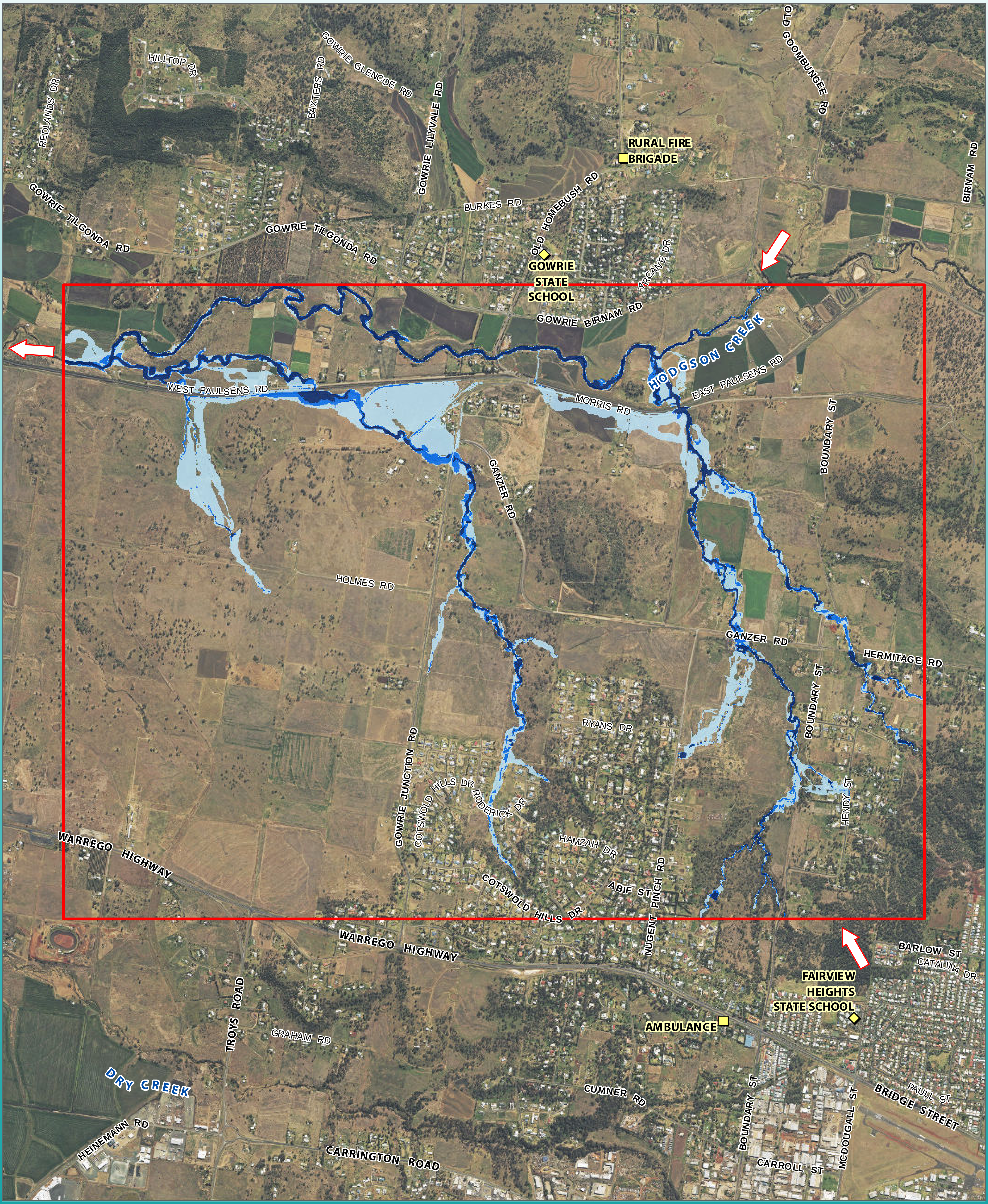
# COTSWOLD HILLS




TOOWOOMBA REGIONAL COUNCIL

1:22,000 (at A3)

0 200 400 600 800

Metres

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## 1% AEP FLOOD DEPTH RIVERINE

| Water Depth (m) | | | |
|---|---|---|---|
| 0 - 0.5 | | Model Extent | |
| 0.5 - 1 | | DirectionFlow | |
| 1+ | | Emergency Services | |
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# Flood Studies

TOOWOOMBA REGION
Rich traditions. Bold ambitions.

## 2D Flood Study for Cotswold Hills (Gowrie Creek Catchment)

August 2014 • *Endorsed on 25 February 2015*

## GENERAL NOTE

These reports/documents are a base source of information that will be continually refined over time.

## DISCLAIMER

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**CLIENT:** Toowoomba Regional Council

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|---|---|---|---|---|
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| FINAL 1 | 1 May 2014 | Final Report | MB/JO/DS | TV/SM |
| FINAL 2 | 16 May 2014 | Final Report (rev 1) | MB/JO/DS | TV/SM |
| FINAL 3 | 21 August 2014 | Final Report (rev 2) | MB/JO/DS | TV/SM |

For and on behalf of
**WRM Water & Environment Pty Ltd**

**Sharmil Markar**
**Director**

# EXECUTIVE SUMMARY

Toowoomba Regional Council (TRC) has appointed WRM Water and Environment Pty Ltd (WRM) in association with DHI Water and Environment Pty Ltd (DHI) to carry out hydraulic investigations of flooding in the town of Cotswold Hills. The hydrologic modelling was undertaken using an XP-RAFTS model and the hydraulic modelling was undertaken using a coupled MIKE FLOOD 1D/2D hydrodynamic model.

The majority of the data for the construction of the hydraulic model was derived from a 1m LiDAR Digital Elevation Model (DEM) provided by TRC. A site visit was undertaken on 4th September, 2013. The purpose of the site visit was to allow the project team to identify key drainage features within the catchment, survey structures with potential significant hydraulic impact and gain a general feel for the floodplain.

The validation data consisted of recorded rainfall and information on two locations where flooding was observed (but not measured) during the January 2011 event. The XP-RAFTS and MIKE FLOOD models were validated to this event iteratively using a joint calibration approach. Flows from XP-RAFTS were adjusted and applied in the MIKE FLOOD model until areas of known flooding were reproduced by the model.

The hydraulic model results show that Cotswold Hills was not affected by flooding during the January 2011 event except for a few low-lying rural areas which were inundated.

Available design rainfall data and associated procedures to determine the design flood discharges for the Cotswold Hills area have been reviewed, and an appropriate methodology and design parameters for this study have been proposed. A comparison of the XP-RAFTS model design discharges against discharges estimated using the Rational Method for 10 year and 100 year Average Recurrence Interval (ARI) shows that the design discharges from the two methods are consistent.

Design flood discharges, flood levels, flood depths, flood velocities and flood hazards for design rainfall events ranging from 2 year ARI to 500 year ARI and for the Probable Maximum Flood (PMF) were predicted using the validated XP-RAFTS and MIKE FLOOD models. In addition, sensitivity analysis on predicted 100 year ARI flood behaviour was undertaken and used to assess the impacts of changes to adopted design discharges (±30%), hydraulic roughness (±30%) and hydraulic structure blockage (50%). Potential impacts of climate change (2ºC, 3ºC and 4ºC temperature increase by 2050, 2070 and 2100 respectively) on 100, 200 and 500 year ARI events were assessed.

The study results show that;

- Design discharges and peak flood levels at all reporting locations increase with ARI up to the 500 year ARI. The 500 year ARI design discharges are 4 to 8 times larger than the 2 year ARI discharges and the 500 year ARI flood levels are 0.2m to 0.67m higher than the 2 year ARI at all reporting locations, with the exception of the Ganzer-Morris Road reporting location, where the 500 year ARI design flood level is up to 1.75m higher than the 2 year ARI design flood level; and
- The crossing at Roderick Drive has low flood immunity, overtopping in a 10 year ARI design event. The Boundary Street crossing has flood immunity up to and including the 500 year ARI event.

- Ultimate-development conditions design discharges and peak flood levels are unchanged at the Roderick Drive reporting location, while ultimate-development conditions design discharges at the Boundary Street reporting location are 25% to 51% larger than for the corresponding existing-development conditions. This is due to the extensive urbanisation of the catchment upstream of this location. As a result, peak flood levels are up to 0.14m higher than for the corresponding existing-development conditions; and
- Ultimate-development conditions design discharges at the remaining reporting locations are up to 21% larger than for the corresponding existing-development conditions. As a result, ultimate-development conditions peak flood levels are up to 0.12m higher than for the corresponding existing-development conditions.

Sensitivity analysis results for the 100 year ARI design event indicate the following:

- A 30% increase and decrease in design discharges results in peak flood levels increasing and decreasing by up to 0.42m and 0.6m respectively;
- A 30% increase and decrease in roughness results in peak flood levels increasing and decreasing by up to 0.2m; and
- A 50% blockage of structures results in peak flood levels increasing by up to 0.15m and 0.2m at the Roderick Drive and Boundary Street reporting locations respectively, while remaining unchanged at the remaining reporting locations.

Climate change scenario results indicate the following:

- For all of the climate change scenarios, peak discharges increase at the reporting locations; and
- Peak flood levels for the 100 year, 200 year and 500 year ARI events increase by up to 0.36m, 0.17m and 0.13m respectively at the reporting locations.

The study limitations and recommendations on how model predictions could be improved are also presented.

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# 1 INTRODUCTION

## 1.1 BACKGROUND

Toowoomba Regional Council (TRC) is a large local government area located in the Darling Downs part of Queensland, Australia. TRC comprises an area of nearly 13,000 $km^2$ with a population of approximately 172,000 people in 33 towns. In 2009 TRC commenced the Toowoomba Regional Planning Project (RPP) to develop one integrated planning scheme policy covering the entire Council area. Later that year TRC commissioned Water Technology Pty Ltd to collate and review the existing flood data in the region and provide advice on the applicability of the data for use in the Planning Scheme. One of the findings from the study was that only a small portion of the Council area is covered by high/medium quality flood mapping.

In 2012 the State Government approved Council adopting the Toowoomba Regional Planning Scheme with a set of conditions to be met to ensure the scheme was compliant with nominated State Planning Policies. To meet the conditions established by the State Government, a scoping study was completed by Council to identify the information required to meet the specified conditions. The study highlighted the need to investigate the flood behaviour and flood risk in several towns in the region.

WRM in association with DHI was commissioned by TRC to undertake a 2D flood study for the township of Cotswold Hills. The 2D flood study which includes both historical and design event modelling will provide Council with information needed for land development control, infrastructure development and management, emergency planning, and emergency response in the study area.

This report describes the methodology, available data, and development of hydrologic and hydraulic models for historical and design event simulations for Cotswold Hills. The report ends with concluding remarks and recommendations to further improve the model accuracy.

## 1.2 SCOPE OF PROJECT

The primary objective of this project was to define the nature and extent of flood behaviour in the Cotswold Hills study area to enable TRC to:

- "*Develop a Flood Risk Management Study and plan to address the flood hazards identified in the flood studies;* and
- *Amend the Toowoomba Regional Planning Scheme to appropriately reflect the flood requirements of State Planning Policy 1/03 and the recommendations of the Queensland Commission of Inquiry*" (TRC, 2013).

The project was divided into a number of phases. The scope of each phase is briefly outlined below.

Information Review and Project Start-Up

- Completion of project briefing;
- Development of stakeholder consultation strategy;
- Site visit; and
- Collection and review of available data.

MIKE FLOOD Model Development

- Development of a coupled 1D/2D MIKE FLOOD model; and
- Adjusting parameters to ensure model stability.

Model Validation

- Adjustment of flows from XP-RAFTS until areas of known flooding are reproduced by the MIKE FLOOD model (a joint calibration approach was used).

Deliverables

- Report detailing methodology and validation results including A3 flood maps for the validation event; and
- Handover of model setup and result files for the validation event.

## 1.3 STUDY AREA

Cotswold Hills is a small town located approximately 10 km north-west of Toowoomba. The Cotswold Hills catchments drain into Gowrie Creek. The primary flood risk to the town is from overland flow and local drainage issues in the catchment.

The focus of this study is local flooding in the tributary catchments of Gowrie Creek draining the Cotswold township. Gowrie Creek flooding is not explicitly modelled in this study.

Land use within the study area is primarily rural. Within the local area of the Cotswold Hills township the land use is predominantly residential. The study area is shown in Figure 1-1.

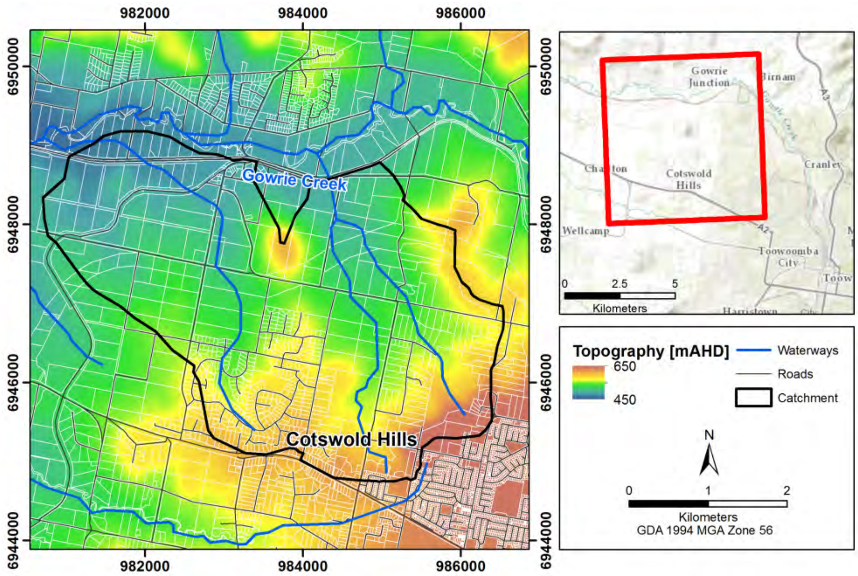


Figure 1-1 Study Area

# 2 AVAILABLE DATA

## 2.1 TOPOGRAPHIC DATA

Tiles of 1 m LiDAR-derived gridded topographic data were provided by TRC. The 1 m tiles were merged to create a seamless 1 m Digital Elevation Model (DEM) of the study area, see Figure 2-1.

## 2.2 GIS LAYERS

The available GIS layers provided by TRC included:

- Aerial photography;
- Cadastral data;
- Road and rail network;
- Structures with a likely hydraulic impact (not including structure geometry, dimensions or levels);
- Land use data;
- Future planning scheme; and
- Location of emergency services.

## 2.3 SITE VISIT

A site visit was undertaken on 4th September, 2013. The purpose of the site visit was to allow the project team to identify key drainage features within the catchment, survey structures with potential significant hydraulic impact and gain a general feel for the floodplain. All information collected during the site visit including photos, structure geometry data and a GIS layer showing the location of the structures was delivered as part of the study.

No topographic survey of drainage structures or critical road or other threshold levels in the catchment was carried out during this project. Drainage structure information used in the study for model development is based on field measurements taken during the site visits.

## 2.4 HISTORICAL FLOOD INFORMATION

Historical pluviograph and daily rainfall data were available from TRC, Bureau of Meteorology (BOM) and the University of Southern Queensland (USQ) for a number of rainfall stations in the vicinity of the Cotswold Hills catchments. Rainfall data available for the January 2011 validation event is described in Section 3.2.2.

Available recorded historical flood level information was supplied by TRC. The available historical flood level information was limited to the January 2011 flood event. All available validation data is summarised in Table 2.1 and shown in Figure 2-1. Please note that TRC has collected flood data for this study from a variety of sources including debris marks, flood marks visible and accessible at the time of survey after the January 2011 flood, eyewitness accounts, community consultation, etc. It is possible that some the flood data available to TRC may not be accurate or complete.

Information used is the best information available at this time for the purposes of this study. Marks observed and other anecdotal information obtained after flood events have been obtained from a range of sources and have varying degrees of uncertainty.

**Table 2.1 Historical Flood Event Data, January 2011 Event**

| Location | Flood Reference |
|---|---|
| Boundary Street (East flood point) | Known flooding (no recorded depth or level) |
| Boundary Street (West flood point) | Known flooding (no recorded depth or level) |

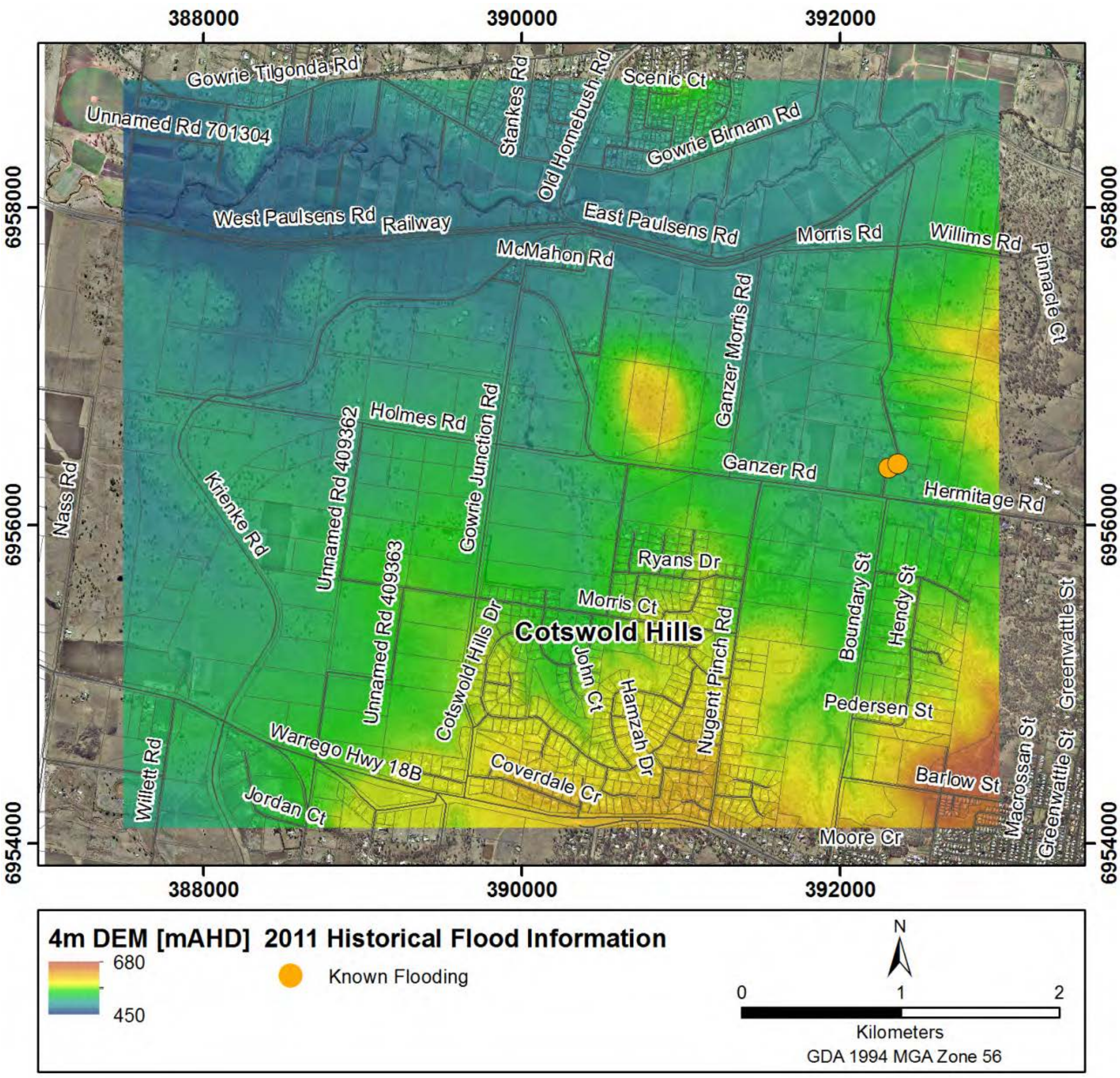


**Figure 2-1 LiDAR-Derived DEM Extent and Available Historical Flood Information**

# 3 HYDROLOGIC MODEL DEVELOPMENT AND VALIDATION

## 3.1 OVERVIEW

Flood discharges within the Cotswold Hills catchments were estimated using the XP-RAFTS runoff-routing model (XP Software, 2009) for existing and ultimate development conditions. The Cotswold Hills catchments and the configuration of the XP-RAFTS model are shown in Figure 3-1.

- The existing-development conditions scenario was based on the Toowoomba Regional Council Zones and Precincts (Version 2) plan and the latest aerial photographs; and
- The ultimate-development conditions scenario assumed that the catchment was fully-developed in accordance with the Toowoomba Regional Council Zones and Precincts plan (Version 2).

The existing-development conditions scenario model was initially validated against the Rational Method estimates at 6 locations within the catchments. The routing parameters of the model were then ‘fine-tuned’ via a joint calibration of the hydrologic and hydraulic models to recorded water levels for the January 2011 event. This has ensured that both XP-RAFTS and MIKE FLOOD models produce consistent discharge hydrographs along the main waterways.

The following section presents the methodology and results of the hydrological model validation for the Cotswold Hills catchments.

## 3.2 MODEL VALIDATION DATA

### 3.2.1 Adopted Validation Event

The largest recent flood event (January 2011) at Cotswold Hills was selected for hydrologic and hydraulic model validation due to availability of rainfall and flood data.

### 3.2.2 Rainfall Data

Figure 3-1 shows the Cotswold Hills catchments and the locations of the two rainfall stations with data suitable for model validation. The Toowoomba Airport Automatic Weather Station (AWS) records rainfall continuously whereas the Moyola station records rainfall on a daily basis. Table 3.1 shows the daily rainfall data recorded at these stations for the January 2011 event.

**Table 3.1 Cotswold Pluviograph and Daily Rainfall Data, 10th and 11th January 2011**

| Station Name | Rainfall Station Type | Station No. | 24-hour Rainfall to 0900 hours (mm) | |
|---|---|---|---|---|
| | | | 10th Jan | 11th Jan |
| Toowoomba Airport AWS | Pluviograph | 041529 | 83.6 | 108.0 |
| Moyola | Daily | 041369 | 55.6 | 82.4 |

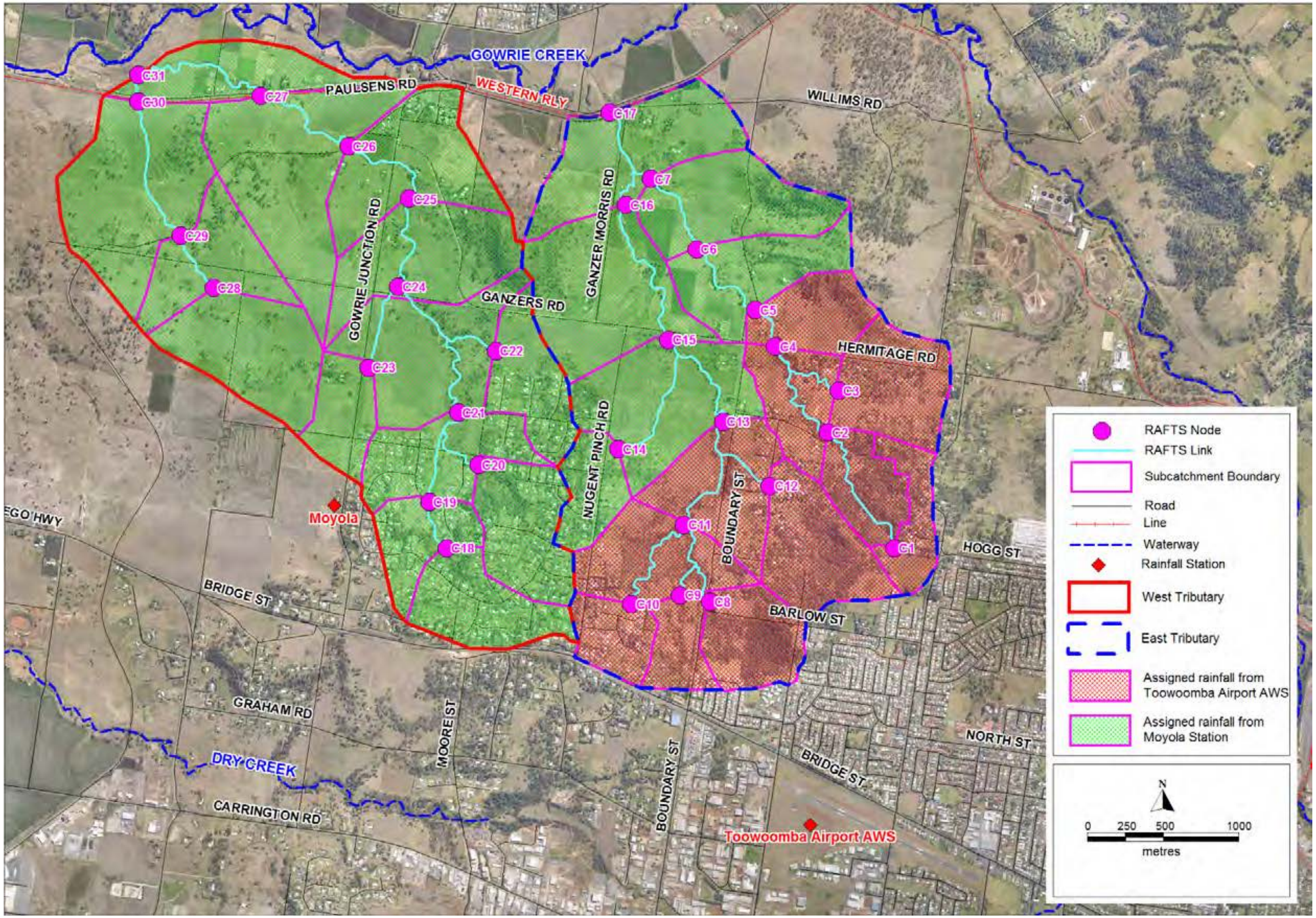


Figure 3-1 XP-RAFTS Model Configuration and Sub-catchment Rainfall Assignment

Figure 3-2 shows the recorded cumulative rainfall at the Toowoomba Airport AWS for the January 2011 flood event. The Moyola data was disaggregated across the days using the Toowoomba AWS temporal pattern.

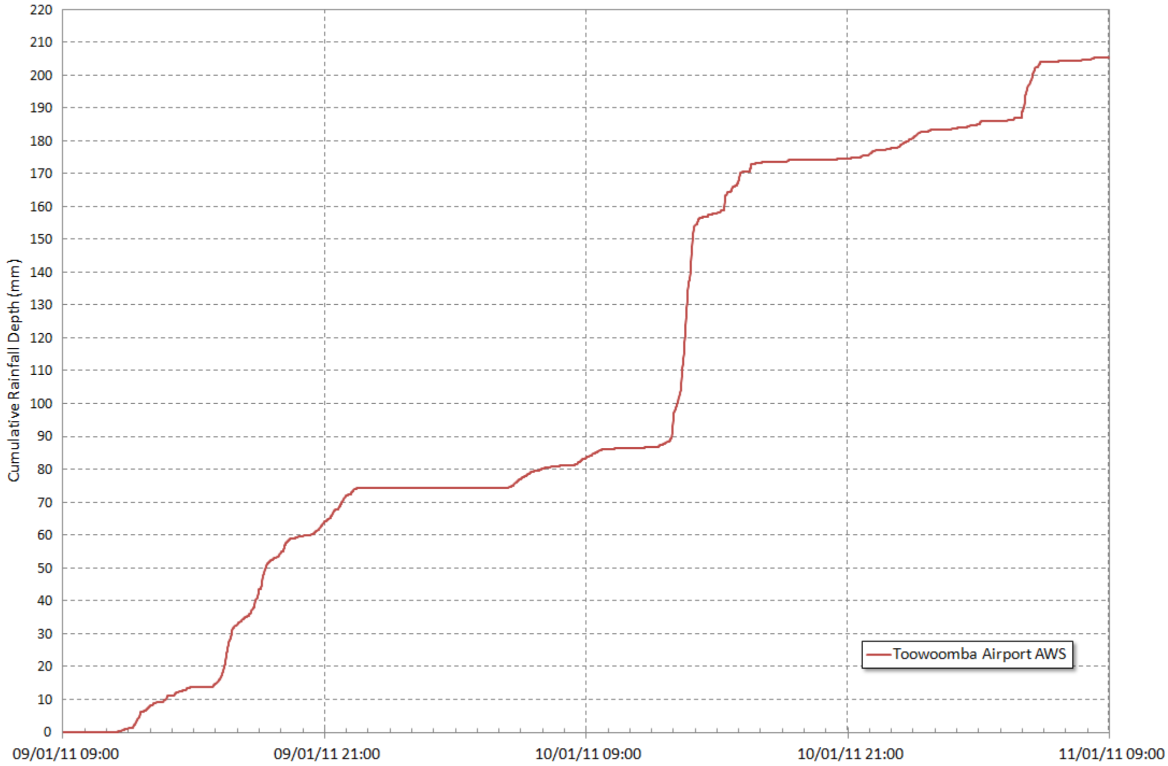


Figure 3-2 Cumulative Rainfall at Toowoomba AWS, January 2011 Event

### 3.2.3 <u>Streamflow Data</u>

There is no streamflow data available within the catchment.

### 3.2.4 <u>Existing and Ultimate Conditions Land use</u>

Figure 3-3 shows the current land uses within the Cotswold Hills catchments. Figure 3-4 shows the ultimate-development conditions land uses within the Cotswold Hills catchments.

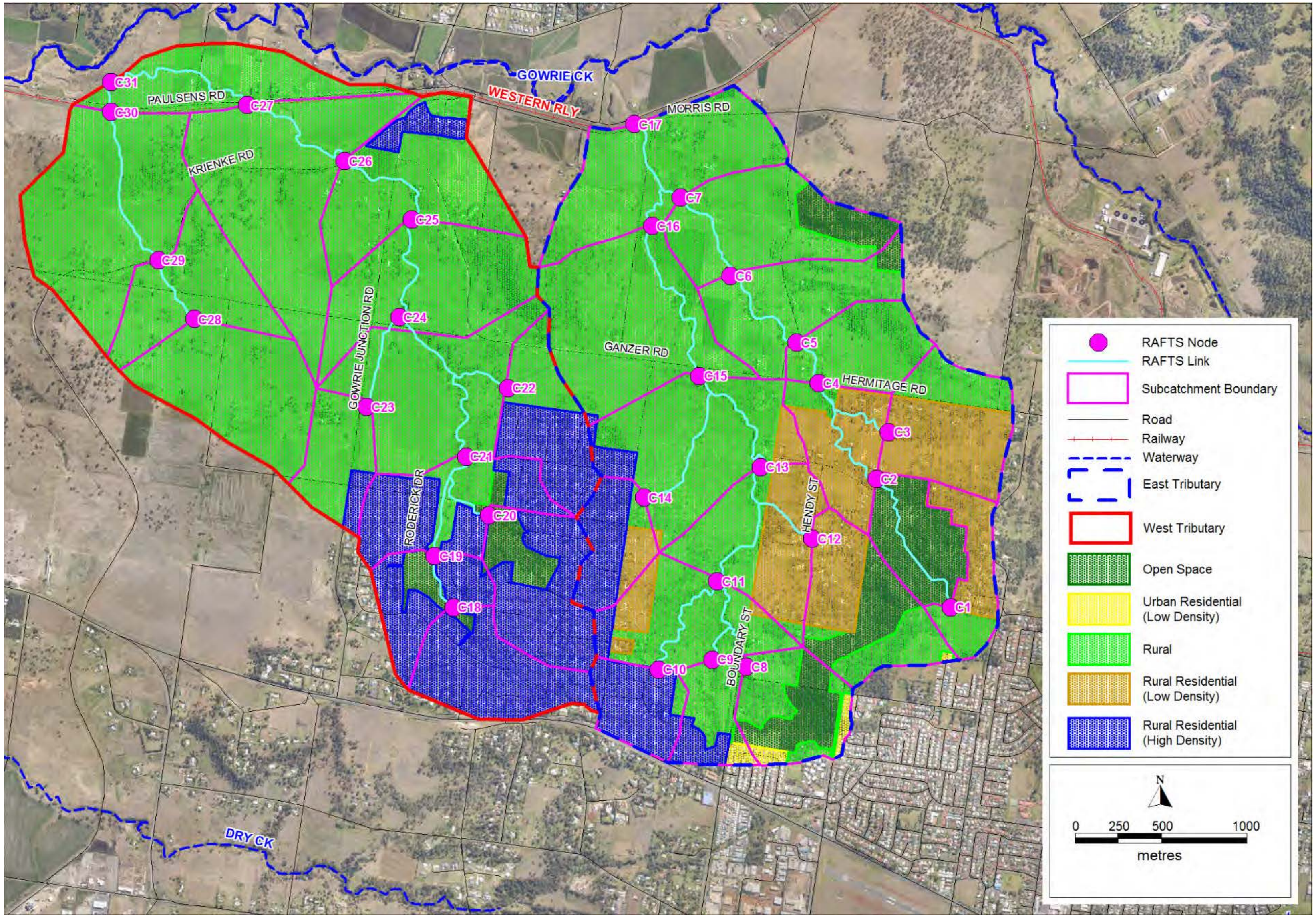


Figure 3-3 Cotswold Hills Land Use – Existing Conditions

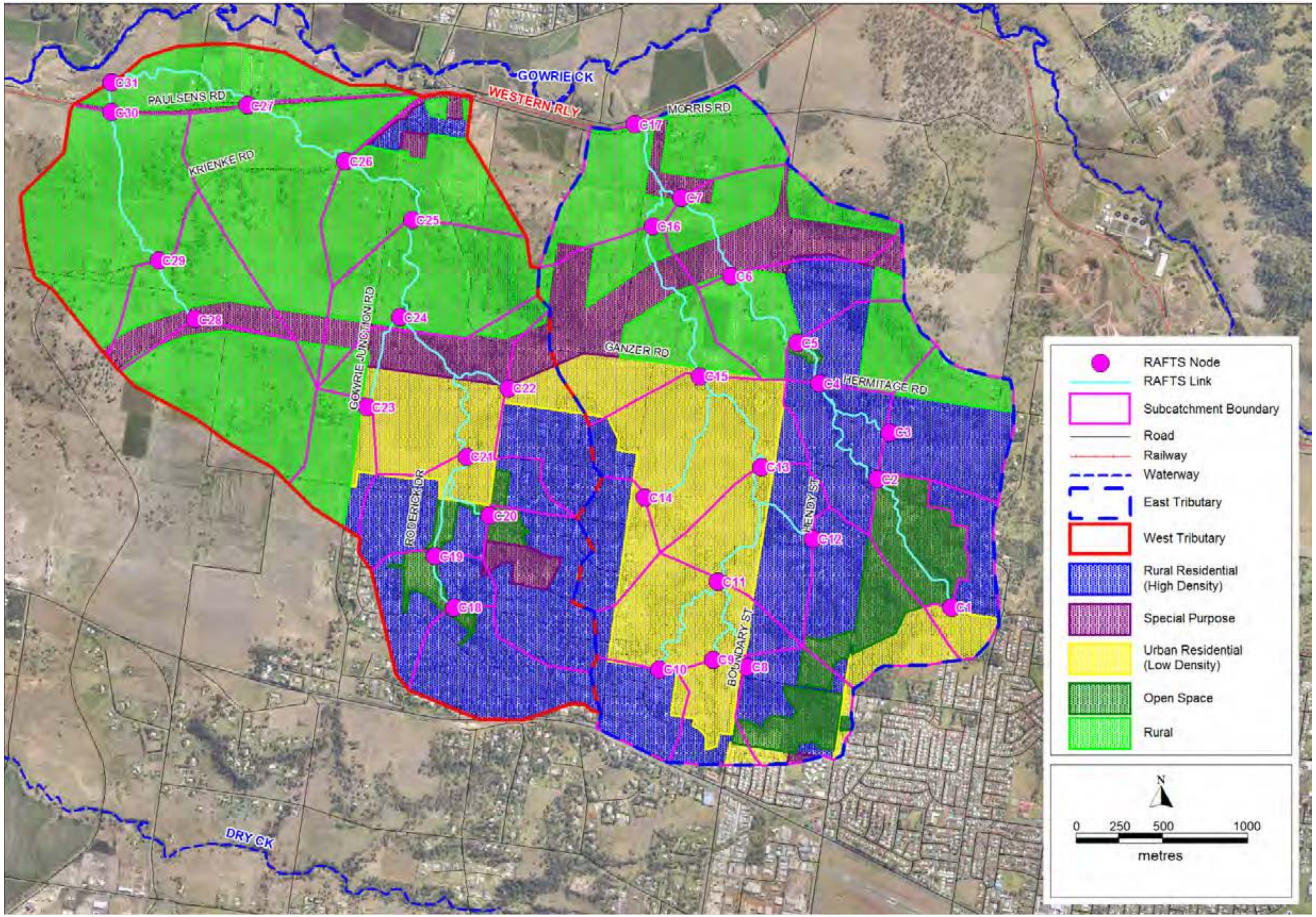


Figure 3-4 Cotswold Hills Land Use – Ultimate Development Conditions

## 3.3 HYDROLOGIC MODEL CONFIGURATION

### 3.3.1 General

Figure 3-3 shows the existing-conditions Cotswold Hills XP-RAFTS model configuration, consisting of 31 Sub-catchments areas including:

- 17 East Tributary sub-catchments totalling 792 ha in area (C1 to C17); and
- 14 West Tributary sub-catchments totalling 803 ha in area (C18 to C31).

XP-RAFTS sub-catchment boundaries were delineated using the available LiDAR data, land use planning information and the available road and stormwater drainage network.

### 3.3.2 RAFTS Model Parameters

Table 3.2 and Table 3.3 show the adopted XP-RAFTS parameters for each land use in the existing and ultimate conditions Cotswold Hills XP-RAFTS model. Table 3.2 also shows the adopted fraction impervious for each land use based on the Queensland Urban Drainage Manual (QUDM) recommendations. The XP-RAFTS model percentage impervious values and catchment PERN 'n' values, which represent the average sub-catchment roughness, were used as a calibration parameter.

Table 3.4 shows the adopted sub-catchment parameters for the existing and ultimate development conditions Cotswold Hills XP-RAFTS models. Where there was more than one land use in each sub-catchment, the percentage impervious and PERN 'n' was factored in proportion to catchment area.

A global 'Bx' factor of 1.0 was adopted. The adopted channel routing parameters were assigned based on the physical channel characteristics including channel length and slope. A channel velocity was assigned to each link based on the MIKE FLOOD model results.

**Table 3.2 Adopted Land Use Parameters, Cotswold Hills XP-RAFTS Model**

| Land Usage | Typical Lot Size (ha) | Fraction Impervious from QUDM ($f_i$) | Adopted RAFTS Parameters | |
|---|---|---|---|---|
| | | | Percent Impervious (%) | Catchment PERN 'n' |
| Commercial/Industrial | Varies | 0.90 | 70 | 0.020 |
| Special Purpose | Varies | 0.75 | 55 | 0.030 |
| Urban Residential | 0.04 to 0.2 | 0.70 | 50 | 0.035 |
| Rural Residential (High Density) | 0.2 to 1.0 | 0.40 | 30 | 0.040 |
| Rural Residential (Low Density) | 1.0 to 10 | 0.20 | 15 | 0.045 |
| Rural | >10 | 0.05 | 5 | 0.060 |
| Open Space | Varies | 0.00 | 0 | 0.070 |

Table 3.3 Initial and Continuing Losses, Cotswold Hills XP-RAFTS Model

| Percentage Impervious | 2 - 5 year ARI | | 10 year ARI | | 20 - 100 year ARI | | > 100 year ARI | |
|---|---|---|---|---|---|---|---|---|
| (%) | IL (mm) | CI (mm) | IL (mm) | CI (mm) | IL (mm) | CI (mm) | IL (mm) | CI (mm) |
| < 25 | 20 | 2.5 | 15 | 2.5 | 15 | 2.5 | 0 | 2.5 |
| 25 to 40 | 20 | 1.5 | 10 | 1.5 | 10 | 1.5 | 0 | 1.5 |
| > 40 | 20 | 1 | 10 | 1 | 5 | 1 | 0 | 1 |

Table 3.4 Adopted XP-RAFTS Sub-Catchment Parameters, Existing and Ultimate Conditions

| | Sub-catchment | Catchment Area (ha) | Existing Conditions | | | Ultimate Conditions | | |
|---|---|---|---|---|---|---|---|---|
| | | | Catchment Slope (%) | Percent Imp. (%) | 'PERN' 'n' | Catchment Slope (%) | Percent Imp. (%) | 'PERN' 'n' |
| East Tributary Catchment | C1 | 25.15 | 11.9 | 11.7 | 0.050 | 11.9 | 36.1 | 0.039 |
| | C2 | 31.47 | 15.2 | 0.0 | 0.070 | 15.2 | 0.1 | 0.070 |
| | C3 | 50.02 | 8.1 | 12.0 | 0.049 | 8.1 | 22.9 | 0.046 |
| | C4 | 35.23 | 5.5 | 13.4 | 0.047 | 5.5 | 30.0 | 0.040 |
| | C5 | 33.34 | 9.6 | 5.0 | 0.060 | 9.6 | 16.4 | 0.051 |
| | C6 | 50.44 | 6.4 | 4.7 | 0.061 | 6.4 | 17.7 | 0.051 |
| | C7 | 61.02 | 7.1 | 4.0 | 0.062 | 7.1 | 31.7 | 0.044 |
| | C8 | 38.89 | 13.7 | 7.4 | 0.062 | 13.7 | 18.7 | 0.053 |
| | C9 | 22.67 | 7.8 | 15.3 | 0.053 | 7.8 | 42.5 | 0.038 |
| | C10 | 24.81 | 8.7 | 28.2 | 0.041 | 8.7 | 31.4 | 0.040 |
| | C11 | 50.8 | 6.9 | 8.1 | 0.056 | 6.9 | 46.6 | 0.036 |
| | C12 | 53.69 | 14.4 | 7.5 | 0.057 | 14.4 | 25.5 | 0.048 |
| | C13 | 54.22 | 6.9 | 10.2 | 0.052 | 6.9 | 39.2 | 0.038 |
| | C14 | 26.93 | 6.0 | 24.3 | 0.043 | 6.0 | 36.3 | 0.038 |
| | C15 | 76.55 | 5.9 | 7.1 | 0.058 | 5.9 | 47.1 | 0.036 |
| | C16 | 85.25 | 8.3 | 5.1 | 0.060 | 8.3 | 33.7 | 0.043 |
| | C17 | 71.16 | 6.7 | 5.0 | 0.060 | 6.7 | 12.2 | 0.056 |
| West Tributary Catchment | C18 | 44.84 | 6.5 | 29.2 | 0.041 | 6.5 | 29.2 | 0.041 |
| | C19 | 30.54 | 8.7 | 23.8 | 0.046 | 8.7 | 24.1 | 0.046 |
| | C20 | 49.3 | 5.1 | 22.7 | 0.047 | 5.1 | 31.2 | 0.041 |
| | C21 | 49.38 | 7.4 | 22.1 | 0.047 | 7.4 | 29.8 | 0.043 |
| | C22 | 39.25 | 7.5 | 20.7 | 0.047 | 7.5 | 38.0 | 0.037 |
| | C23 | 29.01 | 4.8 | 8.3 | 0.057 | 4.8 | 15.6 | 0.053 |
| | C24 | 73.27 | 5.2 | 5.0 | 0.060 | 5.2 | 45.5 | 0.037 |
| | C25 | 68.26 | 9.1 | 5.0 | 0.060 | 9.1 | 8.6 | 0.058 |
| | C26 | 67.92 | 5.6 | 7.6 | 0.058 | 5.6 | 11.7 | 0.055 |
| | C27 | 100.90 | 4.3 | 5.0 | 0.060 | 4.3 | 9.0 | 0.058 |
| | C28 | 63.29 | 4.4 | 5.0 | 0.060 | 4.4 | 7.6 | 0.058 |
| | C29 | 46.82 | 4.2 | 5.0 | 0.060 | 4.2 | 21.3 | 0.050 |
| | C30 | 94.85 | 5.3 | 5.0 | 0.060 | 5.3 | 5.3 | 0.060 |
| | C31 | 45.70 | 1.7 | 5.0 | 0.060 | 1.7 | 8.6 | 0.058 |

## 3.4 XP RAFTS MODEL VALIDATION AGAINST RATIONAL METHOD

### 3.4.1 General

The existing-conditions Cotswold Hills XP-RAFTS model was validated against the Rational Method at 6 locations using the methodology recommended in QUDM (2013) for rural catchments. The model was validated at the following locations (see Figure 3.3 for locations):

- Sub-catchment C7;
- Sub-catchment C16;
- Sub-catchment C17;
- Sub-catchment C18;
- Sub-catchment C30; and
- Sub-catchment C31.

### 3.4.2 Design Rainfalls and Temporal Patterns

Separate design rainfalls were derived for the East Tributary catchment and West Tributary catchment in accordance with Australian Rainfall and Runoff (Pilgrim, 1998). The adopted design rainfalls are presented in Section 6.

To accommodate the separate design rainfalls for the East and West Tributary catchments, the validated Cotswold Hills XP-RAFTS model was partitioned into two separate models. The Cotswold East model consisted of sub-catchments C1 – C17 while the Cotswold West model consisted of sub-catchments C18 – C31.

### 3.4.3 Rational Method Estimates

Table 3.6 shows the Rational Method estimates of 10 year and 100 year ARI discharges at the above 6 locations. The Rational Method discharges were calculated assuming the following:

- A catchment-weighted fraction impervious and catchment slope based on existing conditions land uses (see Table 3.4);
- A catchment-weighted $C_{10}$ value was assigned to each catchment based on the values recommended in QUDM (2013) for the calculated fraction impervious. The adopted 10 year ARI 1-hour duration rainfall intensities for East Tributary catchment and West Tributary catchment are 47.4mm/hr and 47.7mm/hr respectively;
- A $C_{10}$ value of 0.49 was assigned to predominantly rural catchments (with a catchment weighted fraction impervious of 20% or less);
- The stream velocity was calculated as follows:
  - For rural catchments, the stream velocity was selected based on the MIKE FLOOD model results; and
  - For urban catchments, a standard inlet time of 5mins, pipe velocity of 3.0m/s and open channel flow velocity based on the MIKE FLOOD model results.

### 3.4.4 Comparison of Rational Method and XP-RAFTS Results

Table 3.6 compares the 10 year and 100 year ARI peak discharges estimated using the Rational Method with XP-RAFTS model predicted peak discharges at each of the 6 locations. The comparison shows that the XP-RAFTS peak discharges for sub-catchments C7, C16, C17, C18, C30 and C31 match well with the Rational Method discharges and that the differences are less than 20% for all design events up to and including the 100 year ARI event.

Table 3.5 Rational Method 10 Year and 100 Year ARI Design Discharges

| Parameter | Sub-catchment ID | | | | | |
|---|---|---|---|---|---|---|
| | C7 | C16 | C17 | C18 | C30 | C31 |
| Catchment Area (ha) | 287 | 434 | 792 | 44.8 | 205 | 803 |
| Travel Time | | | | | | |
| Standard Inlet Time (mins) | - | - | - | 5 | - | 5 |
| Stream Length (km) | 3.8 | 4.6 | 5.0 | 0.9 | 2.6 | 6.5 |
| Stream Velocity (m/s)[a] | 2.5 | 2.5 | 2.5 | 3.0 | 0.9 | 2.5 |
| **Time of Concentration (mins)** | **25.6** | **30.5** | **33.3** | **10.0** | **48.1** | **48.3** |
| Rainfall Intensity | | | | | | |
| 10 year ARI event (mm/hr) | 79.7 | 71.7 | 74.7 | 124.0 | 57.2 | 57.0 |
| 100 year ARI event (mm/hr) | 118.9 | 106.4 | 133.0 | 189.0 | 84.4 | 84.2 |
| Coefficient of Discharge | | | | | | |
| Fraction Impervious (%)[b] | 9 | 12 | 10 | 39 | 5 | 13 |
| $C_{10}$ (no units) | 0.49 | 0.49 | 0.49 | 0.60 | 0.49 | 0.49 |
| $C_{100}$ (no units) | 0.59 | 0.59 | 0.59 | 0.72 | 0.59 | 0.59 |
| **10 Year ARI Discharge ($m^3/s$)** | **31.1** | **42.3** | **74.7** | **9.3** | **15.9** | **62.3** |
| **100 year ARI Discharge ($m^3/s$)** | **55.7** | **75.4** | **133.0** | **16.9** | **28.3** | **110.5** |

[a] Based on MIKE FLOOD 100 year ARI results
[b] Different to XP-RAFTS Percent Impervious. These values are based on Table 3.2

Table 3.6 Comparison of Rational Method and XP-RAFTS Model Peak Discharges

| Inflow Location | 10 Year ARI Peak Discharge ($m^3/s$) | | 100 Year ARI Peak Discharge ($m^3/s$) | |
|---|---|---|---|---|
| | RM | XP-RAFTS | RM | XP-RAFTS |
| C7 | 31.1 | 32.0 | 55.7 | 57.4 |
| C16 | 42.3 | 46.1 | 75.4 | 80.1 |
| C17 | 74.7 | 81.4 | 133.0 | 141.7 |
| C18 | 9.3 | 11.6 | 16.9 | 19.8 |
| C30 | 15.9 | 15.3 | 28.3 | 28.8 |
| C31 | 62.3 | 64.5 | 110.5 | 108.4 |

## 3.5 XP-RAFTS MODEL JANUARY 2011 RESULTS

### 3.5.1 Assignment of Total Rainfalls and Temporal Patterns

Total rainfalls from pluviograph and daily stations were assigned to each sub-catchment based on the nearest rainfall station. The nearest available pluviograph temporal pattern (at Toowoomba Airport AWS) was applied to the Moyola rainfall station as well. This approach ensures all the available data are used. Figure 3-1 shows the locations of the available daily and pluviograph rainfall stations within and adjacent to the catchment. Sub-catchments assigned rainfall from the Toowoomba AWS are shaded red and those assigned rainfall from Moyola are shaded green. Table 3.7 shows a summary of the adopted rainfall assignment.

Table 3.7 Adopted Rainfall Assignment

| Rainfall source | Sub-Catchments Assigned |
|---|---|
| Toowoomba Airport AWS | C1 –C 5, C8 - C13 |
| Moyola | C6,C7, C14 - C31 |

### 3.5.2 Initial and Continuing Losses

Each sub-catchment was assigned losses based on the catchment percentage impervious. The same initial and continuing losses were adopted for the validation event as were adopted for the 20 to 100 year ARI events shown in Table 3.3. These loss rates are based on the recommended design loss rates for Eastern Queensland (Pilgrim, 1998). Note that the flood peaks are not overly sensitive to lower initial losses, which would have been expected given the very wet conditions experienced prior to this event. The adopted initial loss removes the pre-storm and pre-burst rainfall only.

### 3.5.3 Peak Discharges

Table 3.8 shows a summary of the total upstream catchment (external) and local residual catchment (internal) peak discharges at each hydrodynamic (MIKE FLOOD) model inflow location estimated using the XP-RAFTS model.

Table 3.8 Peak Discharges at Inflow Hydrograph Locations to Hydrodynamic Model

| | Sub-catchment ID | Peak Discharge ($m^3/s$) | |
|---|---|---|---|
| | | Local (Internal) | Total (External) |
| East Tributary Catchment | C1 | 6.0 | 6.0 |
| | C2 | 6.4 | 12.1 |
| | C3 | 10.9 | 10.9 |
| | C4 | 7.6 | 30.1 |
| | C5 | 6.9 | 37.1 |
| | C6 | 6.0 | 42.5 |
| | C7 | 7.1 | 48.4 |
| | C8 | 8.5 | 8.4 |
| | C9 | 5.3 | 5.3 |
| | C10 | 6.4 | 6.4 |
| | C11 | 10.4 | 29.6 |
| | C12 | 11.6 | 11.6 |
| | C13 | 11.4 | 51.7 |
| | C14 | 4.3 | 4.3 |
| | C15 | 9.2 | 62.9 |
| | C16 | 10.4 | 68.6 |
| | C17 | 8.4 | 120.4 |
| West Tributary Catchment | C18 | 7.6 | 7.6 |
| | C19 | 5.1 | 11.9 |
| | C20 | 7.2 | 7.2 |
| | C21 | 7.7 | 25.8 |
| | C22 | 6.1 | 6.1 |
| | C23 | 3.8 | 3.8 |
| | C24 | 7.8 | 40.7 |
| | C25 | 8.7 | 46.2 |
| | C26 | 8.2 | 50.8 |
| | C27 | 9.3 | 57.0 |
| | C28 | 6.5 | 6.5 |
| | C29 | 5.0 | 11.4 |
| | C30 | 9.7 | 20.3 |
| | C31 | 3.0 | 71.1 |

# 4 HYDRODYNAMIC MODEL DEVELOPMENT

## 4.1 OVERVIEW

The following section documents the development and validation of the hydrodynamic model, selection of key model parameters and assumptions made. The hydrodynamic model was developed in the MIKE FLOOD Release 2012 (Service Pack 2), which was the most recent version available at the time of the project. MIKE FLOOD is a software program that allows coupling of a MIKE 11 (1D) model and a MIKE 21 (2D) model to run together in parallel. The fundamental principle of MIKE FLOOD is that features smaller than the MIKE 21 grid resolution (e.g. small channels and structures) can be represented in MIKE 11, with linkages (couples) that transfer water levels and discharges between MIKE 11 and MIKE 21 at each time step.

The MIKE FLOOD model schematisation (DHI, 2013) was proposed and agreed with TRC prior to the commencement of model development. The original assessment was to model the main overland flood channels in MIKE 11 and couple these laterally to the out-of-bank floodplain flows. However, once initial model results were analysed, it became apparent that a 2D representation of the overland flow paths would be sufficient as they are wide and their conveyance and geometry is adequately represented by the 2D grid spacing used. In addition, the initial modelling results showed that out-of-bank flooding would not occur for the majority if not all modelled events (this is demonstrated in Figure 4-1 by an example cross-section with the modelled 100 year ARI peak water level based on preliminary 100 year ARI XP-RAFTS flow estimates).

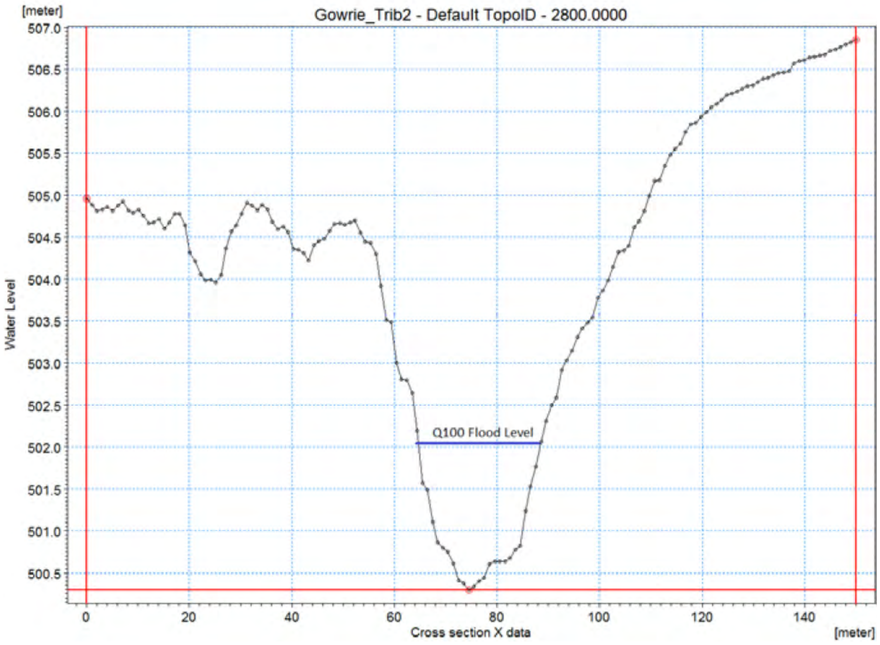


Figure 4-1 Example Cross-Section with Modelled 100 Year ARI Water Level

## 4.2 MIKE 21 MODEL

The 2D model domain for Cotswold Hills extends approximately from Warrego Highway in the south to Gowrie-Tilgonda Road to the north as shown in Figure 4-2. The model domain is approximately 5.5km by 4.7km.

### 4.2.1 Bathymetry

The MIKE 21 model incorporates a detailed elevation model (bathymetry) of the ground surface. The DEM used in this model was created from the 1 m DEM supplied by TRC. The DEM was then resampled to a 4 m grid resolution.

Features of the floodplain likely to influence the flow of floodwaters were included in the 2D model. This included appropriate discretisation of elevated embankments, railway and roads in a form that ensures correct representation of the features both for smaller floods, where the influence on flow behaviour has been observed, and for more extreme events, where the feature may be overwhelmed. Correct and appropriate representation of these features is paramount for the model to correctly extrapolate flood behaviour for extreme events.

The crest levels of major roads and the railway line were incorporated into the model using the following steps:

- Digitise polylines along the crown of roads and railway;
- Create a 10 m buffer around the digitised polylines;
- Extract the elevations within the buffer from the 1 m DEM;
- Convert the extracted raster to points with 1m spacing;
- Interpolate the 1 m points to a 4 m raster by selecting the maximum elevation within each 4 m by 4 m cell; and
- Update the 'base' bathymetry with this raster.

Small flow structures and crossings on secondary flow paths were implemented as they were represented in the source DEM data. These structures were not incorporated as 1D elements, and the 2D bathymetry was not adjusted to reflect any geometry or associated control levels. This implies that most small culverts and structures were assumed to be 100% blocked during a flood, thus producing a conservative estimate of the flood extent.

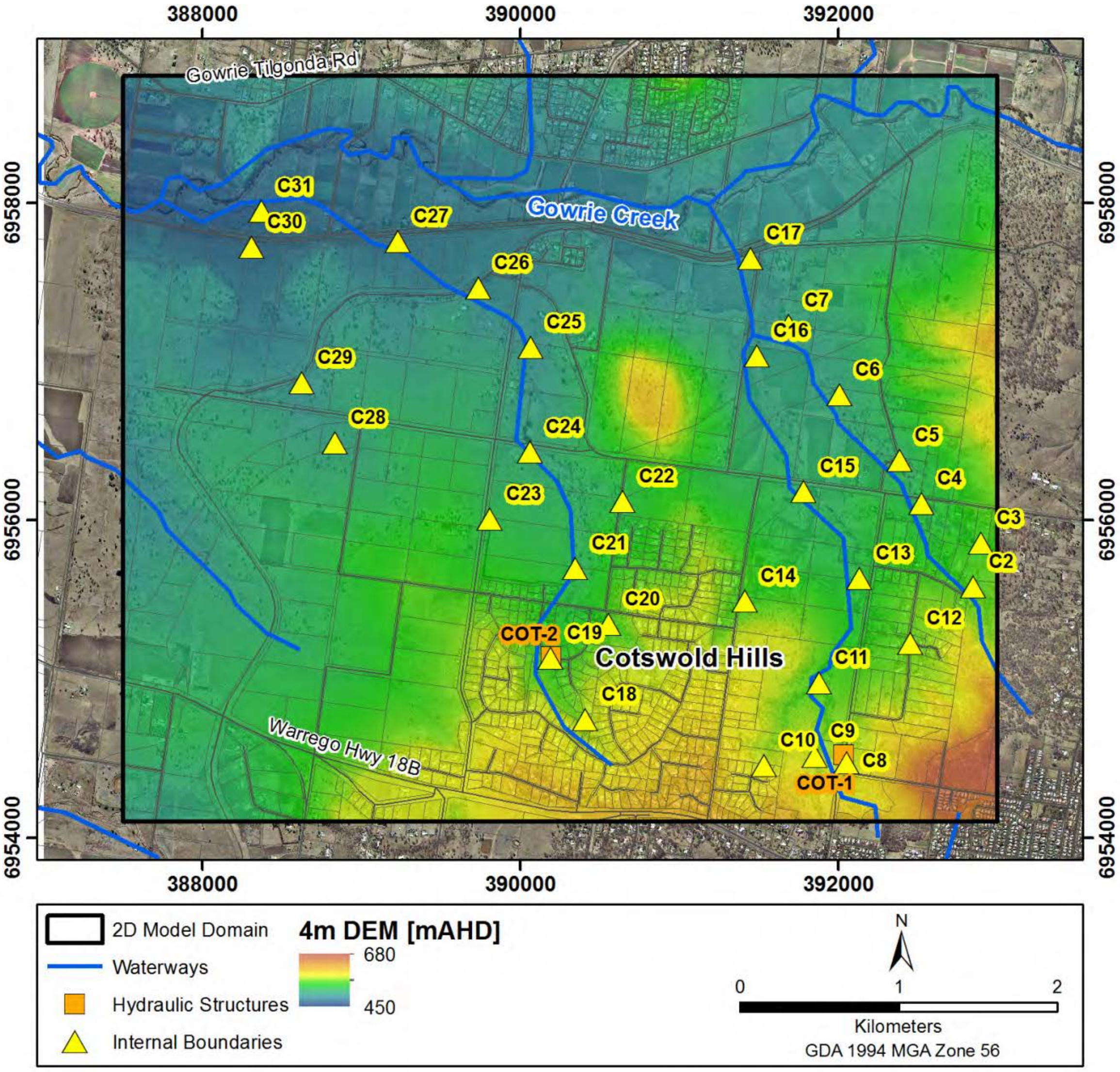


Figure 4-2 MIKE FLOOD Model Setup

### 4.2.2 Hydraulic Roughness

MIKE21 models require the specification of hydraulic roughness to be applied in each cell, either as a constant value or in the form of a map (grid) of roughness values. A spatially distributed roughness map for the model domain was created based on the land uses classes provided by TRC as well as vegetation coverage identified from the aerial photography, also supplied by TRC. Five distinct land use classes were identified within the study area.

The adopted hydraulic roughness values (Manning's 'n') for each class are shown in Table 4.1. These values were based on DHI's previous experience in Queensland, whilst also taking into account the Australian Rainfall and Runoff (ARR) Revision Project's valid Manning's 'n' ranges for different land use types (Smith and Wasko, 2012). It should be noted that the adopted Manning's 'n' value for 'Developed Areas' is slightly lower than the ARR recommended range of roughness values for this land use type. This is due to the coarse delineation of 'Developed Areas' based on land uses classes, resulting in a Manning's 'n' value of 0.083 being applied to buildings as well as some open pervious areas. The spatial distribution of roughness is presented in Figure 4-3.

Table 4.1 Adopted Hydraulic Roughness Values in MIKE FLOOD

| Land Use | Manning's 'n' | Range of Manning's 'n' Values[a] |
|---|---|---|
| Floodplain | 0.04 | 0.03 – 0.05 |
| Roads | 0.025 | 0.02 – 0.03 |
| Developed Areas | 0.083 | 0.1 – 0.2 |
| Waterways | 0.033 | 0.02 – 0.04 |
| Vegetated Waterways | 0.05 | 0.04 – 0.1 |

Notes: [a] (Smith and Wasko, 2012)

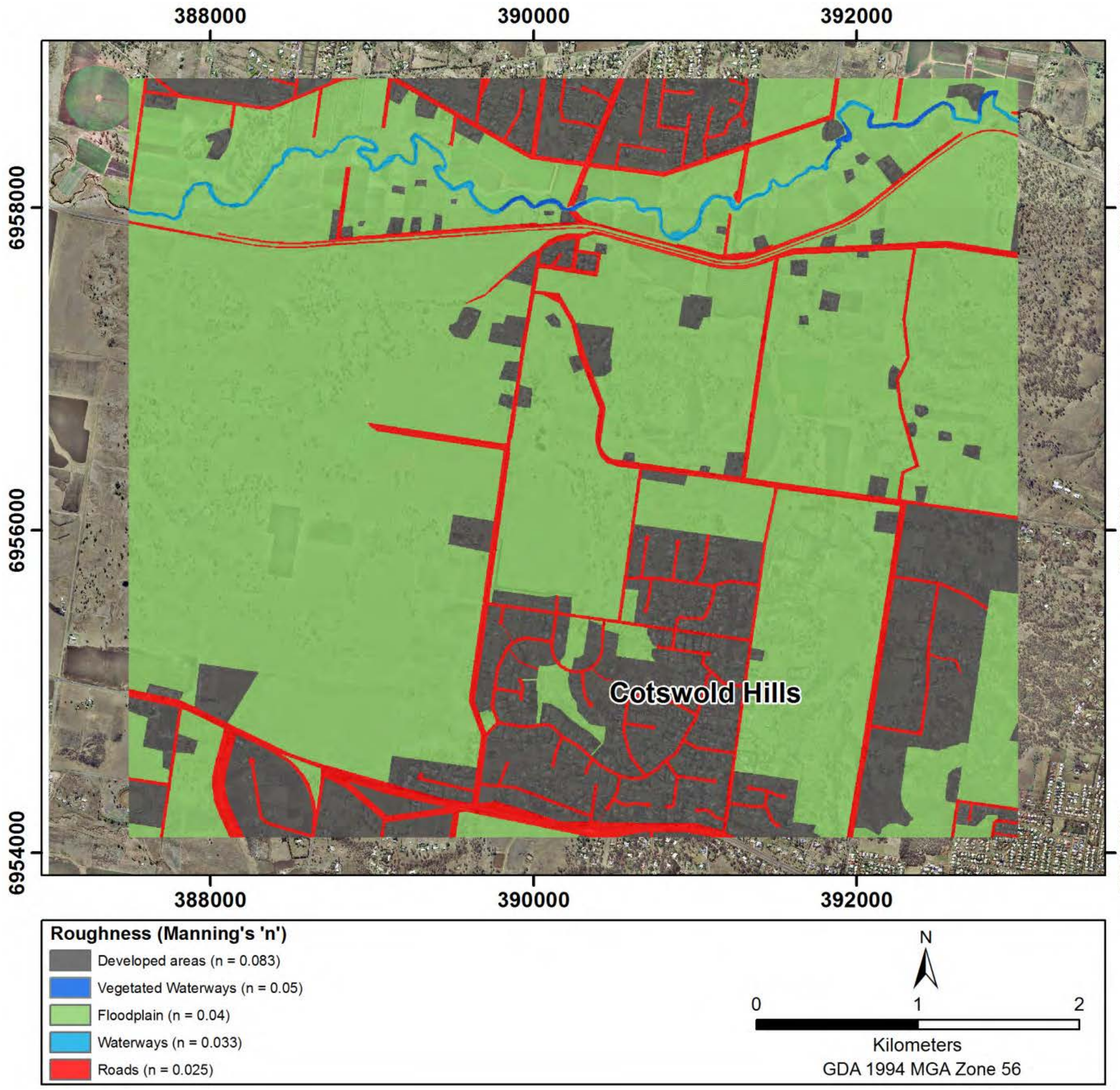


Figure 4-3 Spatial Distribution of Roughness

### 4.2.3 Flooding and Drying Depths

In the MIKE FLOOD Release 2012, there is a new 'Inland Flooding' option available which implements a revised method for phased downscaling of momentum terms in the 2D solution. The new method uses an approach where the transition from full momentum to zero momentum and back into full momentum is achieved using a linear 'Momentum Factor' calculated from the depth of water and the flooding depth and drying depth parameters. This approach ensures a smoother shift from full momentum to less (or zero) momentum and results in much improved mass balances in urban flooding and floodplain applications.

Continuity is fully preserved during the flooding and drying process, as the water depths at the points which are dried out are saved and then reused when the point becomes flooded again. A flooding depth of 0.05 m and a drying depth of 0.02 m were adopted in this study.

### 4.2.4 Eddy Viscosity

Eddy viscosity is used to represent sub-grid scale turbulence to provide the modeller with the opportunity to enhance or retard the natural generation of flow eddies in the solution scheme for the purpose of matching observed flow phenomena. A velocity based eddy viscosity formulation was applied, and this is the recommended approach in floodplain applications.

Values for eddy viscosity can be calculated using a number of empirical formulas related to grid size and time step. Selecting an eddy viscosity value that is too high will result in the modelled flow having a more uniform velocity distribution tending to distribute more of the total flow to the floodplain. Selecting an eddy viscosity value that is too low can result in significant variability in the velocity field, formation of large modelled eddies in areas of no physical manifestation of this hydraulic phenomenon, and contribute to model instability.

In this study, the eddy viscosity was set to 0.5 $m^2/s$, which is consistent with the model resolution and based on DHI's previous experience with selection of secondary model parameters. At a small number of locations associated with 1D/2D couples an eddy viscosity of 5 $m^2/s$ was used to improve model stability.

### 4.2.5 Model Boundaries

Catchment flows from the XP-RAFTS model were applied to the MIKE 21 model at thirty internal boundaries (source points), see Figure 4-2. A Q-h rating was used at the downstream model boundary on Gowrie Creek. The rating curve was derived from a cross-section extracted from the 1m DEM at the location of the MIKE 21 downstream boundary. The cross-section width was set to match the width of the MIKE 21 boundary. An average bed slope of 4 m/km derived from the LIDAR data, and a Manning's 'n' of 0.04 representative of the channel conditions, were used to derive the rating curve. The model boundary was positioned as far downstream of the area of interest as possible to minimise backwater effects on areas of interest to the study from assumptions made at the boundary location.

### 4.2.6 Time Step and Save Step

A 0.4 second time step was used in the Cotswold Hills model based on Courant number considerations. The save step in MIKE 21 was set to 10 minutes.

## 4.3 MIKE 11 MODEL

### 4.3.1 Network and Structures

The MIKE 11 network consists of two branches used to model structures with potential significant hydraulic impact. MIKE 11 was not used to model open overland flow paths as had been originally planned during the project inception, as during model development it was decided a 2D representation was sufficient to represent flow path conveyance and geometry.

The structure locations are shown in Figure 4-2. Structure dimensions for the COT-2 structure were implemented based on the measurements taken during the site visit. Structure dimensions for the COT-1 structure were based on design drawings provided by TRC due to lack of access on site. Invert levels of structures and their waterway length were estimated from the 1 m DEM and aerial photography, respectively. Bridge or culvert headwall railing has not been considered in the structure definition, i.e. it was assumed no blockage of rails occurred during the validation flood event.

### 4.3.2 Cross-Sections

The cross-sections defined at the upstream and downstream ends of each MIKE 11 branch were extracted from the 1 m DEM. Cross-sections upstream and downstream of structures were enlarged if they were smaller than the structure dimensions. This is necessary to ensure a realistic head loss across the structure.

### 4.3.3 Time Step and Save Step

When MIKE 11 and MIKE 21 models are coupled, MIKE 11 is forced to use the same time step as MIKE 21. The MIKE 11 results were saved every 5 minutes.

## 4.4 MIKE FLOOD MODEL

A total of four coupling points were implemented in the MIKE FLOOD model. The structures and couple types are listed in Table 4.2; photographs of the structures taken during the site visit are shown in Appendix A.

Both structures were modelled using the 'Standard' link type, as their waterway length exceeds two MIKE 21 grid cells (8 m). In a 'Standard' link structure submergence and overtopping is modelled in MIKE 11 and MIKE 21, respectively. The two long culverts were modelled as closed cross-sections in MIKE 11. This is to account for the friction effects due to their length.

Table 4.2 Structures Implemented in MIKE FLOOD

| Structure | Link Type | Modelled Structure | Dimensions |
|---|---|---|---|
| COT-1 | Standard | Closed Cross-section | Diameter = 1.8m |
| COT-2 | Standard | Closed Cross-section | 3 x Diameter = 1.22m |

### 4.4.1 Standard/Structure Link Options

The standard/structure link parameters adopted in the MIKE FLOOD model are summarised in Table 4.3. The momentum factor was set to 1 at both links. An exponential smoothing factor of 0.2 was adopted.

Table 4.3 Adopted Standard/Structure Link Parameters

| Parameter | Value/Option |
|---|---|
| Momentum factor | 1 |
| Extrapolation factor | 0 |
| Add/Replace Flow | Replace |
| Depth Adjustment | Yes |
| Exponential Smoothing Factor | 0.2 |

# 5 ASSESSMENT OF MODEL PERFORMANCE

The XP-RAFTS and MIKE FLOOD models were validated for the January 2011 flood event using a joint calibration approach. The fit between modelled and observed flooding is summarised in Table 5.1 and presented in Appendix C.

**Table 5.1 Observed and Modelled Flooding**

| Location | Observed Flooding | Modelled Flooding |
|---|---|---|
| Boundary Street (East flood point) | Flooded | Flooded |
| Boundary Street (West flood point) | Flooded | Flooded |

Both locations known to have been inundated during the January 2011 flood have been reproduced by the model. The predicted head losses at the hydraulic structures are shown in Appendix B, Table B.1.

# 6 DESIGN FLOOD ESTIMATION

## 6.1 ESTIMATION OF DESIGN DISCHARGES

### 6.1.1 Overview

The validated XP-RAFTS model was partitioned into Cotswold East and Cotswold West models as discussed in Section 3.4.2. These were then used to estimate design flood discharges in the Cotswold Hills catchments. The XP-RAFTS model design discharge estimates were compared against Rational Method estimates at 6 sub-catchments for consistency.

The following sections detail the design rainfall data (IFD data, temporal patterns, areal reduction factors, rainfall spatial distribution and design rainfall losses) that have been adopted for the Cotswold Hills catchments. Design flood discharge hydrographs were estimated for a range of storm durations up to 12 hours for the 2, 5, 20, 50, 100, 200 and 500 year Average Recurrence Interval (ARI) and the Probable Maximum Precipitation (PMP) events.

### 6.1.2 Design Rainfalls for Events up to 100 Year ARI

*Rainfall Depth Estimation*

Design rainfall intensities for storms of varying durations (15 minutes to 12 hours) for all ARI events up to and including the 100 year ARI were determined at the centroid of the catchments using BOM's AR&R87 IFDs tool (BOM, 2014). Adopted design rainfall intensities for the two models are provided in Table 6.1 and Table 6.2.

Table 6.1 Cotswold East - Intensity-Frequency-Duration Data (mm/hour)

| Duration | Average Recurrence Interval (years) | | | | | |
|---|---|---|---|---|---|---|
| (Hours) | 2 | 5 | 10 | 20 | 50 | 100 |
| 0.25 | 74.9 | 92.5 | 103.3 | 118.4 | 139.1 | 155.4 |
| 0.5 | 53.4 | 65.1 | 72.2 | 82.4 | 96.1 | 106.9 |
| 1 | 35.6 | 43.1 | 47.7 | 54.2 | 63.1 | 70.0 |
| 1.5 | 27.2 | 33.0 | 36.4 | 41.4 | 48.1 | 53.4 |
| 2 | 22.3 | 27.0 | 29.8 | 33.9 | 39.4 | 43.7 |
| 3 | 16.6 | 20.2 | 22.3 | 25.3 | 29.4 | 32.7 |
| 6 | 10.0 | 12.1 | 13.5 | 15.3 | 17.9 | 19.8 |
| 9 | 7.4 | 9.1 | 10.1 | 11.5 | 13.5 | 15.0 |
| 12 | 6.1 | 7.5 | 8.3 | 9.5 | 11.1 | 12.4 |

Table 6.2 Cotswold West - Intensity-Frequency-Duration Data (mm/hour)

| Duration | Average Recurrence Interval (years) | | | | | |
|---|---|---|---|---|---|---|
| (Hours) | 2 | 5 | 10 | 20 | 50 | 100 |
| 0.25 | 74.5 | 92.2 | 103 | 118.3 | 139.1 | 155.6 |
| 0.5 | 53.1 | 64.9 | 72.1 | 82.3 | 96.2 | 107.0 |
| 1 | 35.3 | 42.8 | 47.4 | 54.0 | 62.8 | 69.7 |
| 1.5 | 26.9 | 32.7 | 36.1 | 41.1 | 47.7 | 53.0 |
| 2 | 22.0 | 26.7 | 29.5 | 33.5 | 39.0 | 43.2 |
| 3 | 16.4 | 19.9 | 22.0 | 25.0 | 29.0 | 32.2 |
| 6 | 9.8 | 11.9 | 13.2 | 15.0 | 17.4 | 19.3 |
| 9 | 7.3 | 8.9 | 9.8 | 11.2 | 13.1 | 14.5 |
| 12 | 5.9 | 7.3 | 8.1 | 9.2 | 10.7 | 11.9 |

***Areal Reduction Factors***

Due to the size of the catchments being considered (792 ha to 803 ha), an areal reduction factor of 1 was adopted for all design events up to and including the 100 year ARI. This will result in slightly conservative results.

***Temporal Patterns***

Temporal patterns for design storm events for durations from 15 minutes to 12 hours for design events up to and including the 100 year ARI were adopted from *Australian Rainfall and Runoff*: *A Guide to Flood Estimation* (Pilgrim, 1998).

The Cotswold Hills catchment is within the transition zone between Zone 2 and Zone 3. For storm durations of 1 hour or less, the temporal patterns for Zones 2 and 3 are identical. Of the events modelled in this study, only the 2 and 5 year ARI events had critical storm durations longer than 1 hour. In this study, Zone 3 temporal patterns were adopted, as the nature of the weather systems that result in large floods in the study area tend to be rainfall from ex-tropical cyclone activity.

***Spatial Distribution***

The design rainfalls for durations from 15 minutes to 12 hours for all ARIs up to and including the 100 year ARI were estimated at the centroids of the two catchments using standard procedures (Pilgrim, 1998), and assumed to be uniform across each catchment.

***Rainfall Losses***

The initial loss (IL) / continuing loss (CL) method of accounting for rainfall losses was adopted for this study. Book II, Section 3 of *Australian Rainfall and Runoff*: *A Guide to Flood Estimation* (Pilgrim, 1998) recommends design initial loss rates of 15-35mm and a median continuing loss of 2.5mm/h for eastern Queensland, up to and including the 100 year ARI event.

For all design events up to and including the 100 year ARI event, initial and continuing losses have been adopted based on the relationship with percentage impervious in Table 3.3. This is consistent with the losses adopted for the January 2011 model validation event (refer Section 3.5.2), and provides results consistent with the Rational Method.

### 6.1.3 Design Rainfalls for 200 and 500 Year ARI Events

*Rainfall Depth Estimation*

Design rainfall depths for the 200 and 500 year ARI events were estimated using the approach recommended in *Australian Rainfall and Runoff*: *A Guide to Flood Estimation* (Pilgrim, 1998). CRC-Forge rainfall (DNRM, 2005) was also considered for design rainfalls for the 200 and 500 years ARI events. The CRC-Forge rainfall intensities were found to be 8-16% lower than the ARR rainfall intensities. The higher ARR rainfall intensities were adopted to produce conservative discharge estimates.

The adopted design rainfalls are provided in Table 6.3 and Table 6.4.

Table 6.3 Cotswold East - Intensity-Frequency-Duration Data (mm/hour)

| Duration | Average Recurrence Interval (years) | |
|---|---|---|
| (hours) | 200 | 500 |
| 0.25 | 175 | 208 |
| 0.5 | 120 | 142 |
| 1 | 79 | 94 |
| 1.5 | 60 | 72 |
| 2 | 49 | 59 |
| 3 | 37 | 45 |
| 6 | 23 | 28 |

Table 6.4 Cotswold West - Intensity-Frequency-Duration Data (mm/hour)

| Duration | Average Recurrence Interval (years) | |
|---|---|---|
| (hours) | 200 | 500 |
| 0.25 | 176 | 209 |
| 0.5 | 120 | 143 |
| 1 | 79 | 94 |
| 1.5 | 59 | 71 |
| 2 | 48 | 58 |
| 3 | 36 | 44 |
| 6 | 22 | 27 |

*Areal Reduction Factors*

Similar to the methodology for design rainfall events up to the 100 year ARI, an areal reduction factor of 1 was adopted for the 200 year and 500 year ARI design events.

*Temporal Patterns*

The temporal patterns for the 200 and 500 year ARI design events for all durations up to and including 6 hours were obtained from The Estimation of Probable Maximum Precipitation in Australia: Generalised Short Duration Method (BOM, 2003).

*Spatial Distribution*

The design rainfalls for durations from 15 minutes to 6 hours for the 200 and 500 year ARI events were estimated at the centroids of the two catchments using standard procedures (Pilgrim, 1998), and assumed to be uniform across each catchment.

*Rainfall Losses*

The initial loss rate adopted for the 200 and 500 year ARI design events is 0 mm. The continuing loss rate has not been reduced from that adopted in Table 3.3.

### 6.1.4 Probable Maximum Precipitation (PMP) Rainfall Estimates

*Rainfall Depth Estimation*

PMP rainfall depth estimates for durations up to 6 hours were obtained using the methodology given in The Estimation of Probable Maximum Precipitation in Australia: Generalised Short Duration Method (BOM, 2003). The notional AEP of the estimated PMP design event is $1 \times 10^{-7}$ (or 1 in 10,000,000) (BOM, 2003).

The PMP initial mean rainfall depths and intensities (unadjusted) for durations up to 6 hours are shown in Table 6.5 and Table 6.6. Design spatial distribution was then applied to the initial mean rainfall depths using the ellipse methodology described in BOM (2003). A spatially averaged design PMP estimate was then applied uniformly across each catchment.

**Table 6.5 Cotswold East - PMP Estimates - Initial Mean Rainfall Depths and Intensities**

| DURATION | PMP Estimate* | |
|---|---|---|
| (hours) | mm | mm/h |
| 0.25 | 180 | 714 |
| 0.5 | 260 | 521 |
| 1 | 380 | 384 |
| 1.5 | 490 | 330 |
| 2 | 580 | 289 |
| 3 | 700 | 233 |
| 6 | 930 | 155 |

Notes: *Initial Mean Rainfall Depth

Table 6.6 Cotswold West - PMP Estimates - Initial Mean Rainfall Depths and Intensities

| DURATION | PMP Estimate* | |
|---|---|---|
| (hours) | mm | mm/h |
| 0.25 | 180 | 714 |
| 0.5 | 260 | 521 |
| 1 | 380 | 384 |
| 1.5 | 490 | 330 |
| 2 | 580 | 289 |
| 3 | 700 | 233 |
| 6 | 930 | 155 |

Notes: *Initial Mean Rainfall Depth

***Areal Reduction Factors***

Areal reduction factors are incorporated in the BOM (2003) PMP rainfall estimation methodology, and as such no ARFs were applied to the rainfalls estimated for the catchments using this method.

***Temporal Patterns***

Temporal patterns for the PMP design storm events for durations up to 6 hours were obtained from The Estimation of Probable Maximum Precipitation in Australia: Generalised Short Duration Method (BOM, 2003).

***Spatial Distribution***

Spatial distribution of rainfall is accounted for in the BOM (2003) PMP rainfall estimation methodology.

***Rainfall Losses***

The initial loss rate adopted for the PMP event is 0 mm. The continuing loss rate was not changed from that adopted in Table 3.3.

***Terrain Category***

A roughness fraction of 1 (i.e. 100% roughness) was adopted, based on the topographical data available.

***Catchment Elevation Adjustment***

An elevation adjustment factor of 1 was adopted based on the mean elevation of the catchment.

***Moisture Adjustment***

A moisture adjustment factor of 0.808 was adopted based on guidelines given in the BOM (2003) PMP rainfall estimation methodology.

### 6.1.5 Design Flow Comparison Using Alternative Methods

Design discharges estimated using the XP-RAFTS models were compared against design discharges estimated using the Rational Method for the 10 and 100 year ARIs, at 6 locations in the Cotswold Hills catchment (refer Figure 3-3 for the locations). Table 3.5 shows the comparison of the XP-RAFTS model and Rational Method peak discharges. Results indicate differences of no more than 20% for both design events at all locations.

## 6.2 FULLY- DEVELOPED CATCHMENT FLOOD BEHAVIOUR

The validated XP-RAFTS and MIKE FLOOD models were modified to reflect the fully-developed catchment flood behaviour, based on the TRC regional planning scheme (refer Figure 3-4).

- The validated XP-RAFTS model was modified to reflect the developed case fraction impervious and catchment PERN ‘n’ as shown in Table 3.4. All other parameters remained unchanged. The modified XP-RAFTS model was then used to estimate 2 to 100 year ARI design discharges for fully-developed conditions.

- The validated MIKE FLOOD model was modified to reflect the fully-developed case hydraulic roughness and design event inflows at the external and internal boundaries. All other model parameters remained unchanged, including bathymetry, flooding and drying depths, eddy viscosity, downstream model boundary, time step and save step; and hydraulic structure setup. The modified MIKE FLOOD model was then used to estimate design peak flood surface elevation, peak water depths and velocities in the Cotswold Hills catchments for 2 to 100 year ARI design events.

## 6.3 ESTIMATION OF DESIGN FLOOD LEVELS, DEPTHS AND VELOCITIES

### 6.3.1 Methodology

The validated MIKE FLOOD model was used to estimate design peak flood surface elevation, peak water depths and velocities in the Cotswold Hills catchments for the nominated design events for both existing and developed catchment conditions. The validated model was modified to reflect design event inflows at the external and internal boundaries.

All other model parameters remained unchanged, including bathymetry, hydraulic roughness, flooding and drying depths, eddy viscosity, downstream model boundary, time step and save step and hydraulic structure setup.

### 6.3.2 Design Flood Levels and Extents

The two XP-RAFTS models were run for a range of storm durations up to 12 hours and the critical durations for the hydraulic model area were found to be 90 minutes for the 2 and 5 year ARIs. For the 10 year ARI, the critical duration in the upper, urbanised portion of the catchment was found to be 60 minutes and 90 minutes in the lower, less-developed portion of the catchment. The critical duration for the 20, 50 and 100 year ARIs was found to be 60 minutes.

Maps of predicted water surface level and flood depth for the following design events for the existing conditions are provided in Appendix D:

- 10 year ARI
- 50 year ARI
- 100 year ARI
- 200 year ARI
- 500 year ARI
- PMP

Tables B6 – B10 in Appendix B shows the predicted affluxes at hydraulic structures for the above events.

Hazard category and flood hydraulic category maps for the 100 year ARI and PMP events for the existing conditions are provided in Appendix F.

### 6.3.3 Peak Flood Levels and Design Discharges at Representative Locations

Table 6.8 shows predicted peak flood levels and discharges at six representative locations within the study area for both existing and fully developed catchment conditions. Table 6.7 and Figure 6-1 show details of the reporting locations.

**Table 6.7 Details of Reporting Locations**

| Reporting Location | Description |
|---|---|
| 1 | Gowrie Junction Road |
| 2 | Roderick Drive[a] |
| 3 | Ganzer – Morris Road |
| 4 | Ganzer Road |
| 5 | Boundary Street[b] |
| 6 | Hermitage Road |

a. Hydraulic structure COT2 location in the MIKE FLOOD model
b. Hydraulic structure COT1 location in the MIKE FLOOD model

Modelling of the existing-development conditions design events showed that:

- Design discharges and peak flood levels at all reporting locations increase with ARI up to the 500 year ARI. The 500 year ARI design discharges are 4 to 8 times larger than the 2 year ARI discharges and the 500 year ARI flood levels are 0.2m to 0.67m higher than the 2 year ARI at all reporting locations, with the exception of location 3 (refer Figure 6-1);
- At reporting location 3, the 500 year ARI design flood level is up to 1.75m higher than the 2 year ARI design flood level; and
- The crossing at Roderick Drive has low flood immunity, overtopping in a 10 year ARI design event. The Boundary Street crossing at reporting location 5 (refer Figure 6-1) has flood immunity up to and including the 500 year ARI event.

Modelling of the ultimate-development conditions design events showed that:

- Ultimate-development conditions design discharges and peak flood levels are unchanged at reporting location 2 (refer Figure 6-1);
- Ultimate-development conditions design discharges at reporting location 5 (refer Figure 6-1) are 25% to 51% larger than for the corresponding existing-development conditions. This is due to the extensive urbanisation of the catchment upstream of this location. As a result, peak flood levels are up to 0.14m higher than for the corresponding existing-development conditions; and
- Ultimate-development conditions design discharges at the remaining reporting locations are up to 21% larger than for the corresponding existing-development conditions. As a result, ultimate-development conditions peak flood levels are up to 0.12m higher than for the corresponding existing-development conditions.

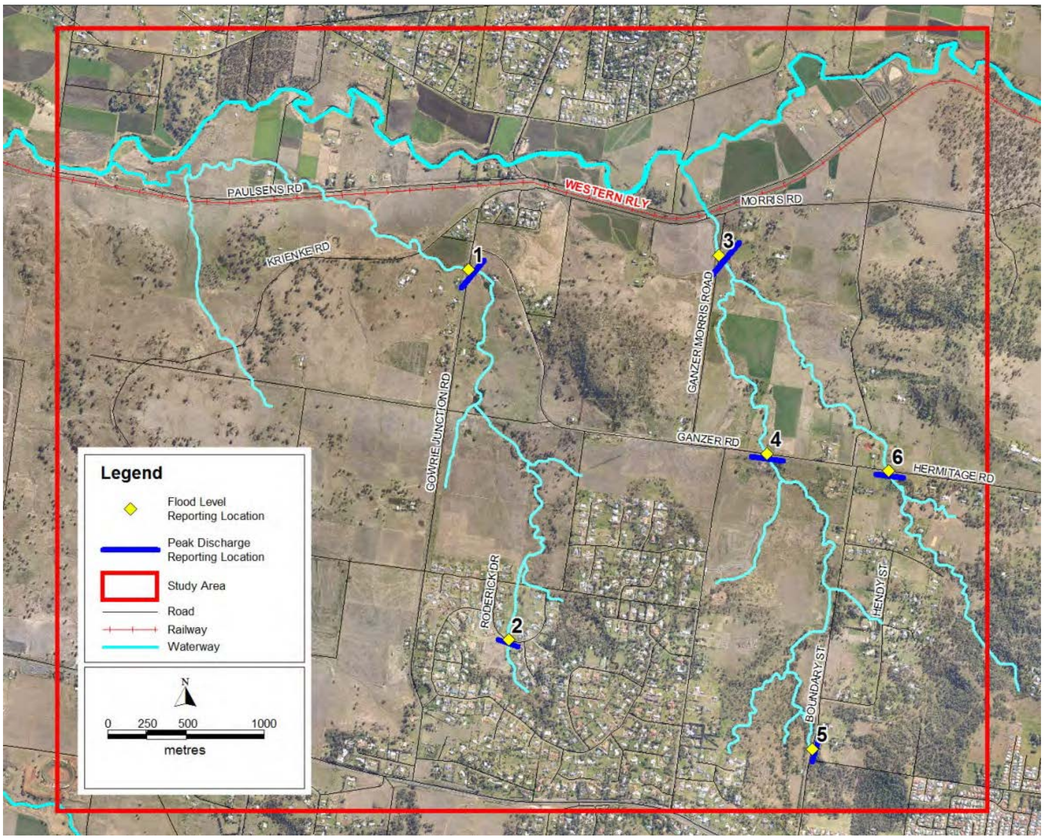


Figure 6-1 Reporting Locations for Peak Flood Levels and Discharges

Table 6.8 Existing and Ultimate Development - Predicted Peak Flood Levels and Discharges at Reporting Locations

| ARI (year) | Case Identifier | Reporting Location: 1 - Gowrie Junction Rd. | | 2 - Roderick Dr. | | 3 – Ganzer-Morris Rd. | | 4 – Ganzer Rd. | | 5 – Boundary St. | | 6 – Hermitage Rd. | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Flood Level (mAHD) | Discharge ($m^3/s$) | Flood Level (mAHD) | Discharge ($m^3/s$) | Flood Level (mAHD) | Discharge ($m^3/s$) | Flood Level (mAHD) | Discharge ($m^3/s$) | Flood Level (mAHD) | Discharge ($m^3/s$) | Flood Level (mAHD) | Discharge ($m^3/s$) |
| | **Validation Event** | | | | | | | | | | | | |
| | COT-V03-E-Jan2011 | 487.89 | 52 | 544.53 | 12 | 494.36 | 116 | 517.96 | 63 | 565.78 | 8.3 | 524.58 | 28 |
| | **Existing-Development Case** | | | | | | | | | | | | |
| 2 | COT-V03-E-002Y-090M | 487.66 | 19 | 544.49 | 6.2 | 493.41 | 25 | 517.54 | 15 | 565.44 | 2.2 | 524.11 | 5.3 |
| 5 | COT-V03-E-005Y-090M | 487.75 | 30 | 544.50 | 10 | 493.65 | 44 | 517.66 | 25 | 565.54 | 3.7 | 524.24 | 10 |
| 10 | COT-V03-E-010Y-001H | 487.89 | 51 | 544.57 | 17 | 493.86 | 65 | 517.79 | 40 | 565.63 | 5.4 | 524.35 | 16 |
| 20 | COT-V03-E-020Y-001H | 487.94 | 60 | 544.60 | 20 | 494.04 | 89 | 517.88 | 52 | 565.69 | 6.4 | 524.45 | 20 |
| 50 | COT-V03-E-050Y-001H | 487.99 | 70 | 544.63 | 23 | 494.33 | 114 | 517.95 | 63 | 565.76 | 7.9 | 524.52 | 24 |
| 100 | COT-V03-E-100Y-001H | 488.04 | 87 | 544.67 | 25 | 494.57 | 130 | 518.00 | 71 | 565.83 | 9.6 | 524.56 | 28 |
| 200[a] | COT-V03-E-200Y-001H | 488.11 | 96 | 544.63 | 23 | 494.99 | 172 | 518.09 | 88 | 565.88 | 10 | 524.68 | 36 |
| 500 | COT-V03-E-500Y-001H | 488.20 | 115 | 544.69 | 27 | 495.16 | 208 | 518.19 | 106 | 565.97 | 12 | 524.78 | 43 |
| PMP | COT-V03-E-PMP-045M | 489.18 | 561 | 545.28 | 122 | 496.37 | 1053 | 519.19 | 517 | 573.23 | 62 | 525.95 | 206 |
| | **Ultimate-Development Case** | | | | | | | | | | | | |
| 2 | COT-V03-D-002Y-090M | 487.67 | 19 | 544.49 | 6.2 | 493.42 | 25 | 517.62 | 18 | 565.50 | 3.0 | 524.12 | 5.6 |
| 5 | COT-V03-D-005Y-090M | 487.76 | 31 | 544.50 | 10 | 493.65 | 44 | 517.72 | 28 | 565.64 | 5.6 | 524.24 | 10 |
| 10 | COT-V03-D-010Y-001H | 487.92 | 55 | 544.57 | 17 | 493.90 | 71 | 517.85 | 43 | 565.77 | 7.9 | 524.36 | 16 |
| 20 | COT-V03-D-020Y-001H | 487.96 | 63 | 544.60 | 20 | 494.04 | 97 | 517.96 | 59 | 565.83 | 9.5 | 524.45 | 21 |
| 50 | COT-V03-D-050Y-001H | 488.03 | 84 | 544.63 | 23 | 494.40 | 121 | 518.02 | 69 | 565.90 | 11 | 524.59 | 29 |
| 100 | COT-V03-D-100Y-001H | 488.13 | 103 | 544.67 | 25 | 494.69 | 143 | 518.09 | 79 | 565.95 | 12 | 524.65 | 34 |

a. Differences between ARR and GSDM temporal patterns result in lower flood level and peak flows in some parts of the catchment when comparing the 100 and 200 year ARI events.

## 6.4 DESIGN FLOOD LEVEL SENSITIVITY ANALYSIS

### 6.4.1 Case 1: Variation in Discharge

Two scenarios investigating a variation in design discharge were undertaken for the existing-development conditions for the 100 year ARI, 60 minute duration design event only, as follows:

- A 30% increase in all inflow hydrographs to the hydrodynamic model; and
- A 30% decrease in all inflow hydrographs to the hydrodynamic model.

### 6.4.2 Case 2: Variation in Roughness

Two scenarios investigating a variation in hydraulic roughness were undertaken for the existing-development conditions, 100 year ARI 60 minute duration design event only, as follows:

- A 30% increase in the hydraulic roughness; and
- A 30% decrease in the hydraulic roughness.

### 6.4.3 Case 3: Hydraulic Structure Blockage

One scenario investigating a 50% blockage of hydraulic structures was modelled for the existing-development conditions, 100 year ARI critical duration (60 minute) design event only, as follows:

- The width of rectangular culverts was halved while maintaining the existing invert and obvert levels;
- The cross-sectional area of circular culverts was halved by reducing the diameter;
- The pier blockage factor was set to 0.5 for bridge openings;
- Bridge handrails were treated as fully blocked. The blockage of handrails was modelled by raising the road level in the MIKE 21 bathymetry file; and
- Culvert headwall railings were not considered strong enough to withstand a flood debris load and so were not treated as blocked.

### 6.4.4 Sensitivity Analysis Results

Table 6.9 shows the peak flood level and discharges estimated in the sensitivity analysis runs at the six reporting locations (refer Figure 6-1).

Sensitivity analysis results for the variation of the 100 year ARI design discharges showed that:

- A 30% increase in design discharges results in peak flood levels increasing by 0.08m to 0.42m across the reporting locations; and
- A 30% decrease in design discharges results in peak flood levels decreasing by 0.09m to 0.6m.

Sensitivity analysis results for the variation of hydraulic model roughness for the 100 year ARI design event showed that:

- A 30% increase in roughness results in peak flood levels increasing by 0.03m to 0.2m across the reporting locations; and
- A 30% decrease in roughness results in peak flood levels either unchanged or decreasing by up to 0.2m.

Sensitivity analysis results for the blockage of hydraulic structures for the 100 year ARI design event showed that:

- A 50% blockage of structures results in peak flood levels increasing by up to 0.15m and 0.2m at the Roderick Drive and Boundary Street reporting locations respectively, while remaining unchanged at the remaining reporting locations.

## 6.5 CLIMATE CHANGE ANALYSIS

Three scenarios investigating climate change effects were undertaken for the existing-development case for each of the 100, 200 and 500 year critical duration design events, as follows:

- A 2ºC temperature increase by 2050;
- A 3ºC temperature increase by 2070; and
- A 4ºC temperature increase by 2100.

The effects of climate change were simulated assuming a 5% increase in rainfall intensity per degree of global warming.

Table 6.9 shows the peak flood level and discharges estimated in the climate change analysis at the 6 reporting locations (refer Figure 6-1). Maps of predicted flood level for the following climate change analysis events are provided in Appendix E:

- 100 year ARI
- 200 year ARI
- 500 year ARI

Results of the climate change scenario analysis showed that:

- For the 100 year ARI climate change scenarios, peak discharges at the reporting locations increase by between 25% and 46%. Peak flood levels increase by up to 0.19m for most reporting locations while peak flood levels at Ganzer-Morris Road increase by up to 0.36m;
- For the 200 year ARI climate change scenarios, peak discharges at the reporting locations increase by between 20% and 30%. Peak flood levels increase up to 0.17m across the reporting locations; and
- For the 500 year ARI climate change scenarios, peak discharges at the reporting locations increase by between 17% and 25%. Peak flood levels increase by up to 0.13m across the reporting locations.

Table 6.9 Sensitivity & Climate Change Analysis - Predicted Peak Flood Levels and Discharges at Reporting Locations

| ARI (year) | Case Identifiers | Reporting Location | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 1 - Gowrie Junction Rd. | | 2 - Roderick Dr. | | 3 – Ganzer-Morris Rd.[a] | | 4 – Ganzer Rd.[b] | | 5 – Boundary St. | | 6 – Hermitage Rd. | |
| | | Flood Level (mAHD) | Discharge (m3/s) | Flood Level (mAHD) | Discharge (m3/s) | Flood Level (mAHD) | Discharge (m3/s) | Flood Level (mAHD) | Discharge (m3/s) | Flood Level (mAHD) | Discharge (m3/s) | Flood Level (mAHD) | Discharge (m3/s) |
| | | | | | | Sensitivity Cases | | | | | | | |
| 100 | COT-V03-E-100Y-001H-Flow-Plus30pct | 488.20 | 119 | 544.75 | 33 | 494.99 | 172 | 518.12 | 92 | 565.97 | 12 | 524.74 | 41 |
| 100 | COT-V03-E-100Y-001H-Flow-Minu30pct | 487.92 | 56 | 544.58 | 18 | 493.97 | 89 | 517.87 | 50 | 565.70 | 6.7 | 524.43 | 19 |
| 100 | COT-V03-E-100Y-001H-Rough-Plus30pct | 488.10 | 82 | 544.71 | 26 | 494.77 | 131 | 518.04 | 72 | 565.92 | 9.6 | 524.59 | 28 |
| 100 | COT-V03-E-100Y-001H-Rough-Minus30pct | 488.04 | 92 | 544.65 | 25 | 494.37 | 128 | 517.98 | 70 | 565.80 | 9.6 | 524.56 | 30 |
| 100 | COT-V03-E-100Y-001H-Blockage-50pct | 488.04 | 87 | 544.82 | 25 | 494.57 | 130 | 518.00 | 71 | 566.03 | 9.6 | 524.56 | 27 |
| | | | | | | Climate Change Cases | | | | | | | |
| 100 | COT-V03-E-100Y-001H-CC2050 | 488.13 | 103 | 544.69 | 28 | 494.77 | 146 | 518.05 | 79 | 565.90 | 11 | 524.67 | 35 |
| 100 | COT-V03-E-100Y-001H-CC2070 | 488.17 | 111 | 544.71 | 29 | 494.87 | 163 | 518.08 | 85 | 565.95 | 12 | 524.71 | 39 |
| 100 | COT-V03-E-100Y-001H-CC2100 | 488.20 | 118 | 544.72 | 30 | 494.93 | 178 | 518.12 | 94 | 565.98 | 13 | 524.75 | 41 |
| 200 | COT-V03-E-200Y-001H-CC2050 | 488.16 | 106 | 544.66 | 25 | 495.09 | 192 | 518.15 | 98 | 565.92 | 12 | 524.73 | 40 |
| 200 | COT-V03-E-200Y-001H-CC2070 | 488.18 | 111 | 544.67 | 26 | 495.11 | 201 | 518.17 | 103 | 565.95 | 12 | 524.76 | 42 |
| 200 | COT-V03-E-200Y-001H-CC2100 | 488.20 | 116 | 544.69 | 27 | 495.16 | 210 | 518.19 | 107 | 565.97 | 13 | 524.78 | 44 |
| 500 | COT-V03-E-500Y-001H-CC2050 | 488.24 | 128 | 544.72 | 30 | 495.21 | 229 | 518.25 | 118 | 566.02 | 14 | 524.83 | 48 |
| 500 | COT-V03-E-500Y-001H-CC2070 | 488.27 | 134 | 544.73 | 31 | 495.24 | 240 | 518.27 | 124 | 566.04 | 14 | 524.85 | 50 |
| 500 | COT-V03-E-500Y-001H-CC2100 | 488.29 | 140 | 544.75 | 33 | 495.29 | 250 | 518.30 | 130 | 566.07 | 15 | 524.88 | 53 |

a. Results at this location show the influence of constricted flow downstream, passing under the Western Railway.
b. Results at this location show the influence of afflux at the upstream side of the road crossing, leading to increased or decreased discharge as water level increases or decreases.

## 6.6 COMPARISON WITH PREVIOUS STUDY RESULTS

Two previous studies were identified for comparison with the current study. The first was a drainage study undertaken by Sinclair Knight Merz (SKM) in 1995 and the second was a stormwater management study, also undertaken by SKM in 2006 (SKM, 2006). The 1995 study was not available for examination, however the 2006 study was reviewed and the results compared with the current study.

SKM (2006) study used DRAINS (ILSAX) and HEC-RAS models for hydrologic and hydraulic modelling.

The SKM study from 2006 was prepared for an area of Cotswold Hills as shown in Figure 6-2. This corresponded with a portion of the East Tributary upper catchment, south of Ganzer Road (refer Figure 3-1). The catchment area to Ganzer Road identified in the 2006 study was similar to the corresponding sub-catchments in the current study (383ha compared to 349ha). The 2006 study identified a catchment outlet at Ganzer Road. This corresponds with reporting location 4 and XP-RAFTS node C15 in the current study.

The 2006 study by SKM and the current study both determined the 100 year ARI critical duration for the existing development case to be 60 minutes. Table 6.10 shows a comparison of estimated maximum discharges and predicted flood levels for existing development conditions at the catchment outlet, for the 100 year ARI, critical duration event.

**Table 6.10 Comparison of Predicted Peak Flood Level and Discharge at Location 4**

| Parameter | SKM (2006) Study | Current Study |
|---|---|---|
| Peak Discharge ($m^3/s$) | 58.4[a] | 72.0[b]<br>71.2[c] |
| Peak Flood Level (mAHD) | 517.7 | 517.1 |

a. DRAINS estimate
b. XP-RAFTS estimate
c. MIKE FLOOD estimate

Reasons for the differences in estimated peak discharge between the SKM (2006) study and the current study are not clear. There is insufficient information reported in the 2006 study to allow the reasons to be determined, for example, no information is given on adopted rainfall loss parameters in the 2006 study. Some possible reasons for the differences between the two may include the following points.

- The 2006 study adopted a percent impervious value of 3.9%, compared with a percent impervious value of 11.2% adopted in the current study. It is likely that the existing development case in 2006 was different from the existing case currently.

- Discharge estimates in the SKM study from 2006 were not validated against the Rational Method or observed events. Estimates of peak discharge in 2006 were obtained using the DRAINS (ILSAX) model, with parameters varied to match Rational Method estimates from the 1995 report. The current study validated peak discharges estimated using XP-RAFTS against Rational Method estimates (refer Table 3.6) and peak flood levels against the January 2011 event. Table 6.10 also shows good agreement between peak discharge estimates in XP-RAFTS and MIKE FLOOD models.

- The SKM (2006) study adopted waterway Manning's 'n' values ranging between 0.05 and 0.10. This is higher than the range of 0.033 to 0.083 adopted by the current study (refer Table 4.1). The combination of higher roughness values and lower discharge could lead to a peak flood level similar to that predicted by the current study.

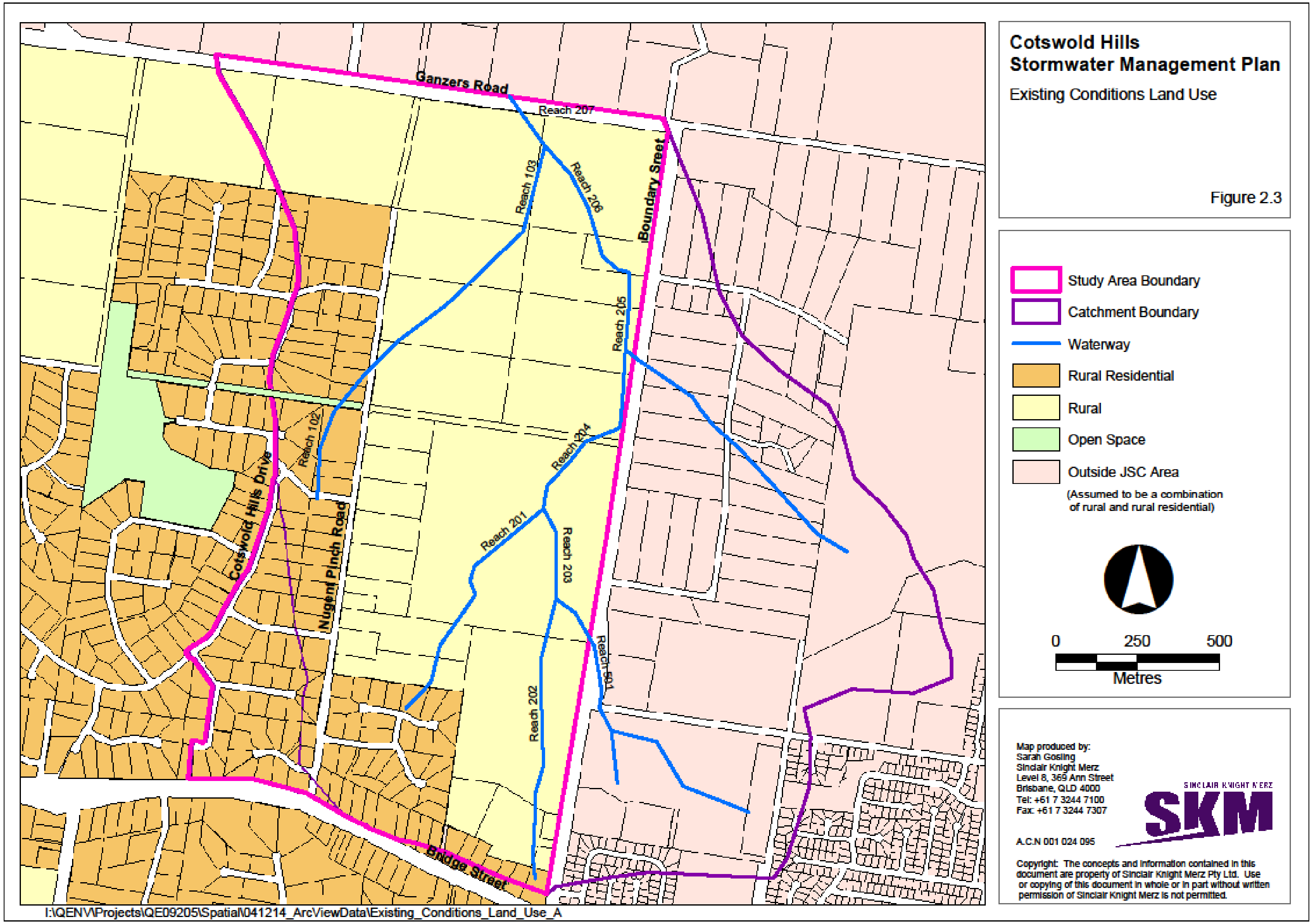


Figure 6-2 Study Area and Existing Conditions used in the SKM (2006) Stormwater Management Study

# 7 SUMMARY AND CONCLUSIONS

The primary objective of this flood study project is to define the nature and extent of flood behaviour in the Cotswold Hills study area to enable Toowoomba Regional Council to develop a Flood Risk Management Study and amend the regional planning scheme to reflect flood requirements of the State Planning Policy and the recommendations of the Queensland Commission of Inquiry.

A hydrologic (XP-RAFTS) model and a coupled 1D/2D (MIKE FLOOD) hydraulic model have been successfully developed and validated for the Cotswold Hills study area. The models were validated for the January 2011 flood event using a joint calibration approach. The validation results were demonstrated graphically and locations of known flooding were reproduced by the hydraulic model.

The hydraulic model results show that residential properties in Cotswold Hills were not affected by watercourse generated flooding during the January 2011 event, except for a few low-lying rural areas which were inundated. Peak flood surface elevation and flood depth maps are included in Appendix C of this report. Digital mapping and model data files were also delivered as part of the study.

The validated MIKE FLOOD hydrodynamic model was used to estimate design peak flood surface elevations, peak water depths and velocities in the Dry Creek catchment for the 2, 5, 10, 20, 50, 100, 200 and 500 year ARI and PMF design events. The sensitivity of predicted 100 year ARI model results was tested to assess the impacts of changes to adopted design discharges, hydraulic roughness and blockage of hydraulic structures. Additionally, the potential impact on flood behaviour was assessed for three climate change scenarios for the 100, 200 and 500 year ARI design events.

The study results show that;

- Design discharges and peak flood levels at all reporting locations increase with ARI up to the 500 year ARI. The 500 year ARI design discharges are 4 to 8 times larger than the 2 year ARI discharges and the 500 year ARI flood levels are 0.2m to 0.67m higher than the 2 year ARI at all reporting locations, with the exception of location 3 (refer Figure 6-1). At reporting location 3, the 500 year ARI design flood level is up to 1.75m higher than the 2 year ARI design flood level; and
- The crossing at Roderick Drive has low flood immunity, overtopping in a 10 year ARI design event. The Boundary Street crossing at reporting location 5 (refer Figure 6-1) has flood immunity up to and including the 500 year ARI event.

- Ultimate-development conditions design discharges and peak flood levels are unchanged at reporting location 2 (refer Figure 6-1), while ultimate-development conditions design discharges at reporting location 5 (refer Figure 6-1) are 25% to 51% larger than for the corresponding existing-development conditions. This is due to the extensive urbanisation of the catchment upstream of this location. As a result, peak flood levels are up to 0.14m higher than for the corresponding existing-development conditions; and
- Ultimate-development conditions design discharges at the remaining reporting locations are up to 21% larger than for the corresponding existing-development conditions. As a result, ultimate-development conditions peak flood levels are up to 0.12m higher than for the corresponding existing-development conditions.

Sensitivity analysis results for the 100 year ARI design event indicate the following:

- A 30% increase and decrease in design discharges results in peak flood levels increasing and decreasing by up to 0.42m and 0.6m respectively;
- A 30% increase and decrease in roughness results in peak flood levels increasing and decreasing by up to 0.2m; and
- A 50% blockage of structures results in peak flood levels increasing by up to 0.15m and 0.2m at the Roderick Drive and Boundary Street reporting locations respectively, while remaining unchanged at the remaining reporting locations.

Climate change scenario results indicate the following:

- For all of the climate change scenarios, peak discharges increase at the reporting locations; and
- Peak flood levels for the 100 year, 200 year and 500 year ARI events increase by up to 0.36m, 0.17m and 0.13m respectively at the reporting locations.

# 8 RECOMMENDATIONS

The following recommendations should be considered to improve the accuracy of the model performance.

1. Detailed calibration for at least two historical flood events should be performed to improve the model accuracy if more data becomes available. As the model has only been validated against two locations of known flooding; the model results should be used with caution.
2. Should a flood event occur it is recommended that as a minimum peak flood levels and extents are collected to further aid the validity of the model.
3. Topographic survey of key structures and flow control thresholds within the catchment should be carried out to validate the assumptions made in developing the hydraulic model.

# 9 REFERENCES


| | |
|---|---|
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# 10 DISCLAIMER

*Information used is the best information available at this time for the purposes of this study. Marks observed and other anecdotal information obtained after flood events have been obtained from a range of sources and have varying degrees of uncertainty.*

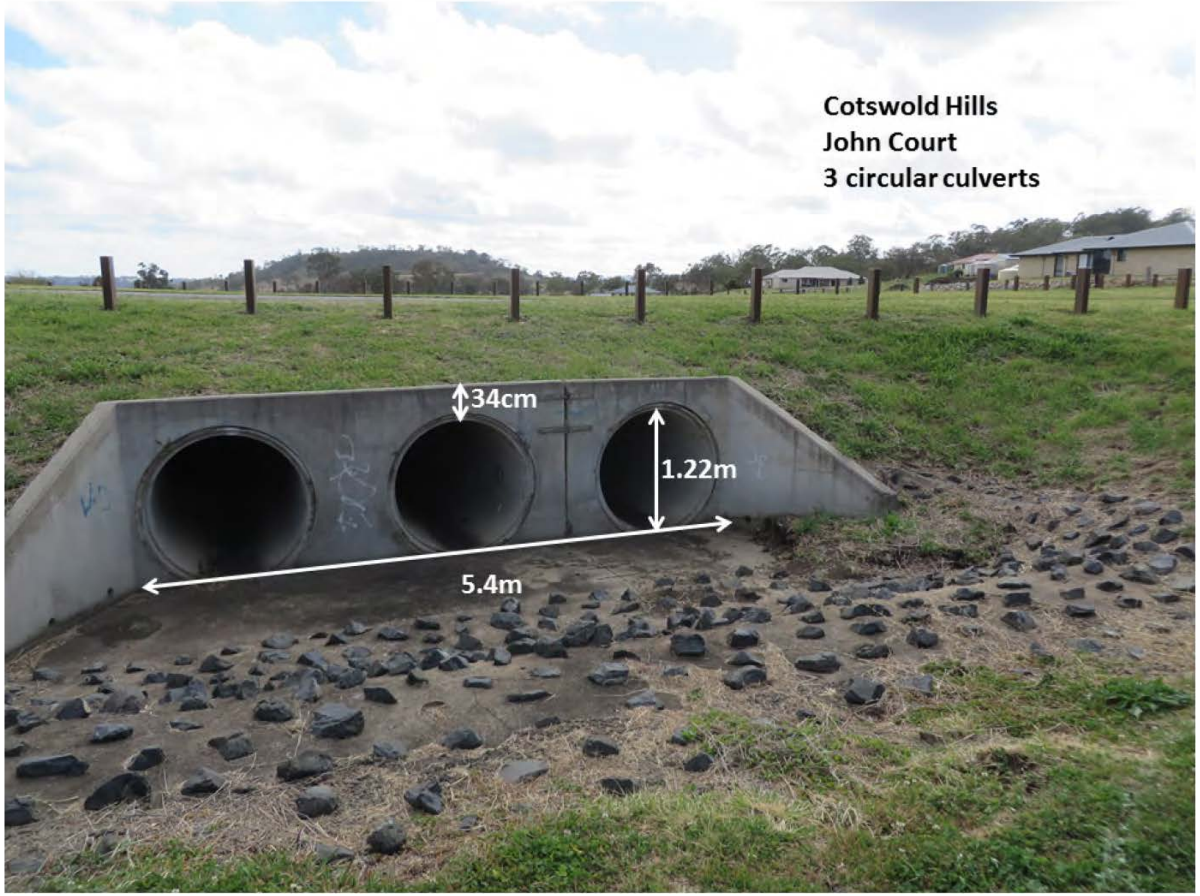
*While every care is taken by the Toowoomba Regional Council (TRC) to ensure the accuracy of the data used in the study and published in the report, Toowoomba Regional Council makes no representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaim all responsibility and all liability whether in contract, negligence or otherwise for all expenses, losses, damages (including indirect or consequential damage) and costs which may be incurred in any way and for any reason as a result of data being inaccurate or incomplete.*

# APPENDIX A

## STRUCTURE ATTRIBUTES

COT-1
Near 330 Boundary Street
(Dimensions taken from design drawings supplied by TRC. 1 circular culvert, Diameter = 1.8m)

Location:
-27.528648
151.906741




COT-2
Near Corner of John Court
and Roderick Drive

Location:
-27.523094
151.888137

# APPENDIX B

STRUCTURE HEAD LOSSES, DISCHARGES AND VELOCITIES FOR VALIDATION AND DESIGN EVENTS

**Table B.1 Head Losses, Discharges and Velocities at Structures for the January 2011 Validation Event**

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.78 | 573.40 | 563.55 | 564.41 US | 566.16 US | 2.23 | 8 | 3.3 | No |
| | | | | 562.60 DS | 564.35 DS | | | | |
| COT-2 | 544.53 | 544.55 | 542.51 | 542.46 US | 543.66 US | 2.02 | 12 | 3.4 | No |
| | | | | 542.06 DS | 543.26 DS | | | | |

[1] The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
[2] Peak discharge is reported through the culvert
[3] Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

**Table B.2 Head Losses, Discharges and Velocities at Structures for the 2 Year ARI Design Event**

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.44 | 573.40 | 563.14 | 564.41 US | 566.16 US | 2.30 | 2 | 0.8 | No |
| | | | | 562.60 DS | 564.35 DS | | | | |
| COT-2 | 544.49 | 544.55 | 542.32 | 542.46 US | 543.66 US | 2.17 | 7 | 2.1 | No |
| | | | | 542.06 DS | 543.26 DS | | | | |

[1] The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
[2] Peak discharge is reported through the culvert
[3] Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.3 Head Losses, Discharges and Velocities at Structures for the 5 Year ARI Design Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.54 | 573.40 | 563.28 | 564.41 US | 566.16 US | 2.26 | 3 | 1.3 | No |
| | | | | 562.60 DS | 564.35 DS | | | | |
| COT-2 | 544.50 | 544.55 | 542.44 | 542.46 US | 543.66 US | 2.06 | 11 | 3.0 | No |
| | | | | 542.06 DS | 543.26 DS | | | | |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.4 Head Losses, Discharges and Velocities at Structures for the 10 Year ARI Design Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.63 | 573.40 | 563.35 | 564.41 US | 566.16 US | 2.28 | 6 | 2.5 | No |
| | | | | 562.60 DS | 564.35 DS | | | | |
| COT-2 | 544.57 | 544.55 | 542.65 | 542.46 US | 543.66 US | 1.92 | 17 | 4.9 | Yes |
| | | | | 542.06 DS | 543.26 DS | | | | |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.5 Head Losses, Discharges and Velocities at Structures for the 20 Year ARI Design Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.69 | 573.40 | 563.48 | 564.41 US | 566.16 US | 2.21 | 7 | 2.9 | No |
| | | | | 562.60 DS | 564.35 DS | | | | |
| COT-2 | 544.60 | 544.55 | 542.89 | 542.46 US | 543.66 US | 1.71 | 18 | 5.0 | Yes |
| | | | | 542.06 DS | 543.26 DS | | | | |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.6 Head Losses, Discharges and Velocities at Structures for the 50 Year ARI Design Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.76 | 573.40 | 563.54 | 564.41 US | 566.16 US | 2.22 | 8.7 | 3.4 | No |
| | | | | 562.60 DS | 564.35 DS | | | | |
| COT-2 | 544.63 | 544.55 | 542.92 | 542.46 US | 543.66 US | 1.71 | 20 | 5.6 | Yes |
| | | | | 542.06 DS | 543.26 DS | | | | |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.7 Head Losses, Discharges and Velocities at Structures for the 100yr ARI Design Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.83 | 573.40 | 563.59 | 564.41 US | 566.16 US | 2.24 | 10 | 3.9 | No |
| | | | | 562.60 DS | 564.35 DS | | | | |
| COT-2 | 544.67 | 544.55 | 542.96 | 542.46 US | 543.66 US | 1.71 | 14 | 4.1 | Yes |
| | | | | 542.06 DS | 543.26 DS | | | | |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.8 Head Losses, Discharges and Velocities at Structures for the 200yr ARI Design Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.88 | 573.40 | 563.64 | 564.41 US | 566.16 US | 2.24 | 10 | 4.1 | No |
| | | | | 562.60 DS | 564.35 DS | | | | |
| COT-2 | 544.63 | 544.55 | 542.91 | 542.46 US | 543.66 US | 1.72 | 20 | 5.7 | Yes |
| | | | | 542.06 DS | 543.26 DS | | | | |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.9 Head Losses, Discharges and Velocities at Structures for the 500yr ARI Design Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.97 | 573.40 | 563.71 | 564.41 US<br>562.60 DS | 566.16 US<br>564.35 DS | 2.26 | 13 | 5.0 | No |
| COT-2 | 544.69 | 544.55 | 542.97 | 542.46 US<br>542.06 DS | 543.66 US<br>543.26 DS | 1.72 | 15 | 4.4 | Yes |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.10 Head Losses, Discharges and Velocities at Structures for the PMP Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 573.23 | 573.40 | 564.48 | 564.41 US<br>562.60 DS | 566.16 US<br>564.35 DS | 8.75 | 39 | 15.5 | No |
| COT-2 | 545.28 | 544.55 | 543.50 | 542.46 US<br>542.06 DS | 543.66 US<br>543.26 DS | 1.78 | 24 | 6.8 | Yes |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.11 Head Losses, Discharges and Velocities at Structures for the 2 Year ARI Ultimate-Development Conditions Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.50 | 573.40 | 563.22 | 564.41 US<br>562.60 DS | 566.16 US<br>564.35 DS | 2.28 | 3.3 | 1.3 | No |
| COT-2 | 544.49 | 544.55 | 542.32 | 542.46 US<br>542.06 DS | 543.66 US<br>543.26 DS | 2.17 | 6.4 | 1.8 | No |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.12 Head Losses, Discharges and Velocities at Structures for the 5 Year ARI Ultimate-Development Conditions Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.64 | 573.40 | 563.38 | 564.41 US<br>562.60 DS | 566.16 US<br>564.35 DS | 2.26 | 5.6 | 2.2 | No |
| COT-2 | 544.50 | 544.55 | 542.41 | 542.46 US<br>542.06 DS | 543.66 US<br>543.26 DS | 2.09 | 11 | 3.0 | No |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.13 Head Losses, Discharges and Velocities at Structures for the 10 Year ARI Ultimate-Development Conditions Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.77 | 573.40 | 563.52 | 564.41 US<br>562.60 DS | 566.16 US<br>564.35 DS | 2.25 | 7.6 | 3.0 | No |
| COT-2 | 544.57 | 544.55 | 542.65 | 542.46 US<br>542.06 DS | 543.66 US<br>543.26 DS | 1.92 | 17 | 4.9 | Yes |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.14 Head Losses, Discharges and Velocities at Structures for the 20 Year ARI Ultimate-Development Conditions Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.83 | 573.40 | 563.60 | 564.41 US<br>562.60 DS | 566.16 US<br>564.35 DS | 2.23 | 9.6 | 3.8 | No |
| COT-2 | 544.60 | 544.55 | 542.89 | 542.46 US<br>542.06 DS | 543.66 US<br>543.26 DS | 1.71 | 18 | 5.0 | Yes |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.15 Head Losses, Discharges and Velocities at Structures for the 50 Year ARI Ultimate-Development Conditions Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.90 | 573.40 | 563.65 | 564.41 US<br>562.60 DS | 566.16 US<br>564.35 DS | 2.25 | 11 | 4.4 | No |
| COT-2 | 544.63 | 544.55 | 542.92 | 542.46 US<br>542.06 DS | 543.66 US<br>543.26 DS | 1.71 | 20 | 5.6 | Yes |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.16 Head Losses, Discharges and Velocities at Structures for the 100 Year ARI Ultimate-Development Conditions Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.95 | 573.40 | 563.71 | 564.41 US<br>562.60 DS | 566.16 US<br>564.35 DS | 2.24 | 11 | 4.4 | No |
| COT-2 | 544.67 | 544.55 | 542.96 | 542.46 US<br>542.06 DS | 543.66 US<br>543.26 DS | 1.71 | 20 | 5.6 | Yes |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.17 Head Losses, Discharges and Velocities at Structures for the Flow Plus 30% Sensitivity Case Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.97 | 573.40 | 563.70 | 564.41 US | 566.16 US | 2.27 | 15 | 5.7 | No |
| | | | | 562.60 DS | 564.35 DS | | | | |
| COT-2 | 544.75 | 544.55 | 543.04 | 542.46 US | 543.66 US | 1.71 | 18 | 5.2 | Yes |
| | | | | 542.06 DS | 543.26 DS | | | | |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.18 Head Losses, Discharges and Velocities at Structures for the Flow Minus 30% Sensitivity Case Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.70 | 573.40 | 563.44 | 564.41 US | 566.16 US | 2.26 | 8 | 3.2 | No |
| | | | | 562.60 DS | 564.35 DS | | | | |
| COT-2 | 544.58 | 544.55 | 542.67 | 542.46 US | 543.66 US | 1.91 | 18 | 5.1 | Yes |
| | | | | 542.06 DS | 543.26 DS | | | | |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.19 Head Losses, Discharges and Velocities at Structures for the Roughness Plus 30% Sensitivity Case Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.92 | 573.40 | 563.65 | 564.41 US<br>562.60 DS | 566.16 US<br>564.35 DS | 2.27 | 9 | 3.6 | No |
| COT-2 | 544.71 | 544.55 | 543.08 | 542.46 US<br>542.06 DS | 543.66 US<br>543.26 DS | 1.63 | 16 | 4.5 | Yes |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.20 Head Losses, Discharges and Velocities at Structures for the Roughness Minus 30% Sensitivity Case Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.80 | 573.40 | 563.56 | 564.41 US<br>562.60 DS | 566.16 US<br>564.35 DS | 2.24 | 8 | 3.3 | No |
| COT-2 | 544.65 | 544.55 | 542.85 | 542.46 US<br>542.06 DS | 543.66 US<br>543.26 DS | 1.80 | 21 | 5.9 | Yes |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.21 Head Losses, Discharges and Velocities at Structures for the 50% Blockage Sensitivity Case Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 566.03 | 573.40 | 563.59 | 564.41 US | 566.16 US | 2.44 | 10 | 3.9 | No |
| | | | | 562.60 DS | 564.35 DS | | | | |
| COT-2 | 544.82 | 544.55 | 542.90 | 542.46 US | 543.66 US | 1.92 | 13 | 3.6 | Yes |
| | | | | 542.06 DS | 543.26 DS | | | | |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.22 Head Losses, Discharges and Velocities at Structures for the 2050, 100 Year ARI Climate Change Scenario Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.90 | 573.40 | 563.65 | 564.41 US | 566.16 US | 2.25 | 9 | 3.6 | No |
| | | | | 562.60 DS | 564.35 DS | | | | |
| COT-2 | 544.69 | 544.55 | 542.96 | 542.46 US | 543.66 US | 1.73 | 11 | 3.2 | Yes |
| | | | | 542.06 DS | 543.26 DS | | | | |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.23 Head Losses, Discharges and Velocities at Structures for the 2070, 100 Year ARI Climate Change Scenario Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.95 | 573.40 | 563.69 | 564.41 US<br>562.60 DS | 566.16 US<br>564.35 DS | 2.26 | 14 | 5.4 | No |
| COT-2 | 544.71 | 544.55 | 543.00 | 542.46 US<br>542.06 DS | 543.66 US<br>543.26 DS | 1.71 | 19 | 5.3 | Yes |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.24 Head Losses, Discharges and Velocities at Structures for the 2100, 100 Year ARI Climate Change Scenario Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.98 | 573.40 | 563.71 | 564.41 US<br>562.60 DS | 566.16 US<br>564.35 DS | 2.27 | 11 | 4.3 | No |
| COT-2 | 544.72 | 544.55 | 543.02 | 542.46 US<br>542.06 DS | 543.66 US<br>543.26 DS | 1.70 | 20 | 5.8 | Yes |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.25 Head Losses, Discharges and Velocities at Structures for the 2050, 200 Year ARI Climate Change Scenario Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.92 | 573.40 | 563.67 | 564.41 US<br>562.60 DS | 566.16 US<br>564.35 DS | 2.25 | 14 | 5.3 | No |
| COT-2 | 544.66 | 544.55 | 542.94 | 542.46 US<br>542.06 DS | 543.66 US<br>543.26 DS | 1.72 | 15 | 4.3 | Yes |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.26 Head Losses, Discharges and Velocities at Structures for the 2070, 200 Year ARI Climate Change Scenario Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.95 | 573.40 | 563.69 | 564.41 US<br>562.60 DS | 566.16 US<br>564.35 DS | 2.26 | 14 | 5.6 | No |
| COT-2 | 544.67 | 544.55 | 542.93 | 542.46 US<br>542.06 DS | 543.66 US<br>543.26 DS | 1.74 | 15 | 4.4 | Yes |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.27 Head Losses, Discharges and Velocities at Structures for the 2100, 200 Year ARI Climate Change Scenario Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 565.97 | 573.40 | 563.71 | 564.41 US | 566.16 US | 2.26 | 11 | 4.4 | No |
| | | | | 562.60 DS | 564.35 DS | | | | |
| COT-2 | 544.69 | 544.55 | 542.98 | 542.46 US | 543.66 US | 1.71 | 16 | 4.4 | Yes |
| | | | | 542.06 DS | 543.26 DS | | | | |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.28 Head Losses, Discharges and Velocities at Structures for the 2050, 500 Year ARI Climate Change Scenario Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 566.02 | 573.40 | 563.73 | 564.41 US | 566.16 US | 2.29 | 16 | 6.2 | No |
| | | | | 562.60 DS | 564.35 DS | | | | |
| COT-2 | 544.72 | 544.55 | 542.98 | 542.46 US | 543.66 US | 1.74 | 17 | 5.0 | Yes |
| | | | | 542.06 DS | 543.26 DS | | | | |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

Table B.29 Head Losses, Discharges and Velocities at Structures for the 2070, 500 Year ARI Climate Change Scenario Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 566.04 | 573.40 | 563.77 | 564.41 US<br>562.60 DS | 566.16 US<br>564.35 DS | 2.27 | 17 | 6.5 | No |
| COT-2 | 544.73 | 544.55 | 542.99 | 542.46 US<br>542.06 DS | 543.66 US<br>543.26 DS | 1.74 | 19 | 5.3 | Yes |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

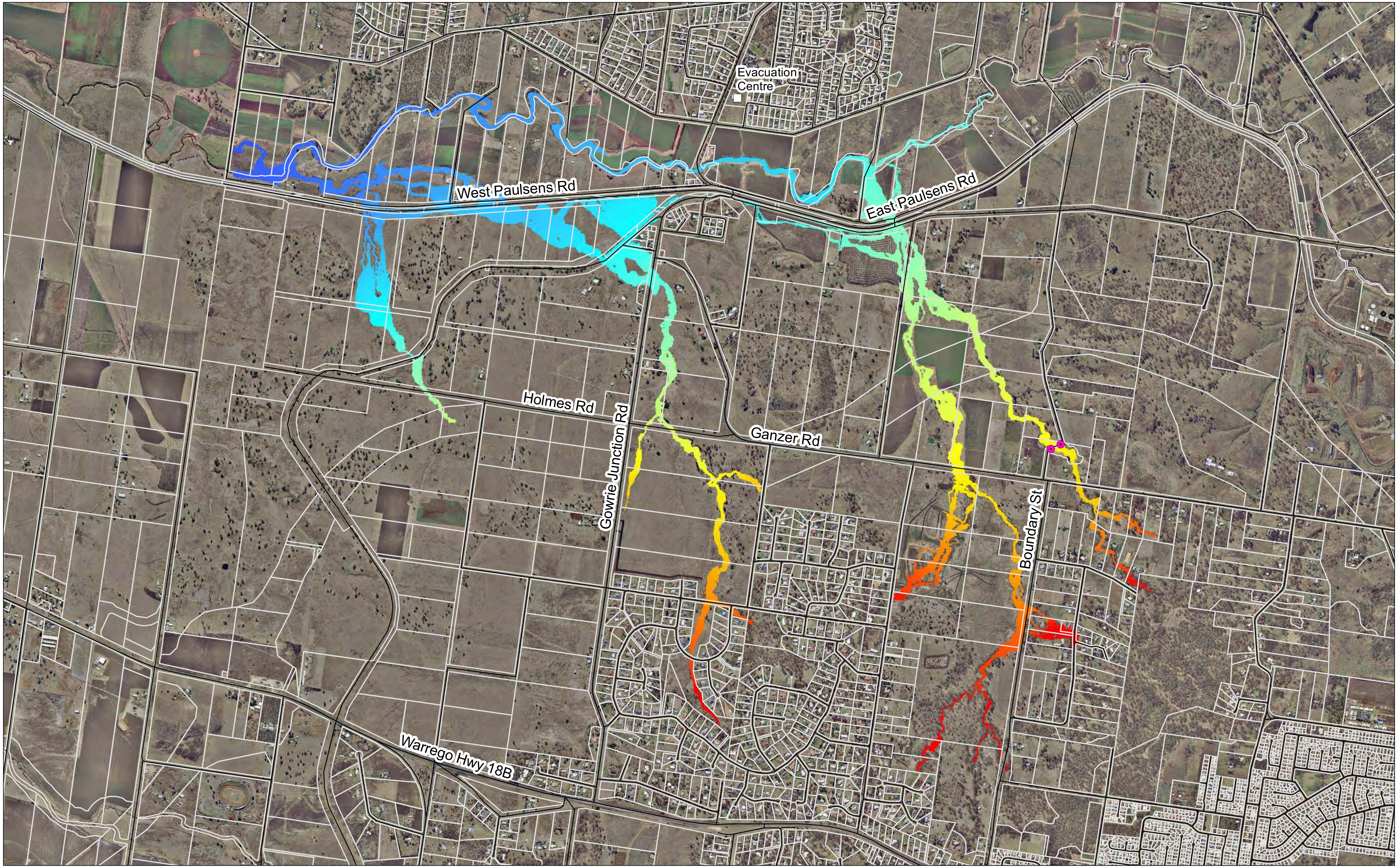
Table B.30 Head Losses, Discharges and Velocities at Structures for the 2100, 500 Year ARI Climate Change Scenario Event

| Structure | Upstream Water Level (mAHD) | Road Level (mAHD) | Downstream Water Level (mAHD) | Structure Invert (mAHD) | Structure Obvert (mAHD) | Head Loss[1] (m) | Peak Discharge[2] ($m^3/s$) | Peak Velocity[3] (m/s) | Road Overtopped (Y/N) |
|---|---|---|---|---|---|---|---|---|---|
| COT-1 | 566.07 | 573.40 | 563.79 | 564.41 US<br>562.60 DS | 566.16 US<br>564.35 DS | 2.28 | 14 | 5.6 | No |
| COT-2 | 544.75 | 544.55 | 543.01 | 542.46 US<br>542.06 DS | 543.66 US<br>543.26 DS | 1.74 | 20 | 5.6 | Yes |

1. The head loss is calculated as the difference between peak water levels at the upstream and downstream MIKE 21 cross-sections
2. Peak discharge is reported through the culvert
3. Peak velocity is calculated as an average based on peak discharge and culvert cross sectional area.

# APPENDIX C

MODEL VALIDATION MAPPING




TOOWOOMBA REGIONAL COUNCIL

1:20,000 (at A3)

0 200 400 800
Meters

GDA 1994 MGA Zone 56

N

**Legend**

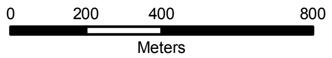
**Surface Elevation [mAHD]**
550
450

Road Centrelines
Cadastre
Emergency Services

**2011 Validation Data**
Known Flooding



**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**January 2011**
**Water Surface Elevation**

1:20,000 (at A3)

0 200 400 800
Meters

GDA 1994 MGA Zone 56

N

**Legend**

**Peak Water Depth (m)**

0 - 0.25, 0.25 - 0.5, 0.5 - 1, 1 - 1.5, 1.5 - 2, 2 - 2.5, 2.5 - 3, 3 - 3.5, 3.5 - 4, 4 - 4.5, 4.5 - 5, >5

Road Centrelines

Cadastre

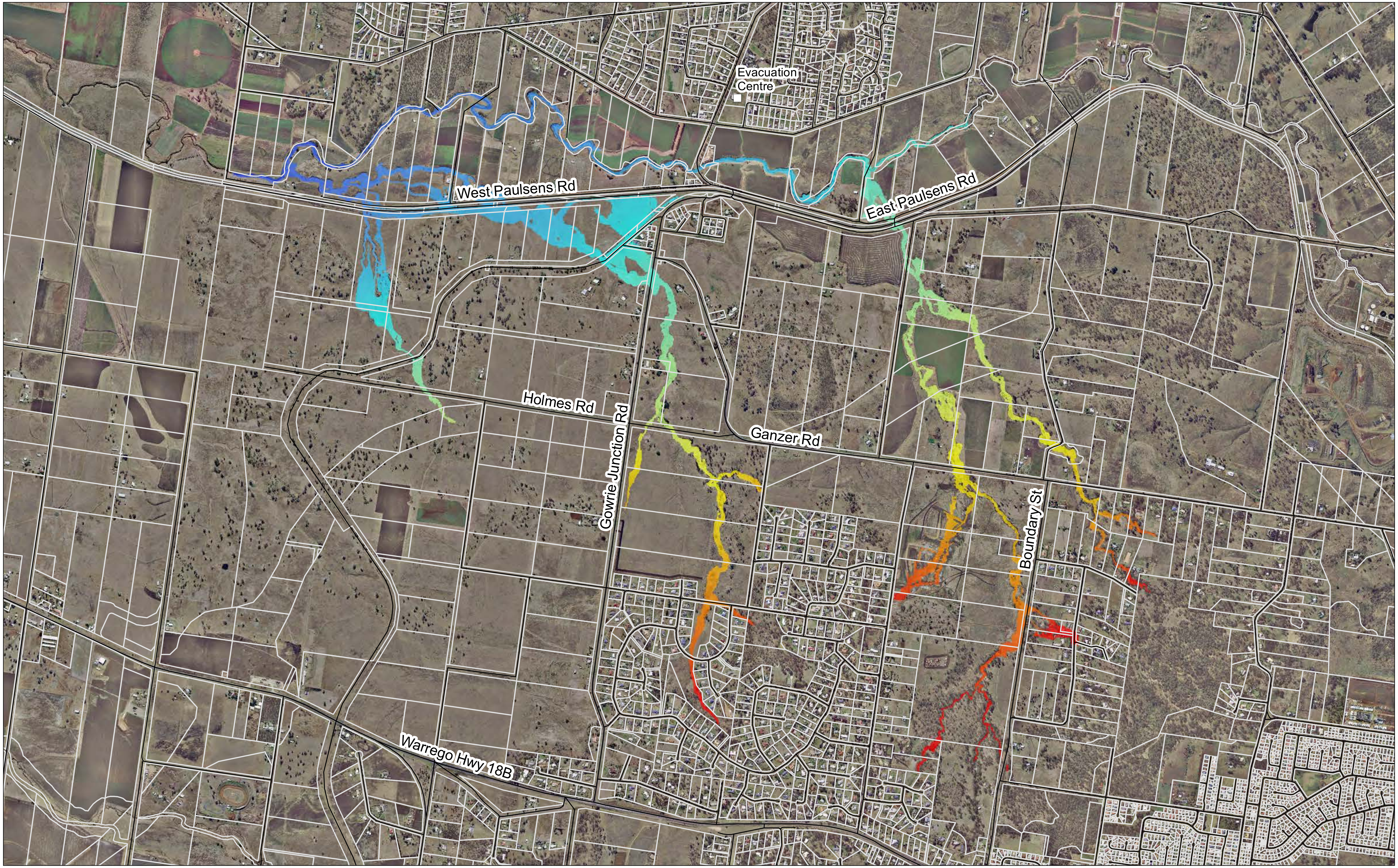
Emergency Services



**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**January 2011**
**Water Depth**

# APPENDIX D

DESIGN EVENT MAPPING




1:20,000 (at A3)

0 200 400 800
Meters

GDA 1994 MGA Zone 56

N

**Legend**

**Surface Elevation [mAHD]**
550
450

Road Centrelines
Cadastre
Emergency Services

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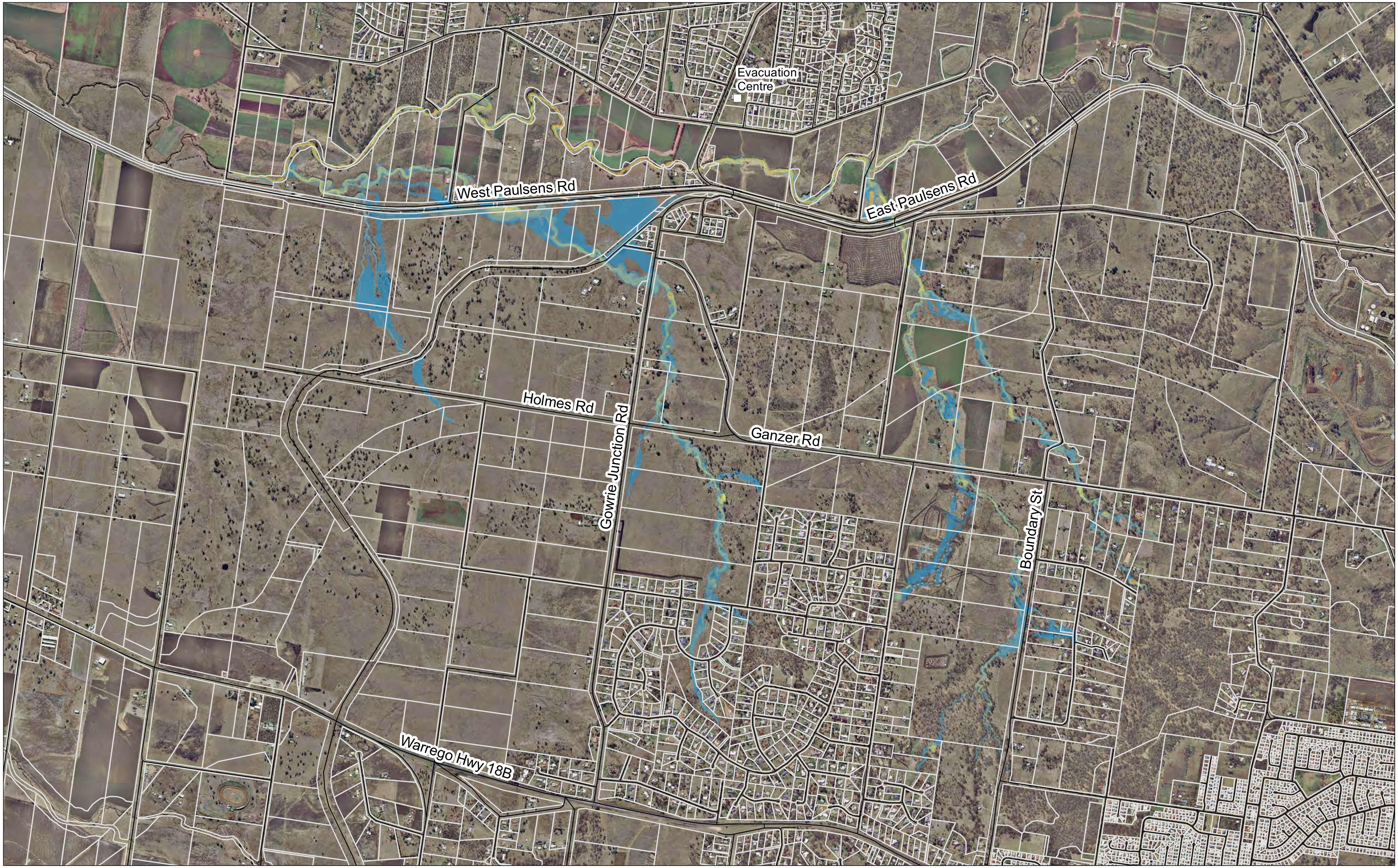
**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**10 Year ARI Event**
**Water Surface Elevation**




TOOWOOMBA REGIONAL COUNCIL

1:20,000 (at A3)

0 200 400 800

Meters

GDA 1994 MGA Zone 56

N

**Legend**

**Peak Water Depth (m)**

0 - 0.25, 0.25 - 0.5, 0.5 - 1, 1 - 1.5, 1.5 - 2, 2 - 2.5, 2.5 - 3, 3 - 3.5, 3.5 - 4, 4 - 4.5, 4.5 - 5, >5

Road Centrelines

Cadastre

Emergency Services

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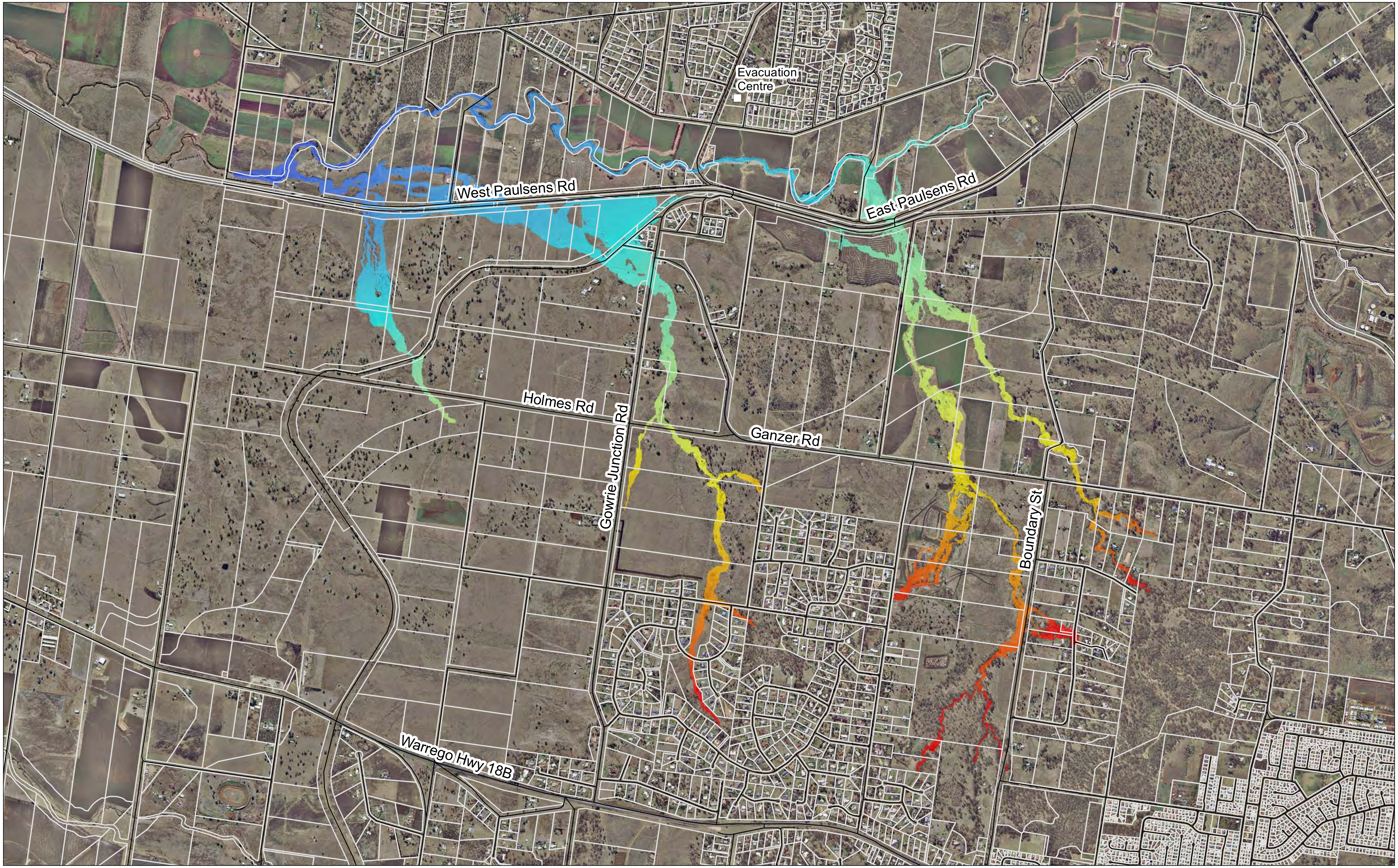
**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**10 Year ARI Event**
**Peak Flood Depths**




1:20,000 (at A3)

0 200 400 800

Meters

GDA 1994 MGA Zone 56

N

TOOWOOMBA REGIONAL COUNCIL

**Legend**

**Surface Elevation [mAHD]**

550

450

Road Centrelines

Cadastre

Emergency Services

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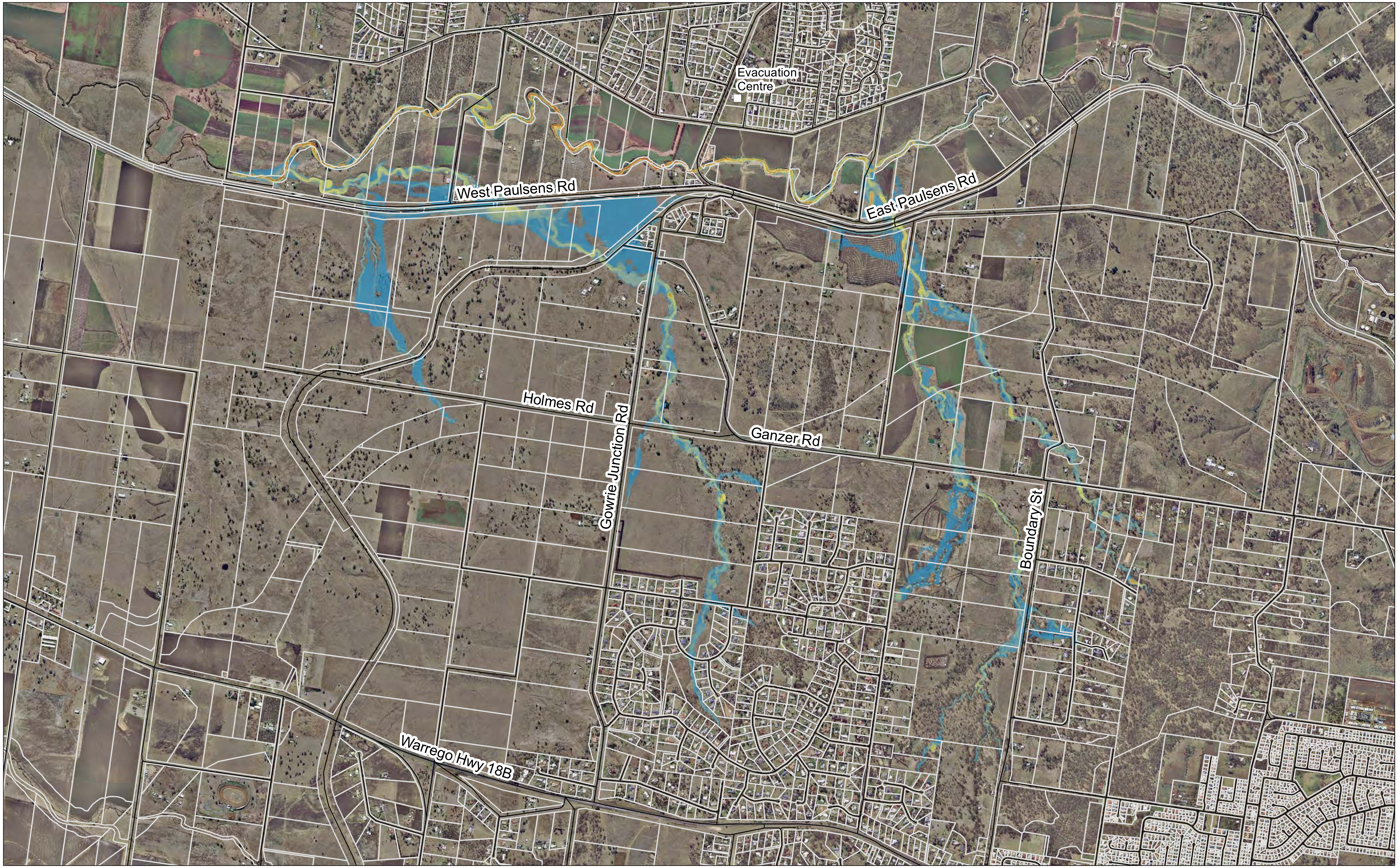
**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**50 Year ARI Event**
**Water Surface Elevation**




TOOWOOMBA REGIONAL COUNCIL

1:20,000 (at A3)

0 200 400 800

Meters

GDA 1994 MGA Zone 56

N

**Legend**

**Peak Water Depth (m)**

0 - 0.25, 0.25 - 0.5, 0.5 - 1, 1 - 1.5, 1.5 - 2, 2 - 2.5, 2.5 - 3, 3 - 3.5, 3.5 - 4, 4 - 4.5, 4.5 - 5, >5

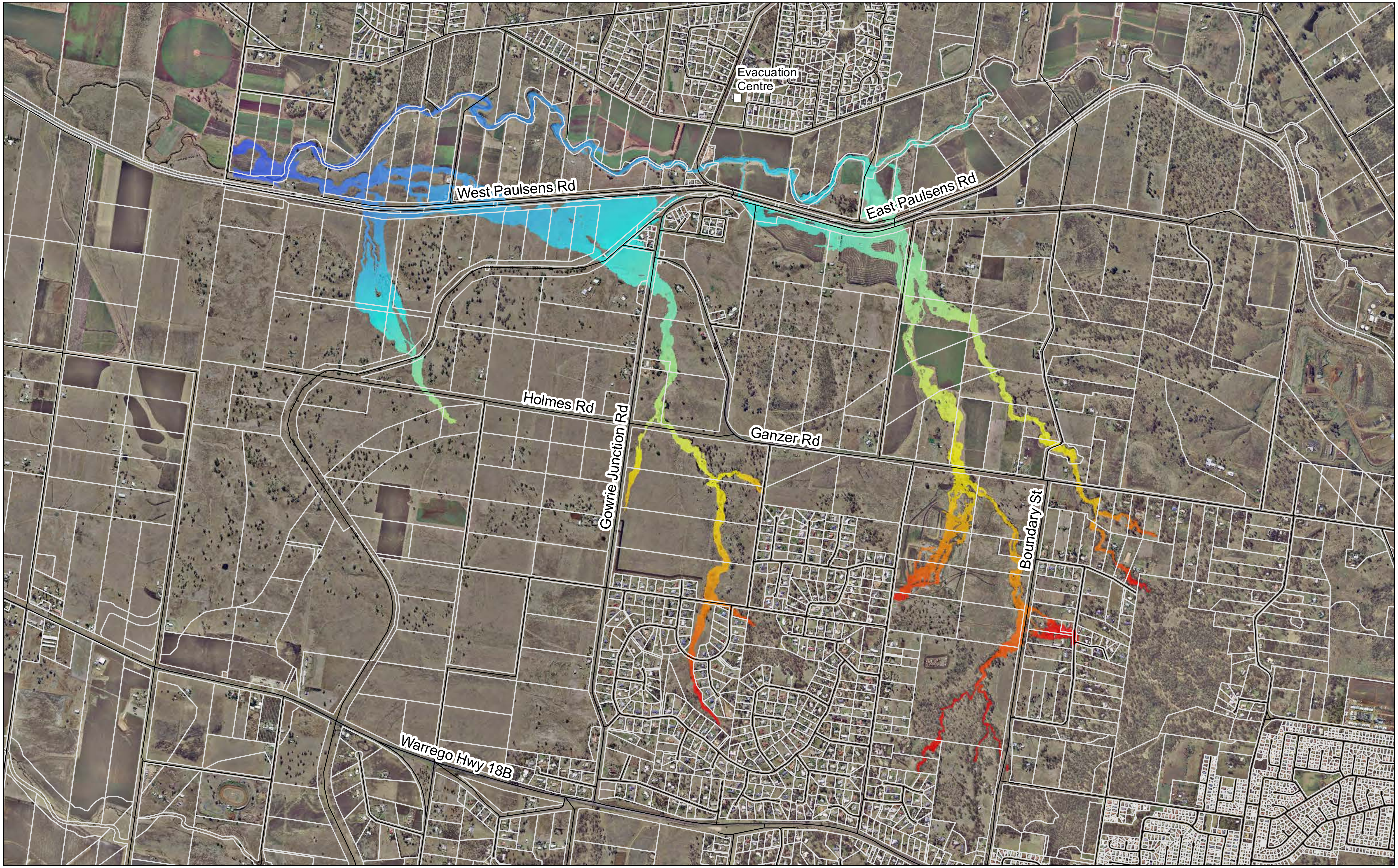
Road Centrelines

Cadastre

Emergency Services



**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**50 Year ARI Event**
**Peak Flood Depths**




TOOWOOMBA REGIONAL COUNCIL

1:20,000 (at A3)

0 200 400 800
Meters

GDA 1994 MGA Zone 56

N

**Legend**

**Surface Elevation [mAHD]**

550

450

Road Centrelines

Cadastre

Emergency Services

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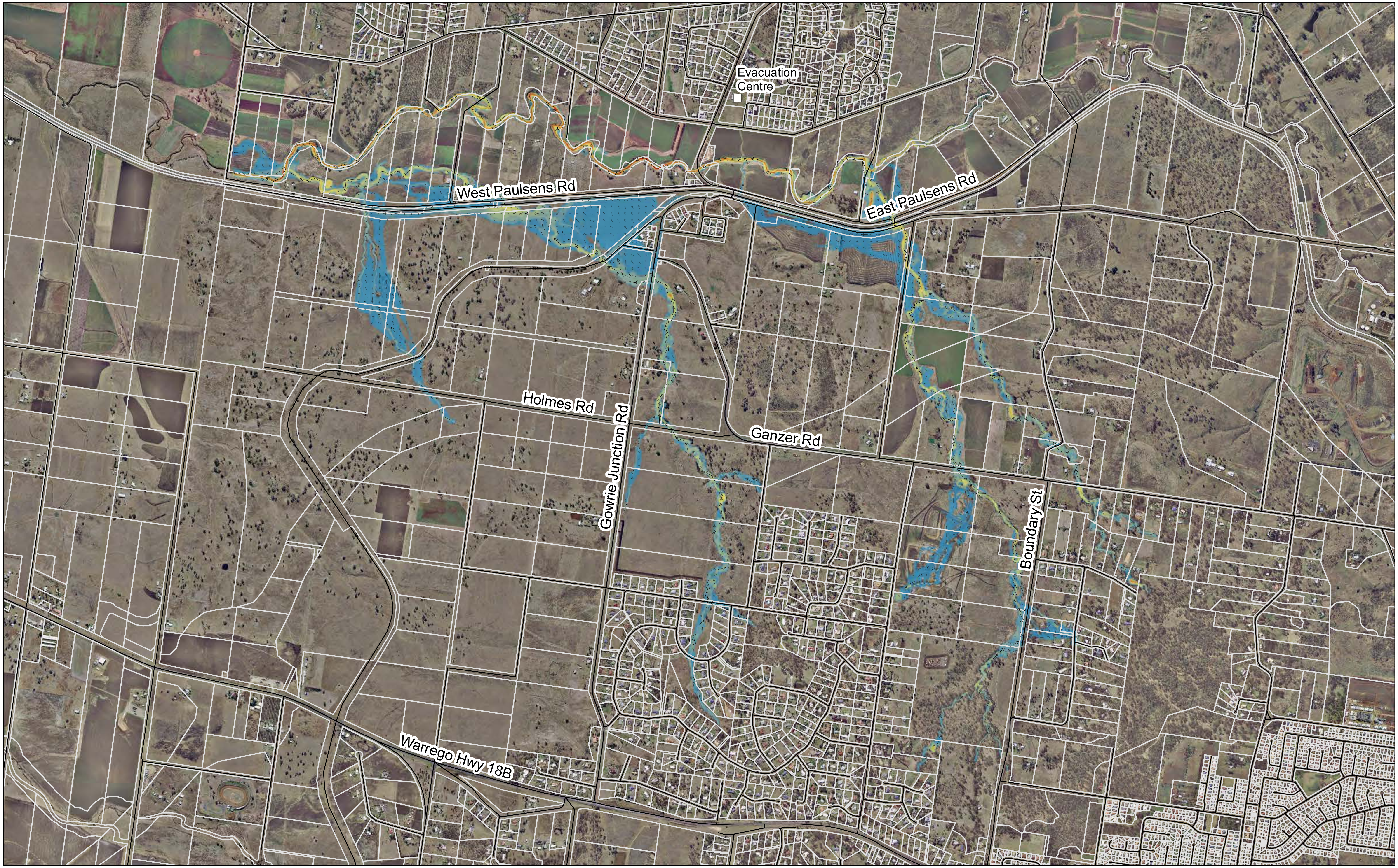
**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**100 Year ARI Event**
**Water Surface Elevation**




TOOWOOMBA REGIONAL COUNCIL

1:20,000 (at A3)

0 200 400 800
Meters

GDA 1994 MGA Zone 56

N

**Legend**

**Peak Water Depth (m)**

0 - 0.25, 0.25 - 0.5, 0.5 - 1, 1 - 1.5, 1.5 - 2, 2 - 2.5, 2.5 - 3, 3 - 3.5, 3.5 - 4, 4 - 4.5, 4.5 - 5, >5

Road Centrelines

Cadastre

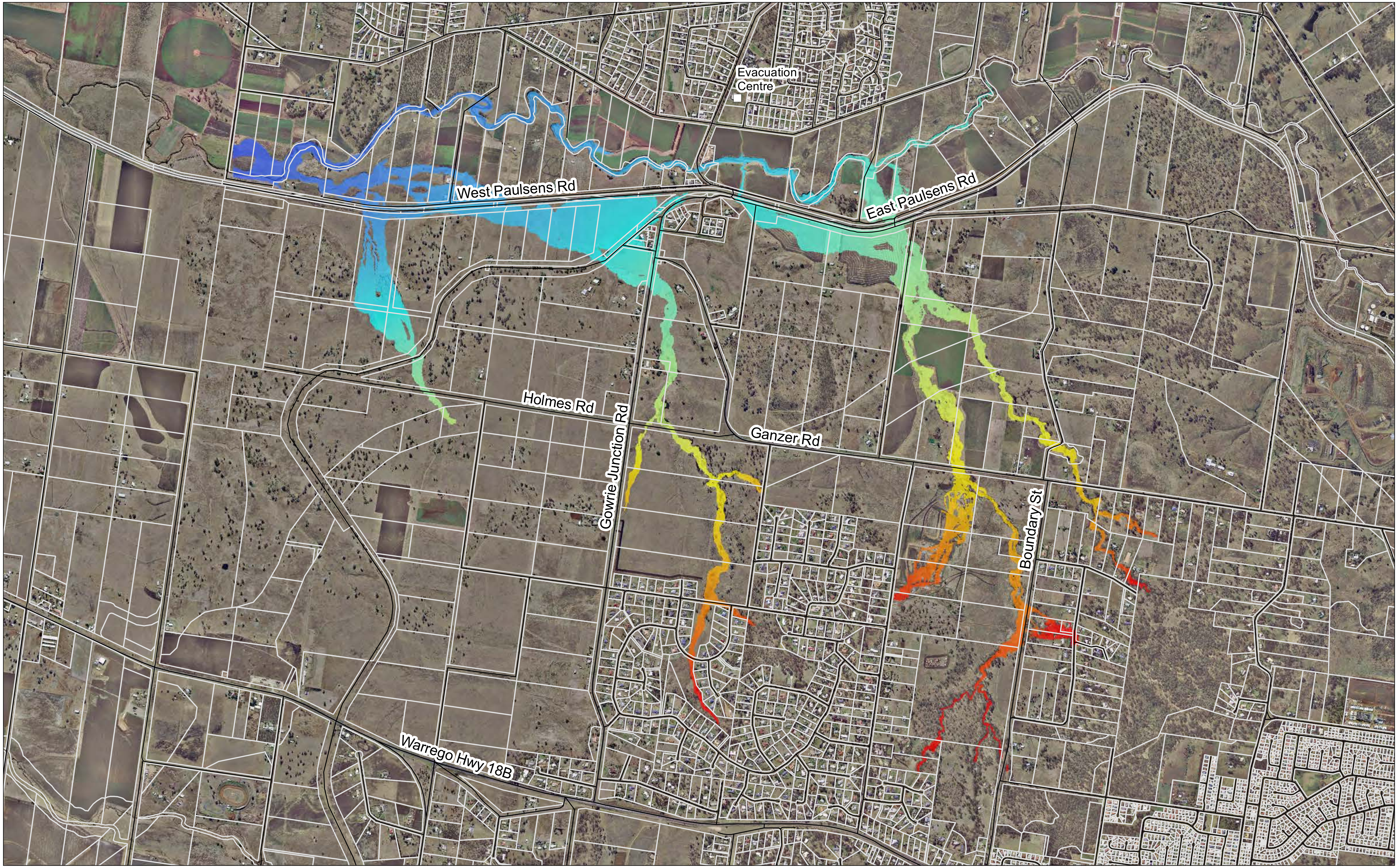
Emergency Services

**Velocity (m/s)**

0 - 1, 1 - 2, >2



**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**100 Year ARI Event**
**Peak Flood Depths and Velocities**




TOOWOOMBA REGIONAL COUNCIL

1:20,000 (at A3)

0 200 400 800
Meters

N

GDA 1994 MGA Zone 56

**Legend**

**Surface Elevation [mAHD]**
550
450

Road Centrelines
Cadastre
Emergency Services

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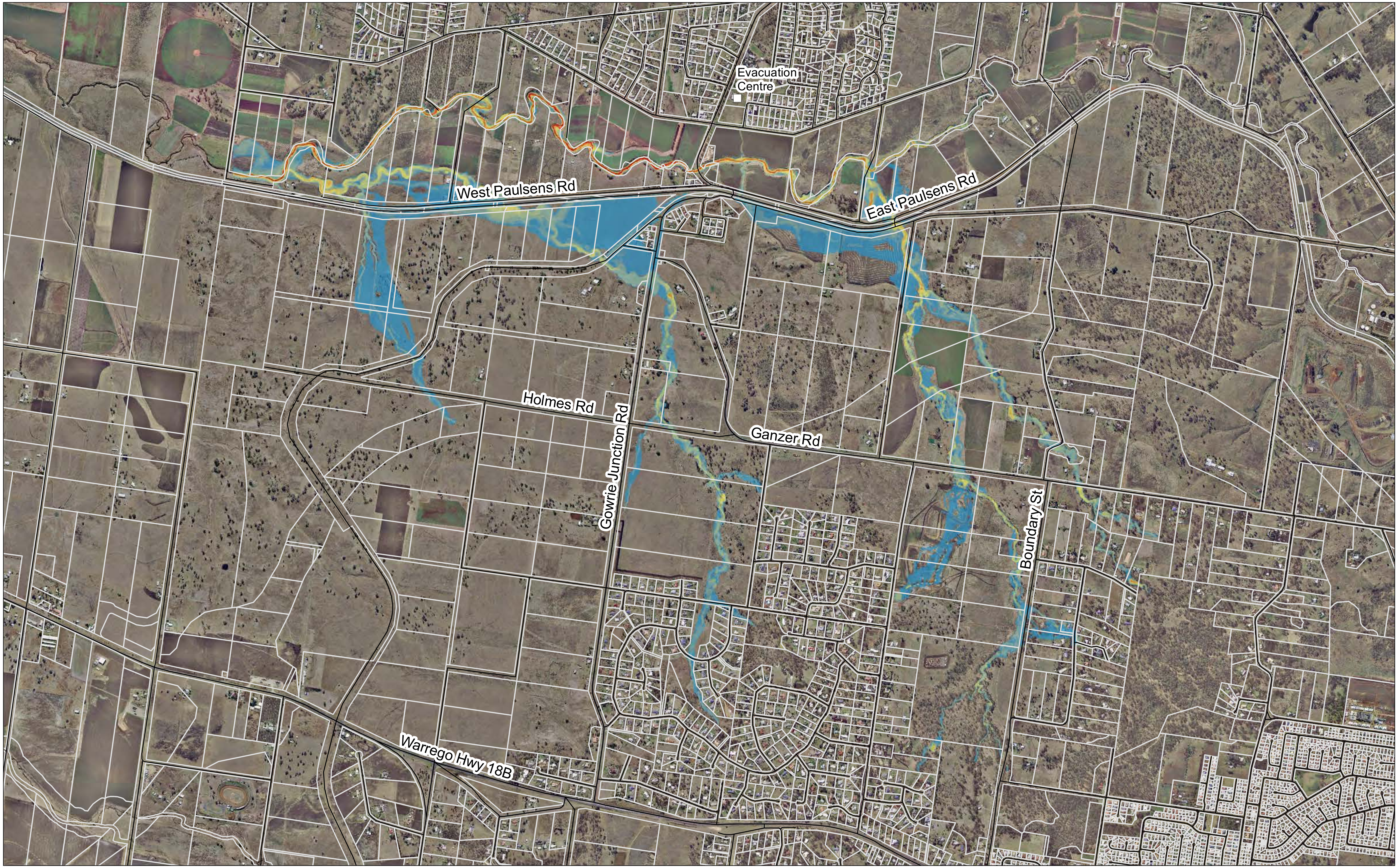
**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**200 Year ARI Event**
**Water Surface Elevation**




TOOWOOMBA REGIONAL COUNCIL

1:20,000 (at A3)

0 200 400 800

Meters

GDA 1994 MGA Zone 56

N

**Legend**

**Peak Water Depth (m)**

0 - 0.25, 0.25 - 0.5, 0.5 - 1, 1 - 1.5, 1.5 - 2, 2 - 2.5, 2.5 - 3, 3 - 3.5, 3.5 - 4, 4 - 4.5, 4.5 - 5, >5

Road Centrelines

Cadastre

Emergency Services

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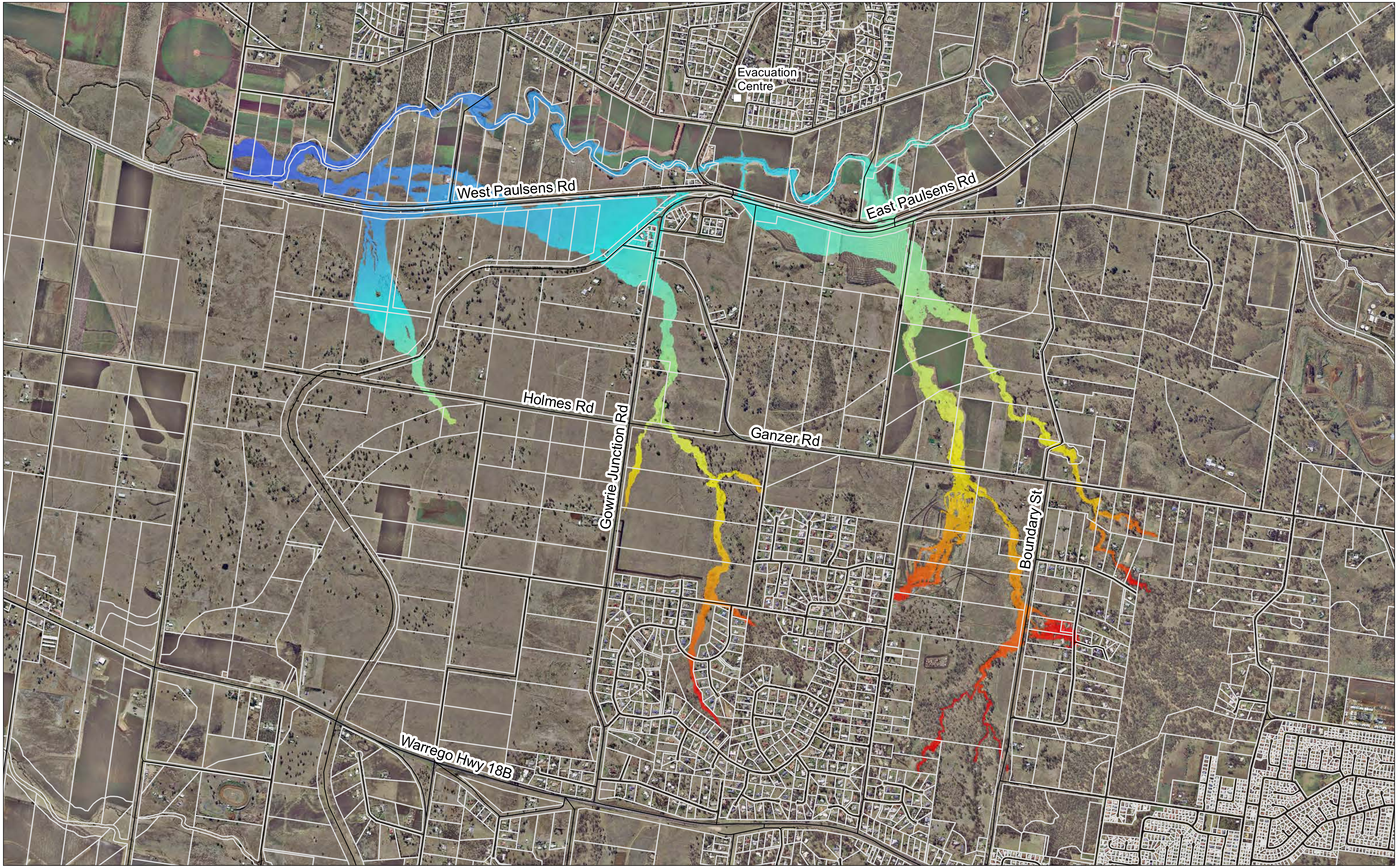
**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**200 Year ARI Event**
**Peak Flood Depths**




1:20,000 (at A3)

0 200 400 800
Meters

GDA 1994 MGA Zone 56

N

TOOWOOMBA REGIONAL COUNCIL

**Legend**

**Surface Elevation [mAHD]**

550

450

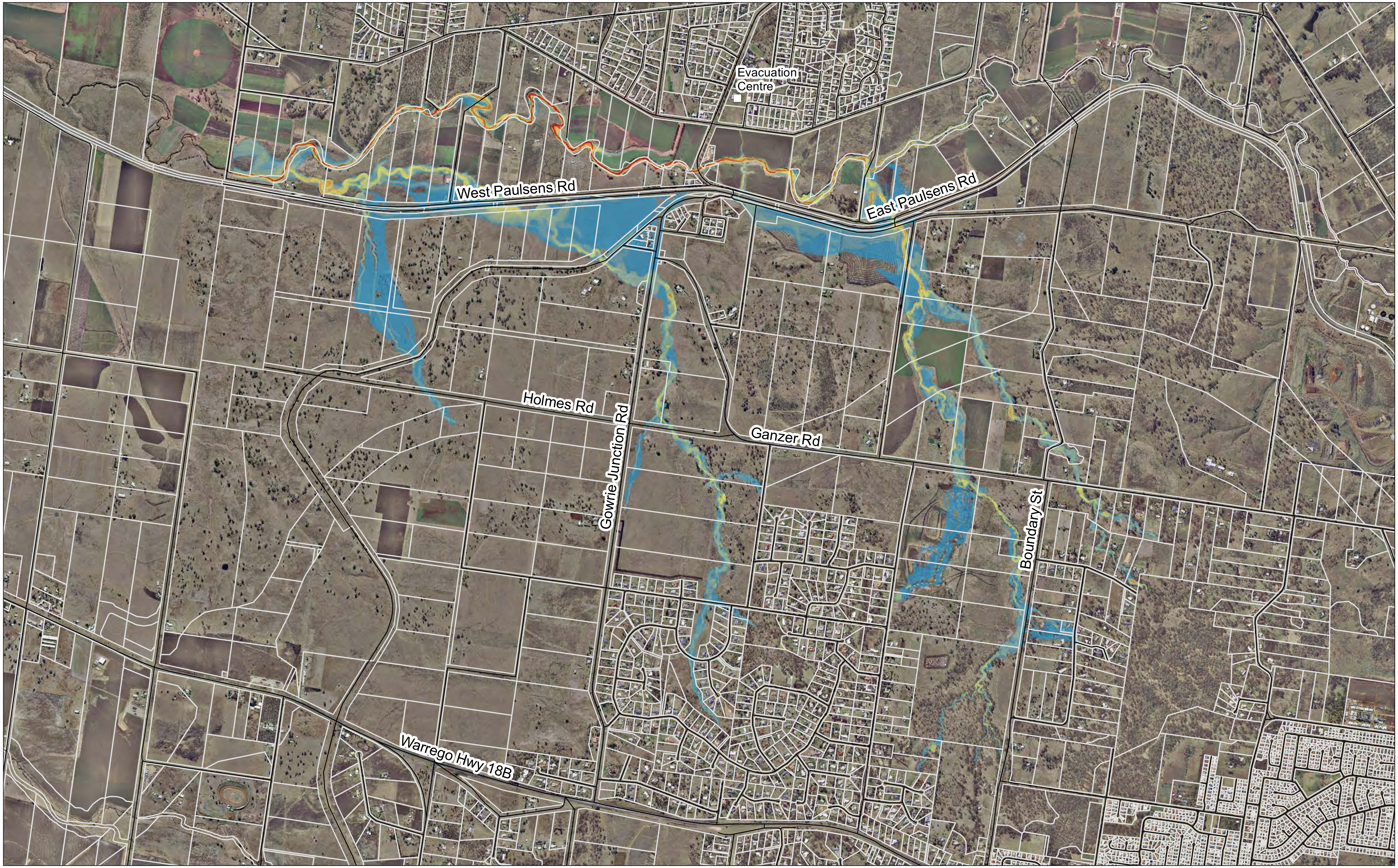
Road Centrelines

Cadastre

Emergency Services



**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**500 Year ARI Event**
**Water Surface Elevation**




TOOWOOMBA REGIONAL COUNCIL

1:20,000 (at A3)

0 200 400 800

Meters

GDA 1994 MGA Zone 56

N

**Legend**

**Peak Water Depth (m)**

0 - 0.25, 0.25 - 0.5, 0.5 - 1, 1 - 1.5, 1.5 - 2, 2 - 2.5, 2.5 - 3, 3 - 3.5, 3.5 - 4, 4 - 4.5, 4.5 - 5, >5

Road Centrelines

Cadastre

Emergency Services

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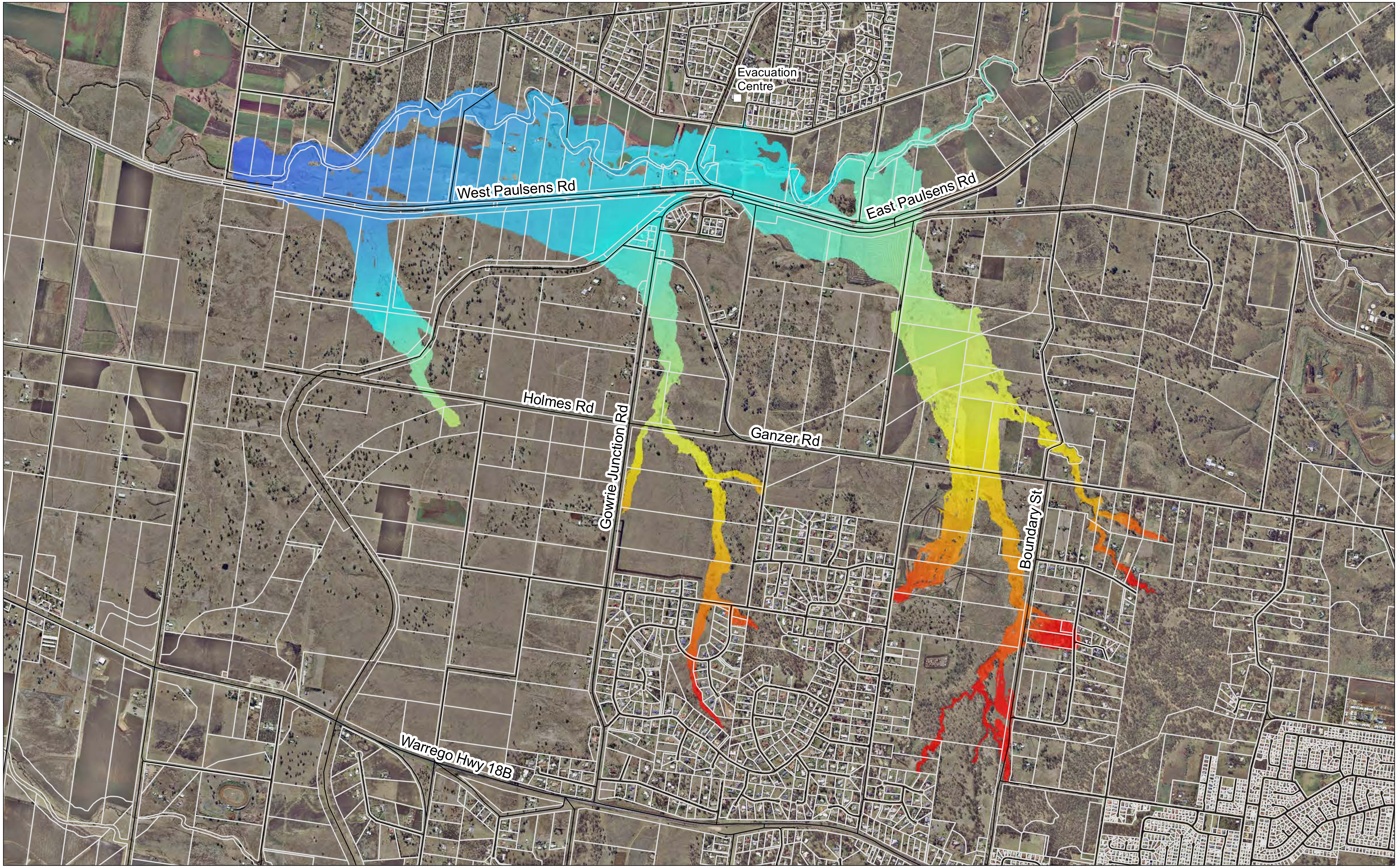
**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**500 Year ARI Event**
**Peak Flood Depths**




TOOWOOMBA REGIONAL COUNCIL

1:20,000 (at A3)

0 200 400 800
Meters

GDA 1994 MGA Zone 56

N

**Legend**

**Surface Elevation [mAHD]**

550

450

Road Centrelines

Cadastre

Emergency Services

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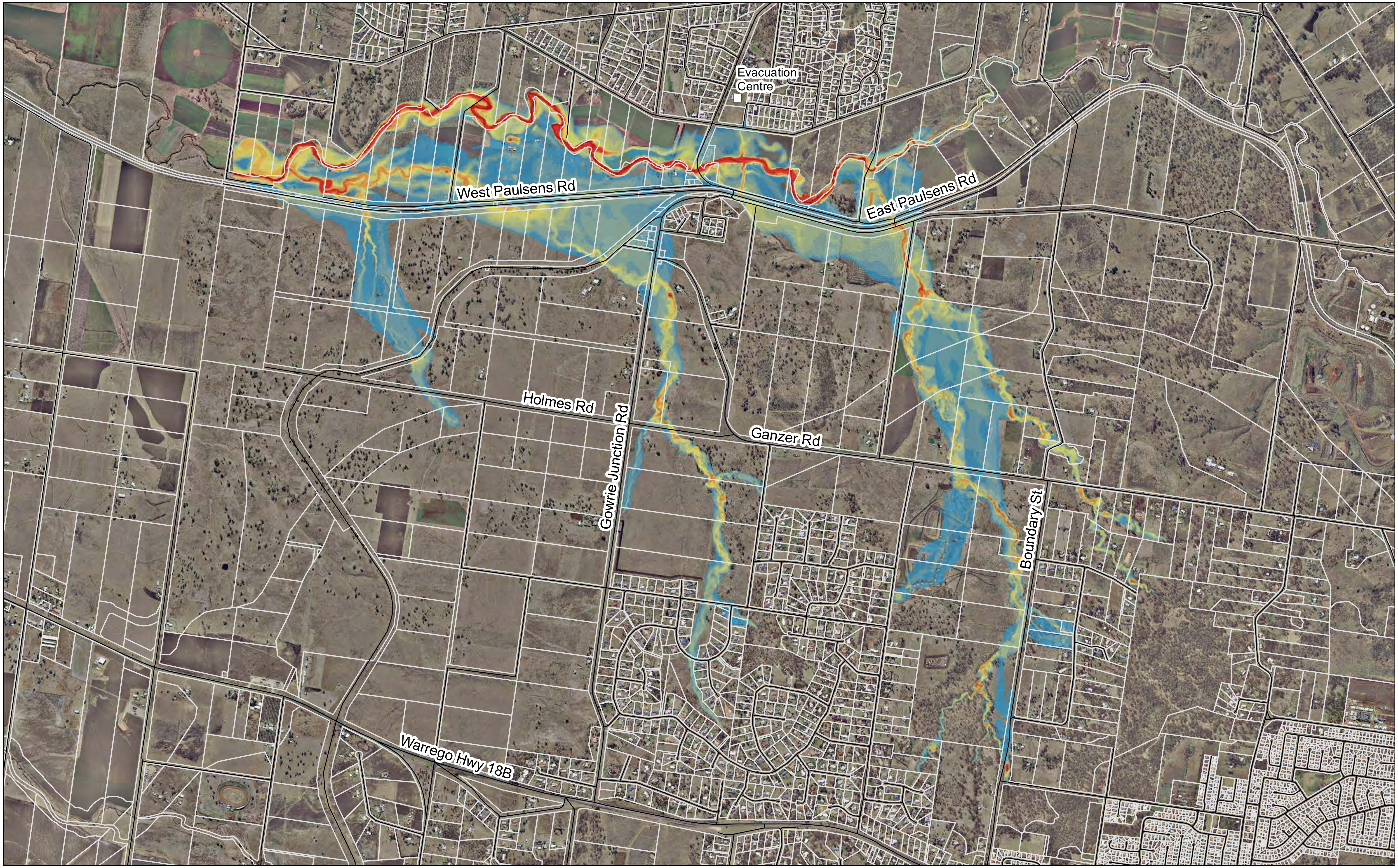
**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**Probable Maximum Flood Event**
**Water Surface Elevation**




TOOWOOMBA REGIONAL COUNCIL

1:20,000 (at A3)

0 200 400 800
Meters

GDA 1994 MGA Zone 56

N

**Legend**

**Peak Water Depth (m)**

0 - 0.25, 0.25 - 0.5, 0.5 - 1, 1 - 1.5, 1.5 - 2, 2 - 2.5, 2.5 - 3, 3 - 3.5, 3.5 - 4, 4 - 4.5, 4.5 - 5, >5

Road Centrelines

Cadastre

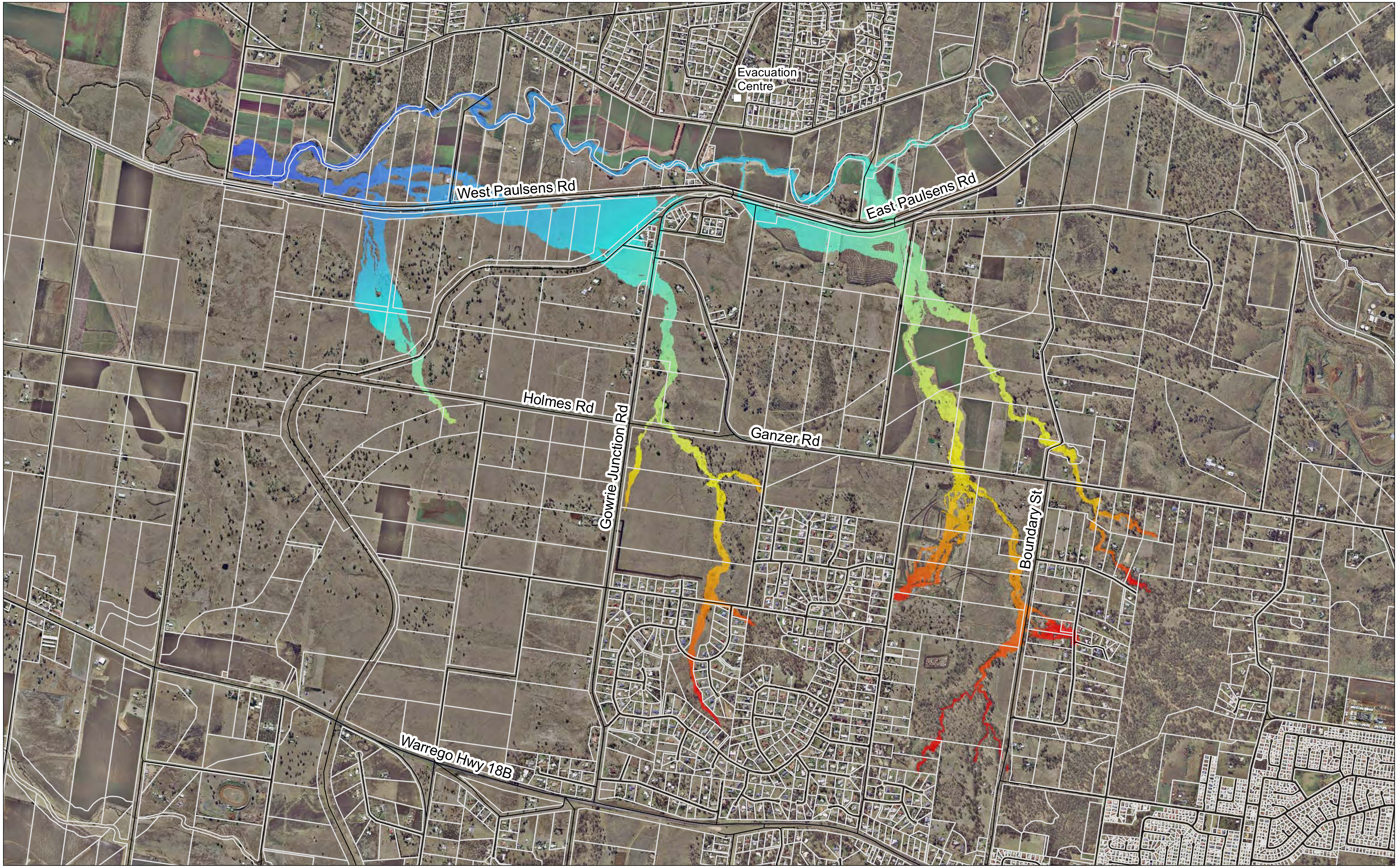
Emergency Services



## SP051 Flood Studies
## Work Package 8 Cotswold Hills
## Probable Maximum Flood Event
## Peak Flood Depths

# APPENDIX E

CLIMATE CHANGE FLOOD MAPPING




TOOWOOMBA REGIONAL COUNCIL

1:20,000 (at A3)

0 200 400 800
Meters

GDA 1994 MGA Zone 56

N

**Legend**

**Surface Elevation [mAHD]**

550

450

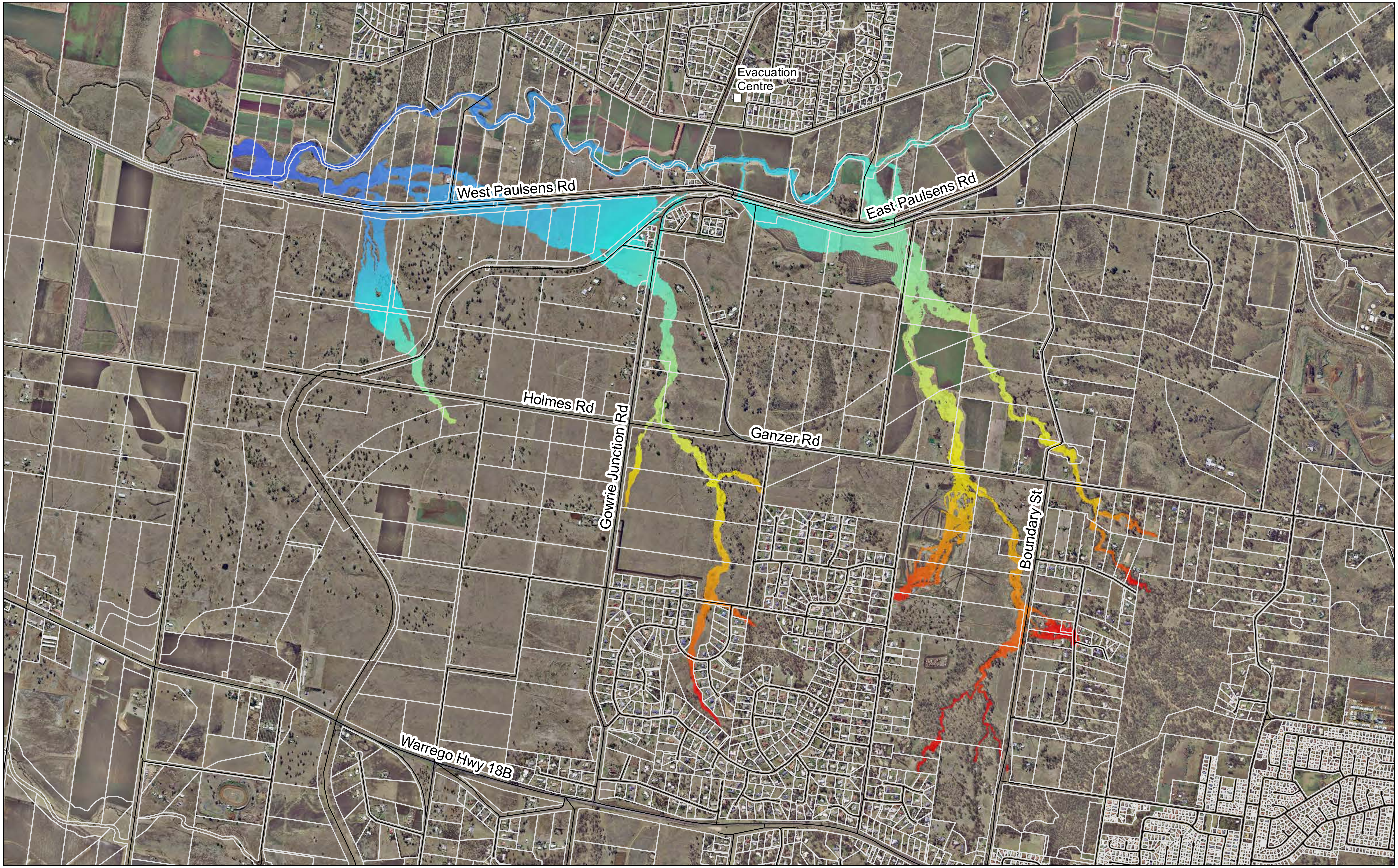
Road Centrelines

Cadastre

Emergency Services



**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**100 Year ARI Event Climate Change 2050**
**Water Surface Elevation**




TOOWOOMBA REGIONAL COUNCIL

1:20,000 (at A3)

0 200 400 800
Meters

GDA 1994 MGA Zone 56

N

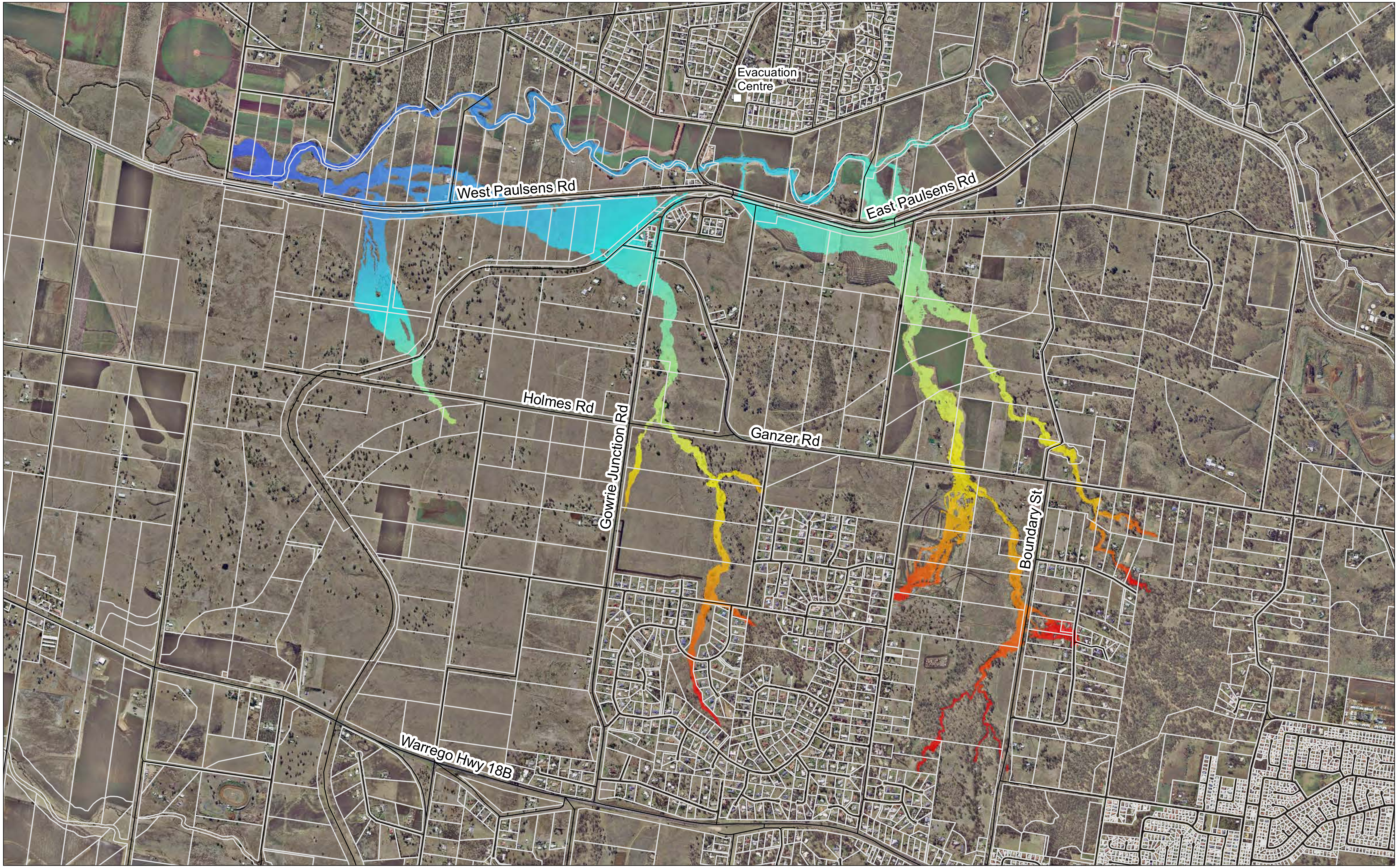
**Legend**

**Surface Elevation [mAHD]**
550
450

Road Centrelines
Cadastre
Emergency Services



## SP051 Flood Studies
## Work Package 8 Cotswold Hills
## 100 Year ARI Event Climate Change 2070
## Water Surface Elevation




1:20,000 (at A3)

0 200 400 800
Meters

GDA 1994 MGA Zone 56

N

TOOWOOMBA REGIONAL COUNCIL

**Legend**

**Surface Elevation [mAHD]**

550

450

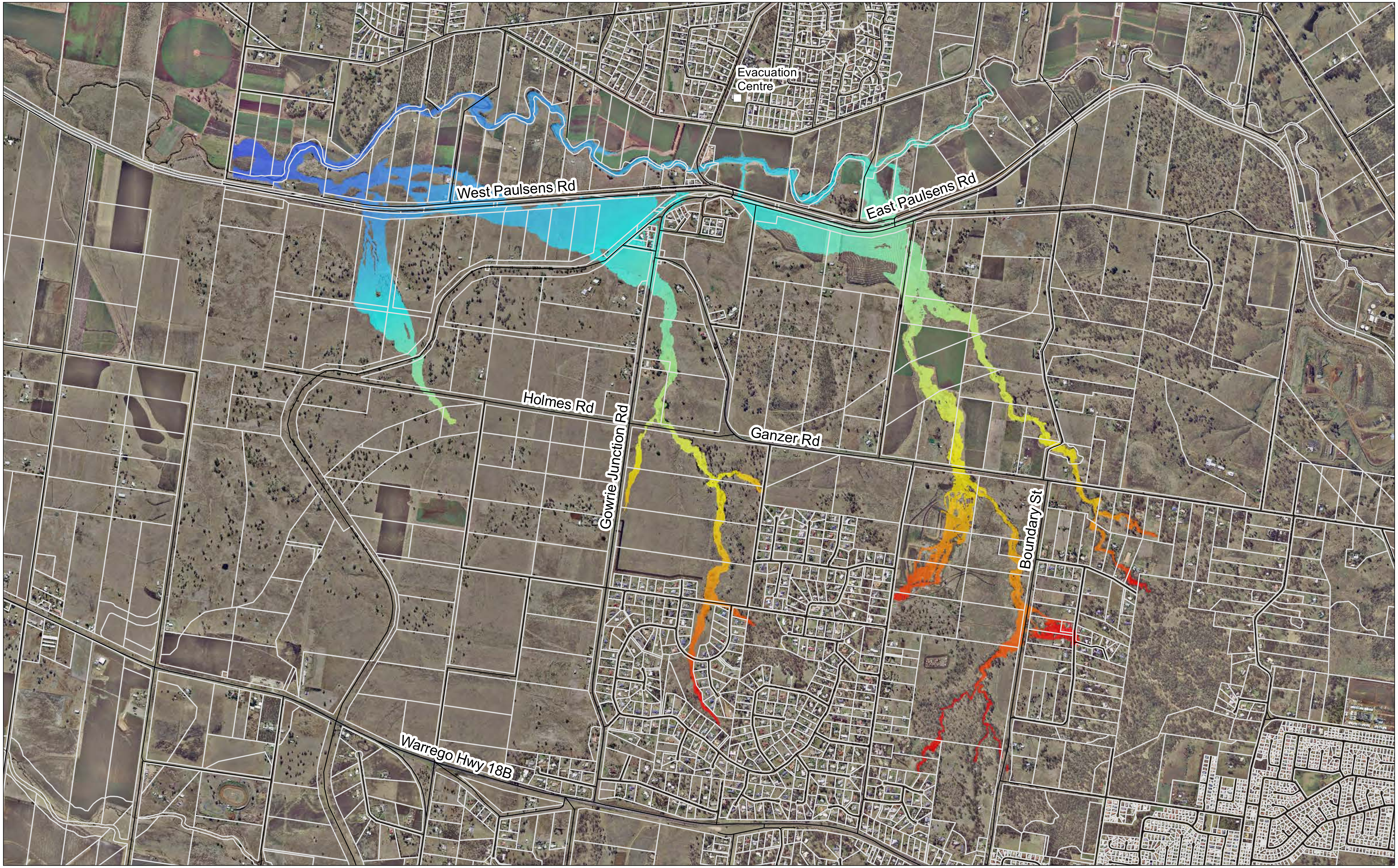
Road Centrelines

Cadastre

Emergency Services



**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**100 Year ARI Event Climate Change 2100**
**Water Surface Elevation**




TOOWOOMBA REGIONAL COUNCIL

1:20,000 (at A3)

0 200 400 800

Meters

GDA 1994 MGA Zone 56

N

**Legend**

**Surface Elevation [mAHD]**

550

450

Road Centrelines

Cadastre

Emergency Services

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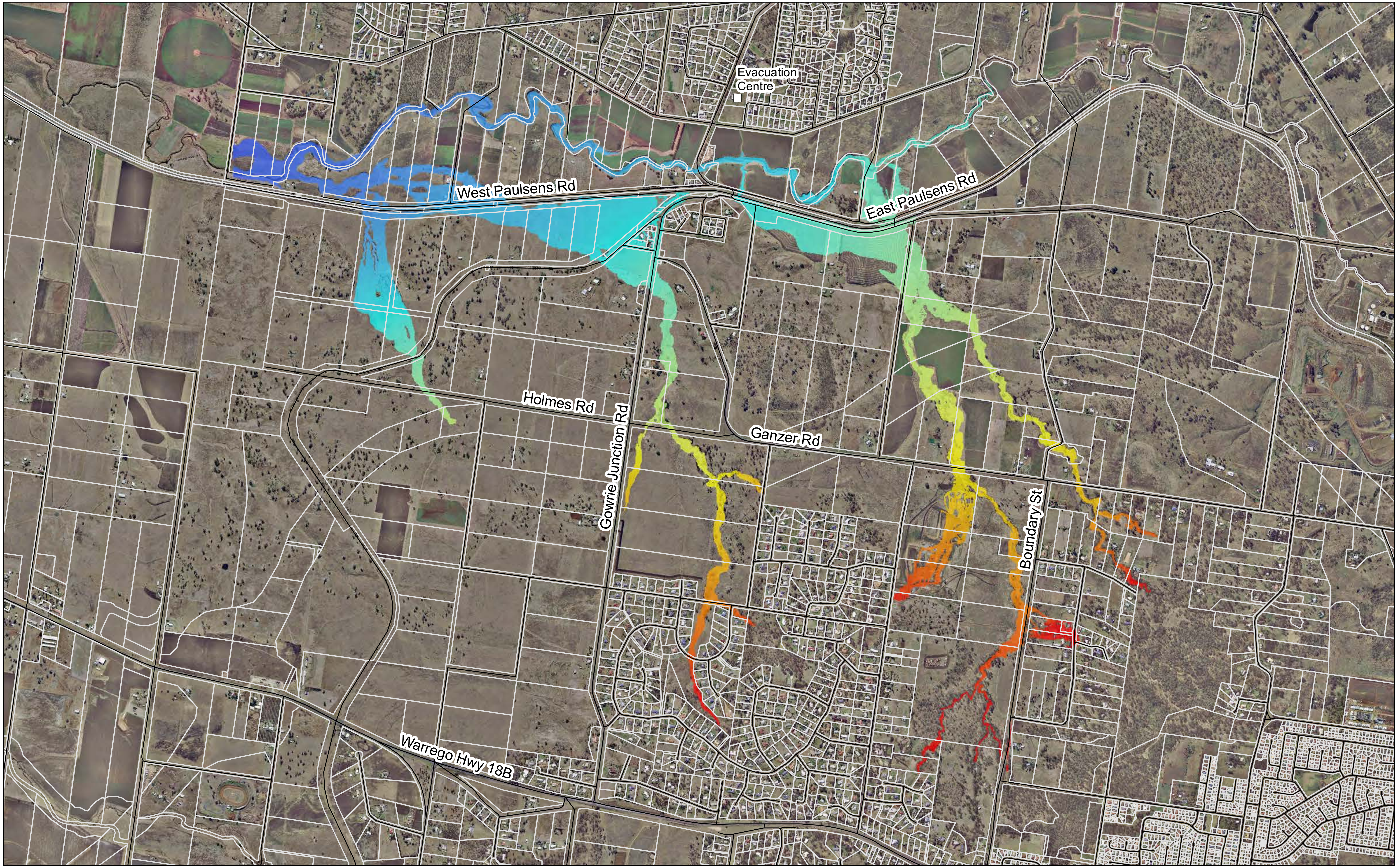
**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**200 Year ARI Event Climate Change 2050**
**Water Surface Elevation**




1:20,000 (at A3)

0 200 400 800

Meters

GDA 1994 MGA Zone 56

N

TOOWOOMBA REGIONAL COUNCIL

**Legend**

**Surface Elevation [mAHD]**

550

450

Road Centrelines

Cadastre

Emergency Services

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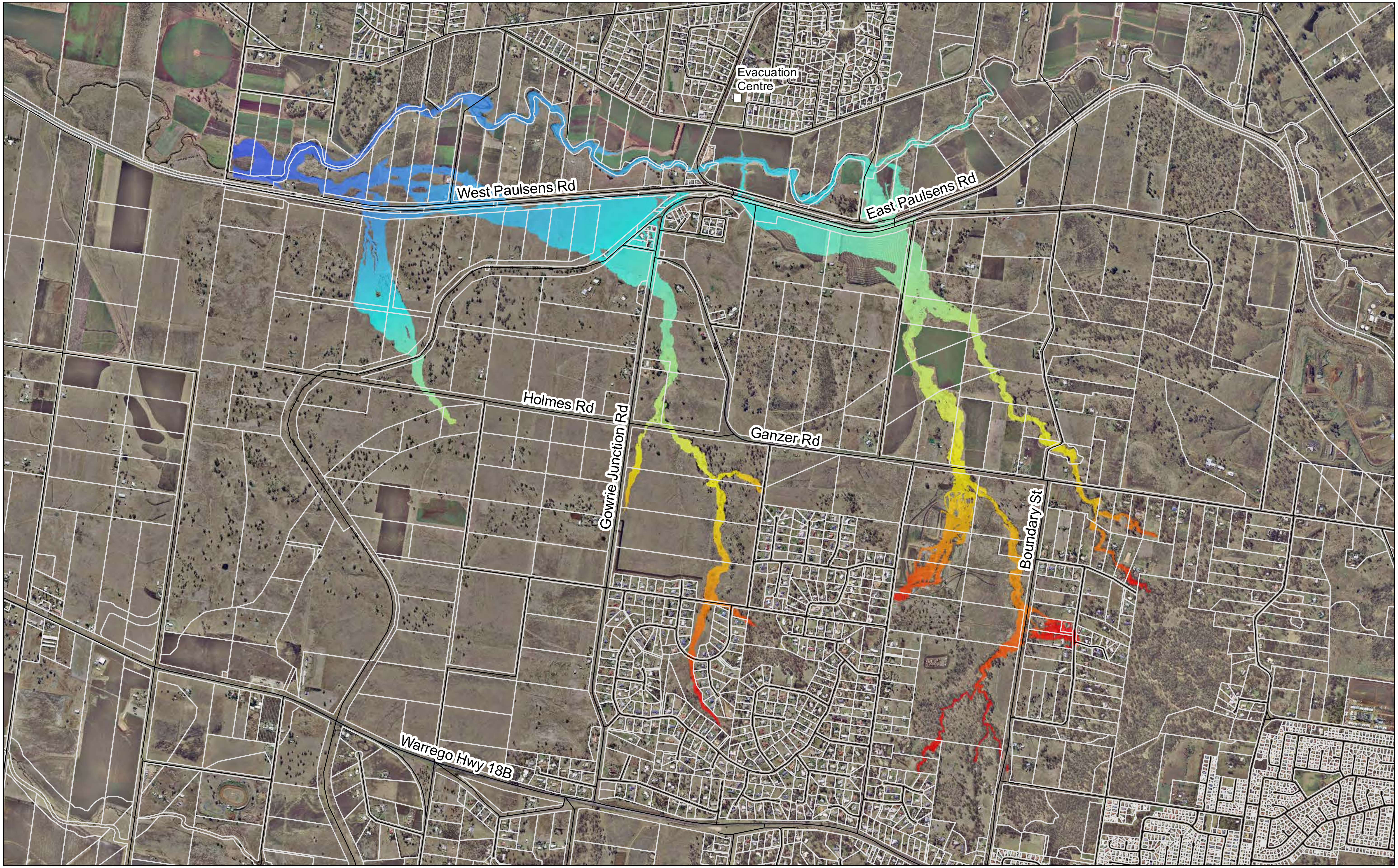
**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**200 Year ARI Event Climate Change 2070**
**Water Surface Elevation**




TOOWOOMBA REGIONAL COUNCIL

1:20,000 (at A3)

0 200 400 800
Meters

GDA 1994 MGA Zone 56

N

**Legend**

**Surface Elevation [mAHD]**
550
450

Road Centrelines
Cadastre
Emergency Services

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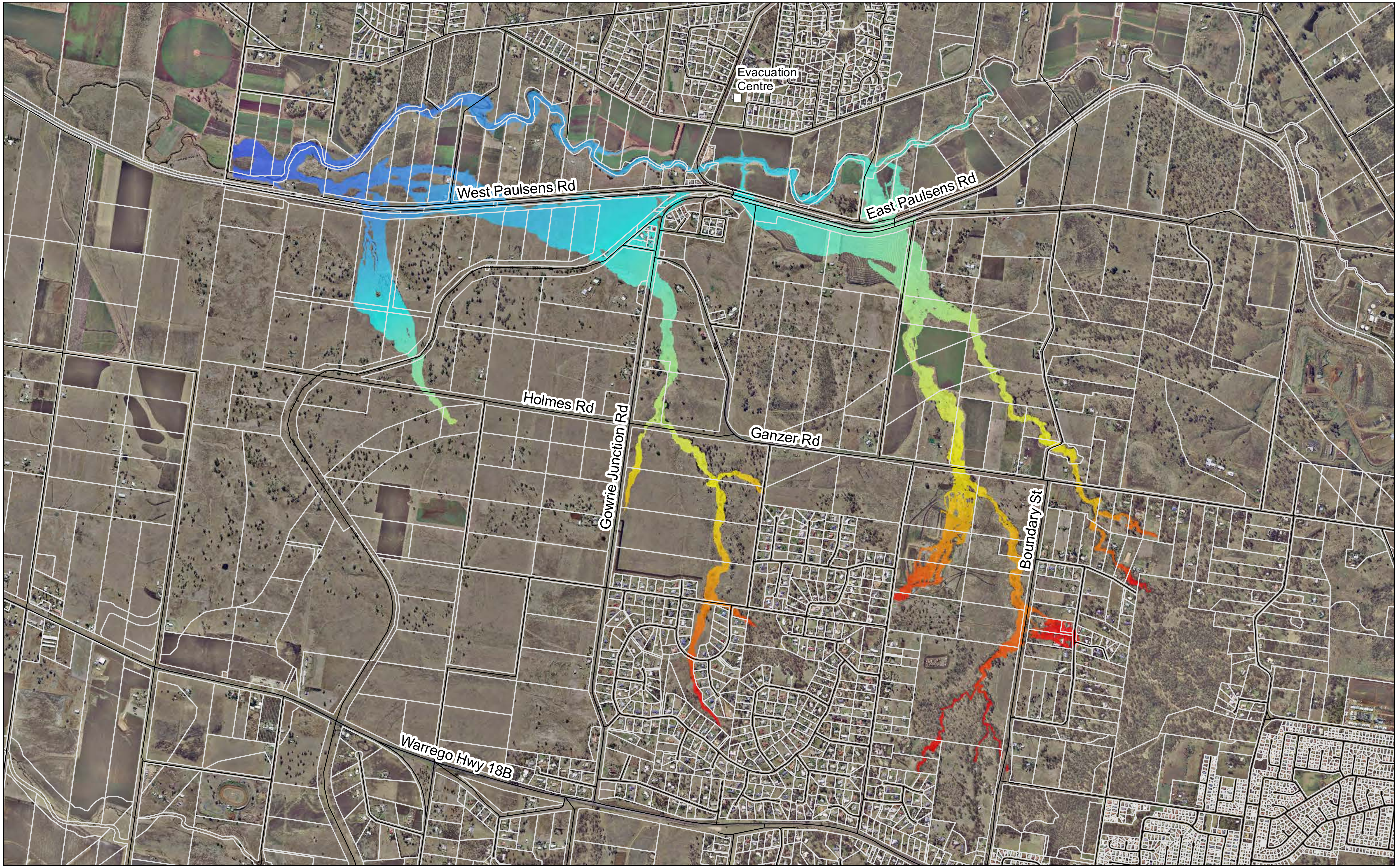
**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**200 Year ARI Event Climate Change 2100**
**Water Surface Elevation**




TOOWOOMBA REGIONAL COUNCIL

1:20,000 (at A3)

0 200 400 800

Meters

GDA 1994 MGA Zone 56

N

**Legend**

**Surface Elevation [mAHD]**

550

450

Road Centrelines

Cadastre

Emergency Services

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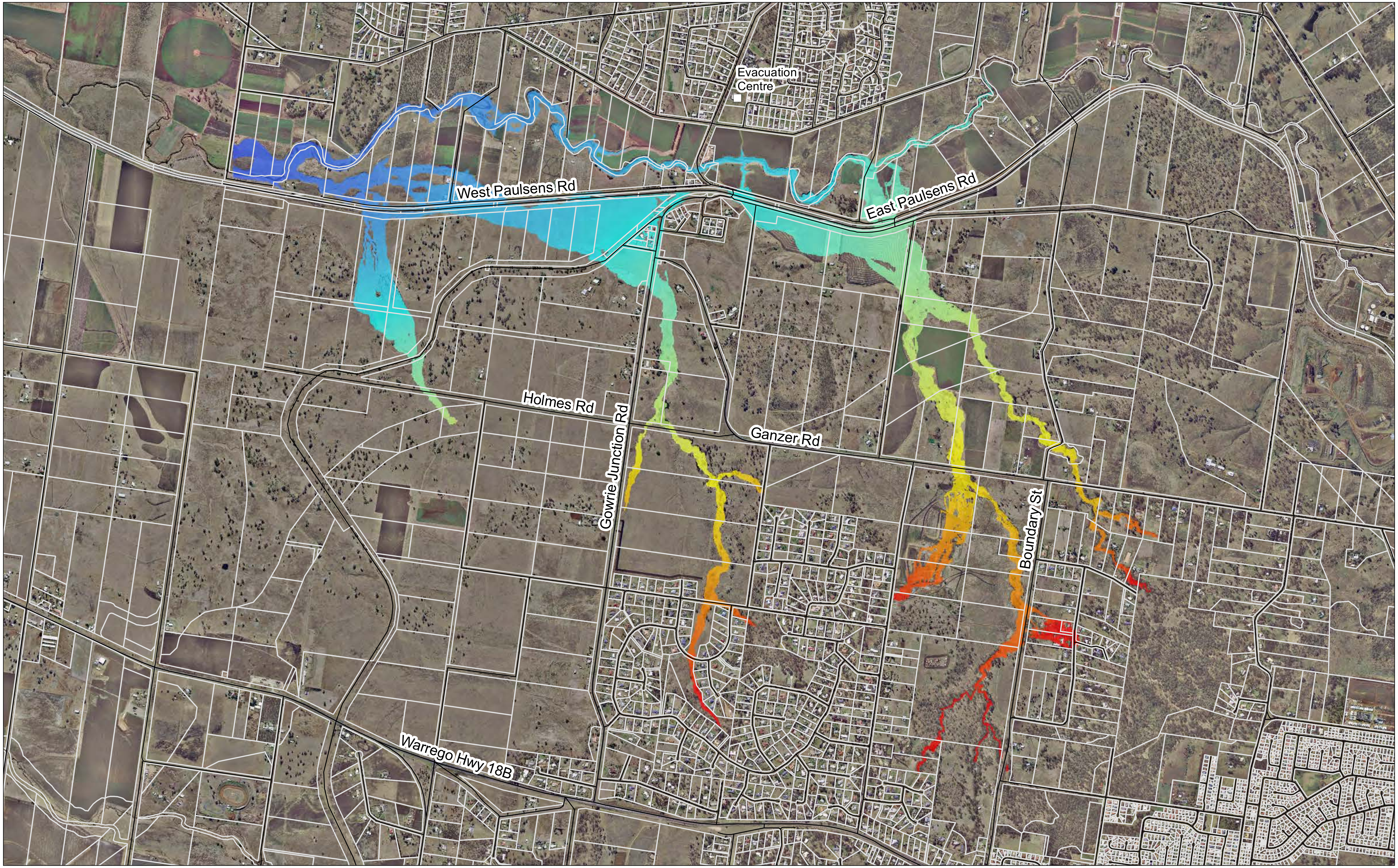
**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**500 Year ARI Event Climate Change 2050**
**Water Surface Elevation**




TOOWOOMBA REGIONAL COUNCIL

1:20,000 (at A3)

0 200 400 800
Meters

GDA 1994 MGA Zone 56

N

**Legend**

**Surface Elevation [mAHD]**
550
450

Road Centrelines
Cadastre
Emergency Services

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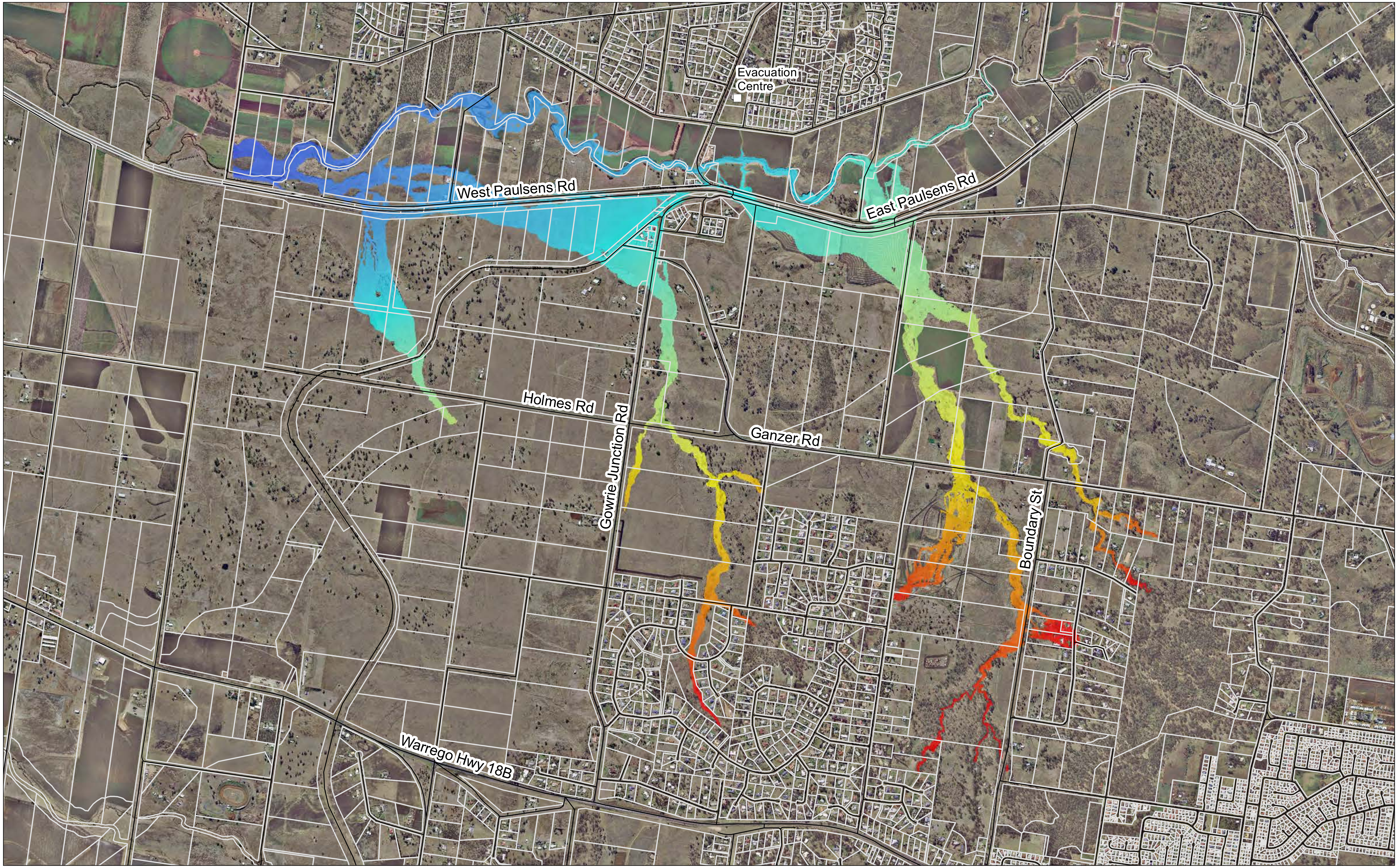
**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**500 Year ARI Event Climate Change 2070**
**Water Surface Elevation**




TOOWOOMBA REGIONAL COUNCIL

1:20,000 (at A3)

0 200 400 800
Meters

GDA 1994 MGA Zone 56

N

**Legend**

**Surface Elevation [mAHD]**

550

450

Road Centrelines

Cadastre

Emergency Services

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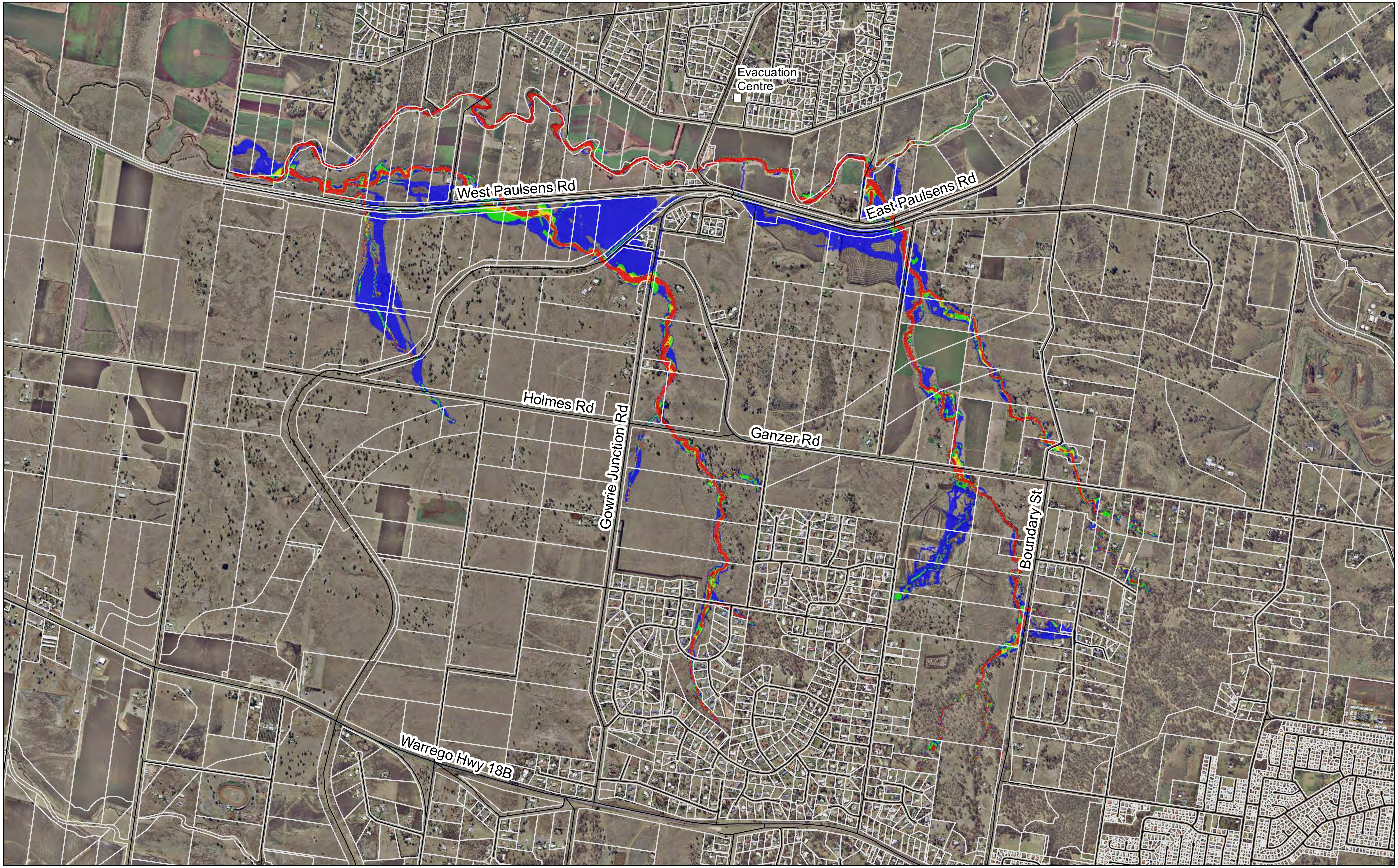
**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**500 Year ARI Event Climate Change 2100**
**Water Surface Elevation**

# APPENDIX F

## HYDRAULIC AND HAZARD CATEGORY MAPPING




TOOWOOMBA REGIONAL COUNCIL

1:20,000 (at A3)

0 200 400 800
Meters

GDA 1994 MGA Zone 56

N

**Legend**

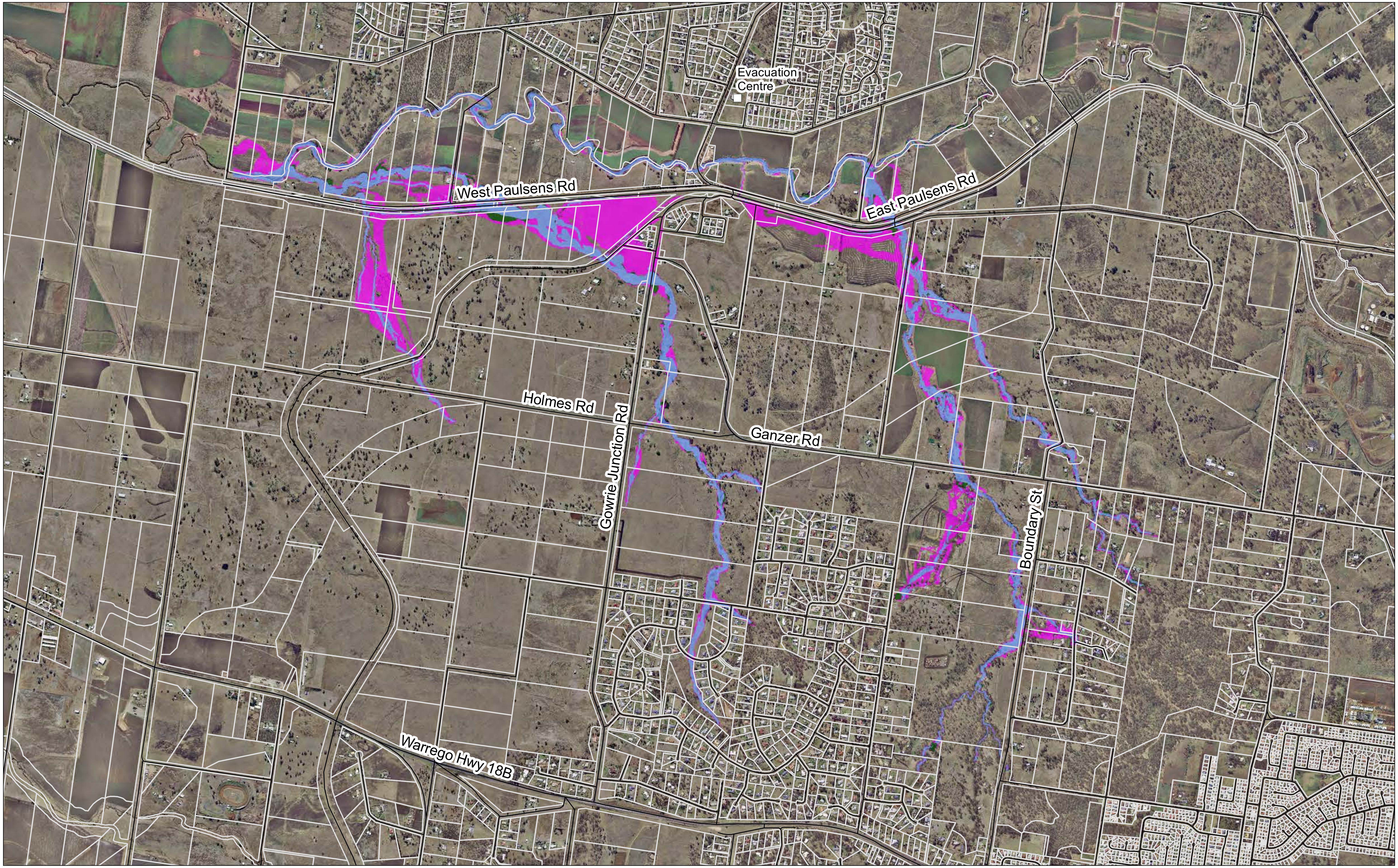
**Hazard Category**

- Low
- Significant
- High
- Extreme

- Road Centrelines
- Cadastre
- Emergency Services



**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**100 Year ARI Event**
**Hazard Category**




TOOWOOMBA REGIONAL COUNCIL

1:20,000 (at A3)

0 200 400 800

Meters

GDA 1994 MGA Zone 56

N

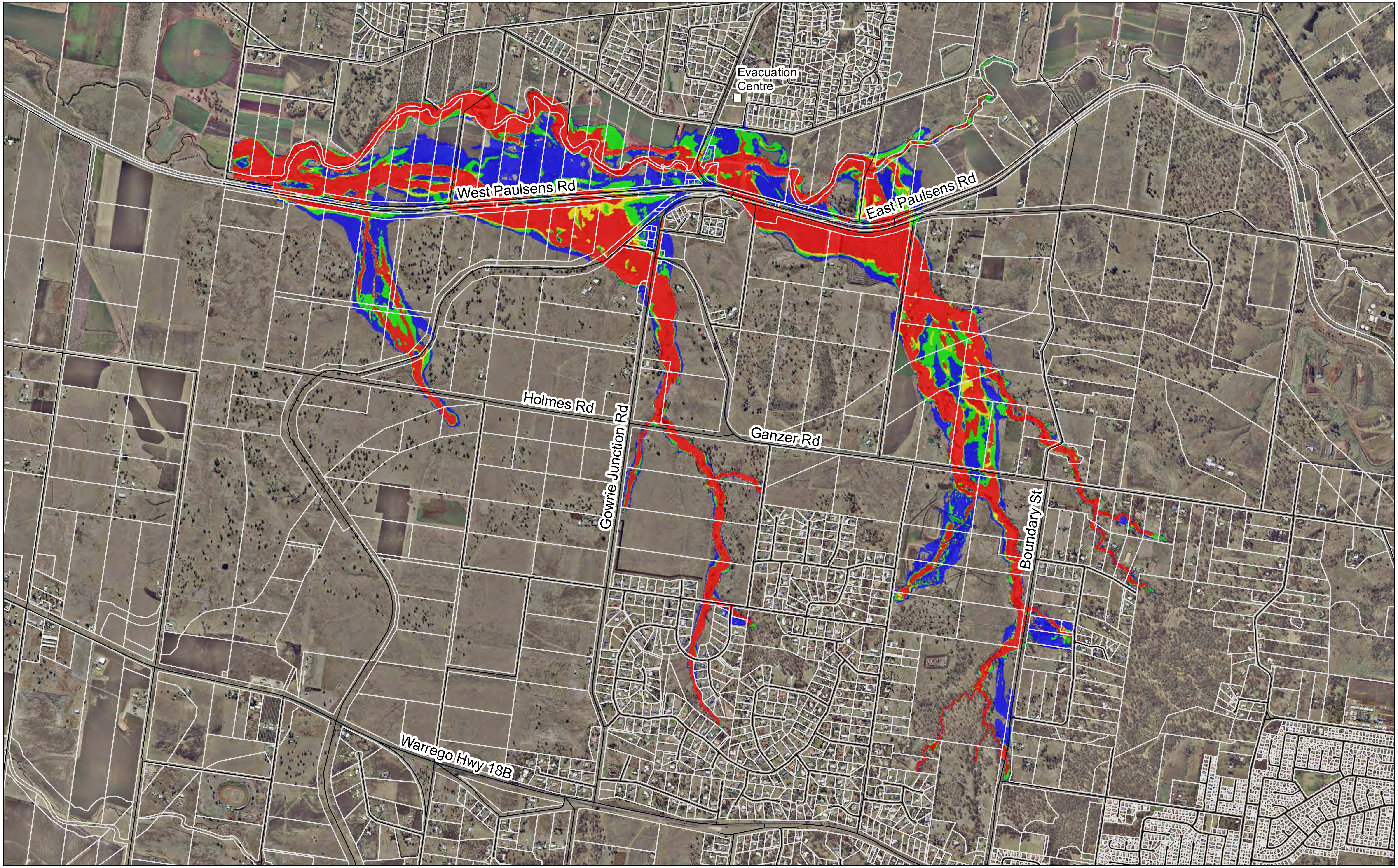
**Legend**

**Hydraulic Category**

- Flood Fringe
- Flood Storage
- Floodway
- Road Centrelines
- Cadastre
- Emergency Services

Disclaimer: The flood information contained in the maps is based on debris lines and marks that were visible and accessible at the time of recording after the January 2011 flood event and may not be accurate or complete and reliance should not be placed on it. Toowoomba Regional Council makes no representations or warranties about the accuracy, reliability, completeness or suitability for any particular purpose and disclaim all responsibility and all liability whether in contract, negligence or otherwise for all expenses, losses, damages (including indirect or consequential damage) and costs which may be incurred in any way and for any reason as a result of the flood information contained in the maps being inaccurate or incomplete.

**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**100 Year ARI Event**
**Hydraulic Category**




TOOWOOMBA REGIONAL COUNCIL

1:20,000 (at A3)

0 200 400 800
Meters

GDA 1994 MGA Zone 56

N

**Legend**

**Hazard Category**

- Low
- Significant
- High
- Extreme

- Road Centrelines
- Cadastre
- Emergency Services

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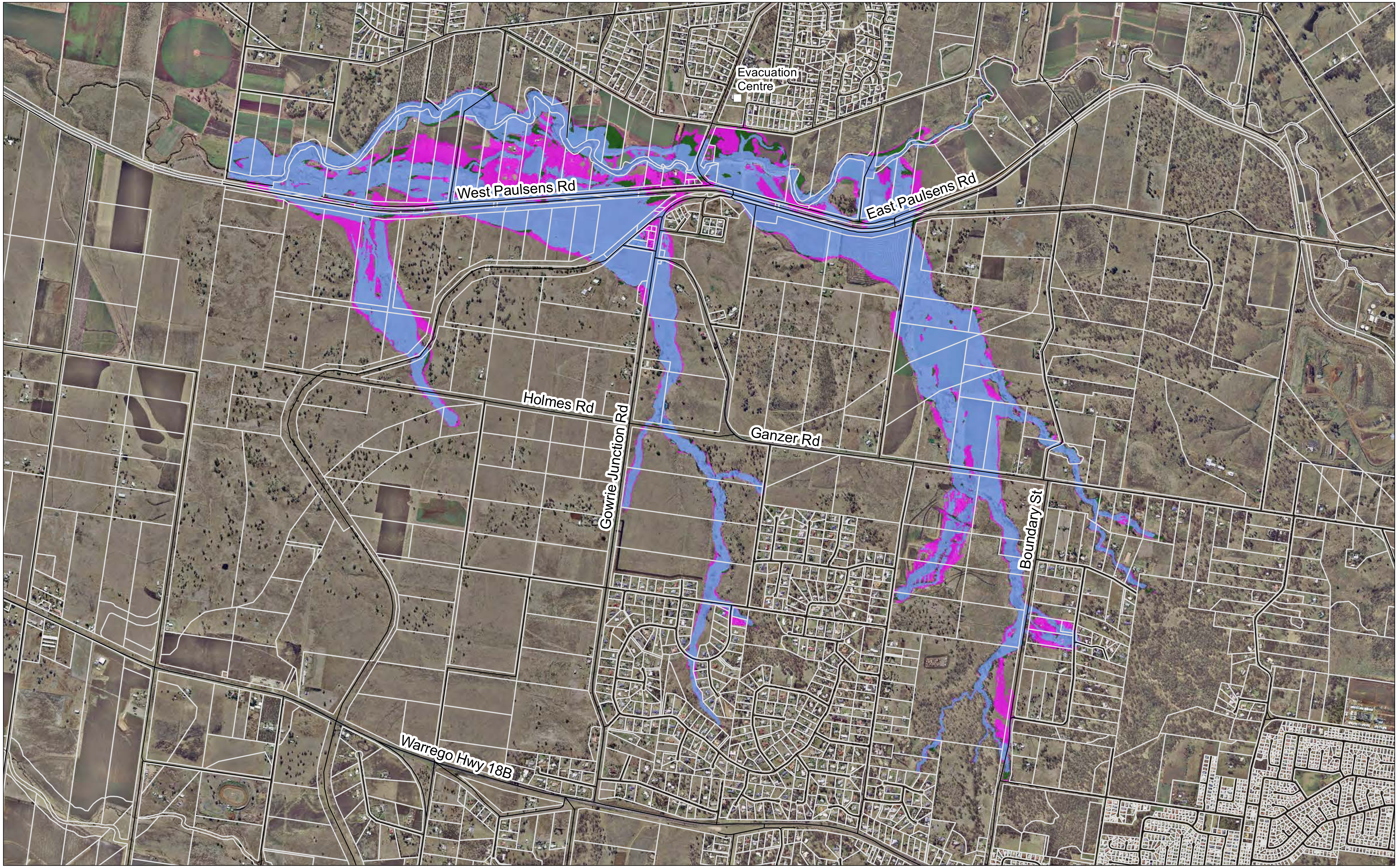
**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**Probable Maximum Flood Event**
**Hazard Category**




TOOWOOMBA REGIONAL COUNCIL

1:20,000 (at A3)

0 200 400 800
Meters

GDA 1994 MGA Zone 56

N

**Legend**
**Hydraulic Category**

- Flood Fringe
- Flood Storage
- Floodway
- Road Centrelines
- Cadastre
- Emergency Services

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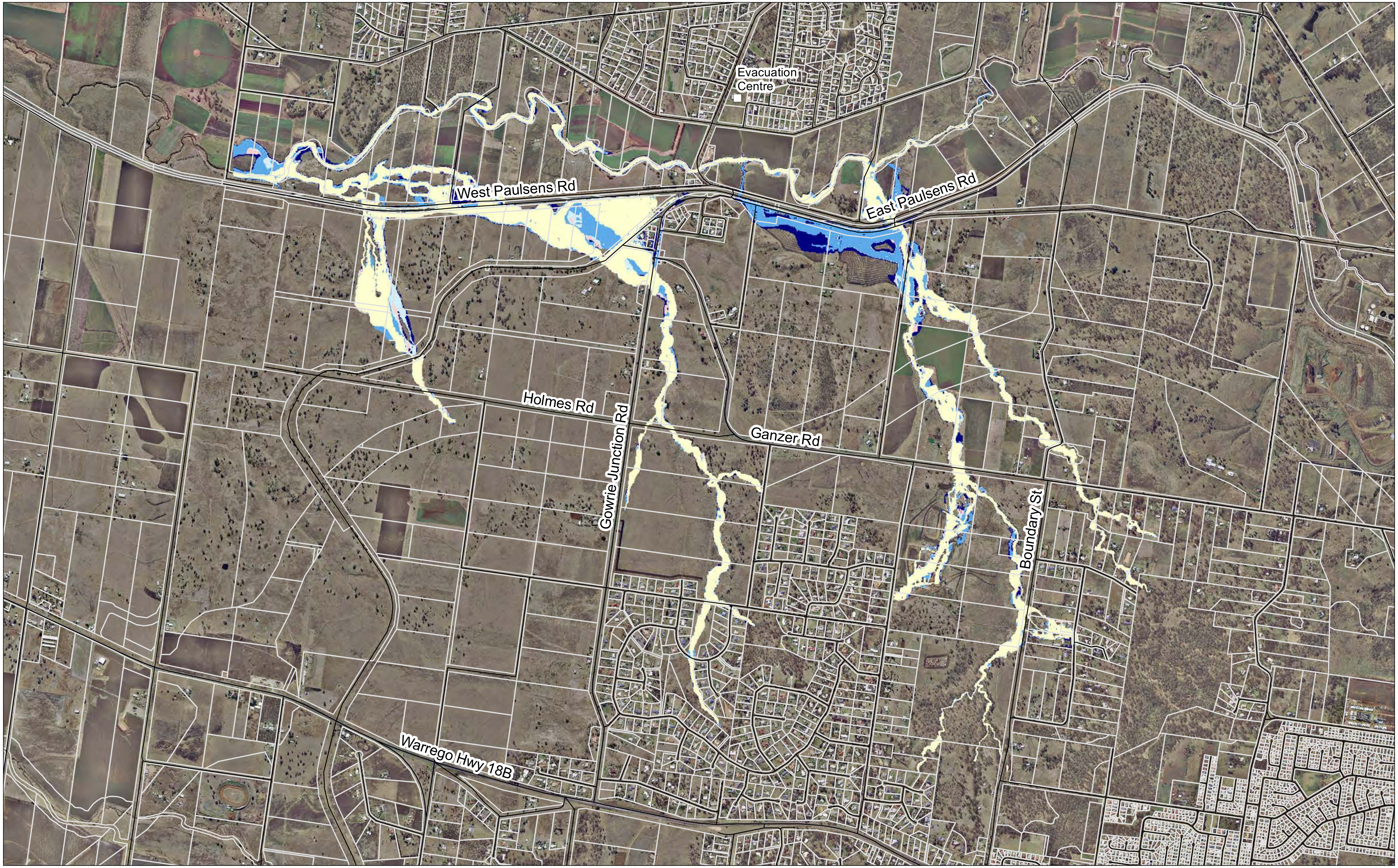
**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**Probable Maximum Flood Event**
**Hydraulic Category**

# APPENDIX G

## SENSITIVITY ANALYSIS MAPPING

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1:20,000 (at A3)

0 200 400 800

Meters

GDA 1994 MGA Zone 56

N

**Legend**

- 30% Reduction in Flow
- 30% Reduction in Roughness
- Baseline
- 30% Increase in Roughness
- 30% Increase in Flow
- Road Centrelines
- Cadastre
- Emergency Services



## SP051 Flood Studies
## Work Package 8 Cotswold Hills
## Sensitivity to Flow and Roughness


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1:20,000 (at A3)

0 200 400 800

Meters

GDA 1994 MGA Zone 56

N

**Legend**

Baseline

50% Blockage of Structures

Road Centrelines

Cadastre

Emergency Services

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**SP051 Flood Studies**
**Work Package 8 Cotswold Hills**
**Sensitivity to Blockage of Structures**

TOOWOOMBA REGION

131 872 | info@tr.qld.gov.au | www.tr.qld.gov.au
PO Box 3021 Toowoomba QLD 4350 | Toowoomba Regional Council

A safer, stronger, more resilient region

Financially, socially and environmentally sustainable

TOOWOOMBA REGION

Rich traditions. Bold ambitions.

# Cotswold Hills Flood Studies Information Sheet

## WHY UNDERTAKE FLOOD STUDIES?

Following extensive flooding across the Toowoomba region, we commissioned a number of flood studies to better understand how flooding can impact our communities. These studies are now complete and available on our website.

The flood studies found that flood behaviour can be complex and vary between locations, depending on landscape, infrastructure and rainfall pattern.

## SOME BASIC FLOOD TERMS

1. **Overland flow** – short duration flooding of backyards, drainage paths, streets and rural properties caused by stormwater as it makes its way into the creek/river system;

2. **Creek flooding** – short to medium duration flooding caused by creeks rising and breaking their banks, which can then flood nearby homes, businesses and rural properties;

3. **River flooding** – longer duration flooding caused by significant rises in a river which can break its banks in the same way as smaller creeks.

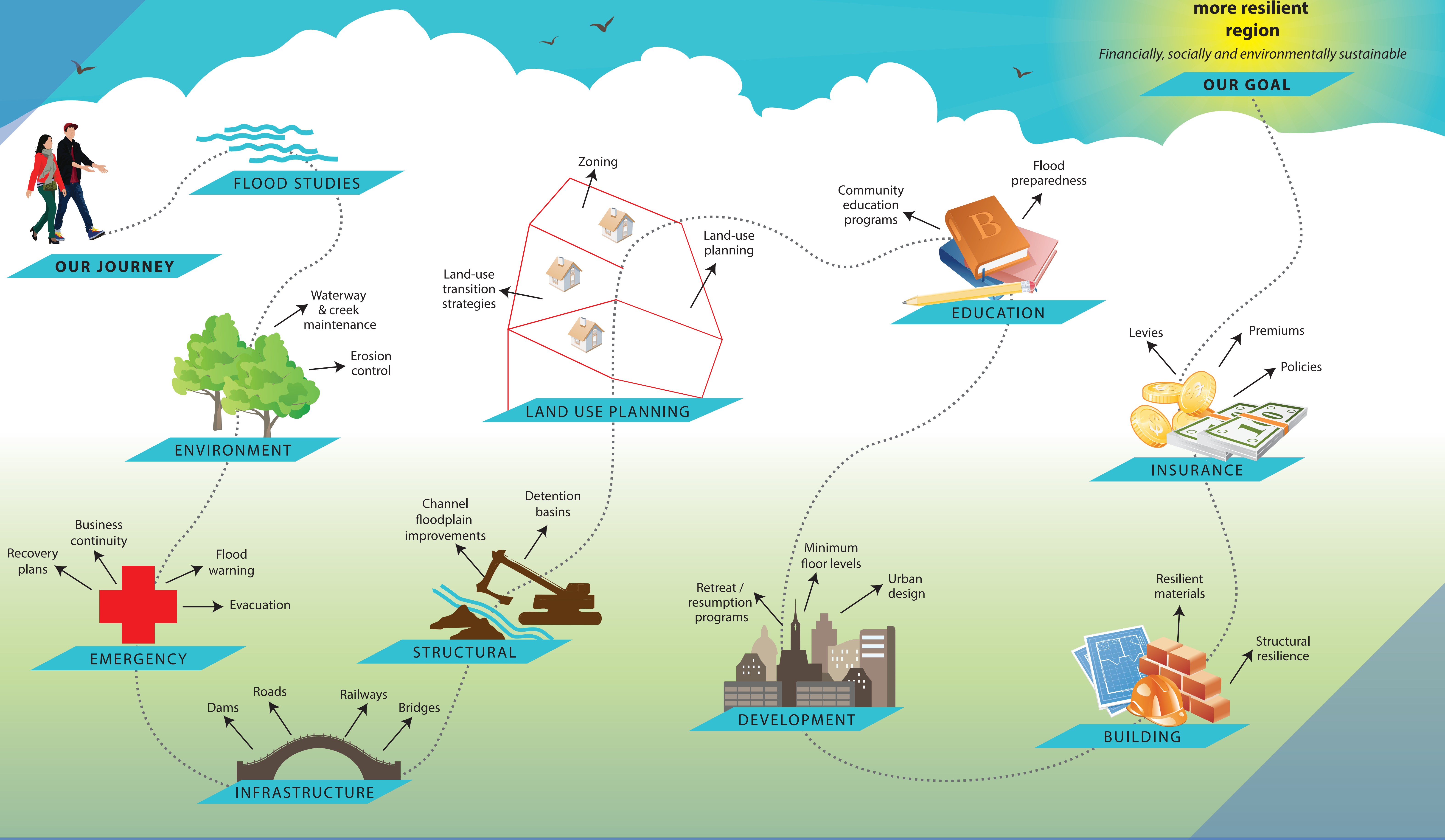
Most of the studies undertaken or commissioned by Council relate to the first two types of flooding – overland flow and creek flooding. It's important to note that these types of flooding can occur separately or together.

## KEY MESSAGES

1. Council has a legislative requirement to undertake flood management and the whole community needs to be involved.
2. Flood studies are a foundation and an essential step towards our goal of a safer, stronger, more resilient region.
3. Flood studies have been undertaken by specialist engineers and incorporate the latest data, modelling techniques and community input.
4. Community consultation enables two-way information sharing about the project to increase community awareness, enhance decision making and help achieve our goal of a safer, stronger, more resilient region.

# Flood + us - our journey

## Steps on the path to achieving our goal



# Cotswold Hills Flood Studies Information Sheet

## WHAT'S COTSWOLD HILLS'S FLOOD STORY?

A flood study and flood maps are now available for Cotswold Hills residents. The primary source of flooding to the town is from overland flow paths draining into Gowrie Creek.

Cotswold Hills is situated on the southern watershed of Gowrie Creek and runoff from the January 2011 rain event contributed to rising waters in the creek. The fast flowing waters were generally contained within the tributaries near Boundary Street, Hermitage and Ganzer roads and behind properties on John Court through the flow path at Roderick Drive. With the exception of flooding in a few low-lying rural areas, no residential properties flooded during the January 2011 event.

The study showed that the crossing at Roderick Drive has low flood immunity, flowing over in a 10% Annual Exceedance Probability event – meaning there is a 10% chance of such an event or larger occurring in a year. Boundary Street crossing is relatively high in the catchment and would rarely flood. This was shown to only occur in large to extreme flood events.

An assessment was made of the size of the 2011 flood by comparing modelled levels at a number of locations. The analysis indicates that the flooding was varied depending on location in the catchment. For the western waterways, the 2011 event was indicative of a 10% Annual Exceedance Probability event – meaning there is a 1% chance in any year of an event of this size or larger occurring. For the eastern waterways, the 2011 flood depths are indicative of between a 2% and a 1% Annual Exceedance Probability event.

Annual Exceedance Probability (AEP) means the chance of a flood of a given size or larger size occurring in any one year, usually expressed as a percentage.

## COMMUNITY INVOLVEMENT

Improving the way we prepare for and respond to flooding as a community is very important to us. Many residents in our region contributed information to build and validate our flood knowledge during the region-wide consultation sessions and other flood studies engagement opportunities.

Community involvement with this project continues to help our region become safer, stronger and more resilient. We encourage you to access the flood study information online and stay up to date with the project by visiting the web address below.

## GET INFORMED

You can access our region's current flood studies and maps by heading to **http://yoursay.toowoombarc.qld.gov.au/flood-resilience**
For more information, please contact the project team by phone, email or post.

**Phone:** 131 872

**Email:** info@tr.qld.gov.au

**Post:** Strategic Planning & Economic Development,
Toowoomba Regional Council, PO Box 3021, Toowoomba Q 4350.

TOOWOOMBA REGION