

**Appendix C – Hydrology and Flooding Assessment**

# HYDROLOGY AND HYDRAULICS STUDY

Doc No: NHA-DR-50500403-CD-01

May 2007



MAUNSELL | AECOM





**Doc No: NHA-DR-50500403**  
**HYDROLOGY AND HYDRAULICS STUDY**  
**FOR**  
**NORTHERN HUME ALLIANCE**

**REVISION CONTROL**

<b>Edition / Revision/</b>	<b>Date</b>	<b>Details</b>	<b>Authorised</b>	
			<b>Name/Position</b>	<b>WVR No.</b>
01	24-05-07	First Issue	Edmond Lee Design Manager	500403-01

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## 1. Introduction

### 1.1. Project Background

The Hume Highway is the main freight route between Sydney and Melbourne. The Federal Government has allocated funding for the upgrade of the highway between Sturt Highway and Table Top to four lane divided carriageways. The RTA has engaged the Northern Hume Alliance to duplicate and upgrade the highway (approximately 35km in overall length) between Sturt Highway and Holbrook under the alliance project delivery framework.

The Northern Hume Alliance (the Alliance) has four separate sections to deliver, including:

- Section N1 Sturt Highway to Tarcutta (also referred to as Tarcutta North) – Approximately 6km duplication and upgrade from Sturt Highway to north of Tarcutta
- Section N2 Kyeamba Hill – Approximately 9km duplication and upgrade south of Tarcutta between two sections of existing dual carriageway
- Section N3 Little Billabong – Approximately 8km duplication and upgrade between two sections of dual carriageway between Tarcutta and Holbrook
- Section N4 Yarra Yarra (also known as Yarra Yarra to Holbrook) – Approximately 12km duplication and upgrade between Yarra Yarra and north of Holbrook

It is noted that the duplication and upgrade of the highway from Holbrook to Table Top is being delivered by another alliance and is outside the scope of this project.

During the Total Outturn Cost (TOC) and project delivery phases, the Alliance will prepare concept and detailed design reports for the following design elements:

- Alignment
- Linemarkings and Signage
- Road Furniture
- Drainage
- Geotechnical
- Pavement
- Bridges / Structures
- Traffic Engineering
- Local Road Works
- Property Adjustments
- Traffic Staging
- Streetlighting

- Urban and Landscaped Design

Additional design elements may be delivered by other parties, for example, services owners. Under those circumstances, the Alliance will interface and coordinate with the parties involved to ensure that the design is integrated with the above elements.

## 1.2. Hydrology and Hydraulics

There are no major rivers or wetlands within the Project area. The Project Sections do however traverse or adjoin a number of named and unnamed creeks as well as farm dams. A number of the dams and creeks within the Project area are spring fed. Named creeks within the vicinity of the Project area include (from north to south):

- Tarcutta Creek
- Dellateroy Creek (tributary of Tarcutta Creek)
- Kyeamba Creek (tributary of Tarcutta Creek)
- Little Billabong Creek (upper reaches of Billabong Creek)
- Billabong Creek
- Yarra Yarra Creek (tributary of Billabong Creek)

Tarcutta, Dellateroy and Kyeamba Creeks (Sections N1 and N2) are part of the Murrumbidgee River catchment. Little Billabong, Billabong and Yarra Yarra Creeks (Sections N3 and N4) are part of the Murray River catchment.

The majority of the waterways within the Project area flow from east to west. Upstream of the Hume Highway (typically the eastern side), the tributaries are generally independent between each individual Section. In this regard, potential upstream impacts from one Section would not have a cumulative effect on another.

The area is characterised by generally high winter and low summer rainfall/runoff. Average monthly rainfall ranges from approximately 50-80 millimetres in the winter months to generally less than 50 millimetres in summer. Approximately 60% of the average annual rainfall typically falls in the six month period from May to October. Available rainfall data indicates that although the total rainfall over the summer months is less than that for the winter months, rainfall in the summer months tends to be more intense. Average monthly evaporation in the region ranges from approximately 30 millimetres in the winter months to over 300 millimetres in summer.

Erosion hazard is generally moderate to high. Gully erosion is evident near stream banks, exacerbated by grazing of livestock, and sheet erosion is common on upper slopes, particularly where cleared. Occurrences of acid soils and salinity in the region are caused by rising groundwater and agricultural practices. Waterlogging is particularly evident at discharge areas which are generally found at the break of slopes and may also be associated with areas where normal drainage is restricted.

### 1.3. Scope of this Report

In accordance with the Design Criteria established for the Project (refer Report No. NHA-0-DR-50500401), a hydrologic and hydraulics study for each bridge structure will be carried out and will include:

- Serviceability effects of afflux on adjacent properties and the stability of the adjacent road embankment for the 100 year average recurrence interval (ARI) event.
- Ultimate limit state for bridge stability for the 2000 year ARI event.
- Ultimate effects of the road on regional flooding for probable maximum flood (PMF) conditions.

Other criteria include:

- The roadworks shall be designed so that the edge line on the pavement surface of the dual carriageways is above the 100 year ARI flood level.
- The project works must be designed to take account of the hydraulic performance so that inundation levels upstream of the project boundary are not increased in the 100 year ARI event by any more than the following:
  - Rural land without buildings or sensitive structures ... 0.25 metres.
  - Where buildings or sensitive structures are already below the 100 year ARI flood level ... 0.05 metres.
  - Where buildings or sensitive structures previously not inundated in the 100 year ARI event would be at increased risk of inundation (including allowance for afflux) ... 0 metres.

In order to satisfy the above requirements, hydrologic and hydraulic assessment of the proposed waterway crossings is to be undertaken. The following report summarises the detailed hydrologic and hydraulic assessment for the proposed major waterway crossings and adjoining creeks (including assessment of any embankment encroachment) along the Hume Highway as indicated in Table 1.3.

**Table 1.3 Major Waterways**

Creek Name	Project Section	Chainage	Catchment Area at the Hume Highway
Dellateroy Creek	Section N1	Ch.39250	5,250 hectares
Kyeamba Creek	Section N2	Ch.75400	1,360 hectares
Little Billabong Creek	Section N3	Ch.93600 (south of duplication)	24,400 hectares
Yarra Yarra Creek	Section N4	Ch.101220	31,200 hectares

The report includes discussion of the various flood related culvert and bridge design issues and the potential effects on creek hydraulics and surrounding floodplain areas.

The RTA has previously assessed the flood behaviour of Little Billabong Creek between Ch.88500 and Ch.90500. The hydraulic model established for this creek was obtained and extended to the north (upstream) for the purpose of evaluating flood levels adjoining the Hume Highway in Section N3.

The above creeks correspond to the largest catchment areas crossed/adjoined by the proposed Hume Highway duplication. The duplication crosses many more smaller waterways, for which the modelling approach is discussed herein (refer report section 3) with the assessment and results documented in the separate reports for each section (refer Report No.'s TBA).

The following attached Appendices also form part of the report:

- Appendix A – Report Figures
- Appendix B – Hydraulic Model Results – Dellateroy Creek
- Appendix C – Hydraulic Model Results – Kyeamba Creek
- Appendix D – Hydraulic Model Results – Little Billabong Creek
- Appendix E – Hydraulic Model Results – Yarra Yarra Creek
- Appendix F – Flood Frequency Analysis

## 2. Bridge and Major Culvert Crossings

### 2.1. Modelling Approach

#### 2.1.1. General

The catchment areas considered in this assessment range in size from approximately 1,360 hectares (13.6 km<sup>2</sup>) to over 30,000 hectares (300 km<sup>2</sup>) at the Hume Highway. Computer based hydrologic and hydraulic models were established for analysis of the existing and proposed flood behaviour at the above crossings.

The Watershed Bounded Network Model (WBNM) software package (WBNM2003 version 103, June 2005) was used for the estimation of design flow hydrographs for the major waterways crossing/adjoining the Hume Highway. WBNM is an event based hydrologic model that represents the tributaries or flow paths of a catchment as a series of sub-catchment areas based on the watershed boundaries. Sub-catchments are linked together to replicate the rainfall runoff process through the natural stream network.

The hydraulic model software package HEC-RAS (version 3.1.3, May 2005) was used to represent the watercourses and floodplain in the areas of the major waterways crossing/adjoining the highway. The program is able to compute hydraulic characteristics such as estimated water surface profiles and flow velocities at specific locations along a channel using steady or unsteady flow conditions.

Due to the particular nature of the topography in the vicinity of the Dellateroy Creek floodplain crossing with the interaction of several waterway openings through the existing highway embankment, the assessment of flow distribution and flood gradients was slightly more complex than a standard 1-dimensional HEC-RAS analysis could reasonably represent with any certainty. This made it all the more difficult to assess the capacity of the different culvert openings and the relative merits of alternative alignment and waterway configurations. It was therefore decided to establish a more sophisticated 2-dimensional hydraulic model of the area using the TUFLOW hydrodynamic software (Build No. 2006-06-BF). TUFLOW is computer program which simulates depth averaged free-surface flows and is specifically orientated towards establishing complex flow patterns essentially 2-dimensional in nature across floodplain areas while dynamically linked to 1-dimensional elements such as defined channels and hydraulic structures.

#### 2.1.2. Calibration

Ideally, hydrologic and hydraulic models should be calibrated against observed historical events to provide some measure of confidence in the results provided. Streamflow data is, or has been, collected at locations within the wider study area as detailed in Table 2.1.2.

**Table 2.1.2 Streamflow Gauges**

Gauge Number	Creek & Location	Catchment Area (hectares)	Date Start (installed)	Date Finish (decommissioned)	Comments
410058	Tarcutta Creek at Westbrook	30,800	1946	1985	
410155	Tarcutta Creek at Belmore Br	40,000	Not available	Not available	
410047	Tarcutta Creek at Old Borambola	166,000	1938	current	
410156	Kyeamba Creek at Book Book	14,500	1985	current	
410048	Kyeamba Creek at Ladysmith	53,000	1938	1987	
410097	Billabong Creek at Aberfeldy	33,100	1967	current	
410089	Billabong Creek at Garryowen	71,400	22/01/1960	12/05/1993	
410099	Yarra Yarra Creek at Yarra Yarra	25,100	23/11/1967	22/02/1977	
410188	Yarra Yarra Creek at Donna Valley	Not available	09/05/2001	current	Staff gauge only – no instrumentation

Source: Pinneena Version 9 (DNR, 2006) and BOM Water Resources Station Catalogue (2007)

The streamflow gauges located on Tarcutta and Kyeamba Creeks have catchment areas that are orders of magnitude larger than the comparatively small catchment areas which cross the Hume Highway in Project Sections N1 and N2. The gauged streamflows for Tarcutta and Kyeamba Creeks are therefore not likely to be representative of the flow characteristics of the comparatively smaller waterway crossings at the Hume Highway.

The available streamflow data for Billabong and Yarra Yarra Creeks would provide a reasonable indication of the typical flow characteristics for the larger watercourses which adjoin (Billabong Creek) and cross (Yarra Yarra Creek) the Hume Highway in Project Sections N3 and N4. However, there is limited continuous rainfall data available for input to the rainfall runoff model for calibration. The nearest locations to these catchments where continuous rainfall has been recorded are Wagga Wagga Airport, Hume Reservoir and at Humula (a short length of record only), located some 50 km to the north west of Holbrook between Tarcutta and Tumbarumba. The storm rainfalls recorded at these locations are unlikely to be representative of the rainfalls received over the Billabong and Yarra Yarra Creek catchments. The streamflow data has, however, been used to undertake a basic flood frequency analysis to provide an order of magnitude check of the more frequent design peak flows determined from WBNM for Billabong Creek (refer report section 2.1.4).

There is limited other available historical flood related data. Observed high flood levels are typically shown on existing bridge plans which are held by the RTA. As mentioned above, the limited available continuous rainfall information precludes calibrating the models to this data. Greater Hume Shire Council, the Murray Catchment Management Authority and Landcare Holbrook have advised they do not hold any flood related reports or other flood related information for the Upper Billabong Creek catchment. Similarly, Wagga Wagga Shire Council

and the Murrumbidgee Catchment Management Authority have indicated limited information is available for the upper reaches of Tarcutta Creek and its tributaries.

### 2.1.3. Modelling Parameters and Data Inputs

The following WBNM hydrologic model parameters were adopted for this study:

Lag Parameter 'C'	=	1.29
Initial Loss	=	15-30 mm for design events up to and including the 100 year ARI 0 mm for the 2000 year ARI and PMF design events
Continuing Loss	=	2.5 mm/h for design events up to and including the 100 year ARI 1.0 mm/h for the 2000 year ARI and PMF design events

The above values are in accordance with the recommendations of Australian Rainfall & Runoff (ARR) (Institution of Engineers, Australia 1987) for ungauged catchments and initial loss recommendations set out in Walsh et al (1991). Furthermore, the above values correspond to calibrated parameters established for Tarcutta Creek as part of previous investigations undertaken for the RTA and generally to parameters adopted for hydrologic investigations undertaken for the Albury Wodonga National Highway project (refer Albury Wodonga Potential National Highway Routes Environmental Impact Statement/Environmental Effects Statement, GHD October 1995).

Design rainfalls, temporal patterns and areal reduction factors were obtained in accordance with ARR. The catchment areas considered by this assessment generally exceed the procedures for estimating probable maximum precipitation (PMP) in accordance with "The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method" (Bureau of Meteorology, June 2003), which is limited to durations of up to 3 hours. An approximate PMF flow has therefore been established for these catchments based on 5 times the 100 year ARI design flow.

HEC-RAS hydraulic models were established for each of the four major creeks being considered in more detail. Steady flow models were established for Dellateroy Creek (Section N1), Kyeamba Creek (Section N2) and Yarra Yarra Creek (Section N4). An unsteady flow model was established for the Little Billabong Creek (Section N3) in order to better represent the implications of floodplain storage effects associated with this reach and the proposed upgrade works.

HEC-RAS hydraulic model parameters include Manning's 'n' values, a friction factor or roughness coefficient affecting the hydraulic efficiency of waterway areas. Manning's 'n' values typically in the range of 0.050 to 0.080 were adopted for floodplain areas and natural watercourses, which were derived from site inspection, review of available aerial photography and by reference to recognised texts (see, for example, ARR and Chow 1959). Hydraulic model expansion and contraction coefficients of 0.1 and 0.3 respectively were adopted, except in the vicinity of bridges and culverts, where they were increased to 0.3 and 0.5 respectively. For culvert structures, an entrance loss coefficient of 0.5 and a Manning's 'n' value of 0.013 were generally adopted.

Similar to the HEC-RAS model, the main parameters for the TUFLOW model relate to hydraulic roughness coefficients and energy losses at structures. The Manning's 'n' values included 0.025 for roads, 0.050 for the pasture areas and 0.080 for the meandering creek channel. Culvert

entry and exit losses were 0.5 and 1.0 respectively with height and width contraction coefficients of 0.6 and 0.9. A model grid resolution of 5m was adopted for definition of the topography and floodplain features.

#### 2.1.4. Hydrologic Frequency Analysis

Billabong Creek at Garryowen (DNR Gauge No. 410089) has a largely complete, continuous record from February 1960 to August 1989 therefore allowing the extraction of observed peak streamflows over a reasonable length of time (refer Appendix F for Locality Sketch and Summary of Data). Data is also available for Billabong Creek at Aberfeldy (DNR Gauge No. 410097) and Yarra Yarra Creek at Yarra Yarra (DNR Gauge No. 410089) (refer Appendix F). However, the available data for Billabong Creek at Aberfeldy contains a fair amount of missing observations and Yarra Yarra Creek at Yarra Yarra consists of a short length of record, and hence flood frequency analysis was not undertaken for streamflows for these two gauges.

A log Pearson Type III distribution was fitted to the annual series data for Billabong Creek at Garryowen. Of the 30 years of record, two years were discarded from the annual maximum time series (1967 and 1982) due to the annual maximum values being close to zero flow. These very low flow values were found to influence the fitted log Pearson Type III distribution to such an extent that it gave an unsatisfactory fit to the observed data, particularly at the higher ARI estimates which are of greater interest in design.

Design flows and confidence limits were calculated in accordance with the procedures set out in ARR. These estimates are provided in Table 2.1.4a.

**Table 2.1.4a Billabong Creek at Garryowen – Flood Frequency Analysis**

	ARI			
	10 year	20 year	50 year	100 year
Peak Flow (m <sup>3</sup> /s)	300	460	750	1050
Upper (5%) Confidence Limit (m <sup>3</sup> /s)	490	850	1720	2880
Lower (95%) Confidence Limit (m <sup>3</sup> /s)	180	240	320	380

Note: Peak flows rounded to the nearest 10 m<sup>3</sup>/s.

To enable comparison of WBNM peak flow estimates to the flood frequency analysis results, the WBNM model established for Yarra Yarra and Billabong Creeks was extended downstream to the location of the Garryowen gauge, located on Billabong Creek some 4 km downstream of its junction with Yarra Yarra Creek (a total catchment area of 71,600 hectares, refer Figure 2.2.4a). Peak flows for the 10 year, 20 year, 50 year and 100 year ARI events were estimated using the adopted model parameters and data inputs set out in report Section 2.1.3. The peak flows are provided in Table 2.1.4b.

**Table 2.1.4b Billabong Creek at Garryowen – WBNM Estimates**

	ARI			
	10 year	20 year	50 year	100 year
Peak Flow (m <sup>3</sup> /s)	232	455	719	962

WBNM files: 10.8.4\Drainage\Flooding\N4\WBNM\Total 10yr Sensitivity; Total 20yr Sensitivity; Total 50yr Sensitivity; Total 100yr Sensitivity

Comparison of Tables 2.1.4a and 2.1.4b shows the WBNM estimates compare well to the flood frequency analysis results, which provides further confidence in the WBNM model parameters and peak flow estimates.

### *2.1.5. Existing Conditions*

In order to assess the effect of the proposed waterway crossings, it was first necessary to establish detailed hydrologic and hydraulic models representing the existing catchment and waterway crossing conditions for each assessed site.

Electronic contour information (10 metre contour interval) and waterway layouts were used to provide an overview of the broader topographic features of the area and define catchment boundaries for hydrologic estimates.

Detailed topographic survey of the existing waterway crossing and surrounds, including floor levels where relevant, were obtained for each crossing for input to the hydraulic model. Site specific conditions were assessed through field observations undertaken in March 2007. Where available, existing bridge and culvert drawings held by the RTA were obtained for use in this study.

Details of the existing crossings considered in this assessment are provided in report Sections 2.2.1 to 2.2.4.

### *2.1.6. Proposed Conditions*

In order to represent the potential hydraulic changes as a result of the proposed Hume Highway duplication, the hydraulic model cross sections at the locations of the proposed works were modified from the existing (baseline) configuration to incorporate the proposed waterway crossings and associated features (embankments, bridge piers and abutments, etc., where relevant).

The waterway dimensions, location and dimensions of piers, abutments, etc., were obtained from the latest available design drawings, reproduced in Appendix A [drawings to be prepared].

Details of the proposed crossings considered in this assessment are provided in report Sections 2.2.1 to 2.2.4.

## **2.2. Analysis and Results**

### *2.2.1. Section N1 – Dellateroy Creek*

#### **Existing Conditions**

Dellateroy Creek and its associated catchment is primarily situated on the south eastern side of the Hume Highway. A couple of contributing sub-catchments are located on the north western side of the highway. After crossing the existing highway alignment, the main channel takes a series of 's' turns in a southerly direction on the western side of the highway and subsequently flows in a north easterly direction downstream of Lower Tarcutta Road prior to joining Tarcutta Creek.

During a field inspection in March 2007 it was noted that a spring was providing a natural low flow condition originating upstream of the Toonga Road Bridge (the precise origin is unknown). It was also evident that the channel and adjacent overbank areas have experienced considerable erosion, which is typical of the surrounding farming areas subjected to long term agricultural use.

There are five (5) existing culvert structures that operate under varying efficiencies and contribute to passing the total discharge from the eastern to the western side of the Hume Highway. These structures are listed in Table 2.2.1a. Under existing conditions the main channel culvert for Dellateroy Creek has insufficient capacity to convey the 20 year ARI event resulting in excess flows breaking out across the upstream floodplain and continuing south towards the other openings. The overall capacity of these culverts combined is still inadequate for events greater than the 50 year ARI and the highway is overtopped at the low point around chainage 39900.

**Table 2.2.1a Dellateroy Creek - Existing Drainage Infrastructure at the Hume Highway**

Chainage	Existing Drainage Infrastructure (mm)	Description
39250	4 x 3000 x 2400 reinforced concrete box culvert (RCBC)	Main channel
39470	3 x 1800 x 450 RCBC	
39610	4 x 2400 x 2130 RCBC	Stock crossing
39800	2 x 2400 x 1850 RCBC	
39920	1 x 1500 dia. reinforced concrete pipe (RCP)	

### Proposed Conditions

#### *Preliminary Assessment of Waterway Options*

The hydraulic assessment presented in this report is primarily based on the design options which incorporate a new northbound carriageway on the western side of the existing Hume Highway as opposed to the original RTA eastern alignment concept presented in the Environmental Assessment.

As part of the 100 day TOC design development process, the actual road alignment details (horizontal and vertical) have been the subject of ongoing modifications and refinements. For the purposes of quantifying the general culvert waterway area requirements, the road design alignment details were initially based on the preliminary western alignment option referred to as N1-7. It was then apparent from preliminary modelling results that some form of culvert augmentation, raising of the vertical alignment and/or a combination of both would be required to address and alleviate the flood related issues through this area. The 1-D hydraulic analysis then considered and developed a number of waterway sub-options as outlined below.

#### **Description of Waterway Options:**

- Option 1 - Base case – Alignment Option N1-7 with no augmentation of the existing culverts.
- Option 2 - Existing 2 x 2400x1850 RCBC augmented to 4x2400x1850 RCBC.
- Option 3 - Existing 2 x 2400x1850 RCBC augmented to 4x2400x1850 RCBC and proposed road raised by 280mm.
- Option 4 - Existing 2 x 2400x1850 RCBC augmented to 6x2400x1850 RCBC.
- Option 5 - Existing 2 x 2400x1850 RCBC augmented with additional 3x3000x2400 RCBC.
- Option 6 - Existing 2 x 2400x1850 RCBC augmented with additional 3x3000x2400 RCBC and proposed road raised by 280mm.

The results of these preliminary waterway considerations were then utilised as the starting point for more detailed investigation of the later alignment option developments (such as Option N1-9a – refer below).

*Adopted 100 Day TOC Alignment Concept*

The current preferred alignment concept adopted for the purposes of the 100 day TOC estimate comprises a new northbound carriageway constructed across the Dellateroy Creek floodplain on the western side of the existing highway which would subsequently become the southbound carriageway. Following several design iterations looking at alternative road safety and geometric issues such as median widths and horizontal curve radii connecting to adjoining alignment constraints, the western alignment referred to as Option N1-9a was adopted for detailed investigation. Based on the results of the preliminary 1D hydraulic analyses, the configuration of culvert waterway areas referred to as Option 6 above was adopted for modelling.

The western alignment designs propose that sections of the existing channel on the western side of the highway would be filled and re-aligned between chainages Ch.39600 and Ch.40000. Details of the proposed creek rehabilitation/realignment works are preliminary at this stage and subject to further consultation with the relevant stakeholder authorities. Details of the proposed creek works and associated drainage infrastructure shall form part of a separate detail design report and submission (refer Design Report No. TBA).

**Results**

*Design Flows*

Estimated design flows for Dellateroy Creek at the location of the existing Hume Highway crossing (catchment area of 5,250 hectares) are summarised in Table 2.2.1b. The WBNM sub-catchment division is shown on Figure 2.2.1a.

**Table 2.2.1b Dellateroy Creek at the Hume Highway – Peak Flows**

	ARI			PMF (5x100y ARI)
	50 year	100 year	2000 year	
<b>Peak Flow (m<sup>3</sup>/s)</b>	114	162	426	810

WBNM files: 10.8.4\Drainage\Flooding\N1\WBNM\DEL 50 yr; DEL 100yr; DEL 2000 yr

*HEC-RAS 1D Relative Assessment of Waterway Options*

The HEC-RAS hydraulic model extent and cross section locations are shown on Figure 2.2.1b. A relatively complex flow regime exists on the upstream side (east side) of the Hume Highway with a reasonable longitudinal hydraulic gradient and unequal distribution of flows between the existing highway culverts (listed in Table 2.2.1a). For preliminary modelling purposes the hydraulic model was divided into two systems, namely Reach A, downstream of the Hume Highway and Reach B, upstream of the highway.

For existing conditions, the preliminary modelling undertaken indicates the main culvert structure does not function to its full capacity (mentioned above) and the remaining culvert crossings have insufficient capacity to accommodate the balance of the design flows for any of

the events considered (i.e., up to and in excess of the 50 year ARI event) and the resulting excess flows overtop the highway.

The model was then modified to represent the 'base case' for the western alignment option (alignment design Option N1-7) assuming no augmentation of the existing culverts and a summary of the relative impacts (compared to existing conditions) are presented in Table 2.2.1c for the 50 year, 100 year, 2000 year ARI and PMF events. Cross sections and detailed model results for the existing and proposed conditions, including peak flows, water levels and velocities, are provided in Appendix B.

**Table 2.2.1c Dellateroy Creek – Preliminary Assessment of Relative Impacts**

Cross Section See Fig. 2.2.1b	Proposed Conditions [Design N1-7] (see Note 1)			
	Relative Impact (m) (see Note 2)			
	50 year ARI	100 year ARI	2000 year ARI	PMF
Reach B (upstream)				
B1	-	-	-	-
B2	-	-	-	-
B3	-	-	-	-
B4	-	-	-	-
B5	-	-	-	-
B6	+0.42	+0.49	+0.52	+0.49
<i>Hwy Overflow</i>	<i>-15 m<sup>3</sup>/s</i>	<i>-20 m<sup>3</sup>/s</i>	<i>-24 m<sup>3</sup>/s</i>	<i>-40 m<sup>3</sup>/s</i>
Reach A (downstream)				
A1	-0.09	-0.09	+0.07	+0.16
A2	+0.06	+0.05	+0.42	+0.43
A3	-0.26	-0.20	-0.19	-
A4	-0.24	-0.12	+0.05	+0.23
A5	-0.03	-	+0.24	+0.44
A6	-	-	-	-
A7	-	-	+0.06	+0.13
A8	+0.02	+0.03	+0.05	+0.03
A9	-	-	-	-

HEC-RAS Files: 10.8.4\Drainage\Flooding\Hec\_Ras\_3.1.3\Dellateroy\Reach\_A.prj;  
existing conditions Reach\_A.p07, Reach\_A.g06, Reach\_A.f03;  
10.8.4\Drainage\Flooding\Hec\_Ras\_3.1.3\Dellateroy\Reach\_A.prj;  
Design conditions Reach\_A.p23, Reach\_A.g17, Reach\_A.f03;

**Notes:**

- (1) The proposed conditions for option N1-7 assume no augmentation of the existing culverts crossing the Hume Highway. This aspect may alter for the different options being considered.
- (2) Only changes greater than +/- 0.01m are shown. A change in flood level of +/- 0.01m or less is indicated by - and is considered to be within the accuracy of the hydraulic model and can therefore be ignored.

The relative flood impacts of the proposed conditions are indicated within the immediate area upstream of the Hume Highway where the road is overtopped (approximate chainage Ch.39925) and the extent of afflux is reasonably limited (i.e., the gradient of the flood profile has a limiting effect on the extent to which any calculated afflux is transmitted upstream).

In assessing the hydraulic performance of the various waterway sub-options under consideration it became apparent from the preliminary results that some form of culvert augmentation, raising of the vertical alignment and/or a combination of both was required to address and alleviate the flood related issues through this area. The key relative impact results

for the different waterway sub-options are summarised in Table 2.2.1d for the 100 year ARI event and discussed below.

In assessing any potential impact on the western side of the proposed road alignment and embankment batters (Reach A) a number of hydraulic models representing a range of proposed conditions were developed on the basis of the potential flow distribution scenarios referred above. The proposed channel was preliminarily represented as a continuation of the 2:1 fill batter and provided with a channel cross section consistent with the existing profile. Preliminary results indicate no significant adverse impacts in relation to flood inundation for both existing and proposed cases. In general, the average flow velocities along this southern Reach A (downstream of the highway) are generally in the order of 1.0m/s to 2.0m/s for both the existing and proposed conditions. Isolated instances of relatively high velocities (>2.5m/s) were noted in the vicinity of the main floodplain constriction around cross section A5 (refer Figure 2.2.1b). Comparison of the modelling results suggests that the proposed roadworks would marginally increase velocities at this location. The proposed channel works including realignment and protection measures will be further considered and developed as the design progresses.

The results provide an indication of the extent of drainage and/or road works required to mitigate flooding of the proposed carriageway and/or adverse afflux impacts upstream of the alignment. Further investigation and refinement of the options will be developed as the design progresses.

**Table 2.2.1d Dellateroy Creek Waterway Options Preliminary Assessment – Design 100 year ARI Flood Results**

Cross Section See Fig. 2.2.1b	Proposed Conditions [Alignment Design N1-7]					
	Relative Impact (m) (see Note 1)					
	100 year ARI					
	Reach B (upstream)					
	Option 1 'Base Case'	Option 2	Option 3	Option 4	Option 5	Option 6
B1	-	-	-	-	-	-
B2	-	-	-	-	-	-
B3	-	-	-	-	-	-
B4	-	-	-	-	-	-
B5	-	-	-	-	-	-
B6	+0.49	+0.35	+0.53	+0.27	+0.20	+0.21
Hwy overflow	-20 m <sup>3</sup> /s	-41 m <sup>3</sup> /s	-52 m <sup>3</sup> /s	-50 m <sup>3</sup> /s	-55 m <sup>3</sup> /s	-61 m <sup>3</sup> /s

HEC-RAS Files: 10.8.4\Drainage\Flooding\Hec\_Ras\_3.1.3\Dellateroy\Reach\_B.prj;  
existing conditions – Reach\_B.p20, Reach\_B.g11, Reach\_B.f04  
10.8.4\Drainage\Flooding\Hec\_Ras\_3.1.3\Dellateroy\Reach\_B.prj;  
design conditions (Base Case) – Reach\_B.p16, Reach\_B.g11, Reach\_B.f04  
10.8.4\Drainage\Flooding\Hec\_Ras\_3.1.3\Dellateroy\Reach\_B.prj;  
design conditions – (Option 6) Reach\_B.p21, Reach\_B.g15, Reach\_B.f04

**Notes:**

- (1) Only changes greater than +/- 0.01m are shown. A change in flood level of +/- 0.01m or less is indicated by - and is considered to be within the accuracy of the hydraulic model and can therefore be ignored.

*TUFLOW 2-D Assessment of Preferred Alignment Option*

The detailed 2D hydraulic modelling of the existing conditions for the surrounding Dellateroy Creek floodplain effectively confirmed the general assessment of flood behaviour and flow distribution indicated by the preliminary 1D approach. The model results confirmed that the main culvert crossing located at chainage Ch.39250 only caters for a relatively low (around

“bank full”) flow condition. The culvert discharge is largely limited by upstream channel capacity, whereby high flows ‘breakout’ of the channel upstream of the highway and flow overland to the adjoining culvert structures. This results in the main culvert cells not reaching their full potential discharge capacity. Additional culvert structures including a stock crossing located further south along the highway appear to function as the main discharge structures or relief points for the broader floodplain. In general the overland flow relief follows a south westerly alignment prior to reaching the highway embankment and eventually overtopping at the low point around chainage 39900. The extent of floodplain inundation is shown on Figure 2.2.1c for the 100 year ARI event under existing conditions along with peak flows in the main channel and culvert structures.

The underlying digital terrain model of the existing floodplain conditions was then modified to incorporate the proposed new embankment and culvert details representing the preferred western Option N1-9a. The TUFLOW model was re-run for the 100 year ARI event and the results compared with the existing. Figure 2.2.1d shows the impacts for the surrounding area in terms of relative change in flood level (afflux). Overall the impacts for the surrounding floodplain areas are generally less than 0.05m with some small areas of up to 0.2m close to the project corridor. The greatest impacts of more than 0.2m are contained within the project boundaries with some isolated values of up to 0.5m evident along the upstream side of the proposed new embankment where flow (water levels) used to overtop the downstream side of the existing highway embankment.

It should be noted that the critical storm duration for the contributing catchment area is around 3 hours and given the prevailing stream slopes the area is expected to be relatively well drained such that inundation or ponding will not be experienced for prolonged periods or adversely affect the pastures.

The distribution of velocities across the floodplain under existing conditions are presented on Figure 2.2.1e which indicates that values are typically in the range of 1.0 to 2.0m/s for the main flow path areas and <1.0m/s elsewhere. There are however some small isolated areas with velocities of up to 3.0m/s near the main floodplain constriction just downstream of the 2x2400x1850mm RCBC and low point around Ch 39900. Figure 2.2.1f shows the relative changes in velocity associated with Option N1-9a proposal which are mostly less than +0.25m/s. However, as to be expected, there are some localised increases of up to +1.0 to +1.5m/s through the existing high velocity area associated with the floodplain constriction. These impacts will require some form of mitigation or treatment to minimise the potential for increased scour or erosion.

#### *TUFLOW 2-D Assessment of Eastern Alignment Option*

In order to compare the relative hydraulic merits of the alternative eastern or western alignment options, the TUFLOW model was also modified to represent construction of the new carriageway embankment on the eastern (upstream) side of the existing highway. Figure 2.2.1g shows the impacts for the surrounding area in terms of relative change in flood level (afflux). Comparison of these results with those of the preferred western option shown on Figure 2.2.1d indicates that the impacts for the eastern alignment would be much greater and generally not acceptable under the design criteria nominated for the project.

2.2.2. Section N2 – Kyeamba Creek

**Existing Conditions**

Kyeamba Creek flows generally due north upstream of the Hume Highway and then follows a similar alignment to the highway downstream of the creek crossing. Kyeamba Creek (1st crossing) is located south of the extent of proposed duplication works. There are a number of minor culvert crossings that feed into Kyeamba Creek to the west of the highway and due north of the main creek crossing.

The main creek crossing consists of four (4) 3000mm x 3000mm RCBC's. During a site inspection in March 2007 the creek crossing was noted to be fairly densely vegetated with trees and tall shrubs. The culvert inlet and outlet appeared to be relatively clear of silt and debris with scattered vegetation that will have minimal effect on low flow events. There was evidence of long term erosion along the banks of the creek with fallen trees noted. The creek was dry at the time of the site inspection.

**Proposed Conditions**

The current design proposes a new carriageway to the west (downstream) of the existing Hume Highway which would cater for northbound traffic. The existing highway would be retained for the southbound carriageway of the duplication. Under this proposal the existing RCBC's would be retained and extended downstream to the proposed carriageway.

**Results**

Estimated design flows for Kyeamba Creek at the location of the existing Hume Highway crossing (catchment area of 1,360 hectares) are summarised in Table 2.2.2a. The WBNM sub-catchment division is shown on Figure 2.2.2a.

**Table 2.2.2a Kyeamba Creek at the Hume Highway – Peak Flows**

	ARI			PMF (5x100y ARI)
	50 year	100 year	2000 year	
Peak Flow (m <sup>3</sup> /s)	41	59	153	295

WBNM files: 10.8.4\Drainage\Flooding\N2\WBNM\KY 50 yr; KY 100yr; KY 2000 yr

The HEC-RAS hydraulic model extent and location of model cross-sections are shown on Figure 2.2.2b. A summary of design flood levels and relative impact results for the 50 year, 100 year, 2000 year ARI and PMF events are presented in Table 2.2.2b. Cross sections and detailed model results for the existing and proposed conditions, including peak flows, water levels and velocities, are provided in Appendix C.

**Table 2.2.2b Kyeamba Creek – Design Flood Results**

Cross Section See Fig. 2.2.2b	Existing Flood Level (m AHD)				Proposed Conditions [Design N2-1] Relative Impact (m) (see Note 1)			
	50 year ARI	100 year ARI	2000 year ARI	PMF	50 year ARI	100 year ARI	2000 year ARI	PMF
XS4500	357.58	357.89	358.91	359.81	-	-	-	-
XS4300	355.64	355.91	356.74	357.47	-	-	-	-
XS4100	352.42	352.75	353.83	354.71	-	-	-	-
XS3900	349.54	349.83	350.75	351.33	-	-	-	+0.02
XS3800	348.49	348.76	349.34	349.70	-	-	+0.05	+0.04
XS3600	345.24	345.52	346.17	346.58	-0.03	-0.02	-0.31	-0.26
XS3500	342.19	342.57	343.79	344.12	-0.09	+0.14	+1.53	+1.68
XS3400	341.74	342.21	343.45	343.91	n/a	n/a	n/a	n/a
XS3300	Existing Hume Highway Culvert				Proposed Hume Highway Culvert			
XS3200	341.22	341.48	342.28	342.96	-	-	-0.03	-
XS3100	337.82	338.17	339.31	340.38	-	-	-	-
XS2900	336.23	336.65	337.83	338.83	-	-	-	-
XS2700	334.18	334.58	335.77	336.80	-	-	-	-
XS2500	331.85	332.24	333.57	334.84	-	-	-	-
XS2300	328.89	329.32	330.86	332.10	-	-	-	-
XS2100	327.22	327.66	328.94	329.80	-	-	-	-
XS1900	324.93	325.26	326.35	327.23	-	-	-	-
XS1700	322.46	322.77	323.84	324.71	-	-	-	-
XS1500	320.07	320.44	321.34	322.08	-	-	-	-
XS1300	318.75	319.12	319.71	320.35	-	-	-	-
XS1000	315.51	315.79	316.50	317.10	-	-	-	-

HEC-RAS Files: 10.8.4\Drainage\Flooding\Hec\_Ras\_3.1.3\Kyeamba\KY.prj;  
existing conditions - KY.p01, KY.g01, KY.f01;  
design conditions - KY.p02, KY.g02, KY.f01

**Notes:**

- (1) Only changes greater than +/- 0.01m are shown. A change in flood level of +/- 0.01m or less is indicated by - and is considered to be within the accuracy of the hydraulic model and can therefore be ignored.

The modelling results indicate no significant impact of flooding for both existing and proposed conditions up to and including the 100 year ARI event. The estimated afflux for the proposed design is less than +0.15m upstream of the crossing for the 100 year ARI event. In more significant storm events the impacts are greater. However, the afflux is confined to the area immediately upstream of the highway.

**2.2.3. Section N3 – Little Billabong Creek**

**Existing Conditions**

Little Billabong Creek runs generally parallel to the Hume Highway on its eastern side. The creek flows to the south and crosses Little Billabong Road (MR284) near its intersection with the Hume Highway, then crosses the highway further downstream at Ch.93600, a short distance to the south of the limit of duplication.

Little Billabong Creek was dry when inspected in March 2007. The creek channel is fairly incised with scattered vegetation along the banks. The majority of the remaining floodplain area has been cleared for agricultural use. The channel is approximately 20m wide and 1.5m deep

upstream of the bridge on Little Billabong Road. The bridge deck level of approximately 319.2 mAHD is some 3.4m above the creek invert. Downstream of Little Billabong Road, a creek anabranch is located at or close to the foot of the highway embankment between chainages Ch.89000 and Ch.90500.

Two rural properties, which include residential dwellings and private infrastructure such as a pump station, adjoin Little Billabong Creek downstream (south) of Little Billabong Road. Key ground and floor levels are provided in Table 2.2.3a (refer Figure 2.2.3a for surveyed floor levels).

**Table 2.2.3a Little Billabong Creek – Property Ground and Floor Levels**

Property	Ground Level (mAHD)	Floor Level (mAHD)
Residential dwelling at Lot 118 DP 753347 (Mr Paul Lubke), south of Little Billabong Road	320.0 at dwelling	320.49
Residential dwelling at Lot 60 DP 753347, "Yammacoona" (Mr Bill Wearn), south of Little Billabong Road	319.0 at dwelling	319.35
Pump station at private property, south of Little Billabong Road	316.4	n/a

Source: RTA survey and "SH2 – Hume Highway, Construction of Dual Carriageways 84.72km to 93.19km South of Gundagai At Little Billabong – Hydraulic Investigation", November 2006.

### Proposed Conditions

The current design proposes the new carriageway to the west of the existing Hume Highway. The existing highway will generally be retained for the southbound carriageway of the duplication, with the exception of a length of some 1600m south of Little Billabong Road, where a new southbound carriageway is proposed (refer design N3-8a). The new carriageway would be located to the east of the existing highway between approximate chainages Ch.89400 and Ch.9000 and would result in filling of the Little Billabong Creek anabranch and floodplain. Proposed widening of the existing highway at its intersection with Little Billabong Road would result in fill earthworks near Little Billabong Creek.

### Results

Estimated design flows for Little Billabong Creek at the location of the Little Billabong Road and the Hume Highway crossings (catchment areas of 14,620 hectares and 24,440 hectares respectively) are summarised in Table 2.2.3b. The WBNM sub-catchment division is shown on Figure 2.2.3b.

**Table 2.2.3b Little Billabong Creek – Peak Flows**

Location	Peak Flow (m <sup>3</sup> /s)			
	ARI			PMF (5x100y ARI)
	50 year	100 year	2000 year	
Little Billabong Road	223	310	868	1550
Hume Highway	322	455	1288	2250

WBNM files: 10.8.4\Drainage\Flooding\N3\WBNM\LB 50 yr LB Road; LB 100 yr LB Road; LB 2000 yr LB Road; LB 50 yr Hume Hwy; LB 100 yr Hume Hwy; LB 2000 yr Hume Hwy; LB 50 yr; LB 100 yr; LB 2000 yr

The HEC-RAS hydraulic model extent and location of model cross-sections are shown on Figure 2.2.3c. An unsteady flow model was established and a summary of design flood levels

and relative impact results for the 50 year, 100 year, 2000 year ARI and PMF events are presented in Table 2.2.3c. The extent of inundation and relative impacts for the 100 year ARI event are shown on Figure 2.2.3d. Cross sections and detailed model results for the existing and proposed conditions, including peak flows, water levels and velocities, are provided in Appendix D.

**Table 2.2.3c Little Billabong Creek – Design Flood Results**

Cross Section See Fig. 2.2.3c	Existing Flood Level (mAHD)				Proposed Conditions [N3-8a] Relative Impact (m) (see Note 1)			
	50 year ARI	100 year ARI	2000 year ARI	PMF	50 year ARI	100 year ARI	2000 year ARI	PMF
2200	331.24	331.45	332.28	332.96	-	-	-	-
2100	330.49	330.66	331.34	331.89	-	-	-	-
2000	326.75	326.87	327.44	327.95	-	-	-	-
1900	324.94	325.12	325.90	326.61	-	-	-	-
1800	323.33	323.51	324.33	324.95	-	-	-	-
1700	320.78	320.90	321.49	322.04	-	-	-0.02	-0.02
1600	319.53	319.89	321.04	321.58	-	-	-0.10	-0.04
1500	318.65	319.09	320.72	320.87	+0.02	+0.02	-0.19	-0.10
Existing Little Billabong Road (MR284) Bridge								
1450	318.39	318.59	319.49	320.19	+0.04	+0.04	+0.08	+0.07
1400	318.33	318.54	319.34	320.01	+0.04	+0.05	+0.06	+0.06
1300	315.98	316.18	317.06	317.83	-	-	-	+0.02
1200	314.20	314.40	315.16	315.83	+0.07	+0.07	+0.17	+0.29
1120* (see Note 2)	312.88	313.05	313.80	314.48	+0.23	+0.24	+0.28	+0.33
1100	312.64	312.81	313.60	314.30	+0.18	+0.19	+0.21	+0.23
1000	311.47	311.69	312.58	313.34	-	-	-	-0.02
900	310.06	310.27	311.13	311.88	-	-	-	-
800	308.67	308.87	309.74	310.54	-	-	-	-
700	305.58	305.83	306.79	307.53	-	-	-	-

HEC-RAS Files: 10.8.4\Drainage\Flooding\Hec\_Ras\_3.1.3\LB Unsteady\LBUnsteady.prj;  
 existing conditions - LBUnsteady.p04, LBUnsteady.p05, LBUnsteady.p08, LBUnsteady.p09,  
 LBUnsteady.g02, LBUnsteady.u01, LBUnsteady.u02, LBUnsteady.u03, LBUnsteady.u04;  
 design conditions - LBUnsteady.p03, LBUnsteady.p06, LBUnsteady.p07, LBUnsteady.p10,  
 LBUnsteady.g01, LBUnsteady.u01, LBUnsteady.u02, LBUnsteady.u03, LBUnsteady.u04.

**Notes:**

- (1) Only changes greater than +/- 0.01m are shown. A change in flood level of +/- 0.01m or less is indicated by - and is considered to be within the accuracy of the hydraulic model and can therefore be ignored.
- (2) The maximum afflux was found to occur at an interpolated model cross-section located approximately 85m upstream of surveyed cross section 1100.

The hydraulic model results indicate the 100 year ARI flood level would be close to the level of the existing Hume Highway north (upstream) of Little Billabong Road between approximate chainages Ch.88450 and Ch.88750 (existing road levels of 320.0 to 319.2 m AHD). The 100 year ARI flood level would be at, or would just inundate the level of the Hume Highway at and to the north of the road sag at approximate chainage Ch.90250 (existing road level of 310.4 m AHD). An earth mound to the north, which adjoins the eastern side of the Highway between approximate chainages Ch.89800 and Ch.90000, provides some flood protection for the road and limits the length over which the 100 year ARI flood level would be at, or just inundate the Highway.

The hydraulic modelling results indicate a potential increase in 100 year ARI flood levels of up to +0.24m in the area of the proposed filling of the Little Billabong Creek anabranch and floodplain between approximate chainages Ch.89400 and Ch.90000. This would correlate to a corresponding minor increase in extent of flood inundation of up to approximately 3.5m. Filling of the creek anabranch would result in a redistribution of flood flows across the floodplain, with a potential increase in peak flows and peak velocities in the main Billabong Creek channel. The hydraulic modelling results show an increase in 100 year ARI peak velocities of up to approximately +0.3m/s (from 0.9m/s to 1.2m/s) at approximate chainage Ch.89700 (refer Appendix D, cross section 1100). Other locations show a minor decrease in 100 year ARI peak velocities.

The above hydraulic modelling results provide a conservative assessment of the potential impacts of the current design option. Additional works including construction of a new channel to replace the creek anabranch proposed to be filled by the design option would be required for effective drainage. Such works would add conveyance to the floodplain and in part assist in compensating for the filling of the anabranch.

#### 2.2.4. Section N4 – Yarra Yarra Creek

##### **Existing Conditions**

The Hume Highway crosses Yarra Yarra Creek north of its intersection with Yarra Yarra Road. The existing bridge crossing comprises five spans and has a total length of approximately 45m. The deck level of approximately 271.1 mAHD is some 8.5 m above the creek invert. The creek meanders significantly upstream (east) and downstream (west) of the bridge and runs generally parallel with the highway for about 400m downstream of the bridge before discharging to Billabong Creek.

The creek is characterised by steep, eroded banks with scattered vegetation. The channel is approximately 15m wide and 3.0m deep at the bridge. The in-bank creek geometry varies considerably upstream and downstream of the crossing. Beyond the immediate creek line, the floodplain is generally cleared and used for grazing.

##### **Proposed Conditions**

Two new bridges are proposed at Yarra Yarra Creek at about Ch.101200 (refer design N4-4b). The bridges will have a total length of about 60m for both the northbound and southbound carriageways, each consisting of 4 x 15m spans and spill through abutments. The new northbound bridge will replace the existing highway bridge to be demolished. The current design places the new northbound bridge a short distance to the west of the existing structure. This shift to the west incorporates the new bridge alignment with geometric improvements on the existing highway immediately to the north and south.

##### **Results**

Estimated design flows for Yarra Yarra Creek at the location of the existing Hume Highway crossing (catchment area of approximately 31,200 hectares) are summarised in Table 2.2.4a. The WBNM sub-catchment division is shown on Figure 2.2.4a.

**Table 2.2.4a Yarra Yarra Creek at the Hume Highway – Peak Flows**

	ARI			PMF (5x100y ARI)
	50 year	100 year	2000 year	
Peak Flow (m <sup>3</sup> /s)	350	470	1362	2350

WBNM files: 10.8.4\Drainage\Flooding\N4\WBNM\YY 50 yr; YY 100 yr; YY 2000 yr

The HEC-RAS hydraulic model extent and location of model cross-sections are shown on Figure 2.2.4b. A summary of design flood levels and relative impact results for the 50 year, 100 year, 2000 year ARI and PMF events are presented in Table 2.2.4b. Cross sections and detailed model results for the existing and proposed conditions, including peak flows, water levels and velocities, are provided in Appendix E.

**Table 2.2.4b Yarra Yarra Creek – Design Flood Results**

Cross Section See Fig. 2.2.4b	Existing Flood Level (mAHD)				Proposed Conditions [Design N4-4b] Relative Impact (m) (see Note 1)			
	50 year ARI	100 year ARI	2000 year ARI	PMF	50 year ARI	100 year ARI	2000 year ARI	PMF
LB80	276.99	277.46	278.28	278.66	-	-	-	-
LB70	275.24	275.53	276.40	276.82	-	-	-	-
LB60	273.75	274.05	274.81	275.28	-	-	-	-
LB50	271.52	271.85	273.18	273.88	-	-	-	-
YY50	273.75	274.03	274.70	275.15	-	-	+0.37	+0.54
YY40	272.49	272.66	273.42	274.05	-	-	+1.44	+1.41
YY30	271.28	271.60	272.64	273.41	+0.02	+0.03	+2.17	+1.97
YY20	270.77	271.42	272.52	273.28	+0.15	+0.06	+2.28	+2.09
YY19	270.66	271.38	272.45	273.20	+0.11	-0.03	+2.28	+2.10
	Proposed Hume Highway Bridge (SB)							
YY17	270.51	271.16	272.38	273.14	-0.03	-0.18	+0.78	+1.88
YY16	270.50	271.14	272.37	273.14	-0.04	-0.18	+0.76	+1.23
	Existing + Proposed Hume Highway Bridge (NB)							
YY14	270.35	270.79	272.35	273.13	-	-	-	-
YY10	270.31	270.77	272.28	273.03	-	-	-	-
LB40	270.02	270.45	271.82	272.43	-	-	-	-
LB30	268.59	269.00	270.40	271.05	-	-	-	-
LB20	267.46	267.84	269.08	269.79	-	-	-	-
LB10	266.17	266.59	268.03	268.86	-	-	-	-

HEC-RAS Files: 10.8.4\Drainage\Flooding\Hec\_Ras\_3.1.3\Yarra Yarra\YY.prj;  
existing conditions - YY.p06, YY.g07, YY.f01; design conditions - YY.p01, YY.g02, YY.f01

**Notes:**

- (1) Only changes greater than +/- 0.01m are shown. A change in flood level of +/- 0.01m or less is indicated by - and is considered to be within the accuracy of the hydraulic model and can therefore be ignored.

The hydraulic model results indicate the 100 year ARI flood level at the existing Hume Highway bridge would be at, or would just inundate the existing bridge deck level of 271.1 mAHD.

The current hydraulic modelling results indicate a potential increase in 50 year ARI flood levels of up to +0.15m and 100 year ARI flood levels of up to +0.06m upstream of the proposed southbound bridge. The relative hydraulic impacts are less for the 100 year ARI event due to the proposed removal of the existing northbound bridge structure and replacing it with an increased waterway area which improves the capacity of the crossing, particularly for higher flows.

The soffit levels of the proposed bridges have been set based on the current peak flood levels and impacts and include a clearance of 0.5m to the superstructure. This will be reviewed when bridge concepts are further developed and the estimated afflux is refined to suit. The resulting high embankment levels limits overtopping of the proposed highway (limiting the volume of overflow and, for the southbound carriageway, the frequency of overtopping), which in turn leads to significant afflux upstream of the proposed bridges for events in excess of the 100 year ARI event.

The estimated peak velocities at the proposed southbound bridge waterway opening are 2.1m/s and 4.3m/s for the 100 year ARI and 2000 year ARI events respectively. Estimated peak velocities at the proposed northbound bridge waterway opening are 2.2m/s and 4.8m/s for the 100 year ARI and 2000 year ARI events respectively.

### 3. Other Crossings

#### 3.1. Modelling Approach

##### 3.1.1. General

The remaining catchment areas are generally less than 500 hectares ( $5 \text{ km}^2$ ) in size at the Hume Highway. The Probabilistic Rational Method (PRM) was used to obtain peak flow estimates for these catchment areas. The performance of the culvert structures were generally assessed by inlet and outlet control calculations using software packages including CulvertW (version 1.1, 1994) and CulvertMaster (Haestad Methods, version 1.0, 1995).

##### 3.1.2. Hydrology

Details of the PRM are presented in ARR. The method is based on data from 308 gauged catchments and is applicable to rural catchments in eastern New South Wales up to 25,000 hectares ( $250 \text{ km}^2$ ) in size.

Peak flows obtained by the PRM are dependent on a runoff coefficient ( $C_{10}$  value) interpolated from a map included in ARR (Figure 5.1, Volume 2). Runoff coefficients for the study area and wider adjoining region, particularly in project Sections N2, N3 and N4, appear to be potentially affected by poor quality data from a gauge at Tumberumba, approximately 50 km to the east of Holbrook. The  $C_{10}$  values (when adjusted in accordance with the procedures set out in ARR) result in peak flow estimates that are generally considered to be very low for catchment areas in the order of 50 hectares ( $0.5 \text{ km}^2$ ) in size and greater.

In order to determine whether the ARR recommended  $C_{10}$  value was leading to an under-estimation of design flow estimates, peak flows from the PRM were compared to WBNM estimates for a number of catchment areas of varying size and location. Additional WBNM models were established using the adopted model parameters and data inputs set out in Section 2.1.3 and supported by the flood frequency analysis results discussed in Section 2.1.4. A summary of the results is provided in Table 3.1.2. Also provided in Table 3.1.2 are the calculated  $C_{10}$  values required to produce PRM peak flow estimates similar to the WBNM estimates.

**Table 3.1.2 Comparison of PRM and WBNM Peak Flow Estimates**

Section	Chainage	Catchment Area at Hume Highway (hectares)	C <sub>10</sub> from Fig 5.1 Vol 2 ARR	ARR PRM Procedure		WBNM 100y ARI Peak Flow (m <sup>3</sup> /s)	Adjusted C <sub>10</sub> required for PRM to match WBNM
				Adjusted C <sub>10</sub> (in accordance with ARR)	100y ARI Peak Flow (m <sup>3</sup> /s)		
N1	Ch.39060	57.0	0.26	0.40	7.31	5.23	0.29
N1	Ch.39250 Dellateroy Ck	5,250	0.26	0.29	146.4	162.0	0.32
N2	Ch.69400	79.9	0.10	0.21	4.83	6.35	0.27
N2	Ch.75400 Kyeamba Ck	1,360	0.10	0.14	25.4	58.9	0.31
N2	Ch.73620	28.1	0.10	0.24	2.51	2.95	0.28
N3	Ch.84970	180.1	0.10	0.18	7.91	12.2	0.28
N3	Ch.90110	13.1	0.10	0.27	1.54	1.66	0.29
N3	Ch.93600 Little Billabong Creek	24,440	0.10	0.10	149.9	454.9	0.30
N4	Ch.101900	41.9	0.10	0.23	3.19	3.80	0.27
N4	Ch.104985	291.6	0.10	0.17	10.5	14.5	0.24
N4	Ch.109020	517.6	0.10	0.16	14.5	22.2	0.24

Table 3.1.2 indicates that in Section N1, the PRM flow slightly under-estimates the WBNM result for the large catchment area and conversely, appears to over-estimate the WBNM result for a relatively small catchment area. However, the PRM flows are shown to consistently under-estimate those obtained from WBNM for a range of catchment sizes in Sections N2, N3 and N4, with the under-estimation generally increasing with an increase in catchment size.

In order to produce PRM peak flow estimates for Sections N2, N3 and N4 that are more consistent with the WBNM estimates, a minimum adjusted C<sub>10</sub> value of 0.25 has been adopted in cases where this value (when adjusted in accordance with the procedures set out in ARR) would otherwise be less than 0.25. This corresponds to catchment areas greater than approximately 20 hectares in size, which represents about 30% of the catchment areas at the Hume Highway in Sections N2, N3 and N4. No change was made for PRM estimates in Section N1.

Design flow estimates are documented in the separate design reports for each section (refer Design Report No.'s TBA).

### 3.1.3. Hydraulics

CulvertW and CulvertMaster calculations are based on the U.S. Department of Transportation (1965) nomographs. These software packages calculate:

- The headwater depth for a pipe or box culvert for a given flow and tailwater level, or calculated tailwater based on a flow in an outlet channel
- Stage/discharges for a pipe or box culvert, a weir or a culvert and weir combination.

The assessment of each culvert is documented in the separate design reports for each section (refer Design Report No.'s TBA)

## 4. References

Bureau of Meteorology 2003, The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method, Commonwealth Bureau of Meteorology, Hydrometeorological Advisory Service, June 2003.

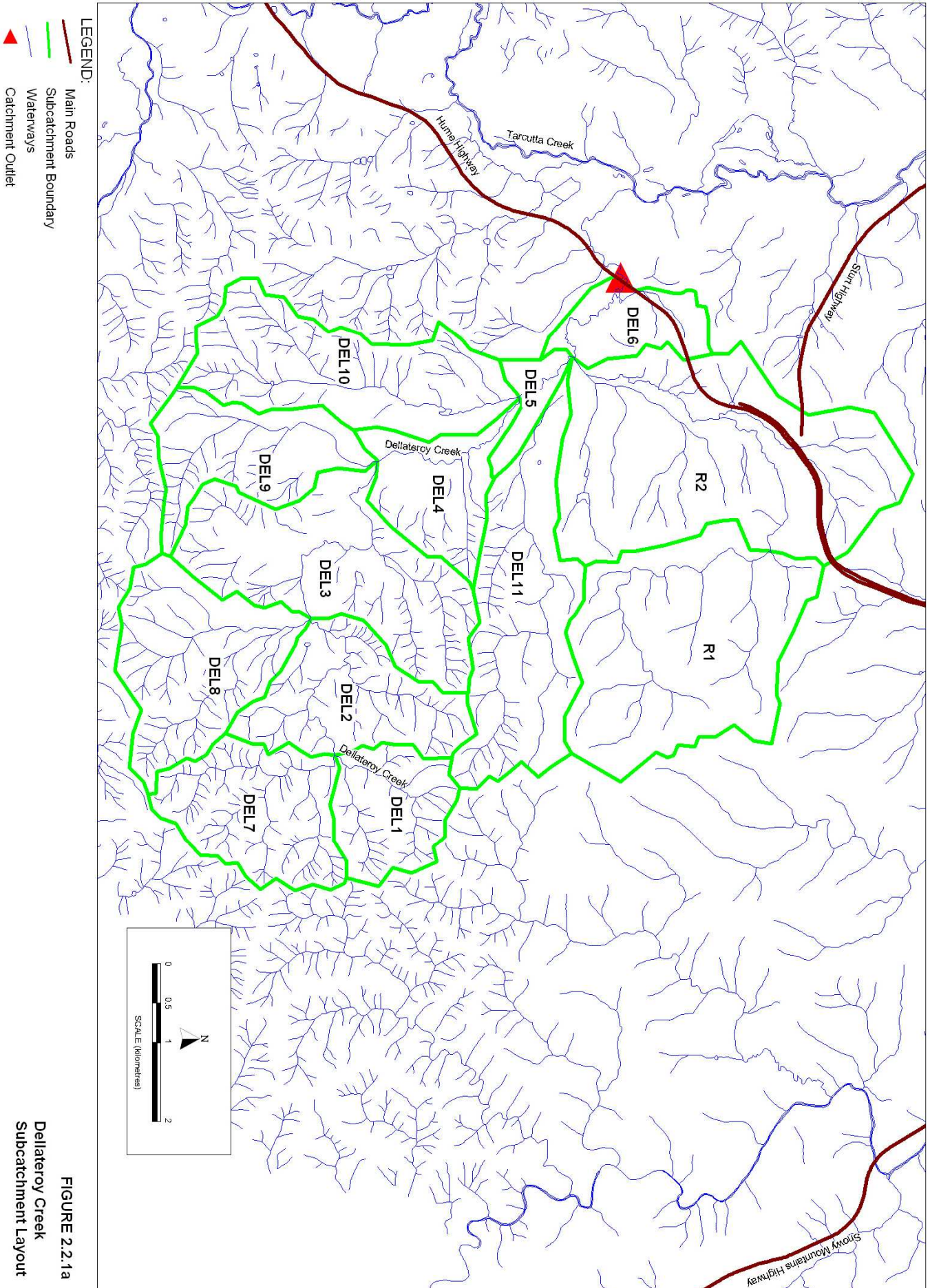
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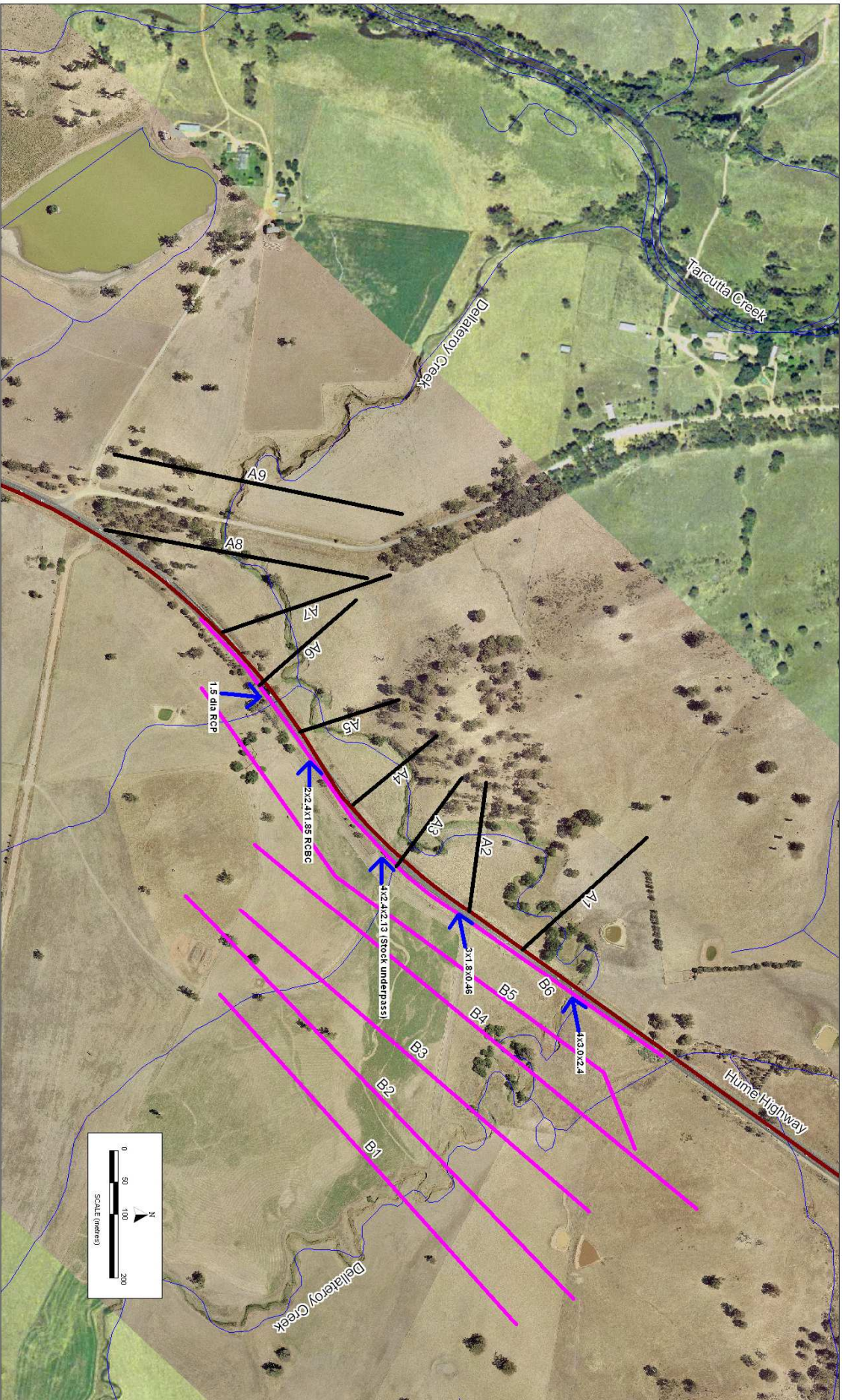
## Appendix A – Report Figures

Drawing No	Rev	Title
Figure 2.2.1a		Dellateroy Creek Subcatchment Layout
Figure 2.2.1b		Dellateroy Creek Hec-Ras Cross Section Layout
Figure 2.2.1c		Extent of Inundation and Flow Distribution – 1 in 100 year ARI Flood – Existing Conditions
Figure 2.2.1d		Relative Change in Flood Level – 1 in 100 year ARI Flood – Western Option N1-9a
Figure 2.2.1e		Velocity Distribution – 1 in 100 year ARI Flood – Existing Conditions
Figure 2.2.1f		Relative Change in Velocity – 1 in 100 year ARI Flood – Western Option N1-9a
Figure 2.2.1g		Relative Change in Flood Level – 1 in 100 year ARI Flood – Eastern Option N1-8
Figure 2.2.2a		Kyeamba Creek Subcatchment Layout
Figure 2.2.2b		Kyeamba Creek Hec-Ras Cross Section Layout
Figure 2.2.3a		House Floor Levels Little Billabong Road
Figure 2.2.3b		Little Billabong Creek Subcatchment Layout
Figure 2.2.3c		Little Billabong Creek Hec-Ras Cross Section Layout
Figure 2.2.3d		Little Billabong Creek Relative Impacts 100 yr ARI Event
Figure 2.2.4a		Billabong and Yarra Yarra Creeks Subcatchment Layout
Figure 2.2.4b		Billabong and Yarra Yarra Creeks Hec-Ras Cross Section Layout

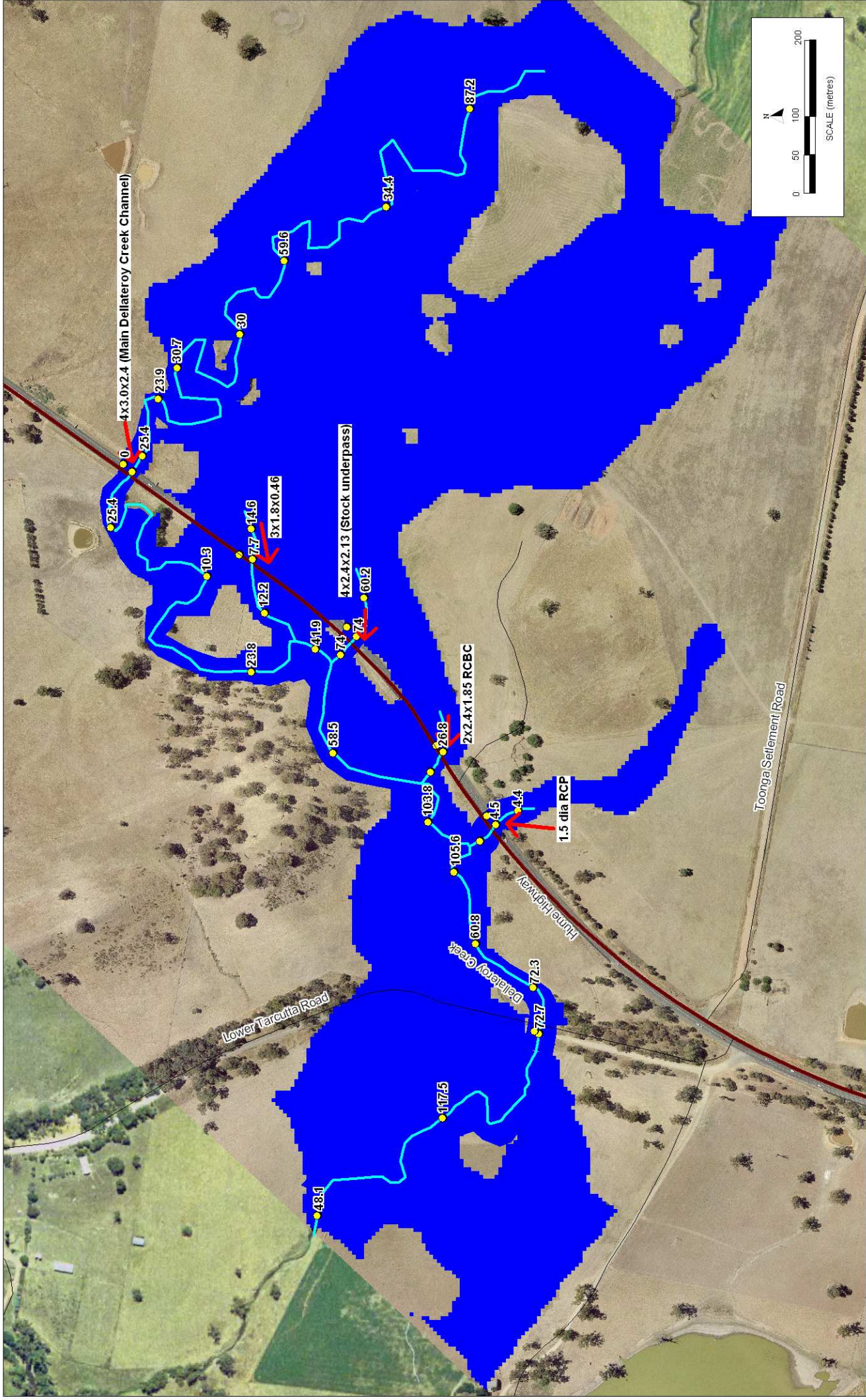


**FIGURE 2.2.1a**  
**Dellateroy Creek**  
**Subcatchment Layout**

- LEGEND:**
-  Main Roads
  -  A2 Reach A Hec-Ras Cross Section
  -  B3 Reach B Hec-Ras Cross Section
  -  Waterways
  -  Culvert Location



**FIGURE 2.2.1b**  
 Dellateroy Creek  
 Hec-Ras Cross Section Layout

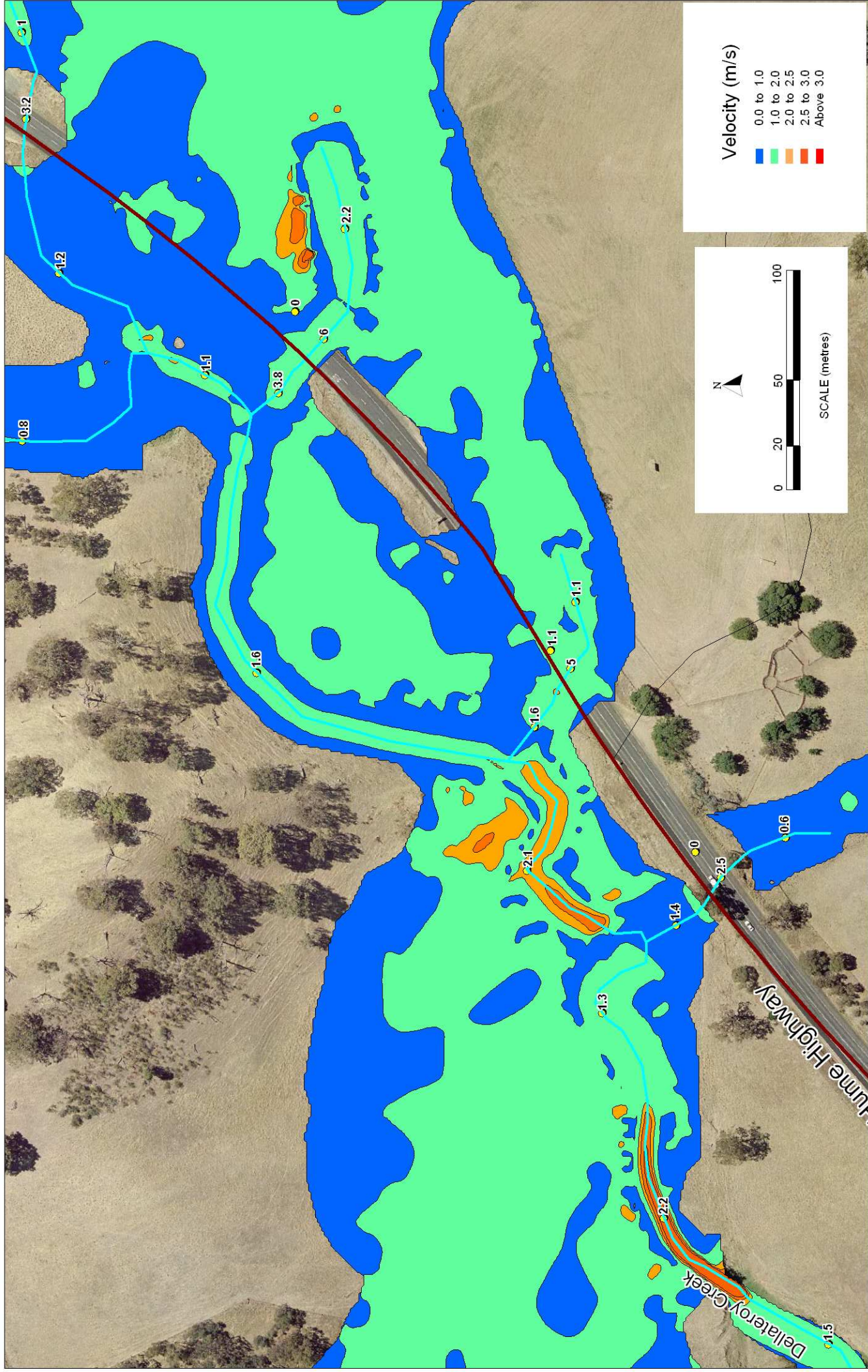


Extent of Inundation and Channel Flow Distribution 1 in 100yr ARI flood Existing Conditions

Figure 2.2.1c

- Extent of Inundation
- Channel Flow (m<sup>3</sup>/s)
- Existing Structure

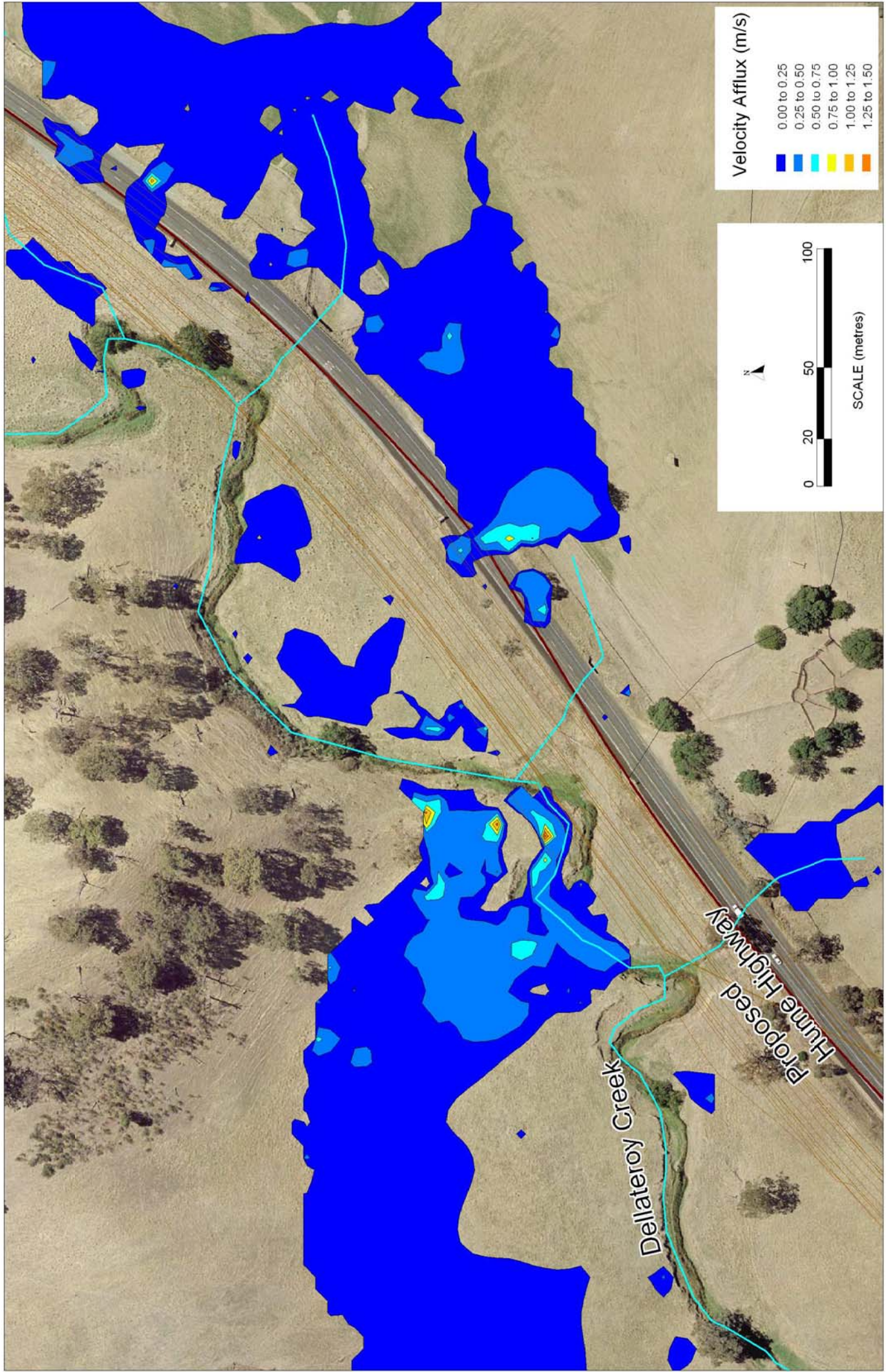




Velocity Distribution  
1 in 100yr ARI Flood  
Existing Conditions

Existing Conditions Channel Velocity (m/s)  
Waterways

Figure 2.2.1e



Relative Change in Velocity  
 1 in 100yr ARI Flood  
 Western Option N1-9A

Figure 2.2.1f