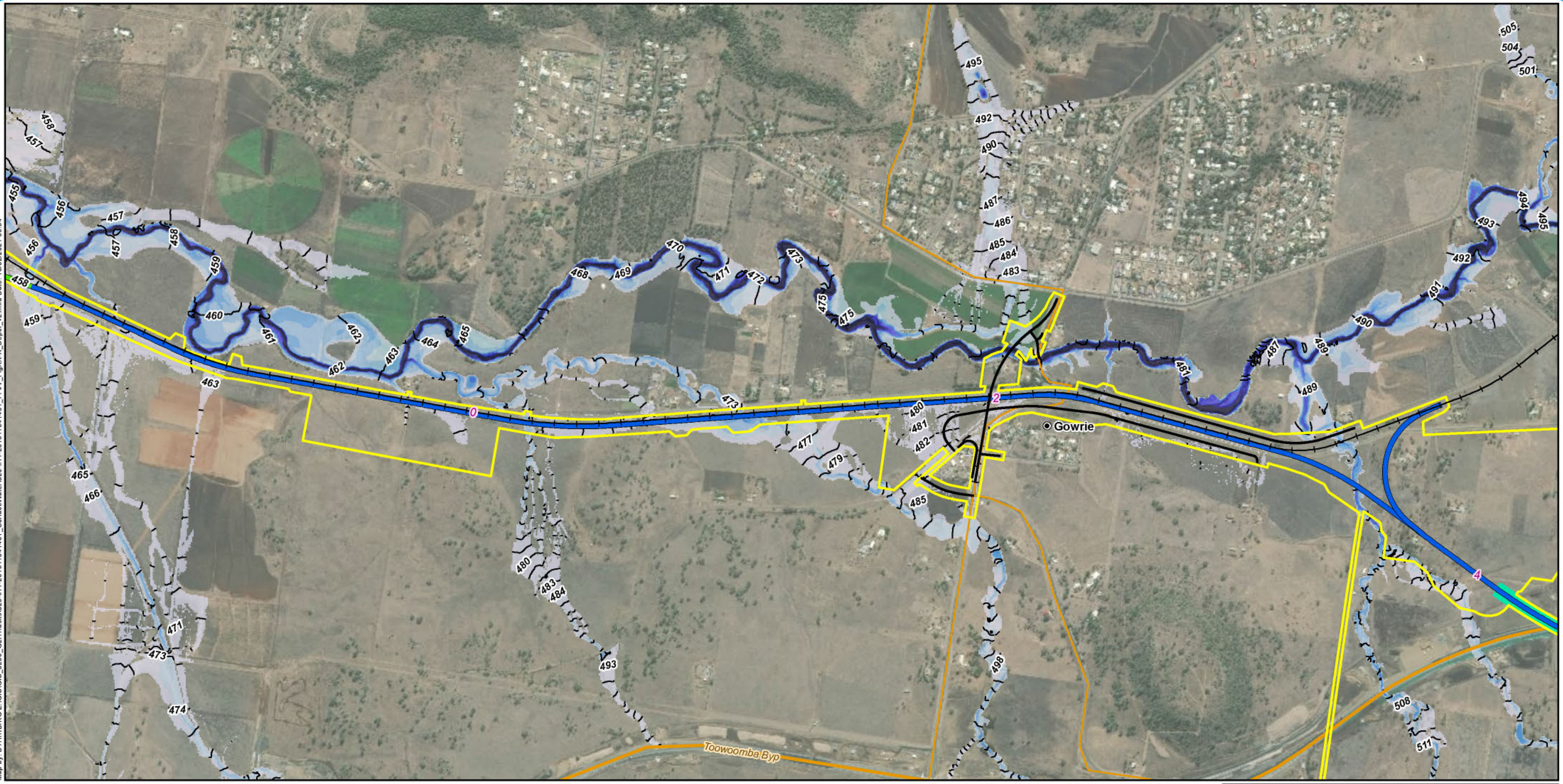
Issue date: 03/02/2021 Version: 3
Coordinate System: GDA 1994 MGA Zone 56

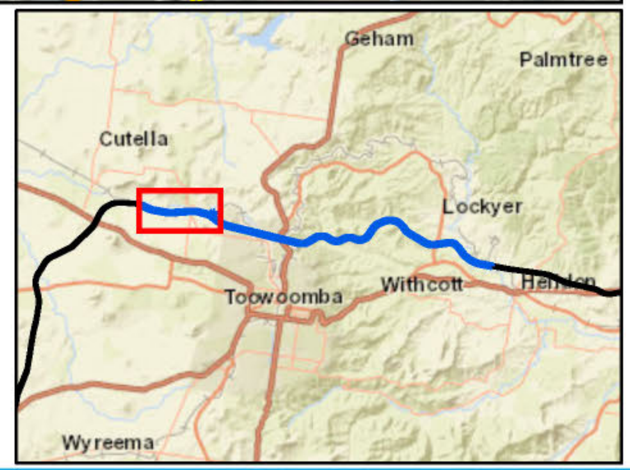
Gowrie to Helidon

Figure A5-B: Developed Case - Afflux - 5% AEP event - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community



Map by: DTH/RB/KG Z:\GIS\GIS_3200_G2H\Tasks\320-IHY-201911041401_SurfaceWater\320-IHY-201911041401_FFJV_FigXX.X_Depth_v2.mxd Date: 10/02/2021 09:54



Legend

- 5 Chainage (km)
- Localities
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Road design
- EIS disturbance footprint
- 1.0m contour mAHD

Depth (m)

- 0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 2.5
- 2.5 - 3.0
- 3.0 - 3.5
- 3.5 - 4.0
- 4.0 - 4.5
- 4.5 - 5.0
- > 5.0

A3 scale: 1:16,000

0 0.1 0.2 0.3 0.4 0.5km

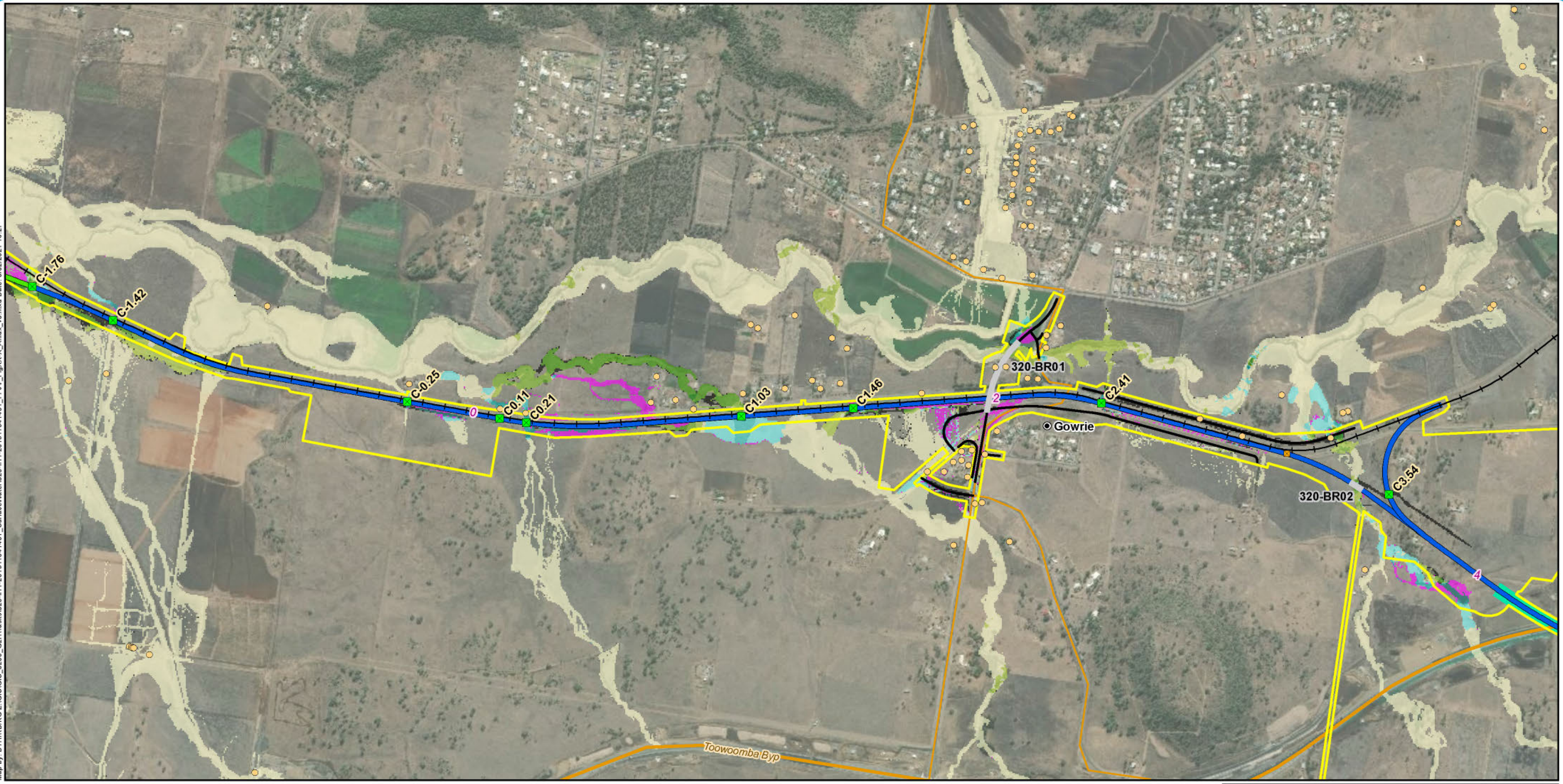
Future Freight
Integrating Community, Environment and Engineering

Issue date: 10/02/2021 Version: 2
Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure A6-A: Existing Case - Inundation Extent - 2% AEP event - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community



Map by: DTH/RB/KG Z:\GIS\GIS_3200_G2H\Tasks\320-IHY-201911041401_SurfaceWater\320-IHY-201911041401_FFJV_FigXX.X_Afflux_v3.mxd Date: 3/02/2021 16:27

Legend

- 5 Chainage (km)
- Localities
- Flood sensitive receptors
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Road design
- EIS disturbance footprint
- Was Wet Now Dry
- Was Dry Now Wet

Drainage Structures

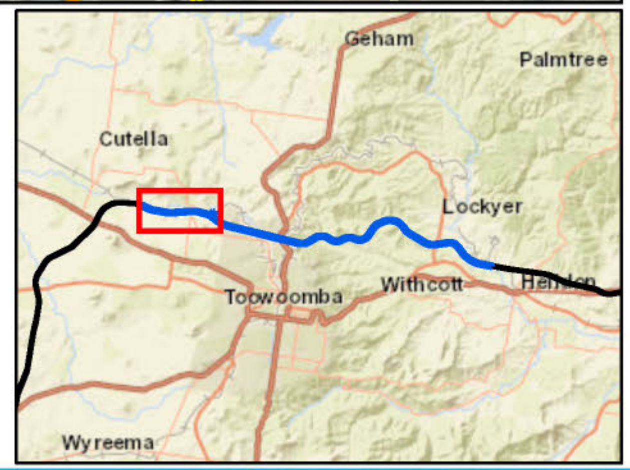
- Floodplain culvert
- Local drainage culvert
- Bridges

Change in peak water levels (m)

- < -0.5
- -0.5 to -0.2
- -0.2 to -0.1
- -0.1 to -0.05
- -0.05 to -0.01
- -0.01 to 0.01
- 0.01 to 0.05
- 0.05 to 0.1
- 0.1 to 0.2
- 0.2 to 0.5
- > 0.5

A3 scale: 1:16,000

0 0.1 0.2 0.3 0.4 0.5km



Future Freight
Integrating Community, Environment and Engineering

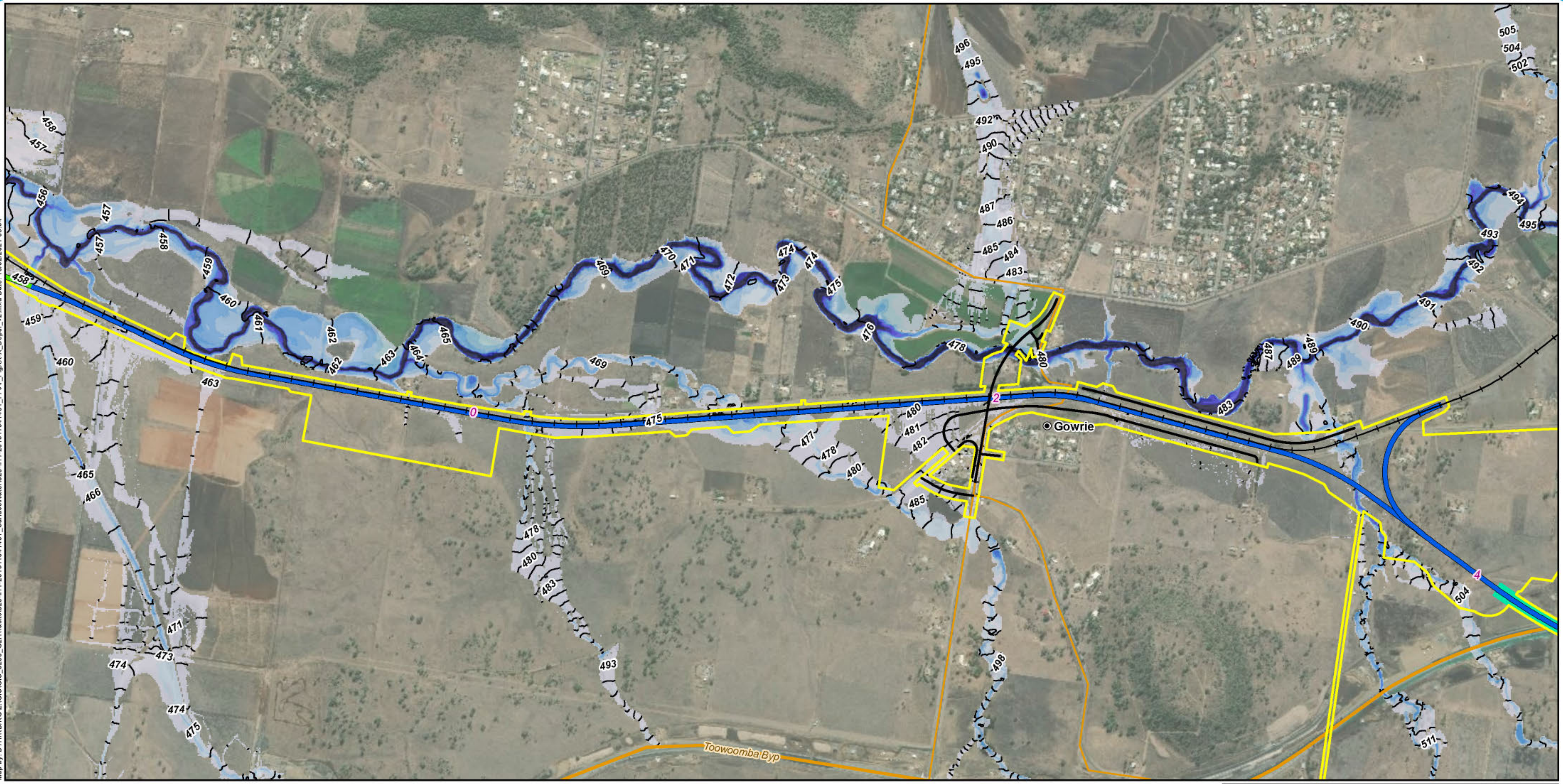
Issue date: 03/02/2021 Version: 3
Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure A6-B: Developed Case - Afflux - 2% AEP event - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community

Map by: DTH/RB/KG Z:\GIS\GIS_3200_G2H\Tasks\320-IHY-201911041401_SurfaceWater\320-IHY-201911041401_FFJV_FigXX.X_Depth_v2.mxd Date: 10/02/2021 09:54

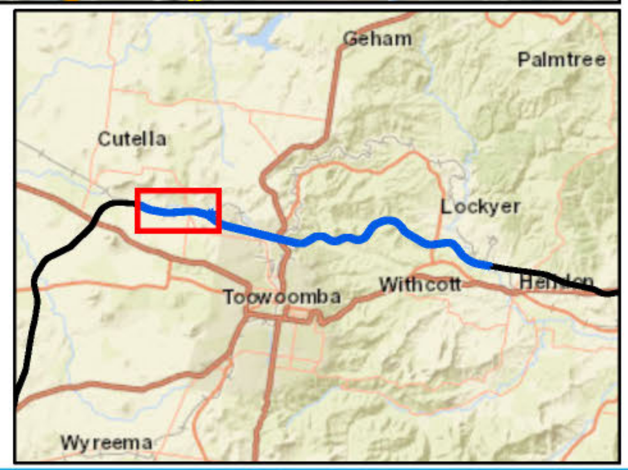


Legend

- 5 Chainage (km)
- Localities
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Road design
- EIS disturbance footprint
- 1.0m contour mAHD

Depth (m)

- 0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 2.5
- 2.5 - 3.0
- 3.0 - 3.5
- 3.5 - 4.0
- 4.0 - 4.5
- 4.5 - 5.0
- > 5.0



A3 scale: 1:16,000

0 0.1 0.2 0.3 0.4 0.5km

Future Freight
Integrating Community, Environment and Engineering

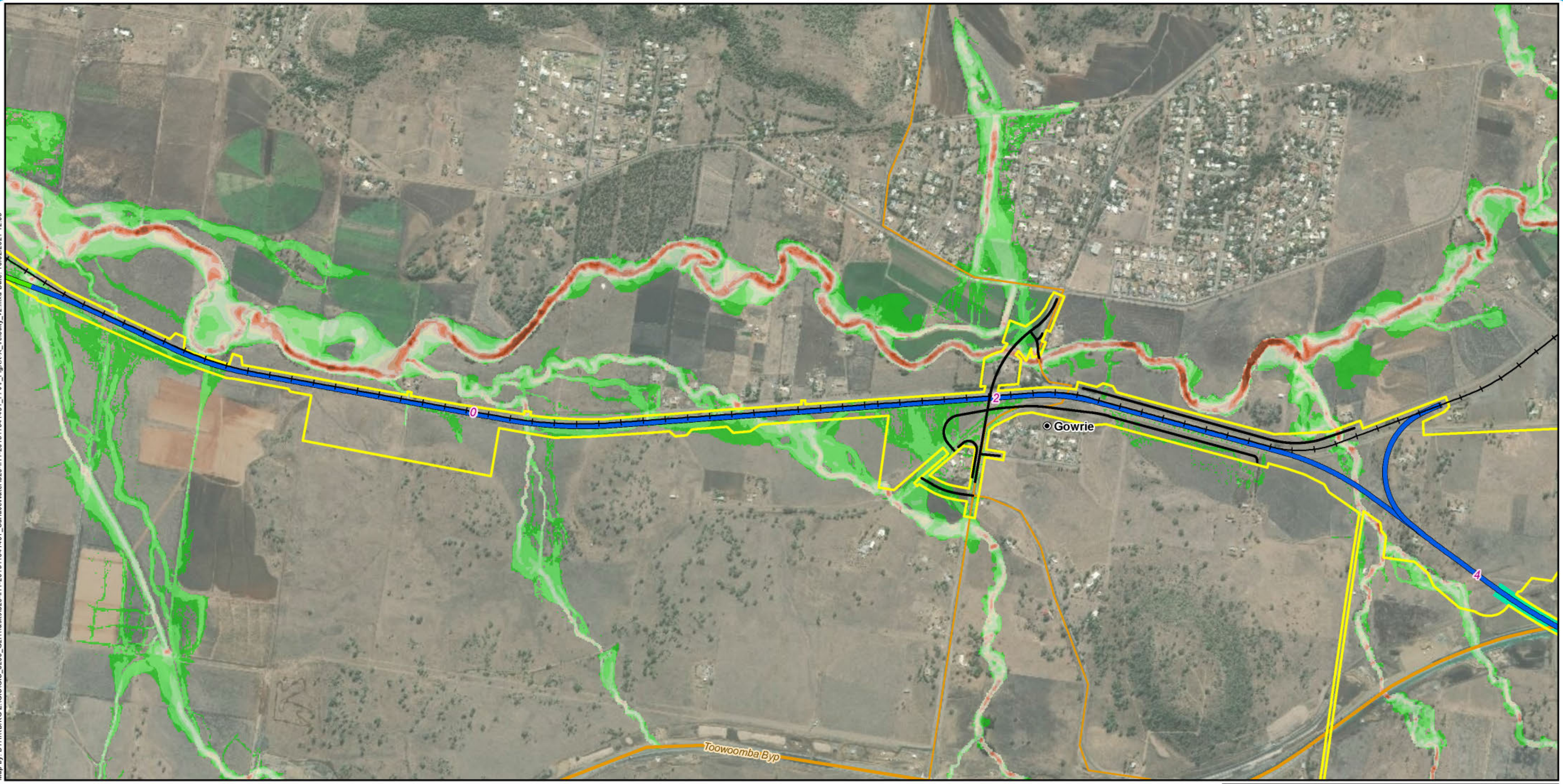
Issue date: 10/02/2021 Version: 2

Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure A7-A: Existing Case - Inundation Extent - 1% AEP event - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community



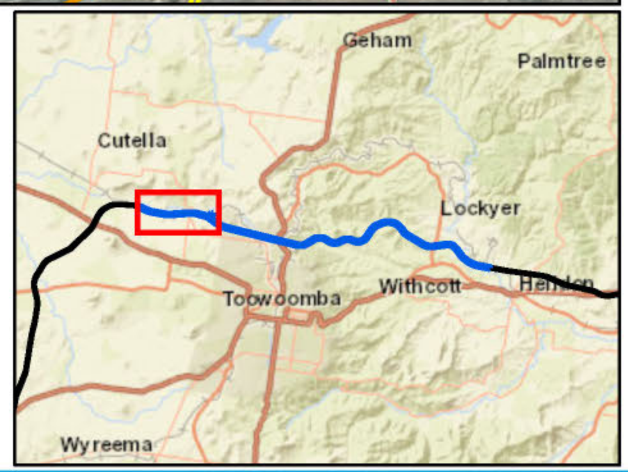
Map by: DTH/RB/KG Z:\GIS\GIS_3200_G2H\Tasks\320-IHY-201911041401_SurfaceWater\320-IHY-201911041401_FFJV_FigXX-X_Velocity_v2.mxd Date: 10/02/2021 12:09

Legend

5 Chainage (km)
Localities
Existing rail
B2G project alignment
G2H project alignment
Tunnel
Major roads
Minor roads
Road design
EIS disturbance footprint

Peak velocity (m/s)

0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0



A3 scale: 1:16,000

0 0.1 0.2 0.3 0.4 0.5km

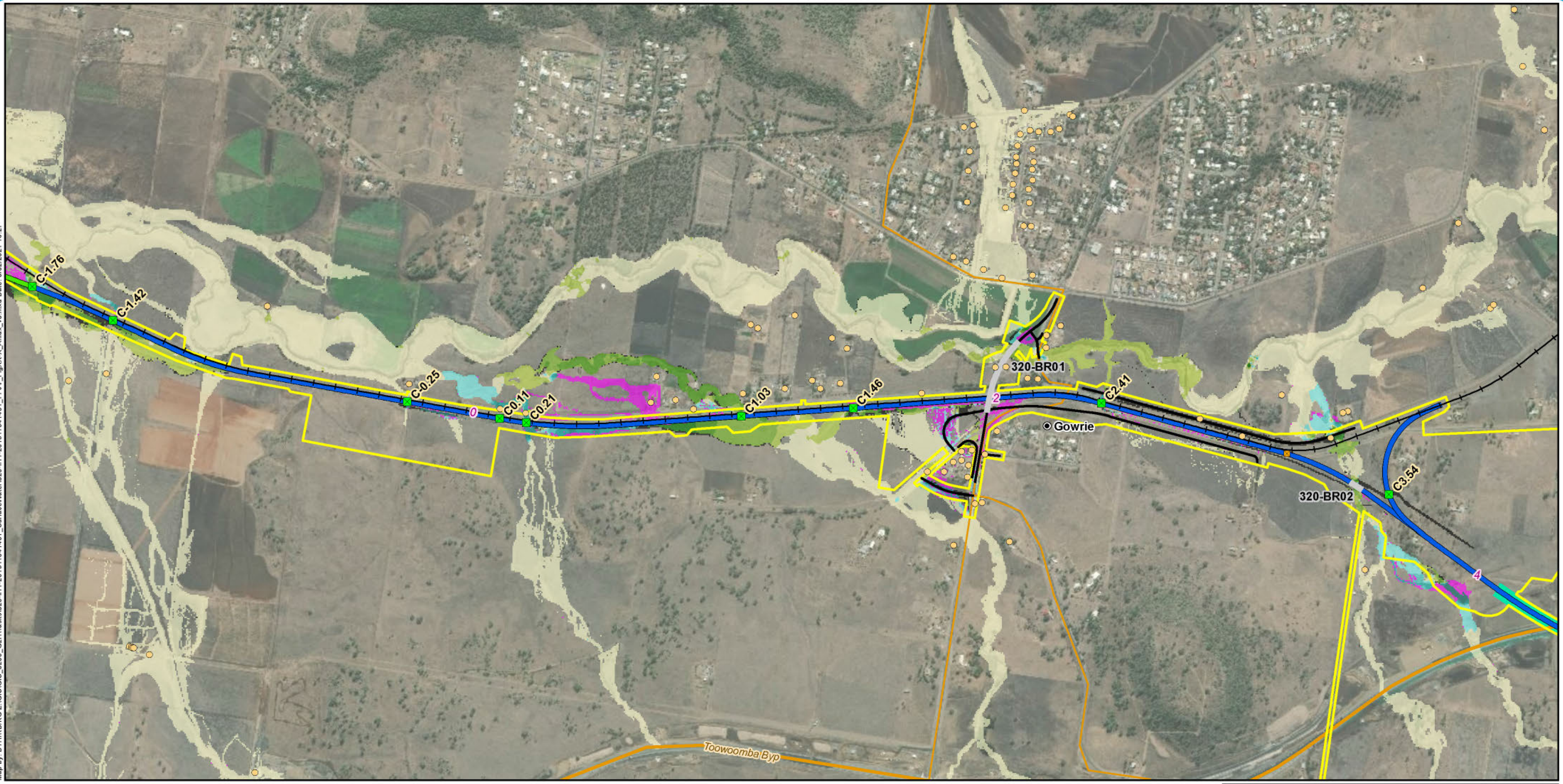
Future Freight
Integrating Community, Environment and Engineering

Issue date: 10/02/2021 Version: 2
Coordinate System: GDA 1994 MGA Zone 56

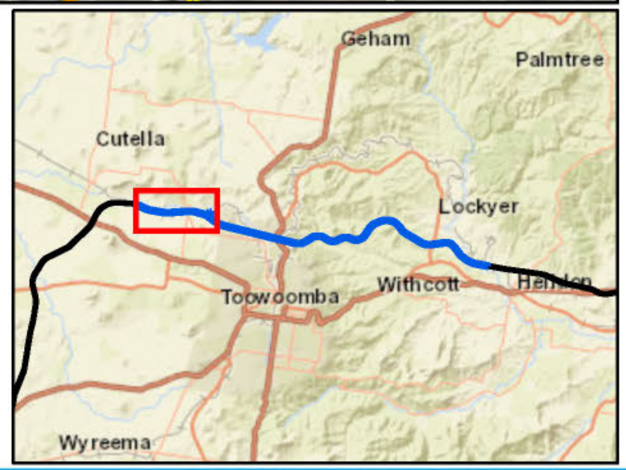
Gowrie to Helidon

Figure A7-B: Existing Case - Velocity - 1% AEP event - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community



Map by: DTH/RB/KG Z:\GIS\GIS_3200_G2H\Tasks\320-IHY-201911041401_SurfaceWater\320-IHY-201911041401_FFJV_FigXX-X_Afflux_v3.mxd Date: 3/02/2021 16:27



Legend

- 5 Chainage (km)
- Localities
- Flood sensitive receptors
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Road design
- EIS disturbance footprint
- Was Wet Now Dry
- Was Dry Now Wet

Drainage Structures

- Floodplain culvert
- Local drainage culvert
- Bridges

Change in peak water levels (m)

- < -0.5
- -0.5 to -0.2
- -0.2 to -0.1
- -0.1 to -0.05
- -0.05 to -0.01
- -0.01 to 0.01
- 0.01 to 0.05
- 0.05 to 0.1
- 0.1 to 0.2
- 0.2 to 0.5
- > 0.5

A3 scale: 1:16,000
0 0.1 0.2 0.3 0.4 0.5km

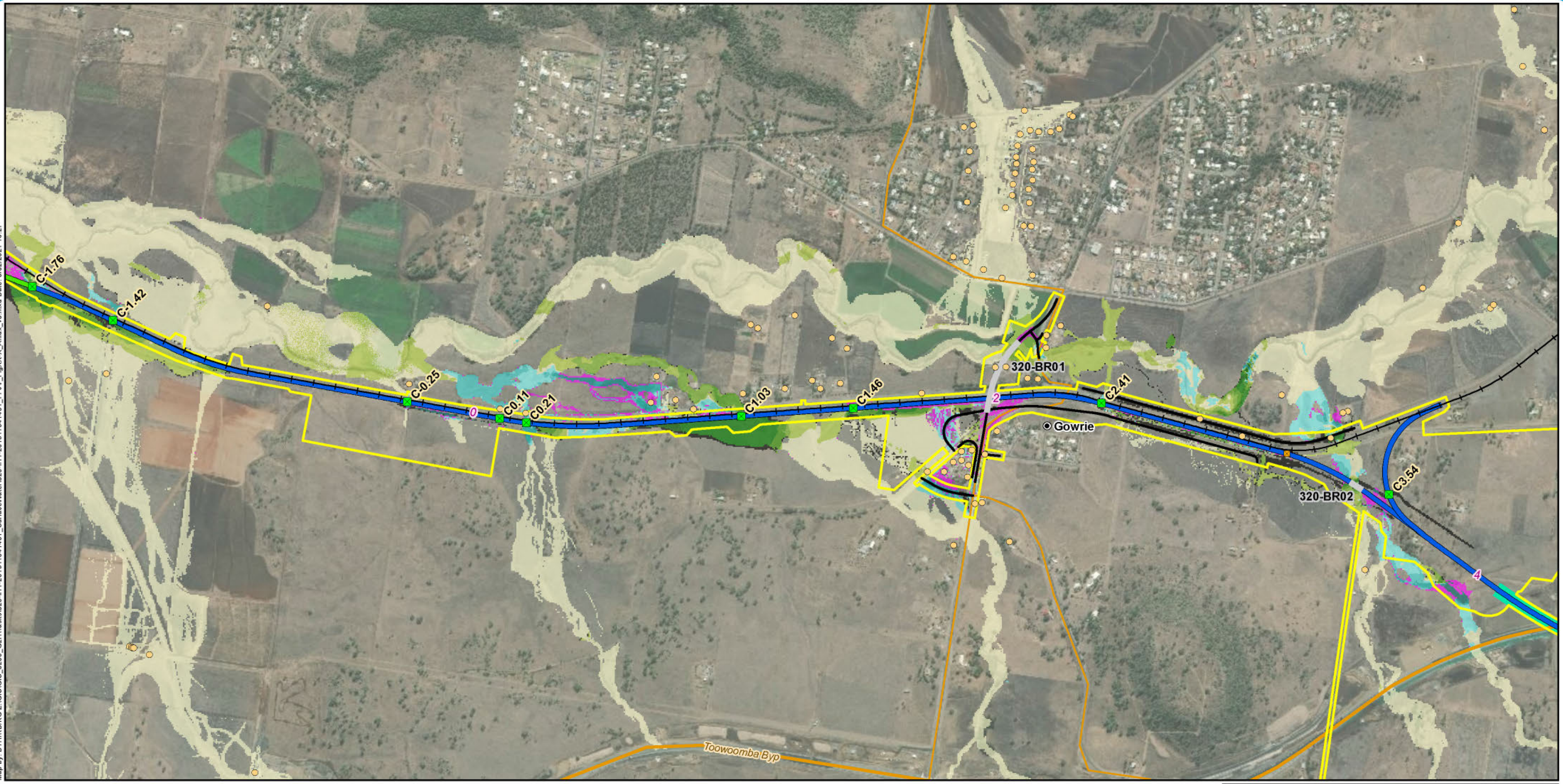
Future Freight
Integrating Community, Environment and Engineering

Issue date: 03/02/2021 Version: 3
Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure A7-C: Developed Case - Afflux - 1% AEP event - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community



Map by: DTH/RB/KG Z:\GIS\GIS_3200_G2H\Tasks\320-IHY-201911041401_SurfaceWater\320-IHY-201911041401_FFJV_FigXX-X_Afflux_v3.mxd Date: 3/02/2021 16:27

Legend

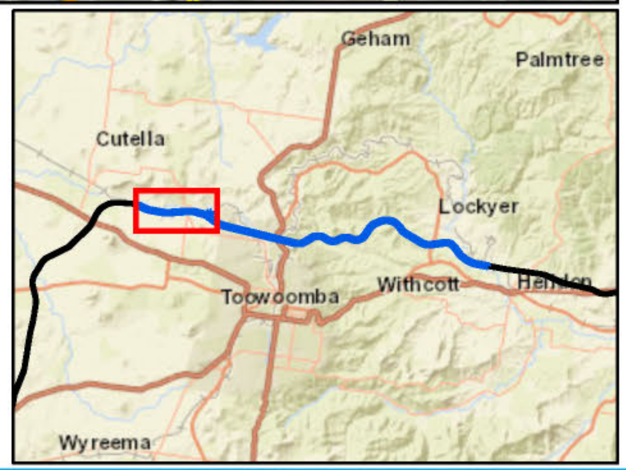
- 5 Chainage (km)
- Localities
- Flood sensitive receptors
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Road design
- EIS disturbance footprint
- Was Wet Now Dry
- Was Dry Now Wet

Drainage Structures

- Floodplain culvert
- Local drainage culvert
- Bridges

Change in peak water levels (m)

- < -0.5
- -0.5 to -0.2
- -0.2 to -0.1
- -0.1 to -0.05
- -0.05 to -0.01
- -0.01 to 0.01
- 0.01 to 0.05
- 0.05 to 0.1
- 0.1 to 0.2
- 0.2 to 0.5
- > 0.5



A3 scale: 1:16,000

0 0.1 0.2 0.3 0.4 0.5km

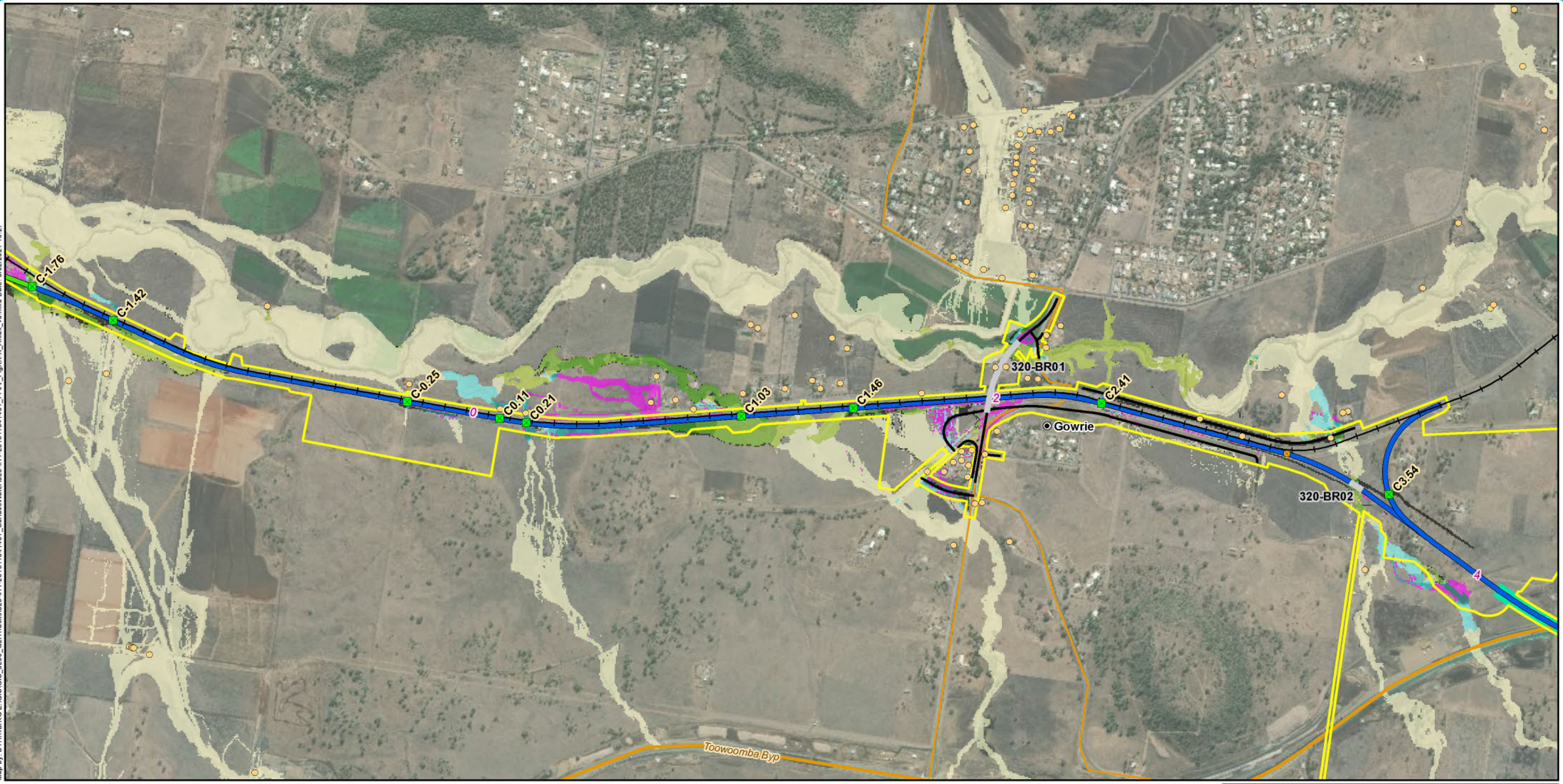
Future Freight
Integrating Community, Environment and Engineering

Issue date: 03/02/2021 Version: 3
Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure A7-D: Climate Change Scenario - Afflux - 1% AEP event - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community



Map by: DTH/RB/KG Z:\GIS\GIS_3200_G2H\Tasks\320-IHY-201911041401_SurfaceWater\320-IHY-201911041401_FFJV_FigXX-X_Afflux_v3.mxd Date: 3/02/2021 16:27

Legend

- 5 Chainage (km)
- Localities
- Flood sensitive receptors
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Road design
- EIS disturbance footprint
- Was Wet Now Dry
- Was Dry Now Wet

Drainage Structures

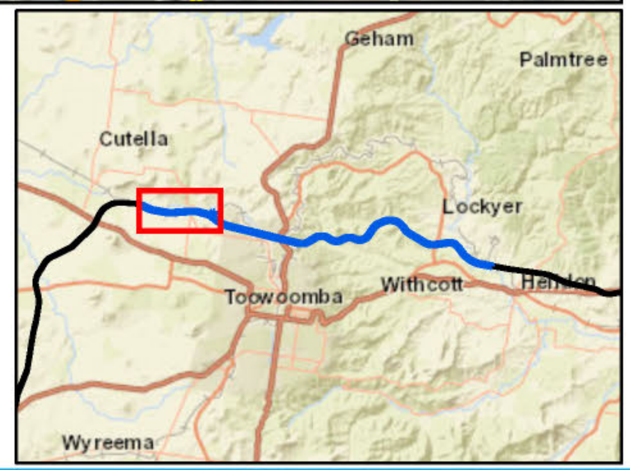
- Floodplain culvert
- Local drainage culvert
- Bridges

Change in peak water levels (m)

- < -0.5
- -0.5 to -0.2
- -0.2 to -0.1
- -0.1 to -0.05
- -0.05 to -0.01
- -0.01 to 0.01
- 0.01 to 0.05
- 0.05 to 0.1
- 0.1 to 0.2
- 0.2 to 0.5
- > 0.5

A3 scale: 1:16,000

0 0.1 0.2 0.3 0.4 0.5km



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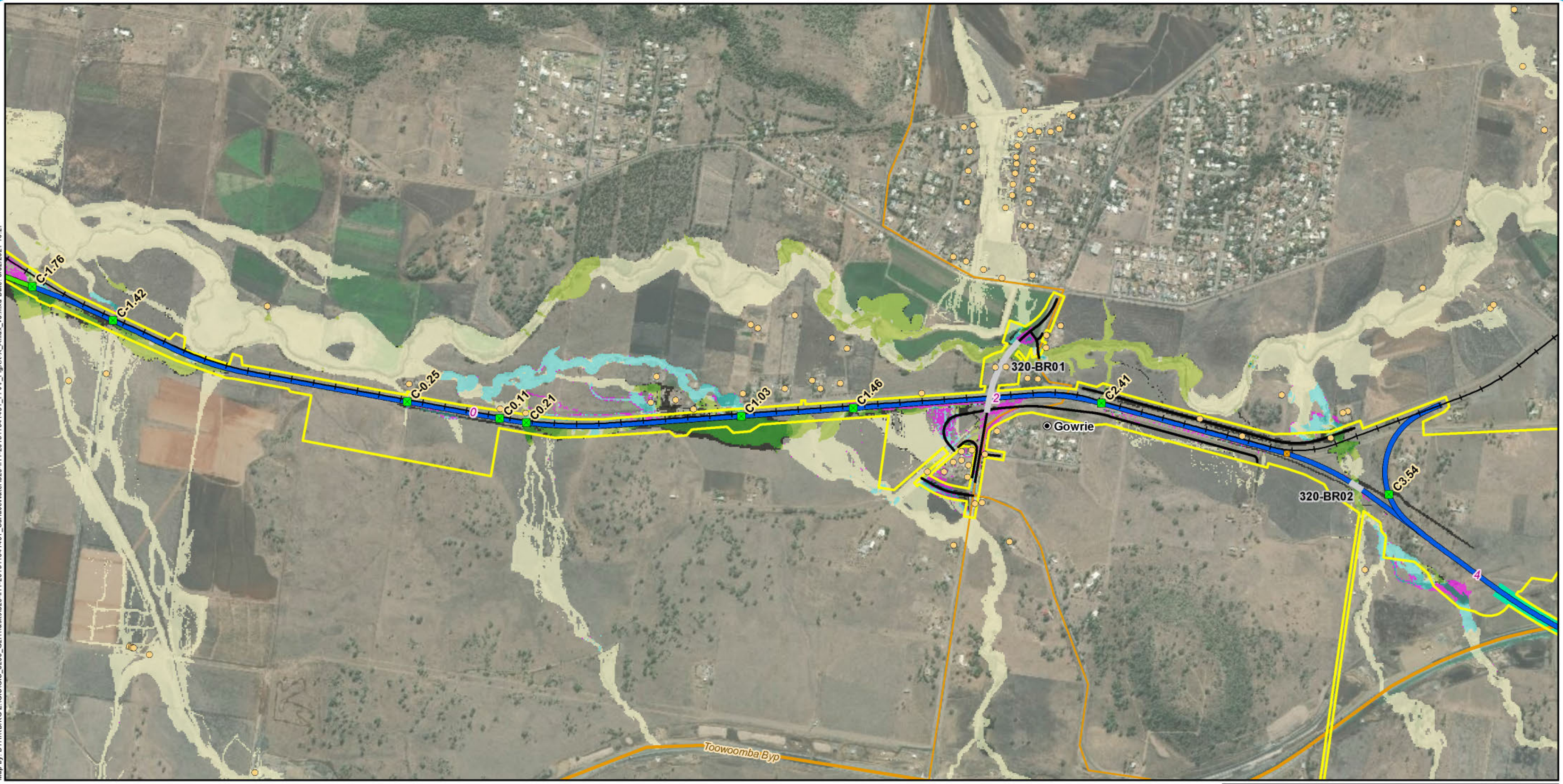
Issue date: 03/02/2021 Version: 3
Coordinate System: GDA 1994 MGA Zone 56

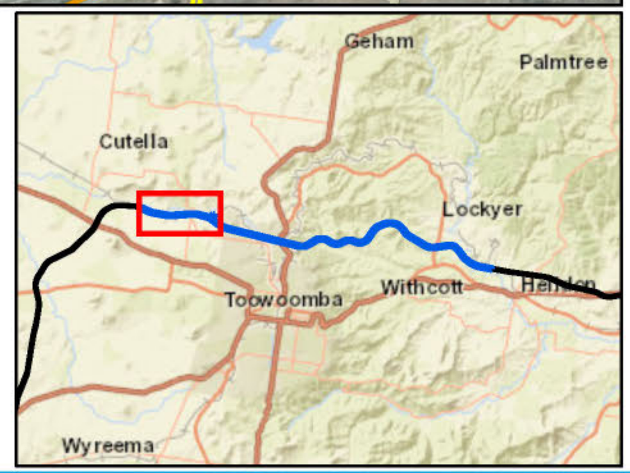
Gowrie to Helidon

Figure A7-E: Blockage 0% Scenario - Afflux - 1% AEP event - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community



Map by: DTH/RB/KG Z:\GIS\GIS_3200_G2H\Tasks\320-IHY-201911041401_SurfaceWater\320-IHY-201911041401_FFJV_FigXX.X_Afflux_v3.mxd Date: 3/02/2021 16:27



Legend

- 5 Chainage (km)
- Localities
- Flood sensitive receptors
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Road design
- EIS disturbance footprint
- Was Wet Now Dry
- Was Dry Now Wet

Drainage Structures

- Floodplain culvert
- Local drainage culvert
- Bridges

Change in peak water levels (m)

- < -0.5
- -0.5 to -0.2
- -0.2 to -0.1
- -0.1 to -0.05
- -0.05 to -0.01
- -0.01 to 0.01
- 0.01 to 0.05
- 0.05 to 0.1
- 0.1 to 0.2
- 0.2 to 0.5
- > 0.5

A3 scale: 1:16,000

0 0.1 0.2 0.3 0.4 0.5km

Future Freight
Integrating Community, Environment and Engineering

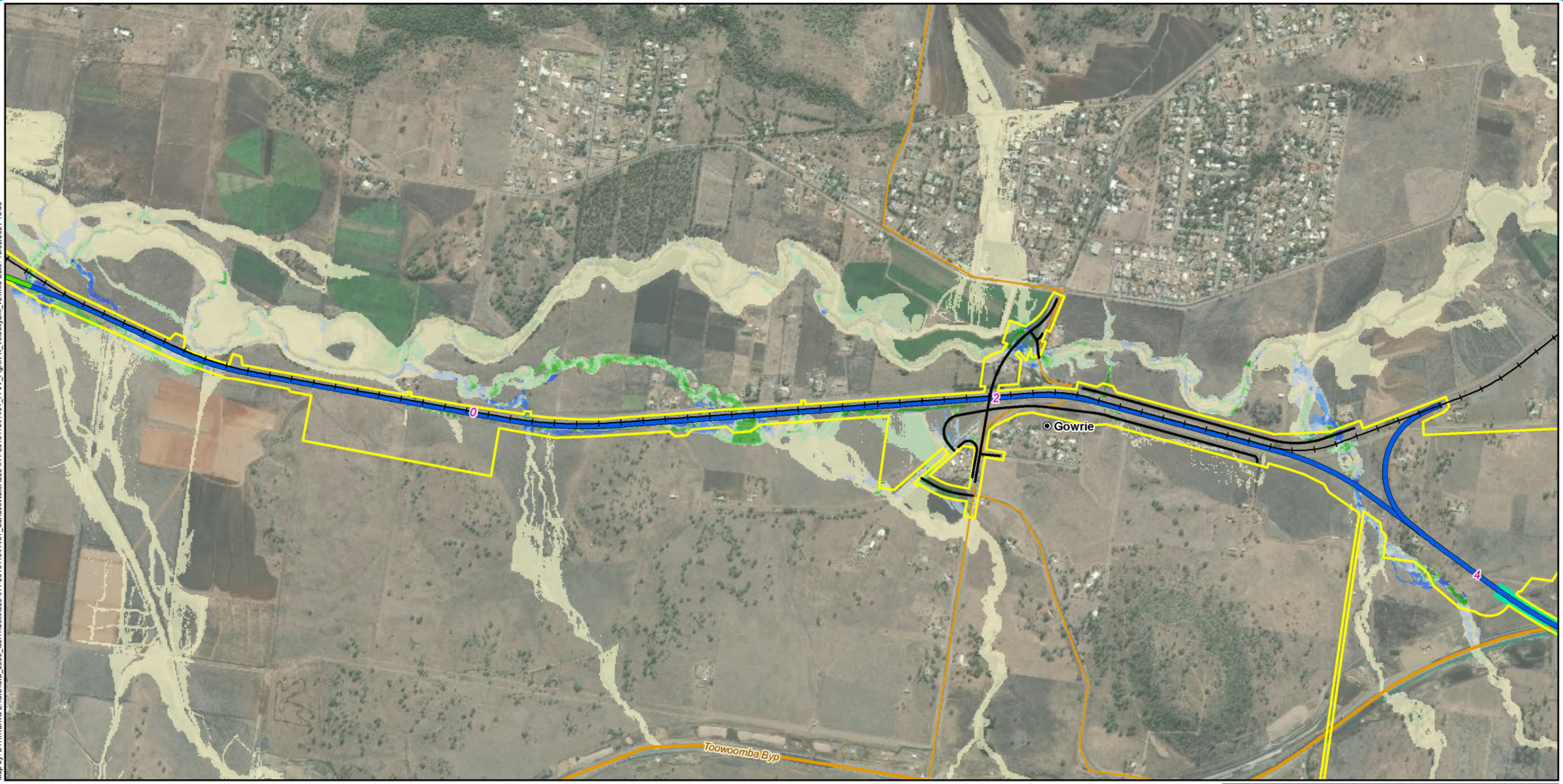
Issue date: 03/02/2021 Version: 3
Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure A7-F: Blockage 50% Scenario - Afflux - 1% AEP event - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community

Map by: DTH/RB/KG Z:\GIS\GIS_3200_G2H\Tasks\320-IHY-201911041401_SurfaceWater\320-IHY-201911041401_FFJV_FigXX-X_VelocityDiff_v2.mxd Date: 10/02/2021 13:09



Legend

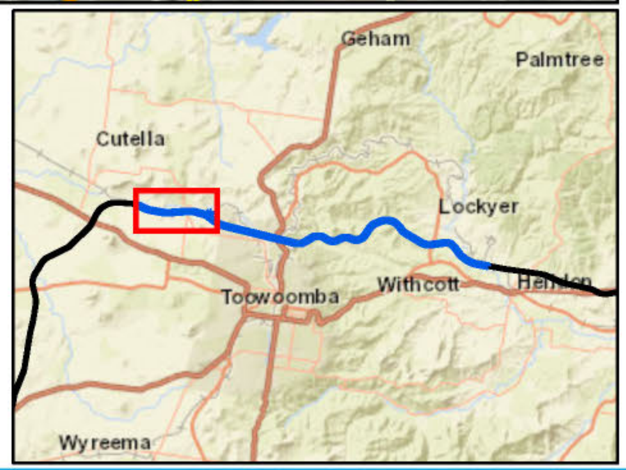
5 Chainage (km)
Localities
Flood sensitive receptors
Existing rail
B2G project alignment
G2H project alignment
Tunnel
Major roads
Minor roads
Road design
EIS disturbance footprint

Change in peak velocity (m/s)

< -0.50
-0.50 to -0.20
-0.20 to -0.10
-0.10 to -0.05
-0.05 to -0.01
-0.01 to 0.01
0.01 to 0.05
0.05 to 0.10
0.10 to 0.20
0.20 to 0.50
> 0.50

A3 scale: 1:16,000

0 0.1 0.2 0.3 0.4 0.5km



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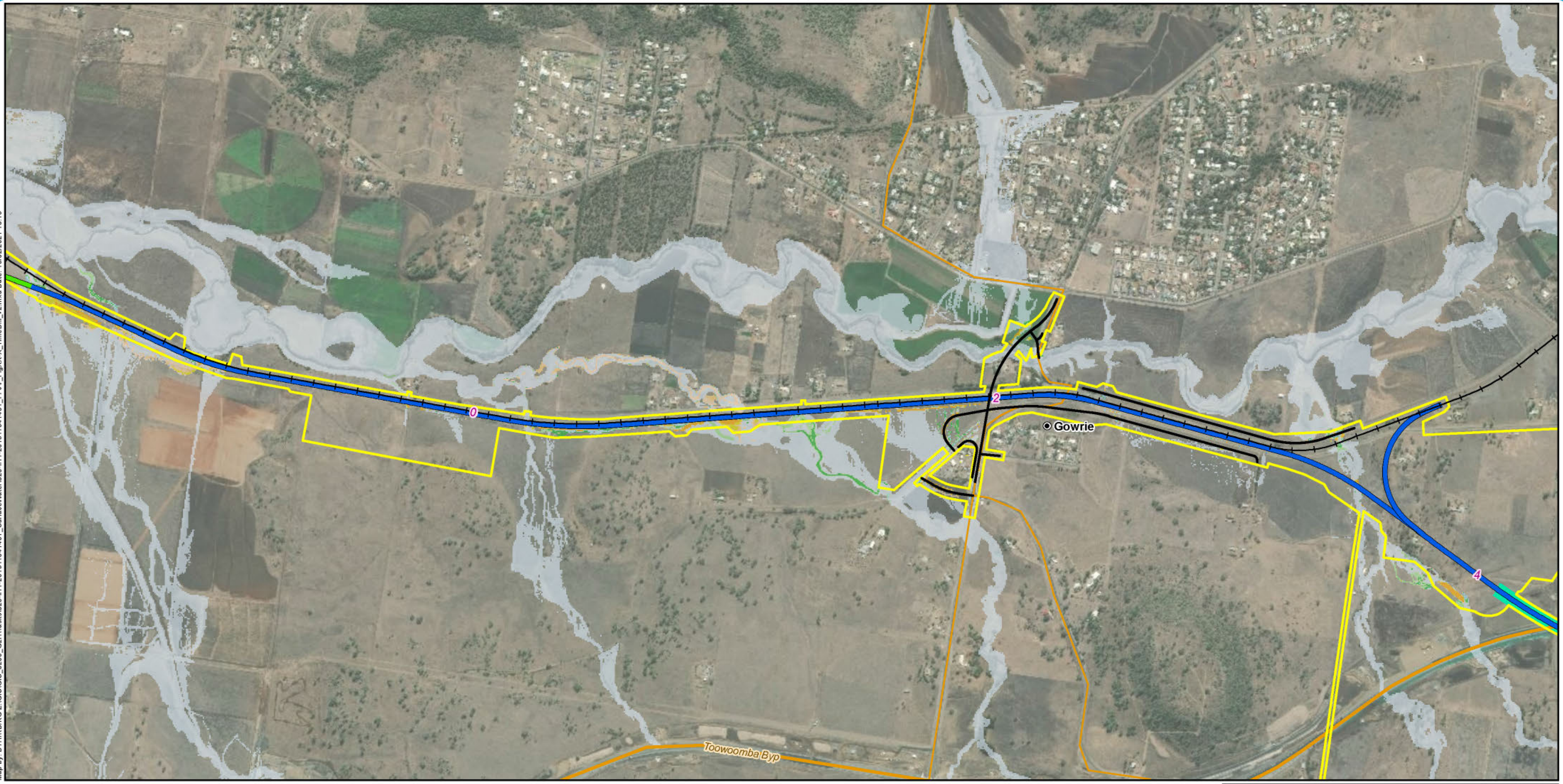
Issue date: 10/02/2021 Version: 2
Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure A7-G: Developed Case - Difference in Velocity - 1% AEP event - Gowrie Creek

Map by: DTH/RB/KG Z:\GIS\GIS_3200_G2H\Tasks\320-IHY-201911041401_SurfaceWater\320-IHY-201911041401_FFJV_FigXX.X_TimeDiff_v2.mxd Date: 10/02/2021 13:15

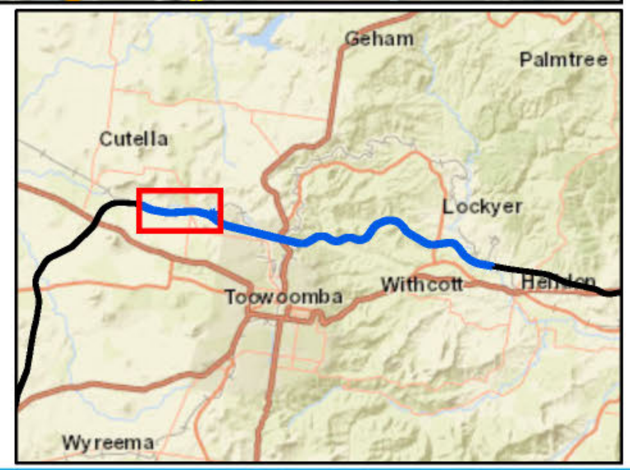


Legend

- 5 Chainage (km)
- Localities
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Road design
- EIS disturbance footprint

Difference in Time of Submergence

- Over 10 hour decrease
- Up to 10 hour decrease
- Up to 5 hour decrease
- Up to 2 hour decrease
- Up to 1 hour decrease
- Up to 0.5 hour decrease
- Within 15 minutes
- Up to 0.5 hour increase
- Up to 1 hour increase
- Up to 2 hour increase
- Up to 5 hour increase
- Up to 10 hour increase
- Over 10 hour increase



A3 scale: 1:16,000

0 0.1 0.2 0.3 0.4 0.5km

Future Freight
Integrating Community, Environment and Engineering

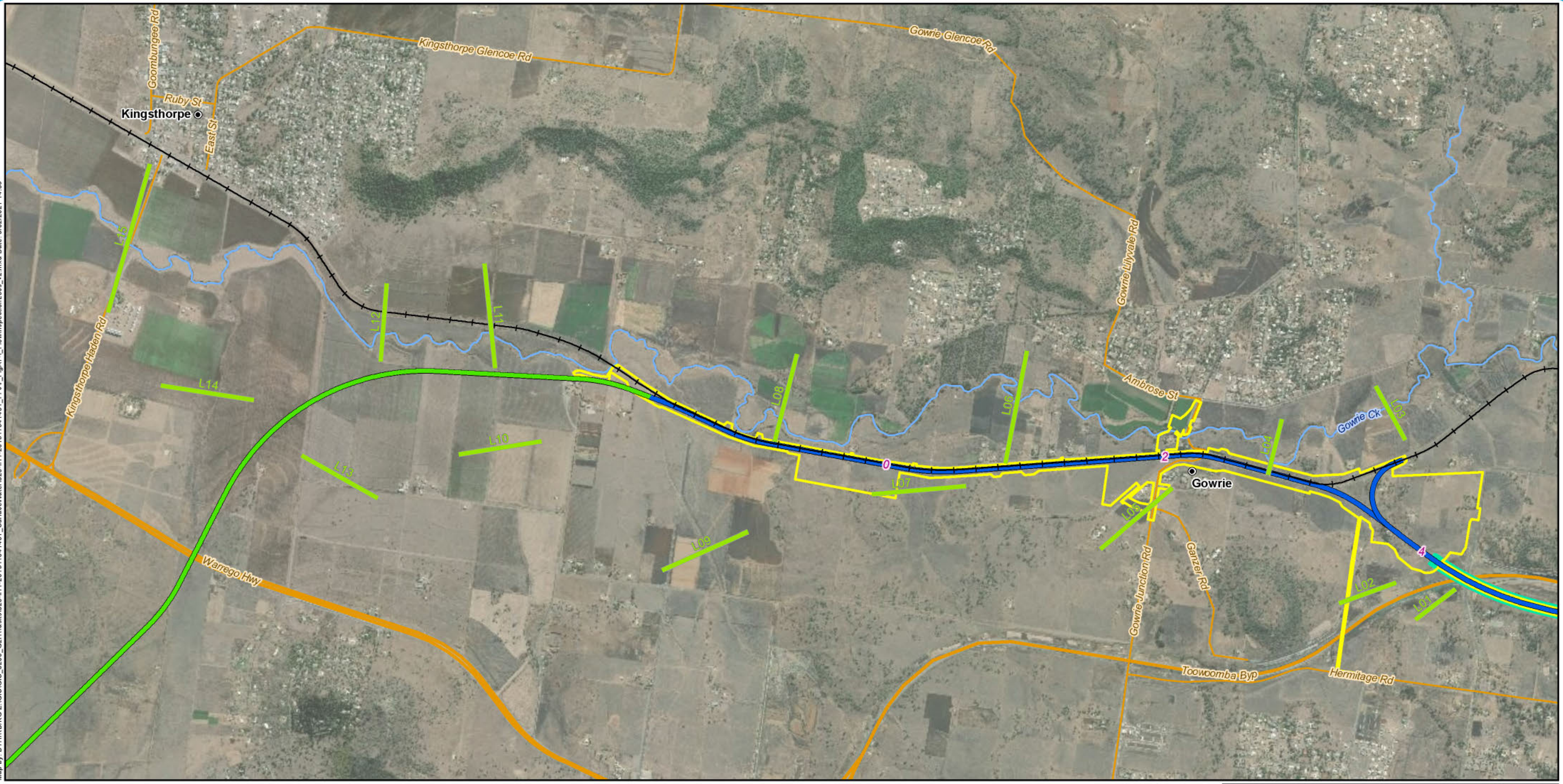
Issue date: 10/02/2021 Version: 2
Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

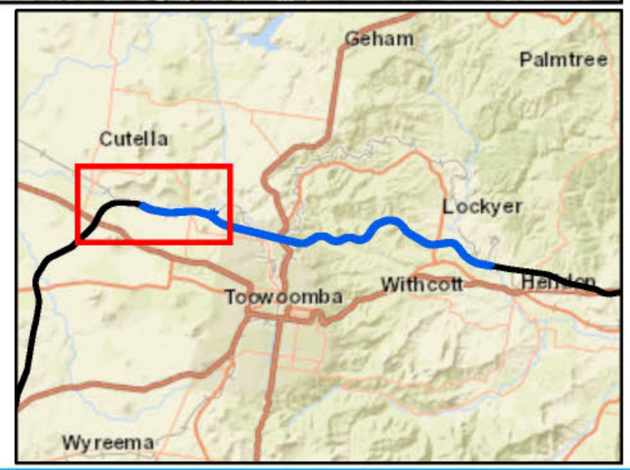
Figure A7-H:

Developed Case - Difference in Time of Submergence - 1% AEP event - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community



Map by: DTH/RB/KG Z:\GIS\GIS_3200_G2H\Tasks\320-IHY-201911041401_SurfaceWater\320-IHY-201911041401_FFJV_FigA7-I_FlowInspectionLocs_v2.mxd Date: 3/02/2021 14:35



Legend

- 5 Chainage (km)
- Localities
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Watercourses
- EIS disturbance footprint
- Flow distribution inspection locations

A3 scale: 1:30,000

0 0.25 0.5 0.75 1 1.25 km

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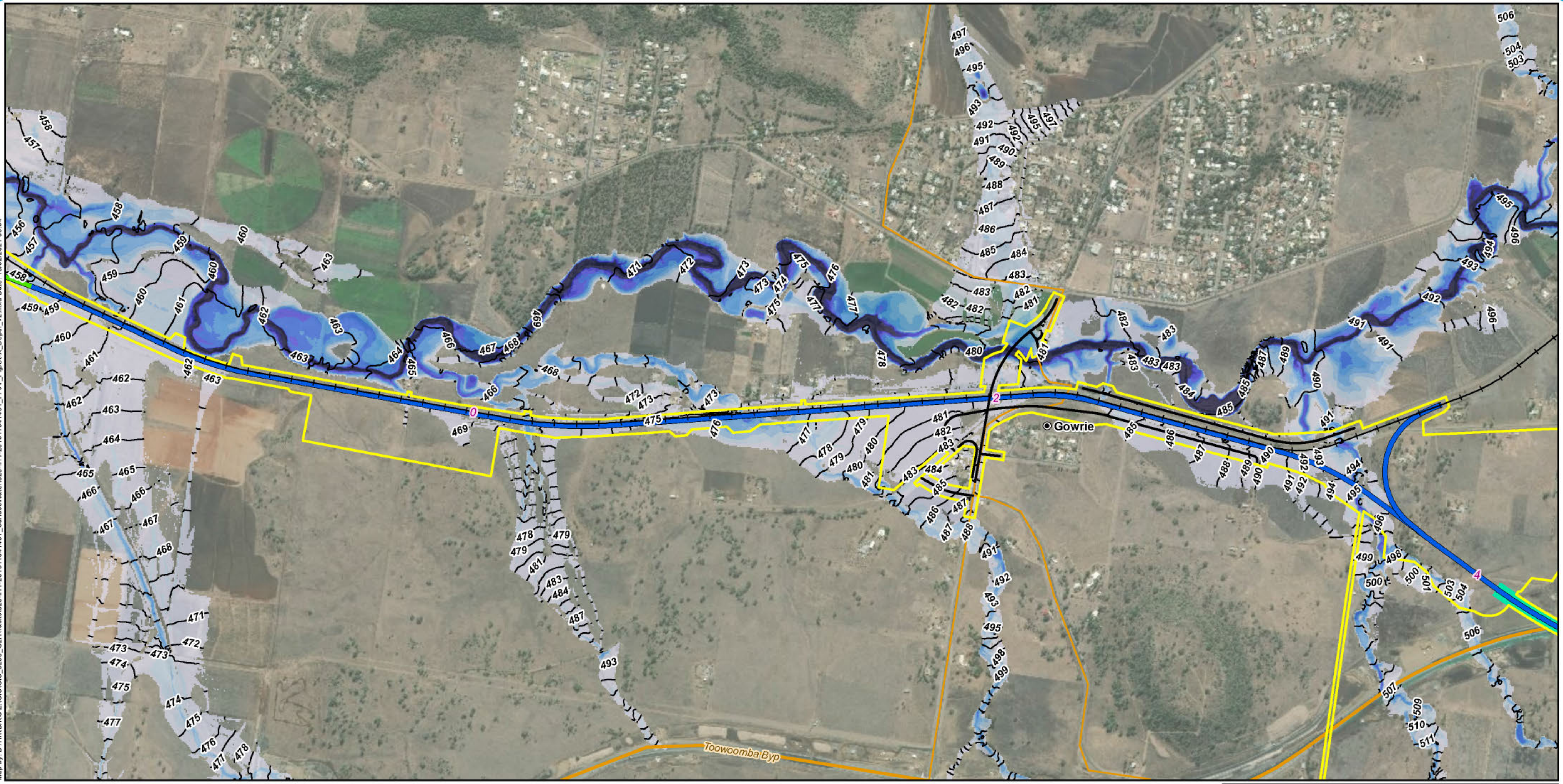
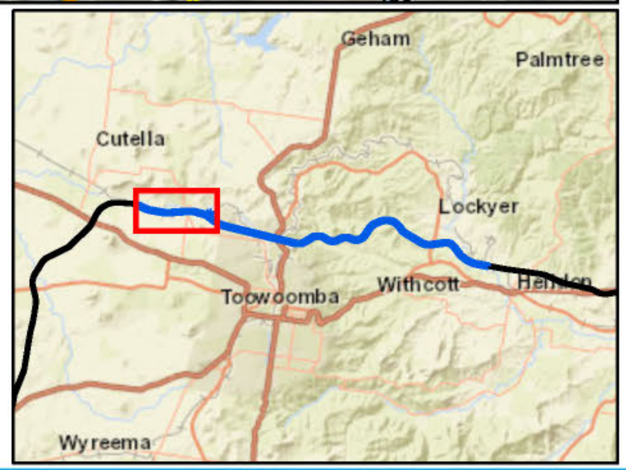
Issue date: 03/02/2021 Version: 2
Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure A7-I: Flow inspection locations - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community





Legend

- 5 Chainage (km)
- Localities
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Road design
- EIS disturbance footprint
- 1.0m contour mAHD

Depth (m)

- 0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 2.5
- 2.5 - 3.0
- 3.0 - 3.5
- 3.5 - 4.0
- 4.0 - 4.5
- 4.5 - 5.0
- > 5.0

A3 scale: 1:16,000

0 0.1 0.2 0.3 0.4 0.5km

Future Freight
Integrating Community, Environment and Engineering

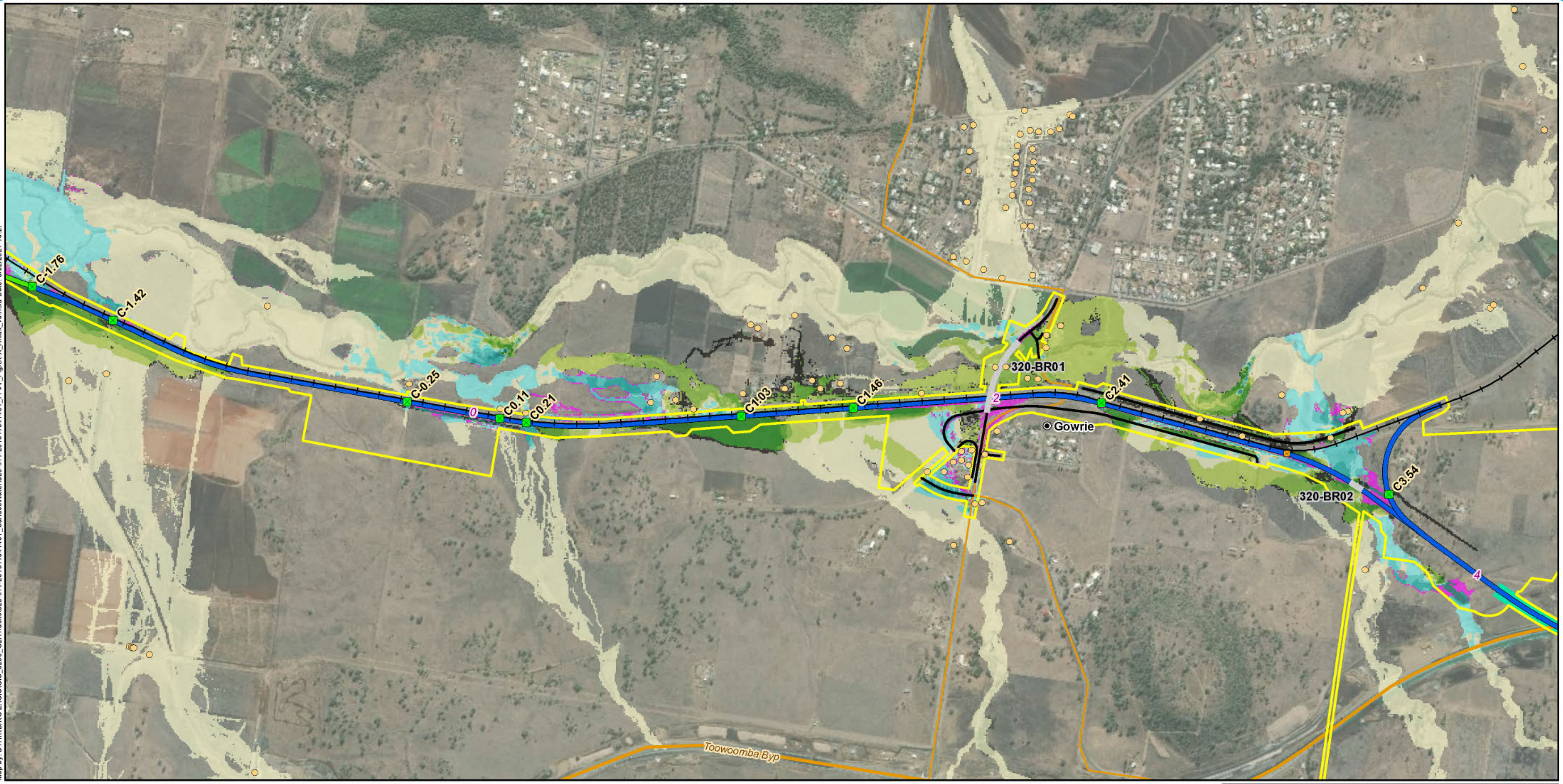
Issue date: 10/02/2021 Version: 2
Coordinate System: GDA 1994 MGA Zone 56

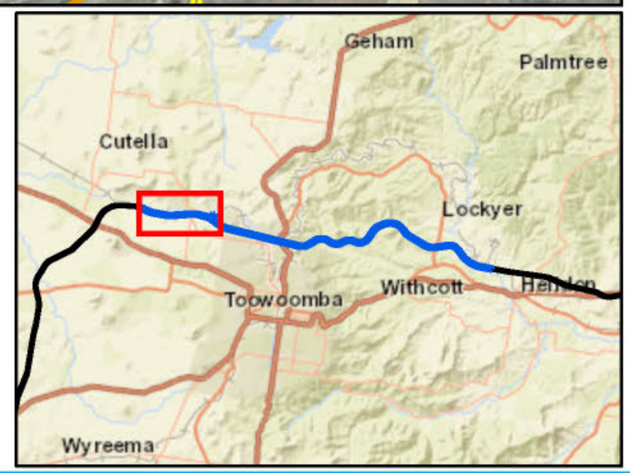
Gowrie to Helidon

Figure A8-A: Existing Case - Inundation Extent - 1 in 2,000 AEP event - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community

Map by: DTH/RB/KG Z:\GIS\GIS_3200_G2H\Tasks\320-IHY-201911041401_SurfaceWater\320-IHY-201911041401_FFJV_FigXX-X_Afflux_v3.mxd Date: 3/02/2021 16:27





Legend

- 5 Chainage (km)
- Localities
- Flood sensitive receptors
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Road design
- EIS disturbance footprint
- Was Wet Now Dry
- Was Dry Now Wet

Drainage Structures

- Floodplain culvert
- Local drainage culvert
- Bridges

Change in peak water levels (m)

- < -0.5
- -0.5 to -0.2
- -0.2 to -0.1
- -0.1 to -0.05
- -0.05 to -0.01
- -0.01 to 0.01
- 0.01 to 0.05
- 0.05 to 0.1
- 0.1 to 0.2
- 0.2 to 0.5
- > 0.5

A3 scale: 1:16,000

0 0.1 0.2 0.3 0.4 0.5km

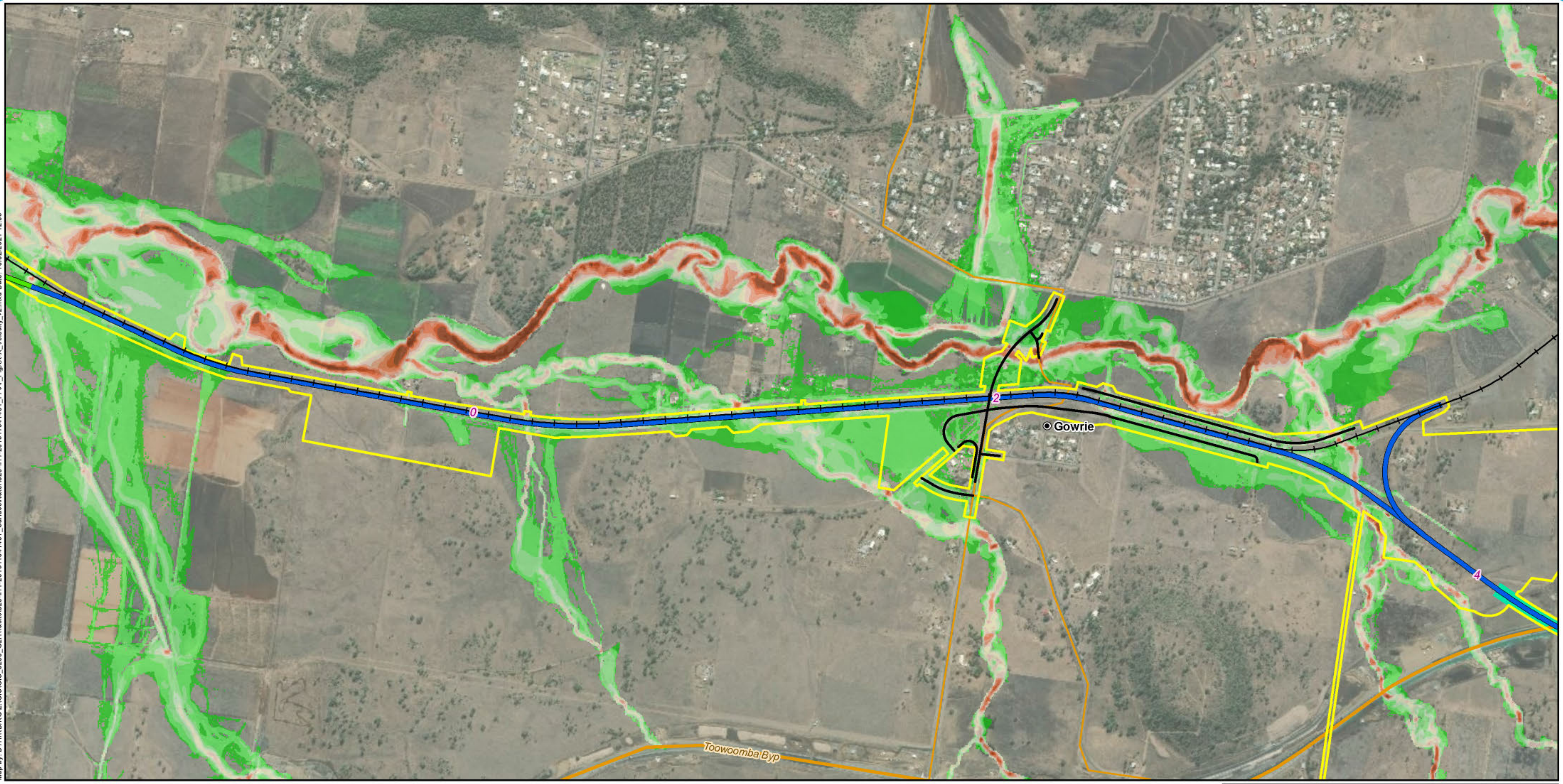
Future Freight
Integrating Community, Environment and Engineering

Issue date: 03/02/2021 Version: 3
Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure A8-B: Developed Case - Afflux - 1 in 2,000 AEP event - Gowrie Creek

Map by: DTH/RB/KG Z:\GIS\GIS_3200_G2H\Tasks\320-IHY-201911041401_SurfaceWater\320-IHY-201911041401_FFJV_FigXX-X_Velocity_v2.mxd Date: 10/02/2021 12:09

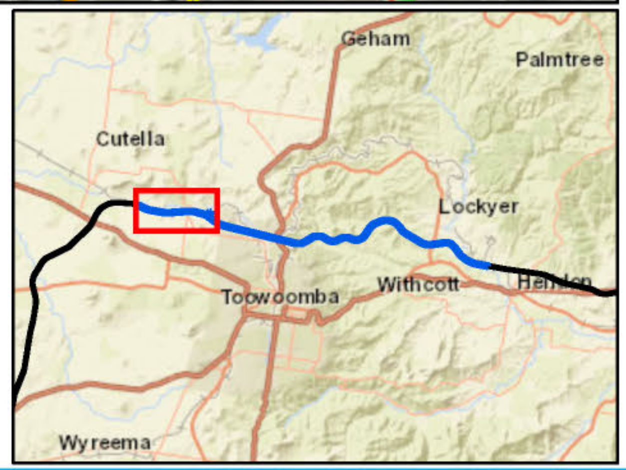


Legend

- 5 Chainage (km)
- Localities
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Road design
- EIS disturbance footprint

Peak velocity (m/s)

0.0	3.0
0.5	3.5
1.0	4.0
1.5	4.5
2.0	5.0
2.5	



A3 scale: 1:16,000

0 0.1 0.2 0.3 0.4 0.5km

Future Freight
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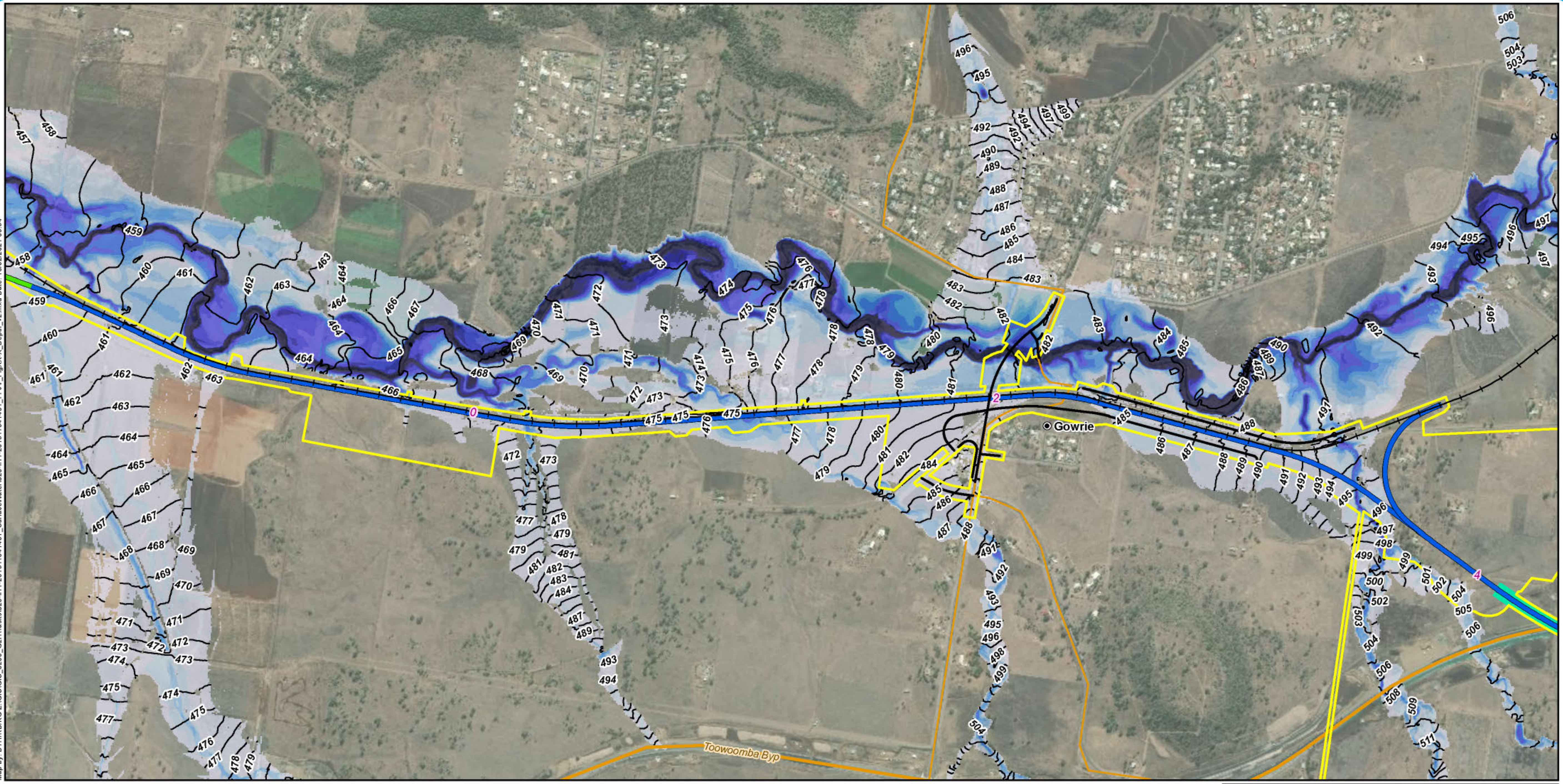
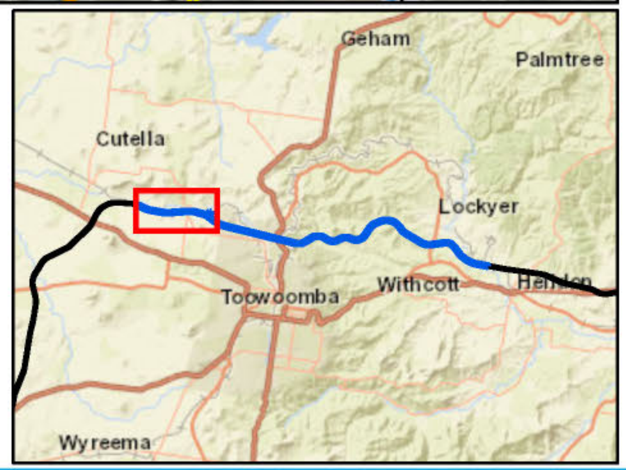
Issue date: 10/02/2021 Version: 2
Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure A8-C: Developed Case - Velocity - 1 in 2,000 AEP event - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community





Legend

- 5 Chainage (km)
- Localities
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Road design
- EIS disturbance footprint
- 1.0m contour mAHD

Depth (m)

- 0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 2.5
- 2.5 - 3.0
- 3.0 - 3.5
- 3.5 - 4.0
- 4.0 - 4.5
- 4.5 - 5.0
- > 5.0

A3 scale: 1:16,000

0 0.1 0.2 0.3 0.4 0.5km

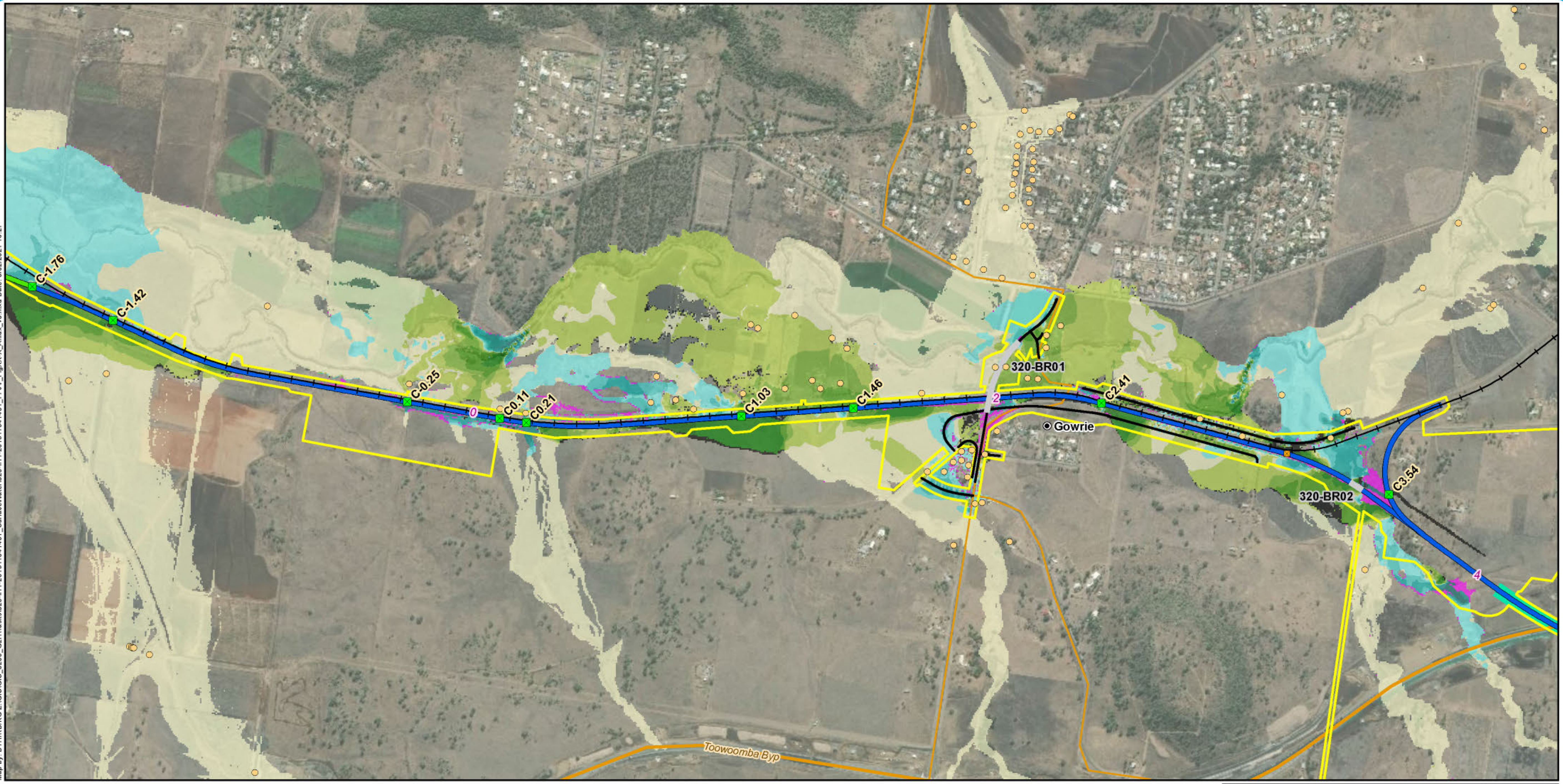
Future Freight
Integrating Community, Environment and Engineering

Issue date: 10/02/2021 Version: 2
Coordinate System: GDA 1994 MGA Zone 56

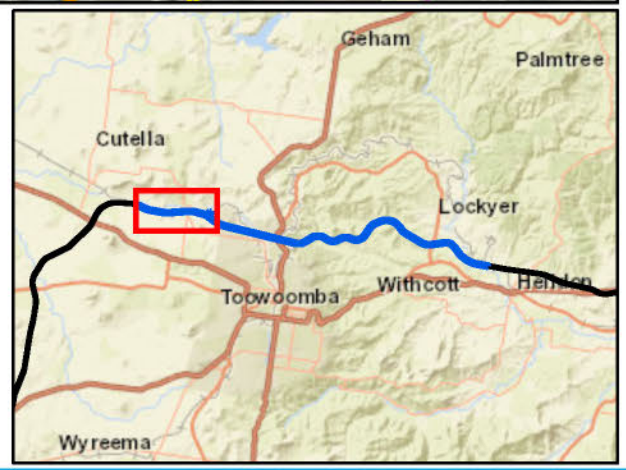
Gowrie to Helidon

Figure A9-A: Existing Case - Inundation Extent - 1 in 10,000 AEP event - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community



Map by: DTH/RB/KG Z:\GIS\GIS_3200_G2H\Tasks\320-IHY-201911041401_SurfaceWater\320-IHY-201911041401_FFJV_FigXX.X_Afflux_v3.mxd Date: 3/02/2021 16:27



Legend

- 5 Chainage (km)
- Localities
- Flood sensitive receptors
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Road design
- EIS disturbance footprint
- Was Wet Now Dry
- Was Dry Now Wet

Drainage Structures

- Floodplain culvert
- Local drainage culvert
- Bridges

Change in peak water levels (m)

- < -0.5
- -0.5 to -0.2
- -0.2 to -0.1
- -0.1 to -0.05
- -0.05 to -0.01
- -0.01 to 0.01
- 0.01 to 0.05
- 0.05 to 0.1
- 0.1 to 0.2
- 0.2 to 0.5
- > 0.5

A3 scale: 1:16,000

0 0.1 0.2 0.3 0.4 0.5km

Future Freight
Integrating Community, Environment and Engineering

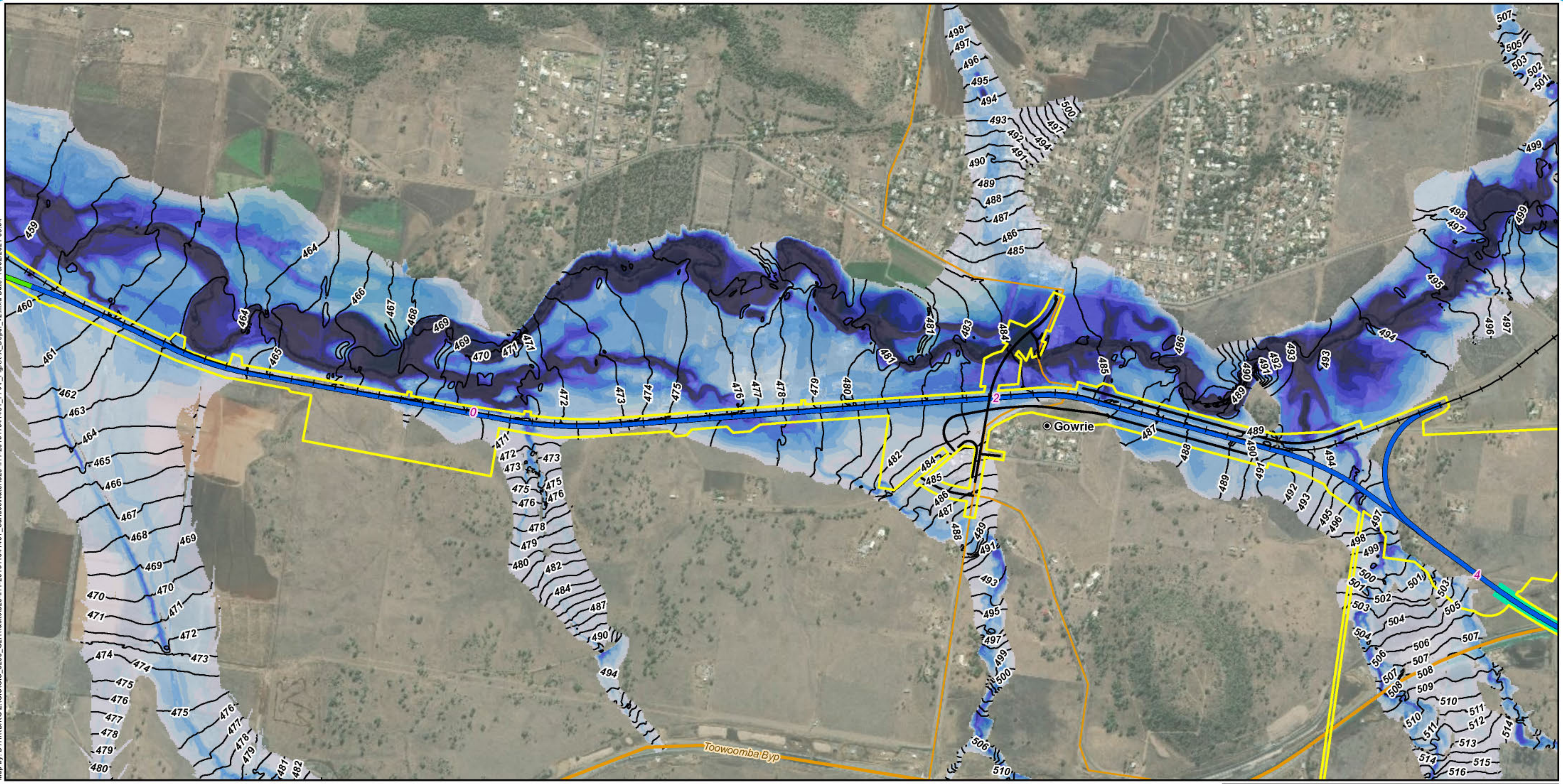
Issue date: 03/02/2021 Version: 3
Coordinate System: GDA 1994 MGA Zone 56

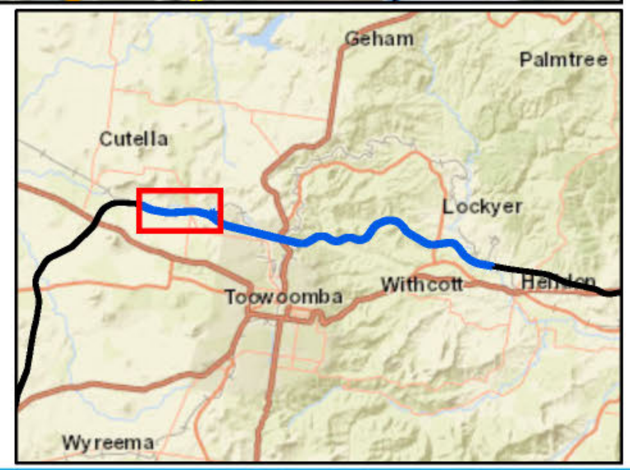
Gowrie to Helidon

Figure A9-B: Developed Case - Afflux - 1 in 10,000 AEP event - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community

Map by: DTH/RB/KG Z:\GIS\GIS_3200_G2H\Tasks\320-IHY-201911041401_SurfaceWater\320-IHY-201911041401_FFJV_FigXX.X_Depth_v2.mxd Date: 10/02/2021 09:54





Legend

5 Chainage (km)
Localities
Existing rail
B2G project alignment
G2H project alignment
Tunnel
Major roads
Minor roads
Road design
EIS disturbance footprint
1.0m contour mAHD

Depth (m)

0 - 0.5
0.5 - 1.0
1.0 - 1.5
1.5 - 2.0
2.0 - 2.5
2.5 - 3.0
3.0 - 3.5
3.5 - 4.0
4.0 - 4.5
4.5 - 5.0
> 5.0

A3 scale: 1:16,000
0 0.1 0.2 0.3 0.4 0.5km

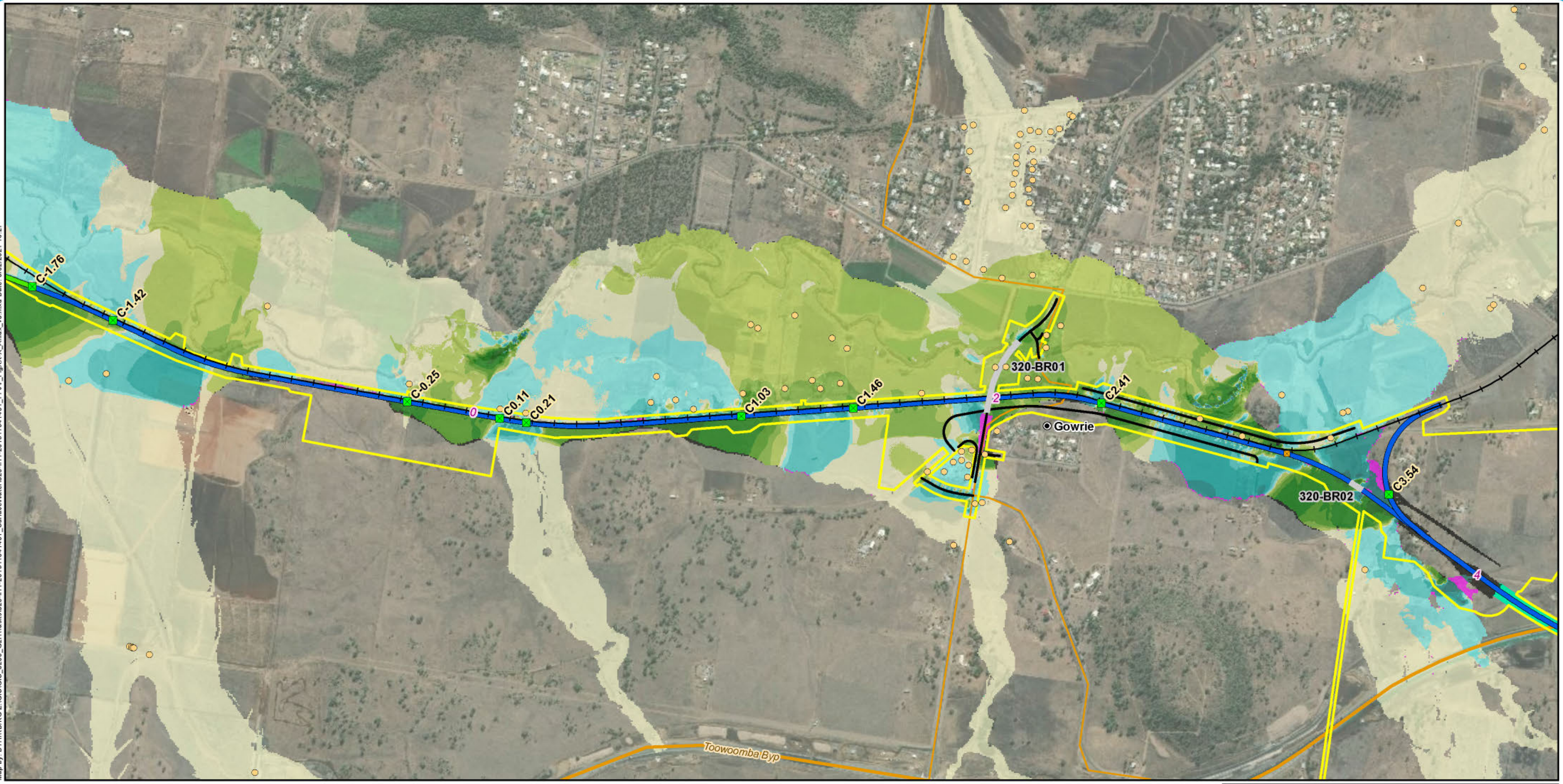
Future Freight
Integrating Community, Environment and Engineering

Issue date: 10/02/2021 Version: 2
Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure A10-A: Existing Case - Inundation Extent - PMF event - Gowrie Creek

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community



Map by: DTH/RB/KG Z:\GIS\GIS_3200_G2H\Tasks\320-IHY-201911041401_SurfaceWater\320-IHY-201911041401_FFJV_FigXX.X_Afflux_v3.mxd Date: 3/02/2021 16:27

Legend

- 5 Chainage (km)
- Localities
- Flood sensitive receptors
- Existing rail
- B2G project alignment
- G2H project alignment
- Tunnel
- Major roads
- Minor roads
- Road design
- EIS disturbance footprint
- Was Wet Now Dry
- Was Dry Now Wet

Drainage Structures

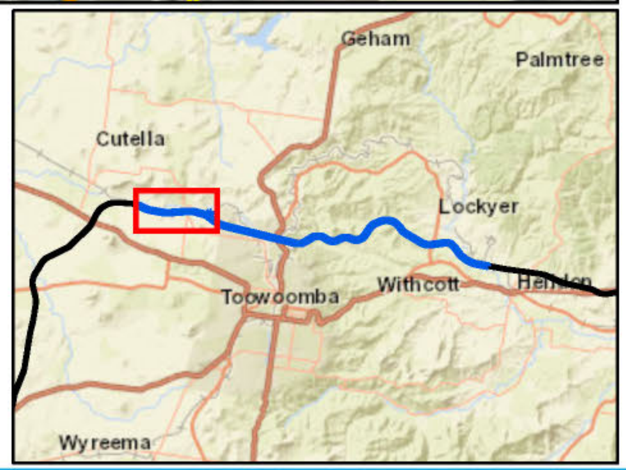
- Floodplain culvert
- Local drainage culvert
- Bridges

Change in peak water levels (m)

- < -0.5
- -0.5 to -0.2
- -0.2 to -0.1
- -0.1 to -0.05
- -0.05 to -0.01
- -0.01 to 0.01
- 0.01 to 0.05
- 0.05 to 0.1
- 0.1 to 0.2
- 0.2 to 0.5
- > 0.5

A3 scale: 1:16,000

0 0.1 0.2 0.3 0.4 0.5km



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Issue date: 03/02/2021 Version: 3
Coordinate System: GDA 1994 MGA Zone 56

Gowrie to Helidon

Figure A10-B: Developed Case - Afflux - PMF event - Gowrie Creek