Whiteheads Creek

Flood Mapping and Intelligence Study

CG150937/V181300

Prepared for
Mitchell Shire Council

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Cover Image: Homewood Road, Whiteheads Creek near Seymour. Taken by Robin Spicer February 27th 2012.

Executive Summary

The Mitchell Shire Council (Council) are currently developing the Seymour Structure Plan. In order to facilitate this plan, the Council require an understanding of the flood risks.

The Whiteheads Flood Mapping and Intelligence Study examines the flood risks on Seymour from the Goulburn River, Whiteheads Creek, Back Creek and South Creek, as well as the risk from local drainage flooding.

The results from this study are expected to be a direct input into the Seymour Structure Plan, and appropriate flood planning and flood intelligence for the township of Seymour.

The flood mapping of Whitehead Creek catchment includes the full extent of the tributaries, however it only focuses on Whiteheads Creek.

Hydrological modelling was undertaken using RORB version 6.15. The full Whiteheads Creek catchment was delineated using the LiDAR information into 160 sub-catchments of roughly equal size. The catchment was split at locations of interest such as the gauge at Whiteheads Creek. Each catchment was assigned a fraction impervious based on the aerial imagery. The calibration of the RORB model was restricted to flows occurring at the Whiteheads Creek at Whiteheads Creek gauge as this was the only recording station with data within the catchment. Design flows were generated for the 20%, 10%, 5%, 2%, 1%, 0.5%, 0.2% and the PMF design events. Design event sensitivity, catchment development and climate change were also assessed.

The WL|Delft 1D2D modelling system, SOBEK, was used to compute the channel (1D) and overland flow (2D) components of the study. The hydraulic model was then calibrated to a number of flood events, the primary event being the 1973 and January 2016. All design events were simulated for a range of durations to ensure the critical flood event was captured.

At the same time as this project, Seymour was developing a levee system that surrounds the township. At the time of this modelling no finalised levee design was completed and levee heights were not finalised. It is understood that the levee is expected to have a minimum of 1% AEP protection with freeboard and as such the level has been set sufficiently high within the hydraulic model to reflect this. From the results, the levee causes some increases to the peak flood levels expected in the 1% AEP. These increases are less than 15 cm in the design 1% AEP flood event and do not impact any additional floors in the floodplain.

A damage assessment was undertaken to estimate the economic impact of flooding. Based on existing conditions, the AAD for the study area is approximately $629,854 per annum. From the information available at the time of this study regarding the levee system, it has been shown that the levee causes some increases in peak flood level in the 1% AEP event.

The Mitigation Report discusses mitigation and remedial works which aim to address unacceptable flood risks will be identified. Treatment options will be mapped and the benefits in terms of mitigating flood risk and cost will be included.

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1 Introduction

The Mitchell Shire Council (Council) are currently developing the Seymour Structure Plan and to facilitate this plan an understanding of the flood risks is required. This investigation aims to examine the flood risks relating to the Goulburn River, Whiteheads Creek, Back Creek and South Creek in relation to Seymour. Examination of the local drainage flooding is also a priority as this has not been examined in previous studies.

The flood mapping for the Whiteheads Creek catchment is expected to be a direct input into the Seymour Structure Plan, and will assist in developing appropriate flood planning and flood intelligence for the township.

1.1 Township

Seymour is located within central Victoria in the Mitchell Shire and is approximately 100 km north of Melbourne along the Hume Highway. It has an estimated population of 6,400 people. The location of Seymour is shown in Figure 1-1.

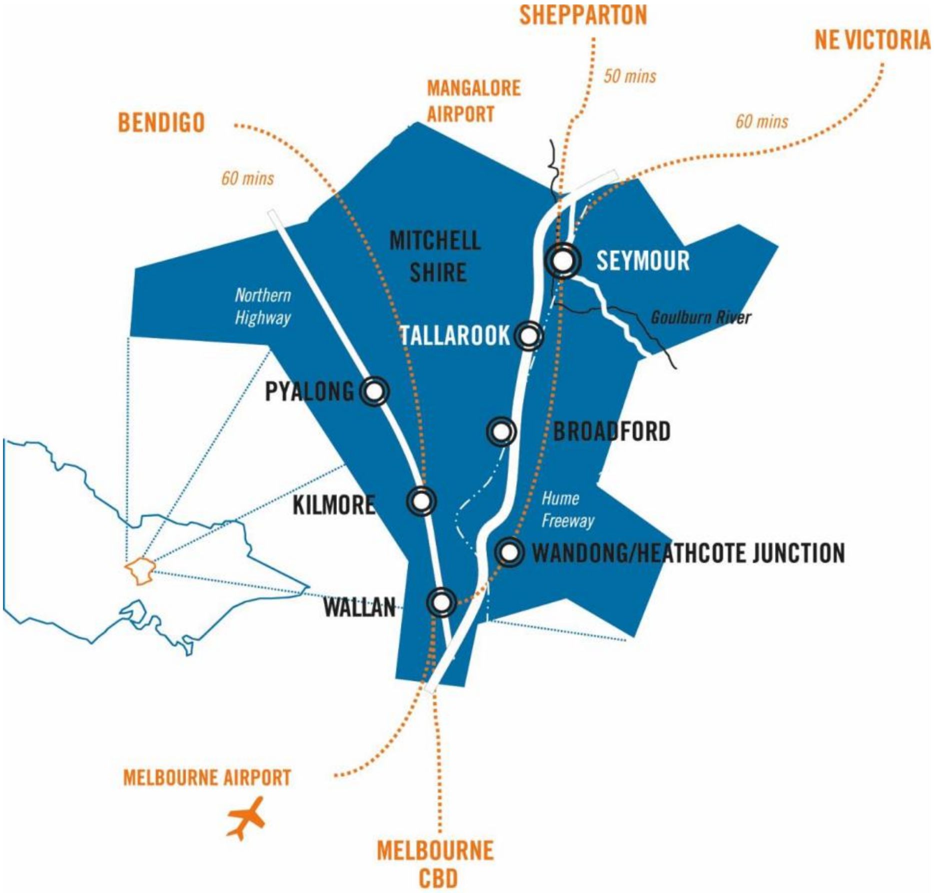


Figure 1-1 Seymour locality plan

Seymour is located along the banks of the Goulburn River. The township also has Whiteheads Creek flowing through the township, Whiteheads Creek including the tributaries of Back Creek and South Creek which join upstream of the main township. The proximity of the township to these Rivers and Creek systems has resulted in a long history of flooding since the mid-1800s. Previous studies has focussed on the flooding due to the Goulburn River and the full extent of flooding relating to Whiteheads Creek and its tributaries is not well understood or recorded.

Initial development of the town occurred on the floodplain of the Goulburn River near Emily Street. In 1870 a significant flood event inundated the entire town and highlighted the vulnerability to flooding. Following this event there was competition as to the location of town centre between Emily Street and Station Street. It was not until 1916 and 1917 when two major floods occurred that heavily impacted Emily Street that the main town centre was formalised at Station Street. The event in 1916 was considerably higher than any other on record and still remains the highest recorded flood event for Seymour.

In more recent times the most significant flood event occurred in 1974, this flooding was primarily caused by the Goulburn River and resulted in some 187 buildings suffering overfloor flooding. A smaller flood event occurred in 1993 which was below the major flood level in the Goulburn River, this resulted in 5 buildings experiencing overfloor flooding. In 2016 rapid flood event claimed the life of one person who was trapped in their car Crossing Delatite Road on Whiteheads Creek when the car was washed from the crossing.

1.2 Study Area

The study area for this hydrological and hydraulic assessment includes the full catchment of Whiteheads Creek of over 100 km^2. The catchment area is shown in Figure 1-2. The upper reaches are fed by the steep slopes of the surrounding ranges which then flow over a gentle sloping catchment which is mainly agricultural in use. Whiteheads Creek is ephemeral in nature and has a noted problem with vegetation growth along the main channels.

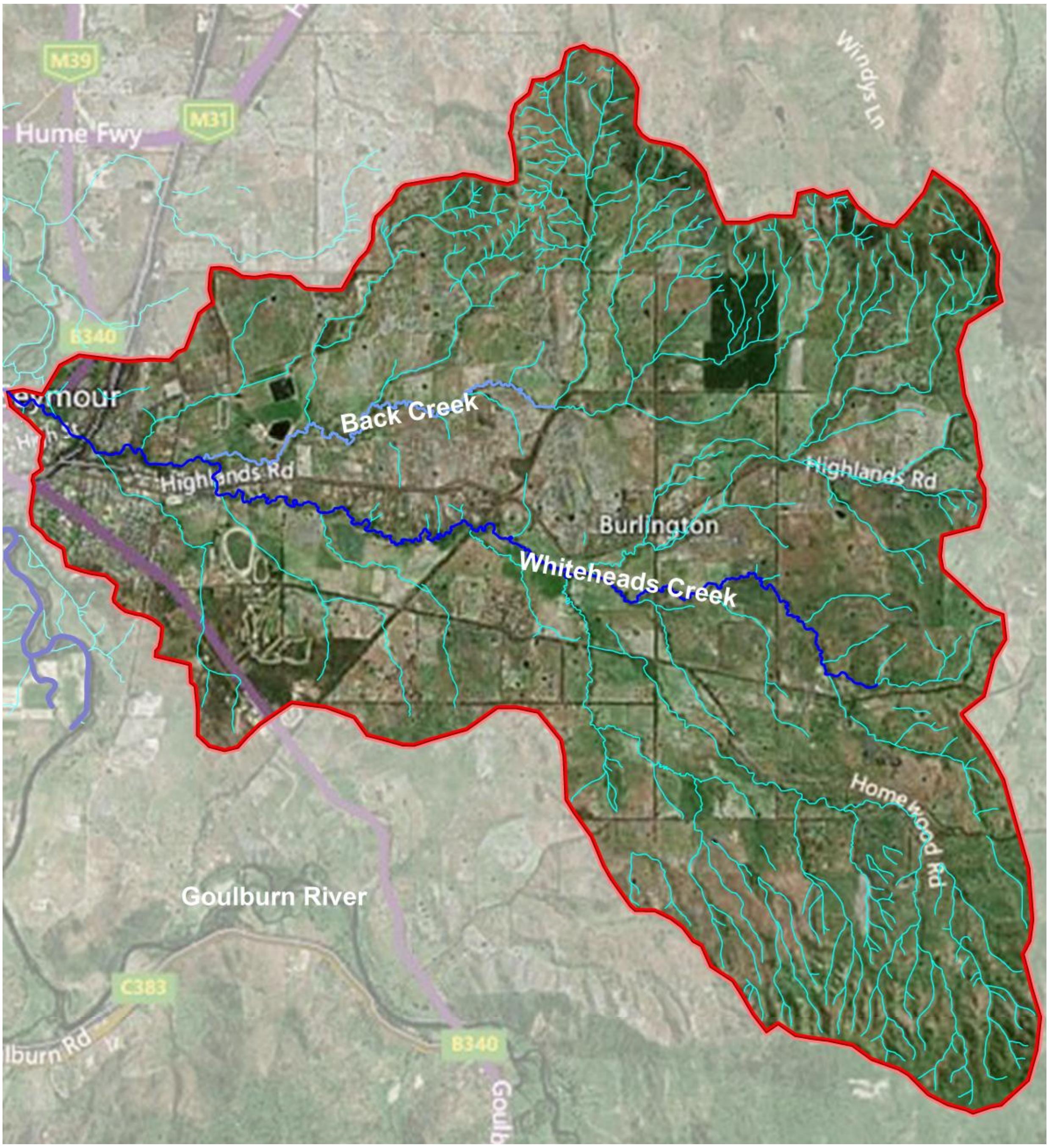


Figure 1-2 Whiteheads Creek catchment

2 Available Data

This section outlines the data collated for this project. A range of historic projects and data sources have been utilised and this aims to provide an outline of where the data was sourced from and what data was missing.

Following the inception meeting previous studies were supplied to Cardno including:

- Seymour Floodplain Mapping Study – BMT WBM completed in 2001 (BMT WBM, 2001)

  This study assessed Seymour for flooding with a primary focus on the influence of the Goulburn River and Sunday Creek. Considerable time was spend assessing the impacts of Lake Eildon and the joint probability of flooding. Preliminary flood protection levee options were established and assessed in this document.

- Seymour Flood Mitigation Communication Investigation – WBM BMT completed in 2006 (WBM BMT, 2006)

  This assessment followed on from the 2001 flood investigation. The project involved communicating the flood risk to the community and developing a mitigation short list for assessment. The report concluded by recommending a levee alignment close to the current preferred alignment.

- Letter reassessing Seymour Levee – WBM BMT completed in 2013 (WBM BMT, 2013)

  This letter updated the 2001 model and assessed the preferred levee alignment. The letter concluded that the levee was likely to have less of an impact than the 2001 and 2006 assessments reported.

- Goulburn River Reach Report – Murray Darling Basin Authority (MDBA)

The other data was supplied and captured from the sources outlined in Table 2-1.

Table 2-1 **Supplied and sourced data for the project**

Item		Format	Comment
Cadastre	Property outlines in the form of cadastral lots	TAB	Covers the full study area
Bridges	Bridge Descriptions and locations	TAB	Limited bridge information, no structure plans or information for delineating openings and cross sections.
Pipes	Drainage pipes through the township	TAB	Includes description, length and diameter only there are no upstream and downstream inverts
Pits	Drainage pit locations	TAB	Pit states if it is closed or open but does not have inverts or pit descriptions.
Planning Overlays	Planning overlays taken from Planning Maps	TAB	These cover the full study area
Land Use	Land use zones taken from Planning Maps	TAB	These cover the full study area
Levee	Alignment of the proposed levee	TAB	This shown the alignment of the revised proposed levee.

Roads	Roads layer	TAB	This covers the full study area and defines the road class.
Study Area	Outline of the study area as defined in the brief	TAB	Includes the detailed study area and Whiteheads Creek catchment area.
Aerial Imagery	Aerial imagery for the study area	ecw, ers, eww	1km x 1km tiles of the study area in detailed aerial imagery
Flow	Streamflow records	xls	Streamflow and gauge level data was sourced from the Bureau of Meteorology (BoM) website and the Department of Environment Land and water (DELWP) water portal.
Rain	Rainfall records	Txt	Rainfall and pluviograph records were purchased from the BoM for their entire records.
Streams	Stream network	TAB	The stream network was sourced directly from the VicMap layers on the DELWP data portal.
VFD	Victorian Flood Database information	TAB	The VFD datasets such as recorded historic flood heights, flood extents and information was sourced from the DELWP data portal.
NASA SRTM	Broad scale elevation data set (20m x 20m cells)	TAB	The NASA SRTM dataset was used to delineate the broad catchments where there was no LiDAR.
LiDAR	LiDAR was supplied by the Goulburn Broken CMA	asc	The LiDAR was captured in 2013 and has a vertical accuracy of 10cm. No other metadata was supplied.

3 Hydrology

The hydrology for Whiteheads Creek is required to generate suitable design flood events for mapping, planning and flood intelligence. This section outlines the approach and outcomes of the hydrological assessment.

3.1 Approach

The approach to the hydrology is to use a number of assessment methods to ensure a clear understanding of the behaviour of Whiteheads Creek. The following steps have been undertaken to develop an understanding of the catchment and subsequent design events for flood mapping.

Table 3-1 **Hydrology approach**

Section	Description
0	Assessment of the available data including streamflow, rainfall and pluviograph information.
3.3	Develop a RORB model for the full catchment
3.4	Calibrate the RORB model to historic events
3.5	Flood Frequency Assessment (FFA) for Whiteheads Creek
3.5.2	Streamflow gauge rating table assessment
3.6	Determine design events
3.7	Determine 0.1% AEP design event
3.8	Probable Maximum Flood
3.9	Design event sensitivity assessment
3.10	Developed scenario
3.11	Climate change assessment

3.2 Available Data

The three key data sources for the hydrological assessment include the daily rainfall gauges, pluviographs and recorded streamflows. These form the basis of calibrating the hydrological model and representing the catchment accurately.

3.2.1 Streamflow data

Within the Whiteheads Creek catchment there are two streamflow gauges. The original gauge is located within the township at the railway bridge and ceased recording data in 1987. The data associated with this gauge was not available for the study. The current streamflow gauge for Whiteheads Creek includes 51 km^2 of the catchment which is approximately half the total whiteheads Creek catchment area. The data availability is summarised in Table 3-2 and the locality of the gauges is shown in Figure 3-1.

Table 3-2 **Streamflow gauge available for the study**

Gauge	Gauge Name	Area (km^2)	Data	Start Date	End Date
405291	Whiteheads Creek at Whiteheads Creek	51	Instantaneous flow and level	15/09/1988	06/01/2016
			Daily Average flow and level	15/09/1988	06/01/2016
405202	Goulburn River at Seymour		Instantaneous flow and level	11/06/1975	15/02/2016
			Daily Average flow and level	11/06/1975	15/02/2016

On the Goulburn River there is a long term streamflow gauge at Seymour. This has been used to set levels for the downstream conditions of the hydraulic model.

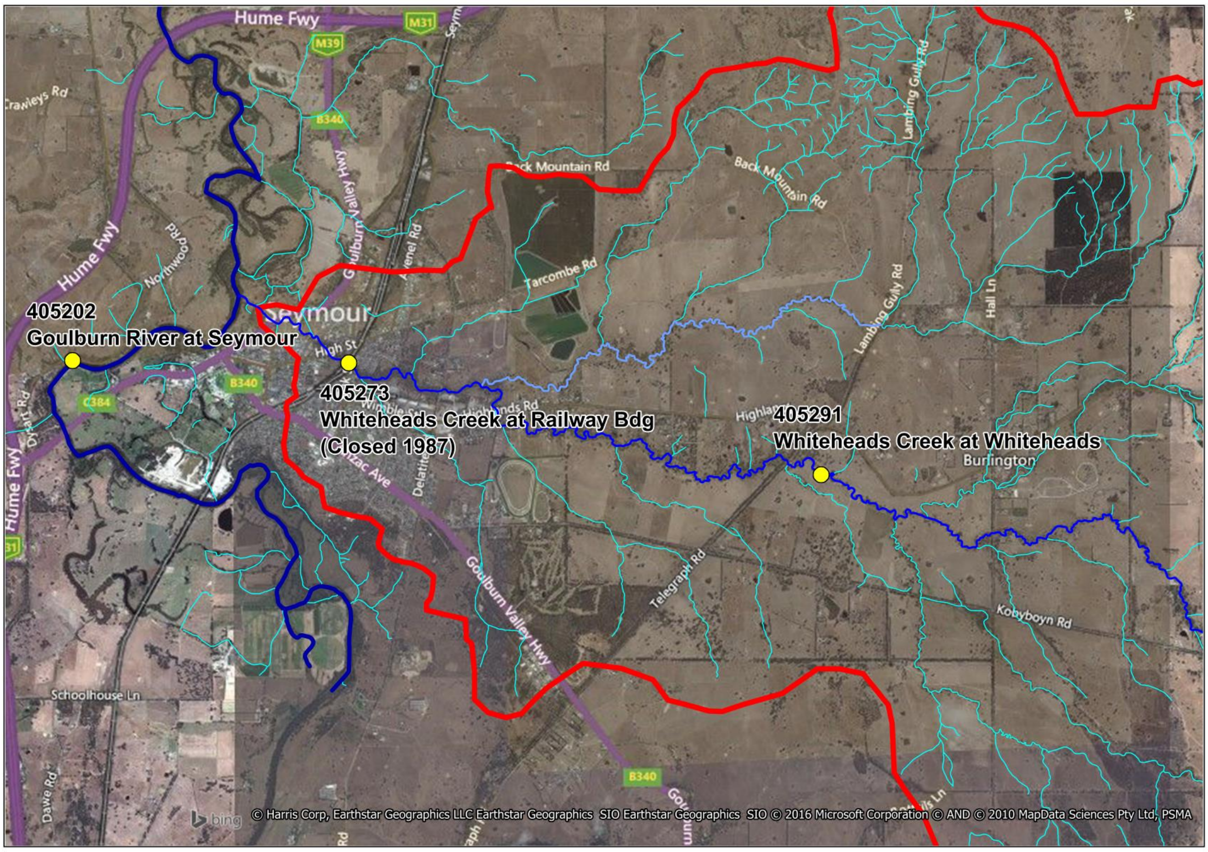


Figure 3-1 Streamflow gauges for the Whiteheads Creek model

The primary gauge for this catchment was Whiteheads Creek at Whiteheads Creek (405291) which had a record of 27 years. The gauge is located midway up the catchment as such does not include important tributaries such as Back Creek and South Creek. From the instantaneous data, the maximum flow annual rate was extracted and this is shown in Figure 3-2.

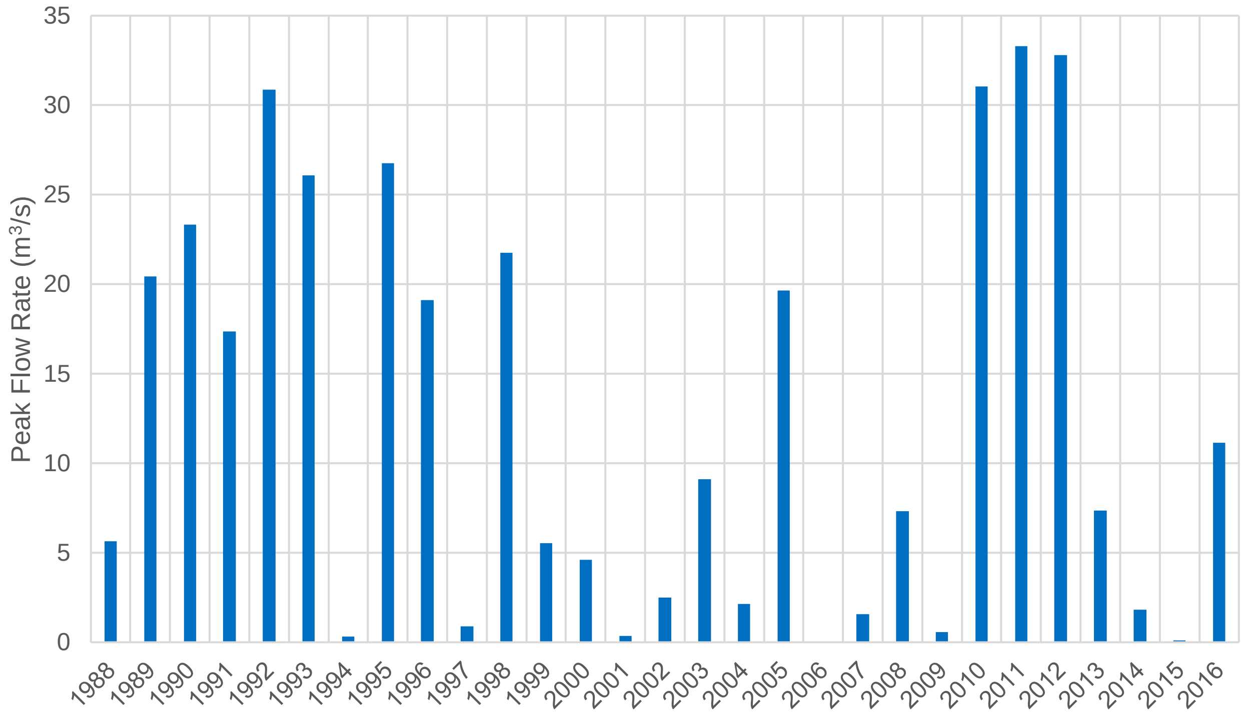


Figure 3-2 Maximum flow rates for Whiteheads Creek at Whiteheads creek (405291)

For Whiteheads Creek the peak flow years were 1992, 2010, 2011 and 2012. These flows were all over 30 m^3/s. In 1993 the Goulburn River flooded Seymour, from the flow record the flooding from Whiteheads Creek was not as significant. Due to the recent nature of the flood events, the 2010 to 2012 period was used to generate the calibration events.

3.2.2 Rainfall and Pluviograph Stations:

The rainfall and pluviograph station used in this assessment are summarised in Table 3-3 and the spatial location of the gauges is shown in Figure 3-3.

The rainfall gauges at Seymour Shire Depot and Avenel cover the lower flats north and south of the catchment, whilst the Highlands (Glentannar) accounts for higher elevation east of the catchment. There are sufficient rainfall records to accurately define rainfall totals for the historic large events.

The majority of the pluviograph gauges close to the catchment were closed for the events between 2010 and 2012. Therefore gauges further away at Heathcote and Strath Creek were required to be used.

Table 3-3 **Rainfall and pluviograph gauges for the Whiteheads Creek catchment**

Number	Name	Data Type	Start Date	End Date
88109	Mangalore Airport	Rainfall	01/01/1957	Open
88002	Avenel (Post Office)	Rainfall	14/12/1900	30/11/2015
88053	Seymour Shire Depot	Rainfall	01/05/1880	Open
88126	Goulbourn River at Seymour	Rainfall	18/10/2001	Open
88031	Highlands (Glentannar)	Rainfall	01/06/1915	27/12/2015
88029	Heathcote	Pluviograph	17/04/1968	31/08/2015
88158	Strath Creek	Pluviograph	01/08/1991	31/08/2015

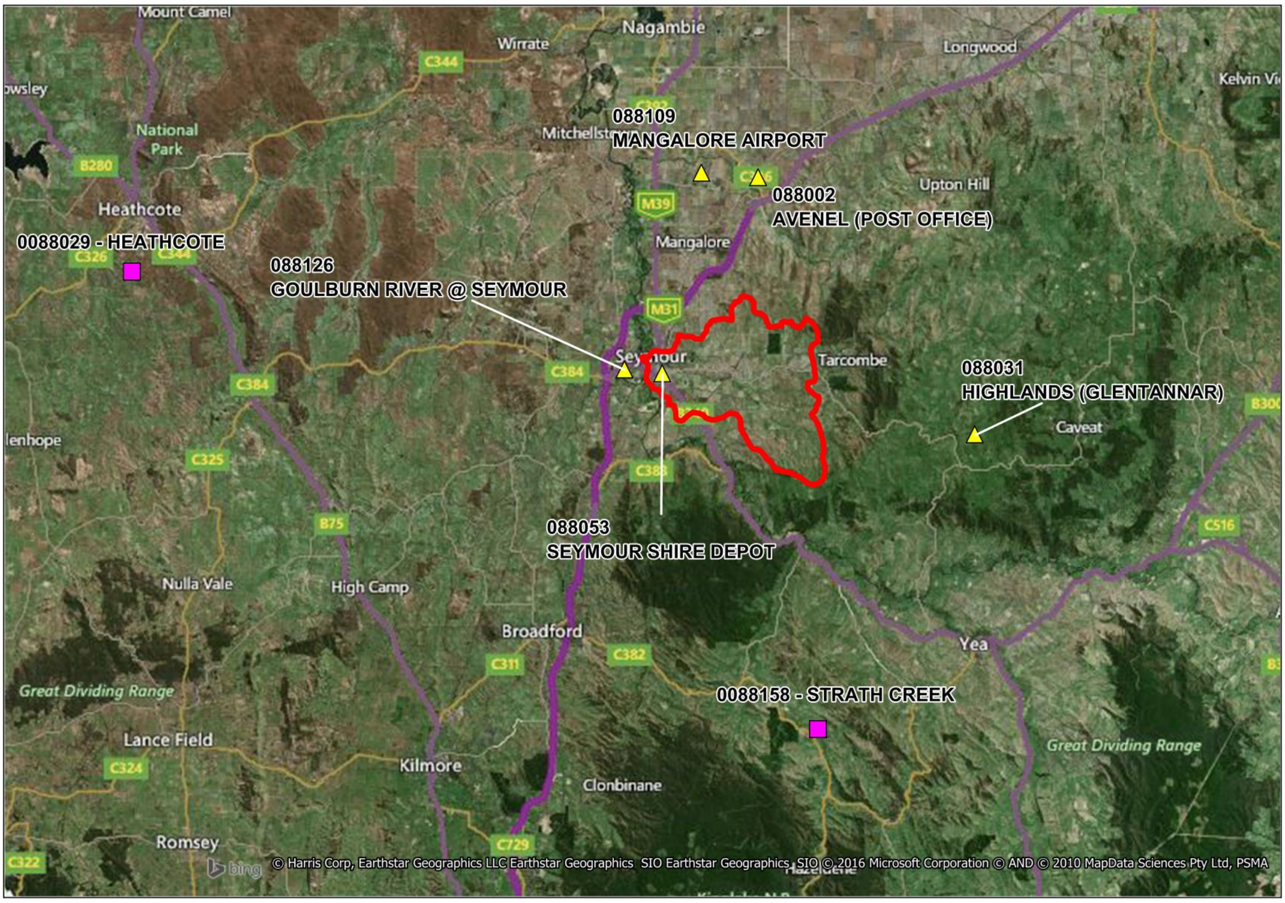


Figure 3-3 Location of the rainfall and pluviograph gauges

3.3 RORB Model Development

The RORB model was developed using RORB version 6.15. The full Whiteheads Creek catchment was delineated using the LiDAR information into 160 sub-catchments of roughly equal size. The catchment was split at locations of interest such as the gauge at Whiteheads Creek. Each catchment was assigned a fraction impervious based on the aerial imagery. For the rural catchments with only minor roads the fraction impervious was set at 2.5%. For the urban catchments the fraction impervious was set based on a rough delineation of the hard surfaces. These ranged from 10% up to 50%.

Flow paths represent the average travel time from the sub area and utilise the streamflow paths where possible.

The RORB model was generated using the MiRORB (version 1.2) tool. This tool produces a CATG file that is spatially referenced.

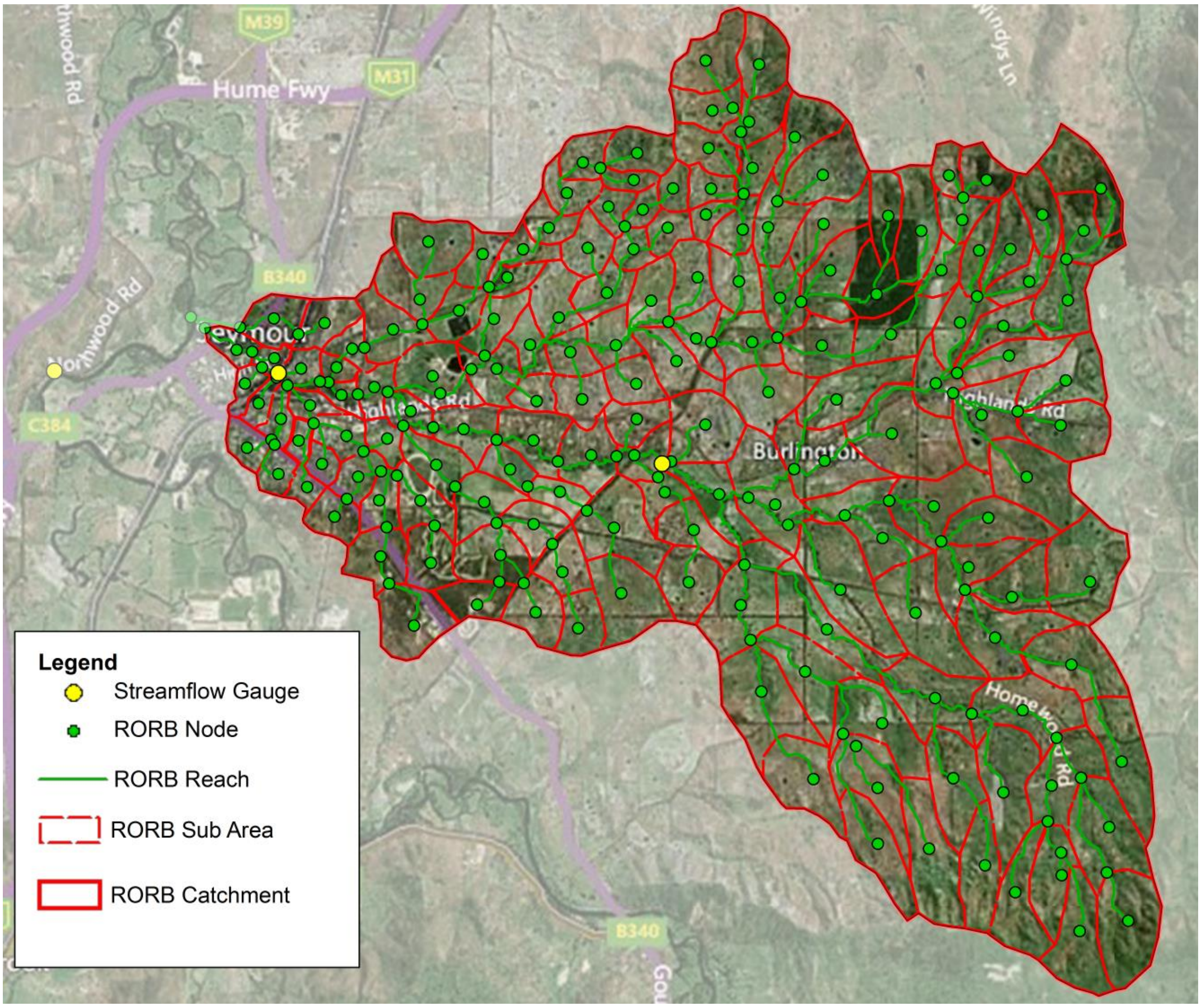


Figure 3-4 RORB hydrology model establishment

3.4 RORB Calibration Events

The calibration of the RORB model was restricted to flows occurring at the Whiteheads Creek at Whiteheads Creek gauge as this was the only recording station with data within the catchment. The calibration events were selected from 2010 to 2012 as these were the most recent events and were more likely to have recorded rainfall and pluviograph data. Where possible events with single catchment responses were selected to assist in identifying the key catchment response parameters.

A summary of the calibration events are shown in Table 3-4 and the peak events are shown in Table 3-5. Each calibration is presented in the following sections. For the purpose of calibration, the original rating table peak flow rates have been used but it should be noted that there have clearly been rating table changes (note Feb-12 has a lower peak flow rate than Jan-11) and there is some concern about the high flow rating.

Table 3-4 **Summary of the streamflow gauge Whiteheads Creek at Whiteheads Creek**

Site	Whiteheads Creek at Whiteheads Creek
Gauge Record	28 years
Dates	1989 – 2017 (still open)
Data Type	Instantaneous Flows

Table 3-5 **Top 10 ranked events for the Whiteheads Creek gauge**

Rank	Date	Peak Level (m)	Recorded Flow (m^3/s)[1]	Updated rating table flow (m^3/s)
1	Feb-12	4.13	26.8	55.8
2	Jan-11	4.05	33.3	38.7
3	Sep-10	4.00	31.1	29.2
4	Oct-92	3.99	30.9	29.2
5	Jun-95	3.88	26.7	28.6
6	Dec-10	3.87	26.2	28.5
7	Oct-93	3.86	26.1	28.5
8	Jul-10	3.81	24.4	28.2
9	Jul-90	3.78	23.3	28.1
10	Sep-93	3.76	22.6	27.9

[1] Flows based on the current rating table at Whiteheads Creek gauge, see Section 3.5.2 for detailed discussion on the rating curve and subsequent adjustments.

3.4.2 March 2010

The March 2010 event was the fourth largest in 2010, however this was a single burst rainfall event which allowed for a robust calibration to be achieved in RORB. The flood event contained one rainfall burst which caused the main flood event to occur on the 7th March 2010. The recorded hydrograph is shown in Figure 3-5.

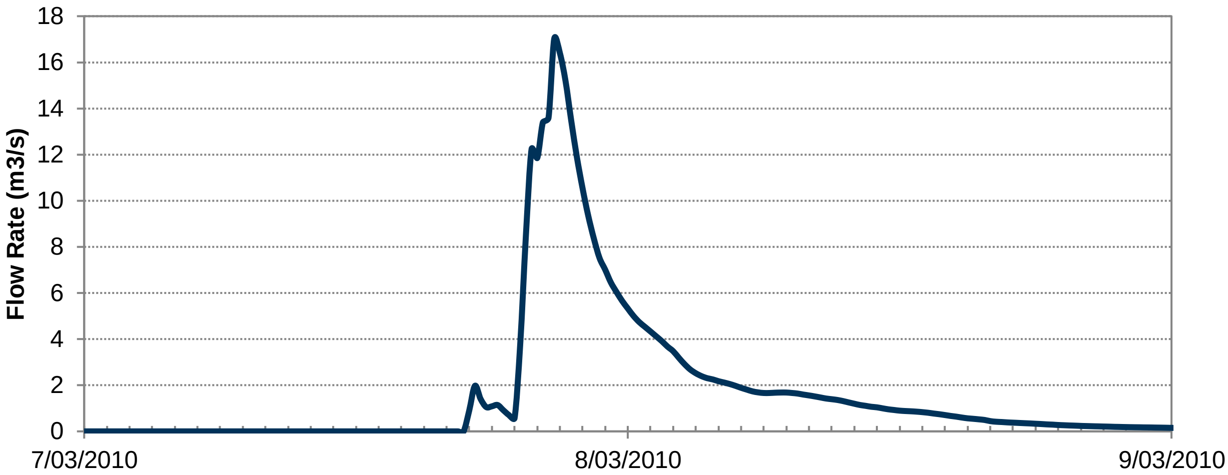


Figure 3-5 March 2010 recorded hydrograph

The rainfall was characterised by an intense burst on the 7th March followed by some sustained rainfall through the 8th March. Table 3-6 shows the rainfall depths for the 7th and 8th March. The streamflow peak flow occurs at around 9pm on the 7th March so the rainfall totals recorded for this event are likely to be between 40 – 70 mm over the two days. There is a notable difference between the rainfall totals for the Seymour Shire Depot and the Goulburn River at Seymour gauge which suggests there may have been some variability in the rainfall across the catchment.

Table 3-6 **March 2010 rainfall depths**

Gauge	7-Mar	8-Mar	Total
88109 - Mangalore Airport	18.4	48.6	**67.0**
88002 - Avenel (Post Office)	-	-	**N/A**
88053 - Seymour Shire Depot	24.0	54.0	**78.0**
88126 - Goulbourn River at Seymour	18.2	34.8	**53.0**
88031 - Highlands (Glentannar)	15.4	55.0	**70.4**

The event was calibrated in RORB with a single burst and a rainfall total across the event of 53.0 mm. The resulting hydrograph is shown in Figure 3-6 and the calibrated parameters are shown in Table 3-7.

Table 3-7 **Calibrated parameters for the March 2010 event**

Parameter	Value
k_c	4.0
m	0.8
IL (mm)	11.4
CL (mm/hr)	5.8

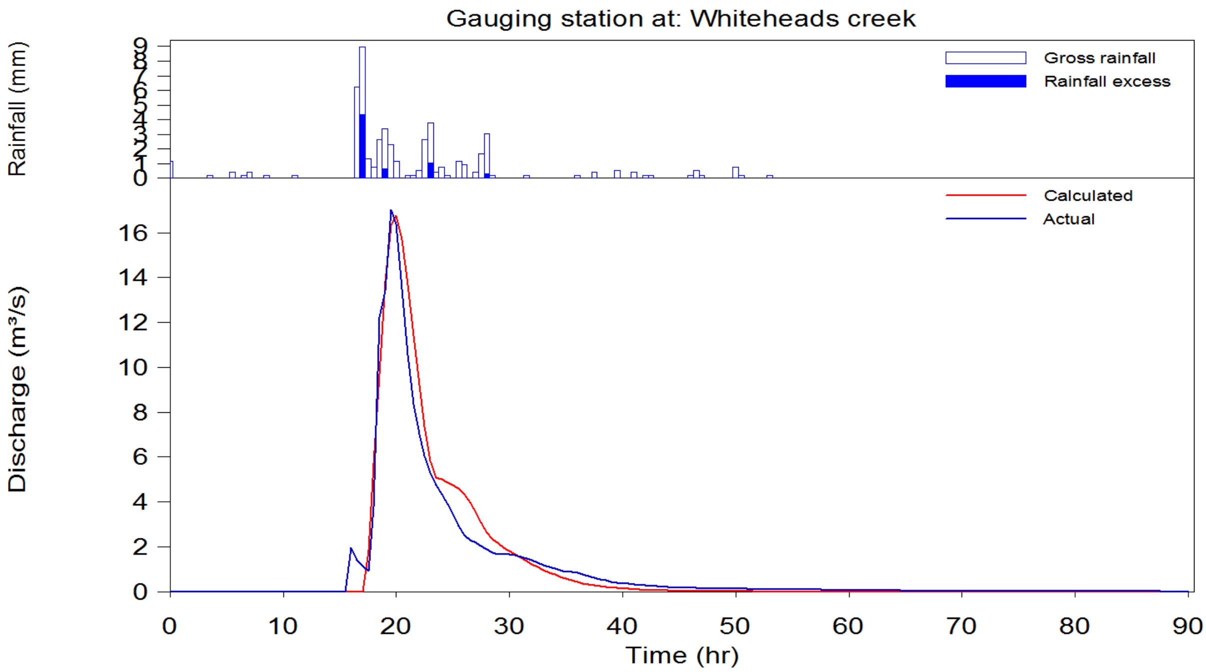


Figure 3-6 Calibrated RORB results in the March 2010 event

The calibration was good with the peak matched to within 1%, the timing of the peak was also well matched. The overall volume of the hydrograph was within 5% of the recorded hydrograph. The resulting high continuing loss rate suggests that the depth of rainfall applied may be too high based on the recorded rainfall totals.

3.4.3 July 2010

The July 2010 event peaked at 24.4 m^3/s at approximately midnight on the 14th July 2010 as shown in in Figure 3-7. The event was characterised by a sharp rising limb to the peak which then receded at a slightly decreased rate before a second small rise in flow on the 15th July 2010.

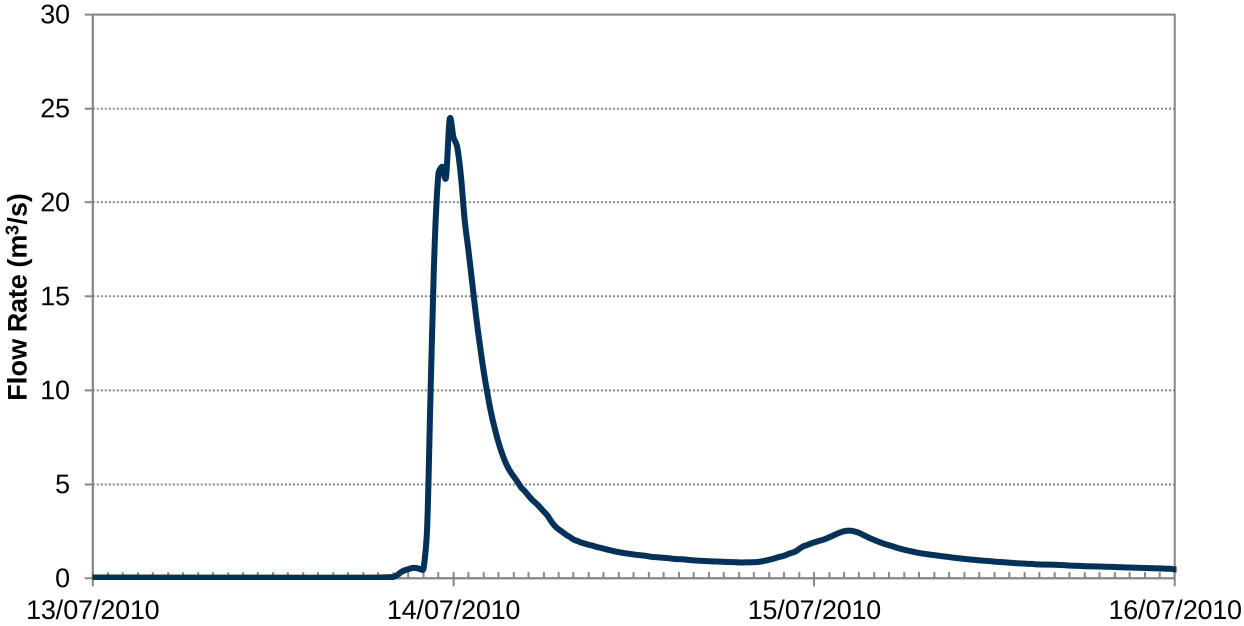


Figure 3-7 July 2010 recorded hydrograph

The rainfall for this event was much lower than the totals recorded in March 2010 and the peak flow was higher. This suggests that the catchment antecedent conditions were wet and the rainfall was quickly converted to runoff. The rainfall total for this event was in the range of 30 to 55 mm.

Table 3-8 **July 2010 rainfall depths**

Gauge	14-July	15-July	Total
88109 - Mangalore Airport	25.8	4.2	**30.0**
88002 - Avenel (Post Office)	27	4.8	**31.8**
88053 - Seymour Shire Depot	32	8.2	**40.2**
88126 - Goulbourn River at Seymour	0	27.8	**27.8**
88031 - Highlands (Glentannar)	43.2	12.4	**55.6**

The pluviograph data for this event captured all of the days. The Heathcote gauge was used for this event, and was close to matching Mangalore Airport and Avenel (Post Office) gauges with a total rain fall of 32 mm between the 13th and 15th of July. The Mangalore Airport data was used for the RORB calibration due to the similar daily and pluviograph rainfall amounts and the continuity in the data set.

Table 3-9 **Calibrated parameters for the July 2010 event.**

Parameter	Value
k_c	5.0
m	0.8
IL (mm)	13.7
CL (mm/hr)	1.6

The RORB model was well calibrated with a close match to the recorded hydrograph. The peak flow rate was matched well with an identical peak reached with similar timing. The volume for the event was within 5% of the recorded hydrograph. The timing and response for the secondary peak was not exactly matched however as the pluviograph is a fair distance away from the site this was considered acceptable.

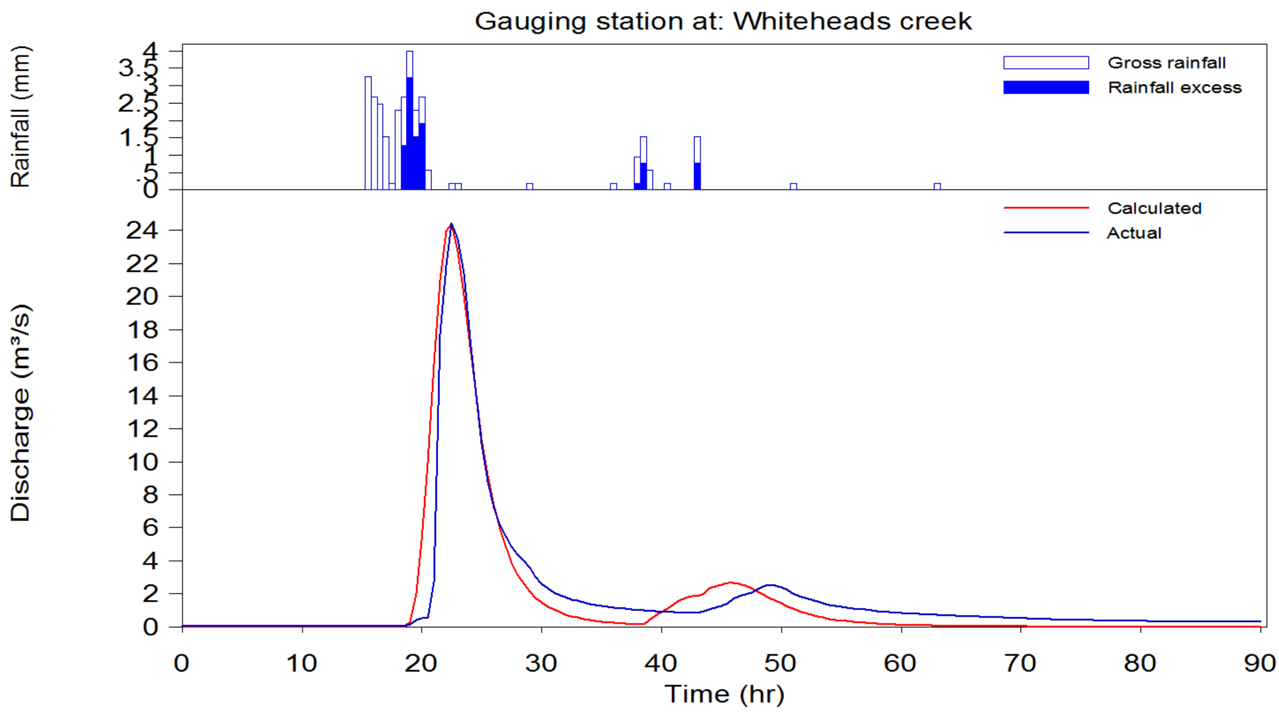


Figure 3-8 Calibrated RORB results in the July 2010 flood event

3.4.4 December 2010

The December 2010 event had a peak flow rate of 26.0 m^3/s which occurred at around 3pm on the 8th December as shown in Figure 3-9. The event has multiple peaks with the main event occurring on the 8th December with a secondary peak on the 9th December.

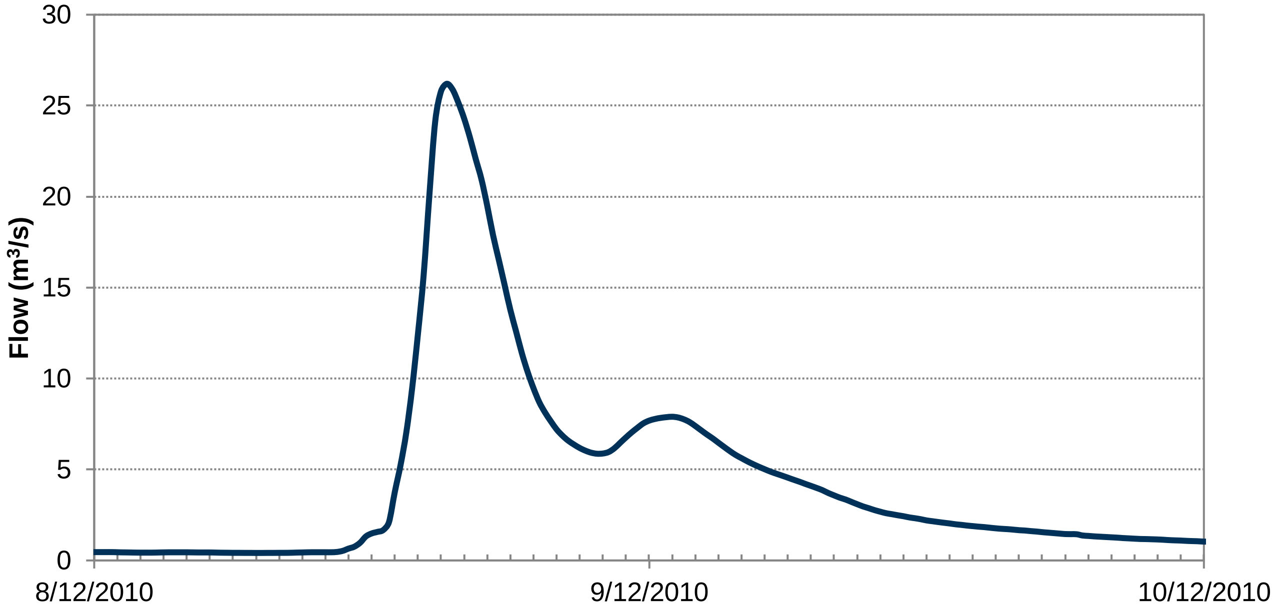


Figure 3-9 December 2010 recorded hydrograph

The rainfall for this event was concentrated on the 8th and 9th of December with more rainfall recorded in the study area of Seymour than in the surrounding rainfall gauges. No rainfall was collected at the Goulbourn River at Seymour gauge. The rainfall totals are shown in Table 3-10 and from these recorded totals it is evident that the Seymour Shire depot gauge recorded higher rainfalls at 71.4 mm than the surrounding areas. This rainfall total was used in the RORB calibration for the catchment.

Table 3-10 **December 2010 rainfall depths**

Gauge	7-Dec	8-Dec	9-Dec	10-Dec	Total
88109 - Mangalore Airport	0	23.8	20.0	0.2	**44.0**
88002 - Avenel (Post Office)	0	21.4	23.8	0	**45.2**
88053 - Seymour Shire Depot	0	34.2	37.2	0	**71.4**
88126 - Goulbourn River at Seymour	-	-	-	-	**N/A**
88031 - Highlands (Glentannar)	0	30.6	34.8	0	**65.4**

The Strath Creek pluviograph gauge was used to distribute the rainfall. The total rainfall recorded at the pluviograph was 60.4 mm which suggests that the rainfall event passed over this location. The calibrated RORB parameters are summarised in Table 3-11.

Table 3-11 **Calibrated parameters for the December 2010 event**

Parameter	Value
k_c	5.0
m	0.8
IL (mm)	52.7
CL (mm/hr)	1.9

The RORB model offered a close match to the recorded hydrograph. However, the pluviograph has a burst early on in the event which is not represented in the hydrograph, as a result the initial loss is very high to remove this portion of the rainfall series. The timing of the first peak and the decline to the second peak were very close. Although the initial loss is quite high, the RORB model managed to match the peak and the timing well for the event. The overall hydrograph volume was lower than the recorded event, however this was thought to be due to the pluviograph pattern rather than the event calibration. This is shown in Figure 3-10.

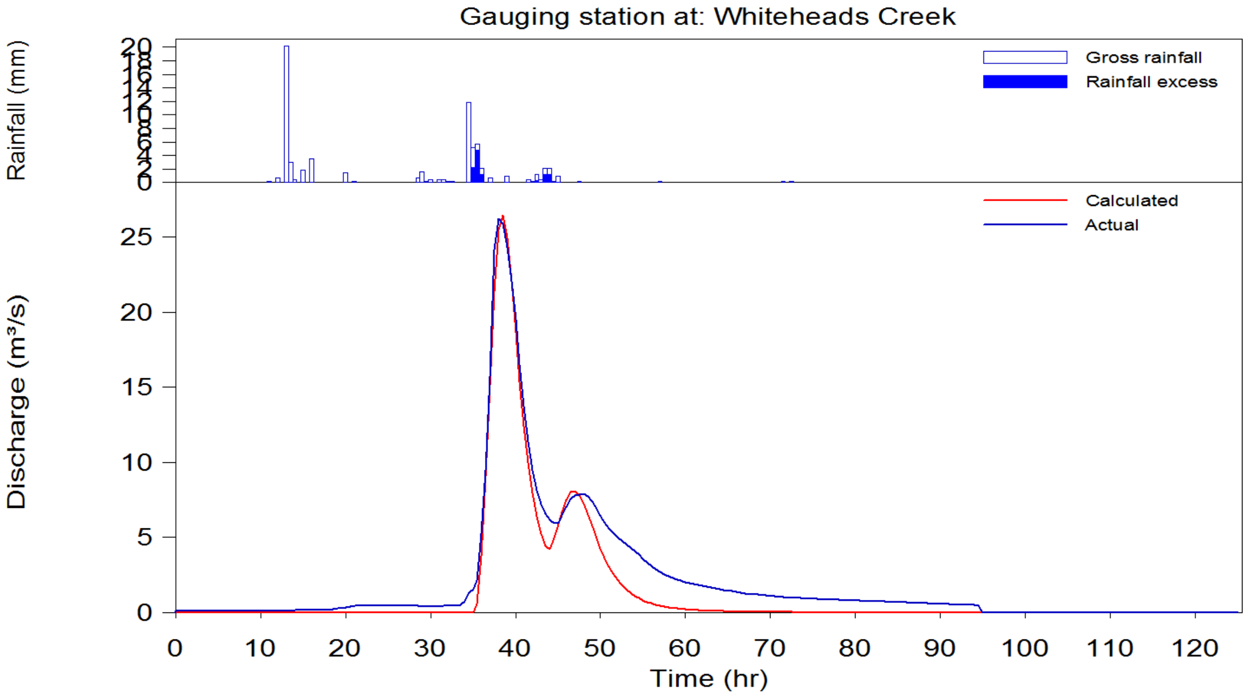


Figure 3-10 Calibrated RORB results for the December 2010 event.

3.4.5 January 2011

The January 2011 event had two large peaks of 26.5 and 33.3 m^3/s occurring on the 14th January. The first peak rose and reduced sharply, whilst the second, larger peak held its large flow rate for 1.5 hours before falling away. The hydrograph for this event is shown in Figure 3-11.

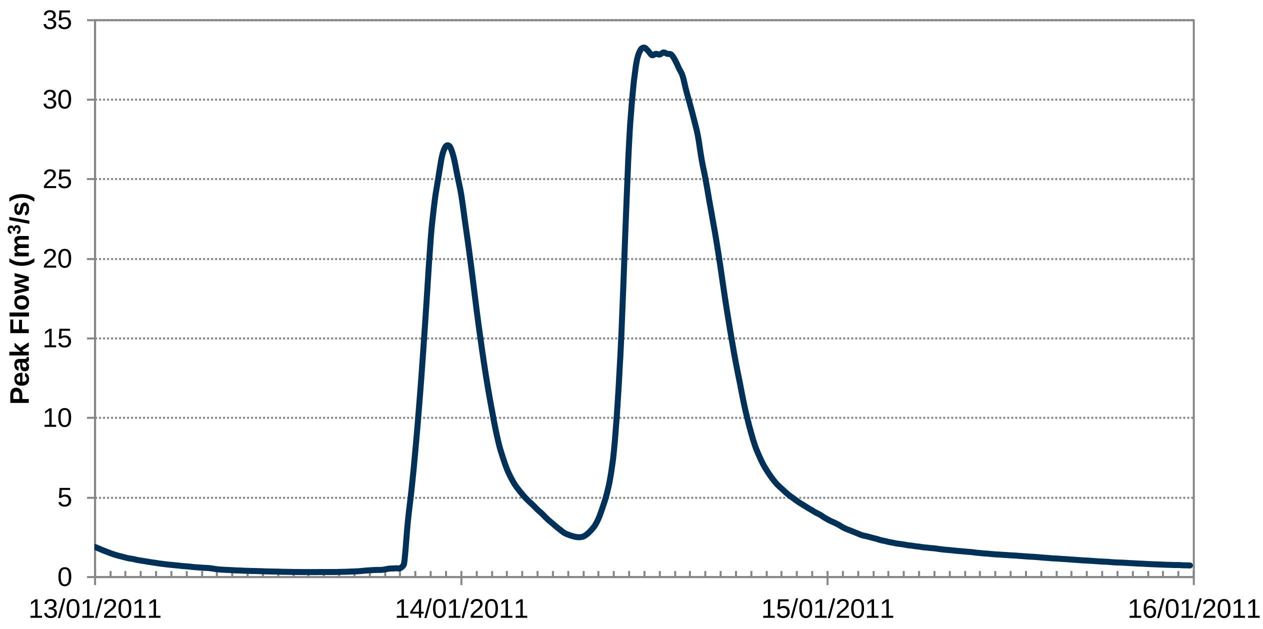


Figure 3-11 January 2011 recorded hydrograph

The rainfall for this event was varied across all gauges. Two of the gauges, Seymour Shire Depot and Goulbourn River at Seymour had missing records of the period, so were unable to be used in calibration. It was decided to use the Avenel (Post Office) gauge for the calibration.

Table 3-12 **January 2011 rainfall depths**

Gauge	13-Jan	14-Jan	15-Jan	Total
88109 - Mangalore Airport	41.4	70.8	16.8	129.0
88002 - Avenel (Post Office)	50.2	42.6	15.8	108.6
88053 - Seymour Shire Depot	20	35	-	55
88126 - Goulbourn River at Seymour	-	-	-	N/A
88031 - Highlands (Glentannar)	31.4	56	28.6	116.0

The pluviograph data for this event was limited. The Strath Creek gauge had no data for the event, whilst the Heathcote gauge had no data for the 15th January 2011. Pluviograph data for the 13th-14th January was available using the Heathcote pluviograph and this was used. The Heathcote pluviograph recorded 73 mm of rainfall over this period which indicates the event passed this location.

The calibrated parameters are summarised in Table 3-13. The calibration was undertaken using a multi burst approach as the hydrograph and rainfall patterns have two distinct bursts that required different loss rates.

Table 3-13 **Calibrated parameters for the January 2011 event.**

Parameter	Value
k_c	5.0
m	0.8
Burst 1	
IL (mm)	58.3
CL (mm/hr)	2.0
Burst 2	
IL (mm)	0.0
CL (mm/hr)	5.8

The calibration of the January 2011 event is not as well matched compared to the previous calibrations. The RORB model replicated the first peak of the hydrograph and matched the receding limb of this event closely. The second rainfall burst does not seem to have enough sustained rainfall for the RORB model to match the recorded hydrograph and the distribution of the rainfall does not match the hydrograph response. The peak flow rate and gauged level recorded for this event exceeds the highest gauging for the streamflow gauge. This implies the flow rate has some uncertainty for the largest peak. This gauging assessment has been discussed in detail in Section 3.5.2.

The recorded and RORB modelled hydrographs are shown in Figure 3-12. The overall calibration shows the catchment responding at a similar rate to the recorded hydrographs, the second hydrograph has insufficient volume to match the recorded hydrograph but this was thought to be more about the rainfall distribution based on the pluviograph data and potentially due to the rating table of the gauge.

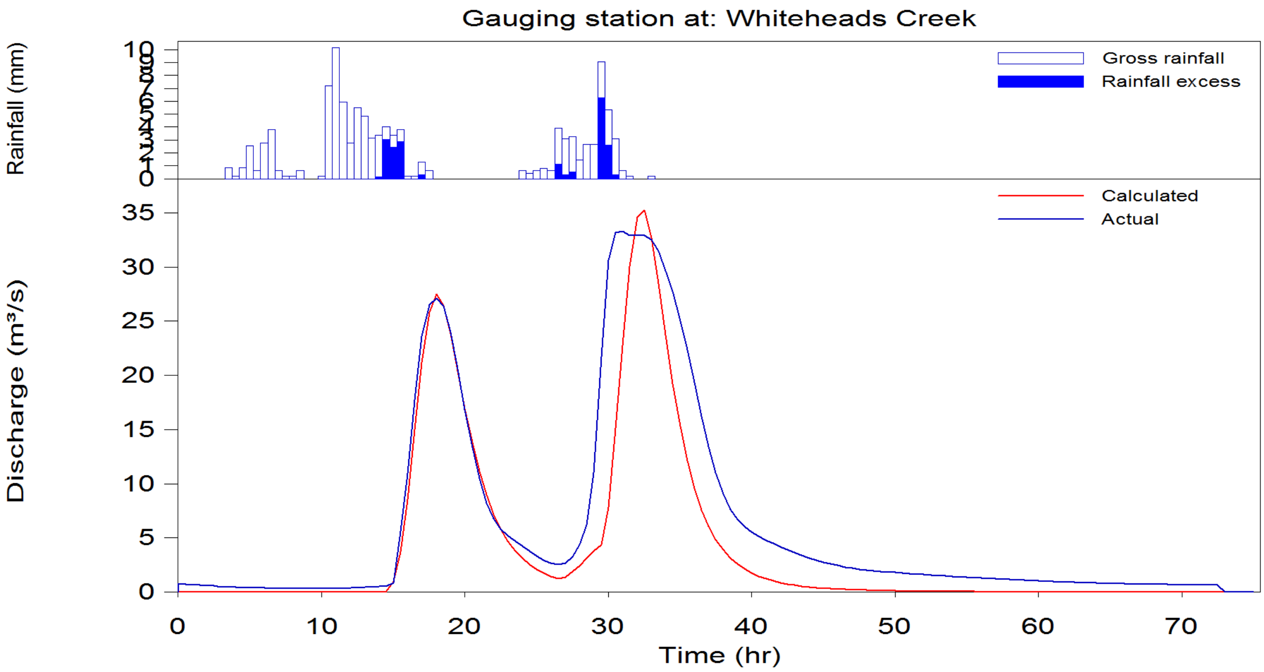


Figure 3-12 Calibrated RORB results for the January 2011 flood event.

3.4.6 March 2012

The 2012 calibration event was an isolated event occurring on the 1st March with a peak flow rate of 18.2 m^3/s occurring at 7:30am. The hydrograph for this event is shown in Figure 3-13.

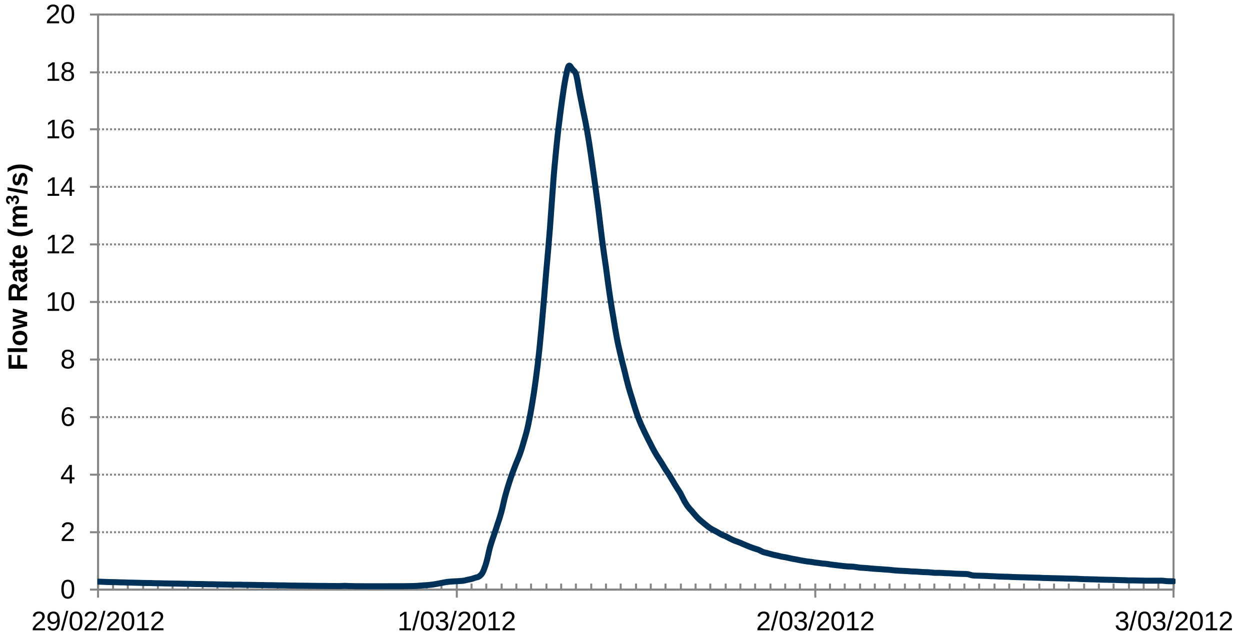


Figure 3-13 March 2012 recorded hydrograph

All of the rainfall gauges show that the bulk of the rainfall fell on the 1st March. Rainfall totals are higher to the north of the catchment and there was between 45 to 70 mm of rainfall depth recorded for this event. As the Seymour Shire Depot recorded 45 mm of rainfall, this total was used in the calibration. A summary of the rainfall depths can be seen in Table 3-14.

Table 3-14 **February 2012 rainfall depths**

Gauge	29-Feb	1-Mar	2-Mar	Total
88109 - Mangalore Airport	0.2	71.4	0	**71.6**
88002 - Avenel (Post Office)	0	66.4	0	**66.4**
88053 - Seymour Shire Depot	0	45.0	0	**45.0**
88126 - Goulbourn River at Seymour	-	-	-	**N/A**
88031 - Highlands (Glentannar)	0	44.2	1.4	**45.6**

The pluviograph data was limited for this event. The pluviograph at Strath Creek provided a partial record for the event with data recorded from the 29th February - 1st March, the 2nd of March was not available, however it is not expected that any rainfall fell on this day relating to this event. The data however gave a total rainfall of 26 mm over the period which is much lower than the recorded totals for Whiteheads Creek. The gauge at Heathcote had no recorded data so this could not be used to verify the pluviograph time series.

The calibrated parameters for the February 2012 event are summarised in Table 3-15. The 'k_c' and 'm' values were consistent with the calibrated 2010 events.

Table 3-15 **Calibrated parameters for the February 2012 event.**

Parameter	Value
k_c	5.9
m	0.8
IL (mm)	28.9
CL (mm/hr)	1.63

The calibration of the February 2012 event was well matched to the recorded hydrograph. The timing of the peak matched well with the recorded hydrograph, with the peak occurring at the same time. The calibration is shown in Figure 3-14. The overall event volume was underpredicted however the shape of the hydrograph was well matched.

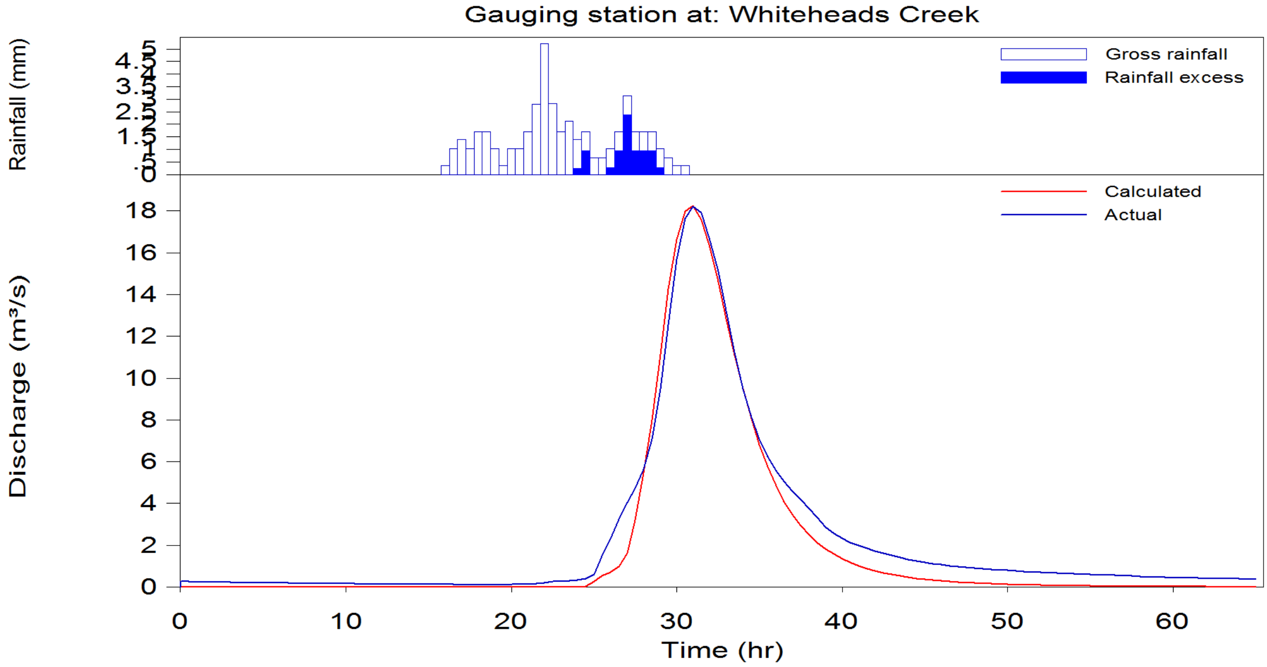


Figure 3-14 Calibrated RORB results for the February 2012 flood event.

3.4.7 Calibration Summary

A summary of the calibration parameters k_c and m, as well as the loss rates for each event is shown in Table 3-16.

Table 3-16 **Summary of calibration parameters and loss rates for each flood event**

Event	k_c	m	IL (mm)	CL (mm/hr)
Mar-10	4.0	0.8	11.4	5.80
Jul-10	5.0	0.8	13.7	1.60
Dec-10	5.0	0.8	52.7	1.90
Jan-11	5.0	0.8	58.3	2.00
			0.0	5.80
Feb-12	5.9	0.8	28.9	1.63

The k_c value ranged from 4.0 up to 5.9 for the calibration events with the majority of event being replicated well using a k_c of 5.0. Loss rates varied for each event and were related to antecedent conditions. Initial loss rates ranged from 11 mm up to 58 mm and continuing loss rates ranged from 1.6 mm/hr up to over 5 mm/hr. There was some uncertainty in the calibration events due to the pluviograph data not being within the catchment, however the five calibration events allow for the catchment runoff parameters to be estimated.

A key observation from the calibration was that the initial and continuing loss rates for the fitted hydrographs seem higher than expected, however the fitted hydrograph shape were quite good in most cases. This may suggest that the peak flow rates associated with the gauge may be underestimated by the rating curve because the shape of the hydrograph was well matched but the volume of runoff removed via the loss rates was high compared to typical event loss rates. This is examined further via the Flood Frequency Assessment and design flow estimation in the following sections.

3.5 Flood Frequency Assessment

The FFA is required to determine the statistical peak flow rates for the design events as determined from a fitted distribution to the streamflow gauge at Whiteheads Creek. For this assessment the distribution fitting program FLIKE has been used and the gauge was assessed using two distributions. The distributions used to fit the annual streamflow data include:

- Log Pearson Type III (LPIII)
- Generalised Extreme Value (GEV)

The FFA has been undertaken using the available data on the BoM Water Data for Whiteheads at Whiteheads Creek which has 27 years of continuous record from 1989-2015.

FLIKE fits a distribution to the annual maximum flow data from a calendar year. The optimisation method for fitting the distributions was based on a Bayesian approach.

The LPIII distribution censored the 2006 data as there was no flow in the creek. For the GEV distribution the flow threshold of 2.5 m^3/s was used to censor the years where the peak flow was lower than this value (that is to ignore the peak flow but include the year in the length of record count to estimate the ARI of the recorded events). This censored 9 years of the gauged record. The purpose of censoring the record is to increase the weighting of the fit to the larger events for the GEV distribution. This gives a more reliable fit to the peak flows that are of interest for this investigation.

The results of the FFA are summarised in Table 3-17. The two distributions are summarised for the 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEPs. The lower and upper 90% Monte Carlo percentile bounds are reported to show the range of uncertainty for the fitted distributions.

Table 3-17 **Flood Frequency Analysis results (1989-2015)**

ARI (years)	5	10	20	50	100	200	500
AEP	20%	10%	5%	2%	1%	0.50%	0.20%
Log Pearson Type III	**24.3**	**31.2**	**35.5**	**38.7**	**39.9**	**40.7**	**41.2**
Lower 90% Monte Carlo Probability	18.6	25.7	30.6	34.3	35.7	36.4	36.8
Upper 90% Monte Carlo Probability	32.2	39.9	48	57.7	63.9	69.5	75.3
Generalised Extreme Value	**29.3**	**32.8**	**35.2**	**37.3**	**38.4**	**39.1**	**39.8**
Lower 90% Monte Carlo Probability	25.6	29.4	31.5	32.9	33.3	33.4	33.5
Upper 90% Monte Carlo Probability	31.7	37.8	43	49	53.1	56.7	60.9

From the assessment of the gauge data it is thought that the FFA was not producing an accurate estimate of the peak flow events for the rarer exceedence probabilities. Figure 3-15 shows the fitted distributions against the annual peak data. The distributions are well fitted to the data with the LPIII distribution matching the expected data better than the GEV.

However, it is evident from the fitted distributions that the FFA is not predicting the expected extreme flood events. The LPIII distribution states that the 5% AEP event has a flow rate of 35.5 m^3/s which appears reasonable, however an 0.2% AEP peak flow rate of just 41.2 m^3/s does not appear reasonable in relation to the 5% AEP and the 51 km^2 catchment upstream of the gauge.

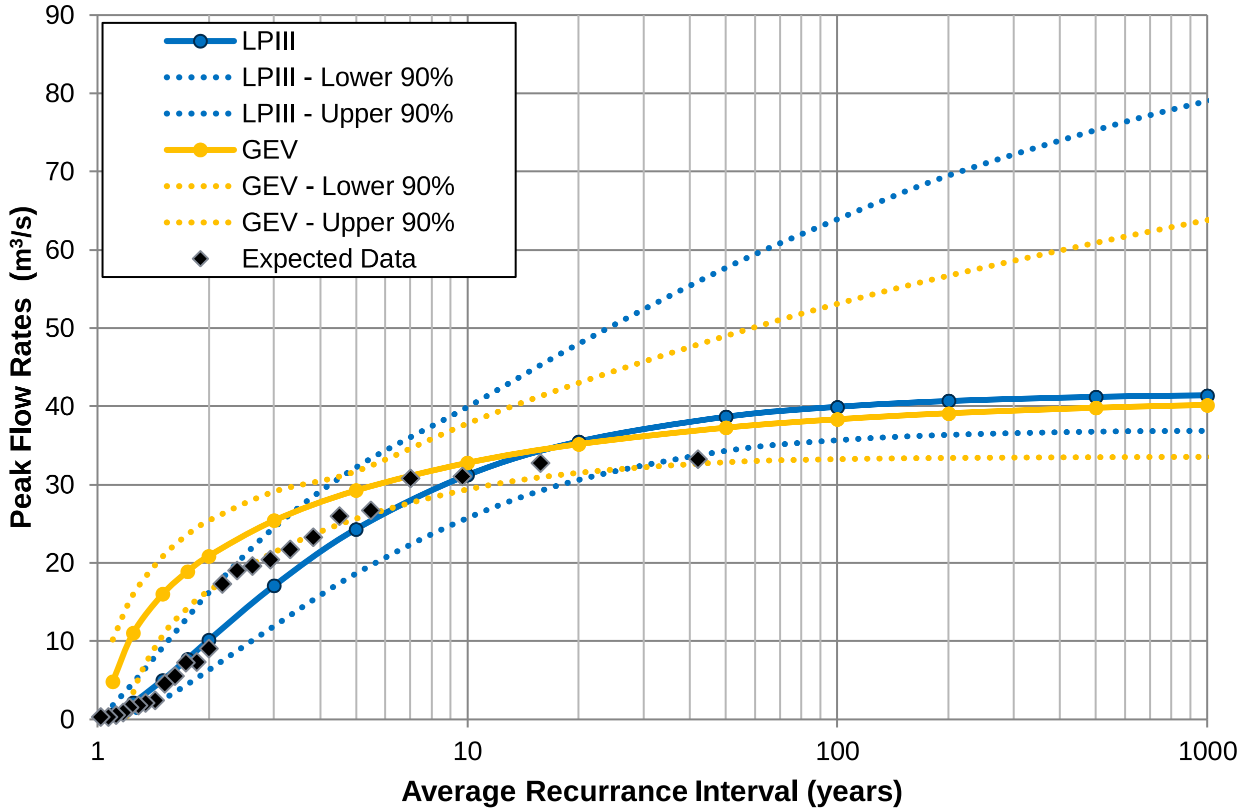


Figure 3-15 Flood Frequency Analysis results

The FFA process relies on the underlying annual peak data and the period of record and uses this information to assign an approximate recurrence interval to the recorded peak flood events. The assigned ARI is shown for the ranked events in Table 3-18. The statistical ARI estimate is determined based on the rank and the length of record and for Whiteheads Creek there have been 6 years where the annual peak flows recorded are in a narrow band of 26 to 33 m^3/s. The rank method for assigning the ARI does not consider this and hence a peak flow rate of 26 m^3 is assigned an approximate ARI of 1 in 4.7 years and a peak flow of 33.3 m^3/s is assigned an ARI of 1 in 42 years. In reality a change in peak flow rate of 7 m^3/s is unlikely to have such a significant difference in return period.

Given that 6 events have occurred around the 30 m^3/s flow rate this would suggest that this type of event is around a 1 in 10 year ARI based on the length of the record.

The cause for this effect could be that during this period

- There have been no large flood events which results in a poor representation of the higher end of the FFA fitted distributions.
- The gauge rating table may be inaccurately predicting the peak flow rates, particularly at higher levels recorded at the gauge.

To examine the likely cause of the poor FFA predictions additional analysis of the streamflow gauge has been undertaken, as well as a revised approach to the FFA.

Table 3-18 **Annual ranked event for Whiteheads Creek**

Rank	Year	Peak Flow Rate (m^3/s)	Assigned ARI
1	2011	33.3	42.0
2	2012	32.8	15.8
3	2010	31.1	9.7
4	1992	30.9	7.0
5	1995	26.7	5.5
6	1993	26.0	4.5
7	1990	23.3	3.8
8	1998	21.7	3.3
9	1989	20.4	2.9
10	2005	19.7	2.6
11	1996	19.1	2.4
12	1991	17.3	2.2
13	2003	9.1	2.0
14	2013	7.4	1.9
15	2008	7.3	1.7
16	1999	5.6	1.6
17	2000	4.6	1.5
18	2002	2.5	1.4
19	2004	2.2	1.4
20	2014	1.8	1.3
21	2007	1.6	1.2
22	1997	0.9	1.2
23	2009	0.6	1.1
24	2001	0.3	1.1
25	1994	0.3	1.0
26	2006	0	-

3.5.2 Gauge Assessment

Due to the difficulties in achieving a reliable FFA prediction for peak flood events, the Whiteheads Creek at Whiteheads Creek gauge has been assessed. This was to inform the project of the reliability of the rating table and subsequent estimates of gauged flow from the recorded levels, particularly at higher out of bank levels.

From the DELWP water portal information is supplied regarding the streamflow gauge and is summarised in Table 3-19 and the measured gaugings are shown in Figure 3-16. It should be noted that it appears that the gaugings are measured using a cable spanning the main channel only to estimate flow.

Table 3-19 **Summary of Whiteheads Creek gauge information (DELWP, 2016)**

Information	Summary
Control	120 degree V-Notch Weir
Cease to flow level	1.1 m
Maximum gauged level	3.915 m
Area	51 km^2
Gaugings	59 samples between June 1988 and July 2014
Rating Table	States it is reliable up to 4.2 m (35.6 m^3/s).
Gauge zero flow level[1]	Estimated at 157.78 mAHD from LiDAR (corresponds to 1.1 m on the rating table)

[1] Estimate only taken from LiDAR as no surveyed level was available

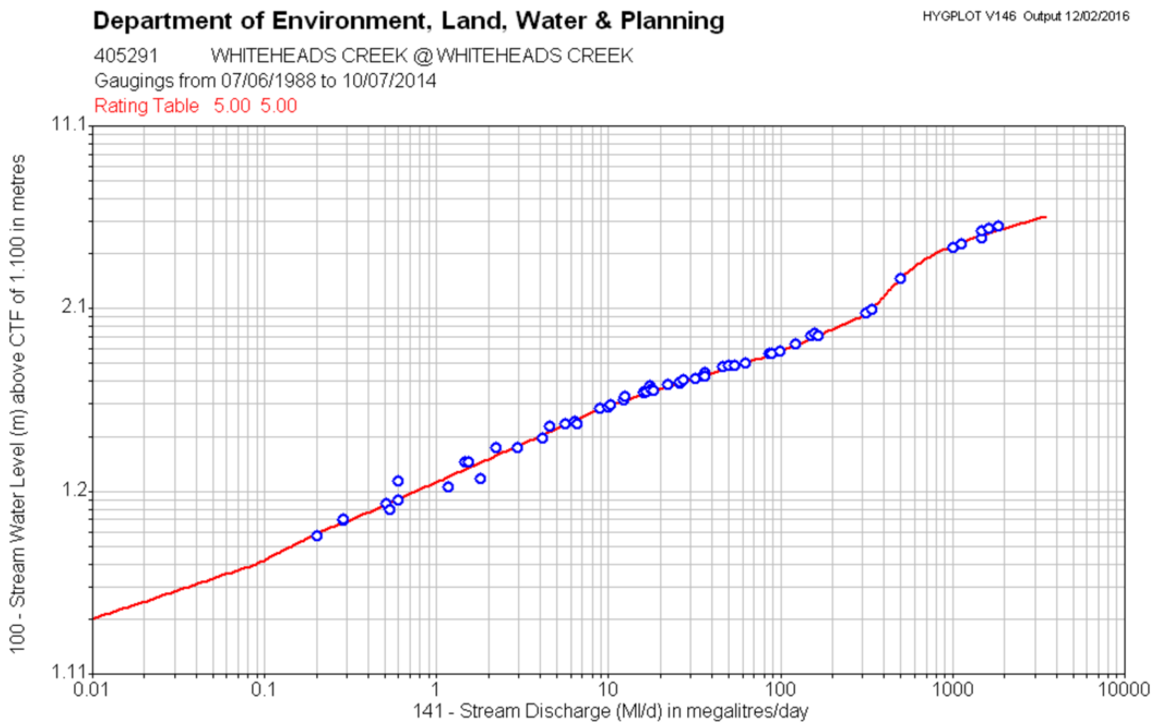


Figure 3-16 Gauging versus rating for Whiteheads Creek

The cross section at the gauge site has been extracted from the supplied LiDAR and is shown in Figure 3-17. The cross section shows the assumed gauge zero flow level of 157.78 mAHD (relates to 1.1 m on the rating table). Also shown is the peak measured gauging and the maximum recorded flood event. The gauge zero was estimated from the LiDAR as this was not available for the gauge.

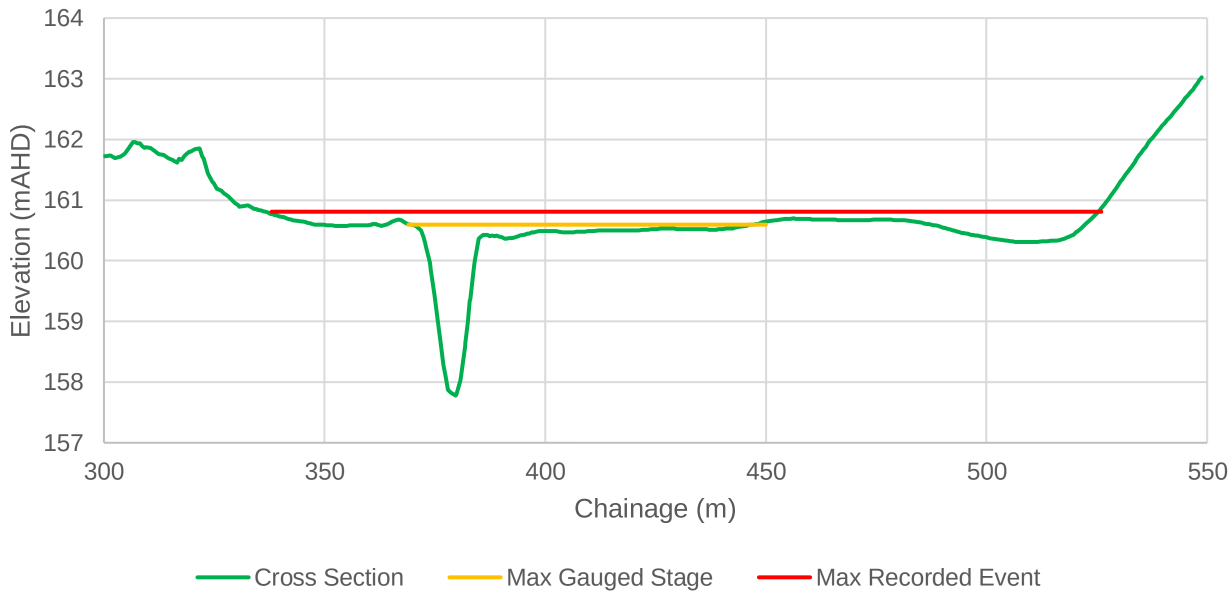


Figure 3-17 Whiteheads Creek gauge cross section

The maximum gauged stage shows that the gaugings have been recorded up to the top of bank level. This suggests that any peak flood levels recorded above this have some uncertainty due to the inclusion of an additional 250 m of floodplain.

To assess the peak flow rates the cross section has had a flow versus stage profile estimated using the Manning's Equation. This is discussed in the following section.

3.5.2.2 *Manning's Calculation*

In order to assess the gauge against the cross section obtained the Manning's Equation has been used to estimate the peak flow rates associated with the recorded flood levels at the gauge. The main purpose of this assessment includes:

- Cross check of the rating table
- Assessment of the sensitivity of the rating table relationship
- Assess the peak flow rates for out of bank peak flows which exceed the maximum measured gauging.

The Manning's equation requires the cross section (see Figure 3-17), the surface roughness and general floodplain slope to provide an estimate of the peak flow rate. Using the LiDAR the floodplain general slope was estimated at 0.7% across the area which is very low grade. The Manning's 'n' for the floodplain was set at 0.06 for the main channel as the channel has trees and shrubs present, and 0.05 for the floodplain as this is predominantly open pasture for the floodplain.

The resulting predicted rating table is shown in Figure 3-18. The existing gauge rating table is shown in black and noticeably shows a smooth increase in peak flow with depth regardless of whether the flood extent is within the main channel. At an elevation of 160.7 mAHD the flood waters escape the main channel and hence the peak flow rate increases significantly.

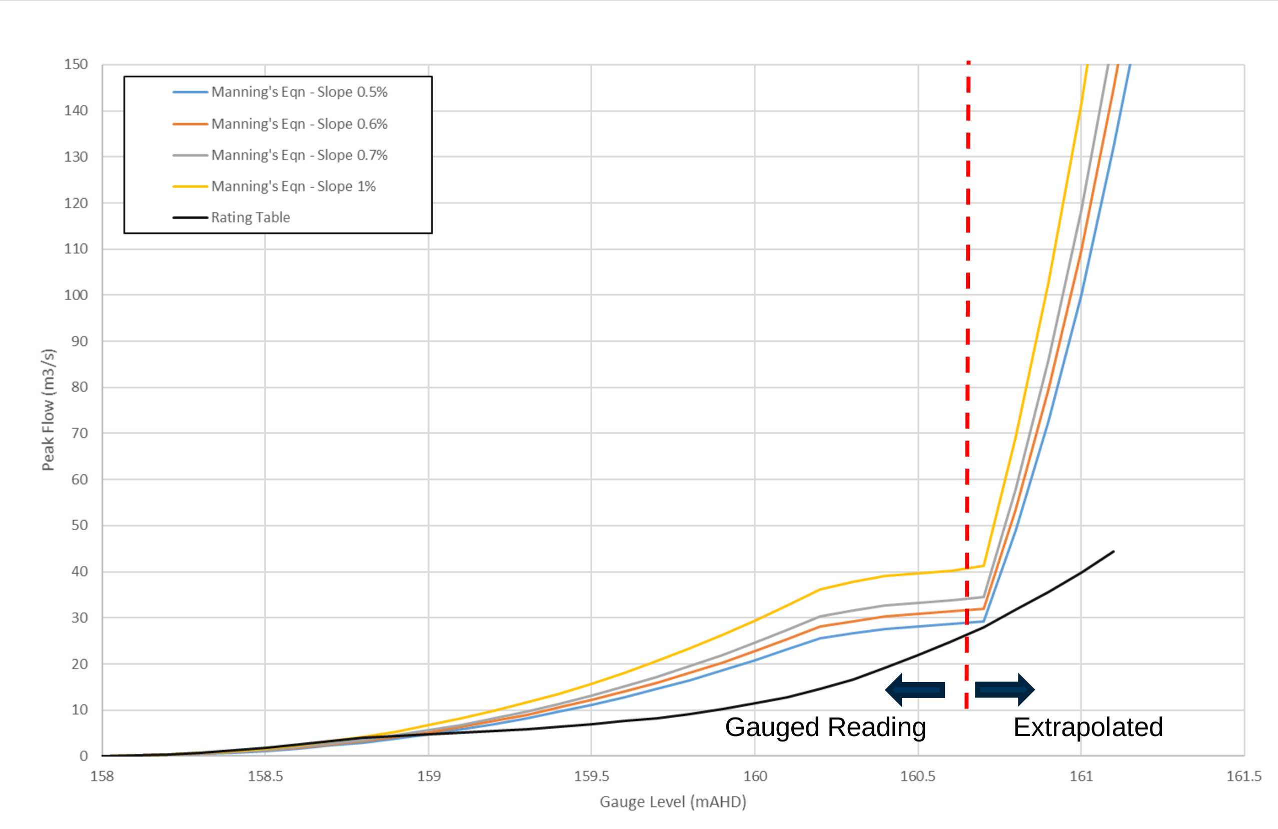


Figure 3-18 Rating table comparison to Manning's Equation

The Manning's calculation predicts that there would be additional flow at the gauge site even within a conservative roughness and channel slope. The portion of interest in the rating table assessment is when the floodwaters exceeds the banks of Whiteheads Creek. Once this occurs then the Manning's estimate of flow increases rapidly as the floodplain increases to approximately 300 m in width.

This Manning's Equation flow rates were used to revise the peak flow rates for the annual peak events and subsequently generate a revised FFA for discussion purposes. The updated peak flow rates are shown in Figure 3-18. The highest level recorded was in 2012 and the peak flow rate for this event was increased from 32.8 m^3/s up to 55.3 m^3/s, an increase of 69%. The events occurring below 160.7mAHD (i.e. the largest gauging) are deemed to be accurately recorded and the focus of the rating table revision was within the extrapolated section of the curve.

Table 3-20 **Revised peak flow estimates for Whiteheads Creek gauge**

Year	Level (m)	Peak Flow (m^3/s)	Peak Flow based on Manning's Rating Table (m^3/s)	Difference (%)
1988	2.56	5.7	8.4	49%
1989	3.69	20.4	32.1	57%
1990	3.78	23.3	32.9	41%
1991	3.56	17.3	30.6	76%
1992	3.99	30.9	34.2	11%
1993	3.86	26.1	33.3	28%
1994	1.51	0.3	0.2	-19%
1995	3.88	26.7	33.4	25%
1996	3.63	19.1	31.6	65%
1997	1.66	0.9	0.6	-33%
1998	3.73	21.7	32.6	50%
1999	2.54	5.6	8.1	46%
2000	2.34	4.6	5.6	21%
2001	1.52	0.3	0.3	-21%
2002	1.92	2.5	1.9	-26%
2003	3.06	9.1	17.6	93%
2004	1.86	2.2	1.4	-33%
2005	3.66	19.7	31.8	62%
2006	0.85	0.0	0.0	0%
2007	1.77	1.6	1.0	-35%
2008	2.85	7.3	13.3	83%
2009	1.59	0.6	0.4	-27%
2010	4.00	31.1	34.2	10%
2011	4.05	33.3	36.4	9%
2012	4.13	32.8	55.3	69%
2013	2.86	7.4	13.6	85%
2014	1.80	1.8	1.1	-37%
2015	1.37	0.1	0.1	-20%
2016	3.28	11.1	22.9	106%

The FFA was recalculated using FLIKE and the revised annual peak flow data. The resulting predicted peak flows for each recurrence interval are summarised in Table 3-21. The 100 year ARI peak was predicted at around 66 m^3/s with 90% confidence intervals spanning from 50 to 132 m^3/s. The revised FFA is shown in Figure 3-19.

Table 3-21 **Revised Flood Frequency Analysis results (1989-2015)**

ARI (years)	5	10	20	50	100	200	500
AEP	20%	10%	5%	2%	1%	0.50%	0.20%
Log Pearson Type III	**32.5**	**45.5**	**54.7**	**62.4**	**65.9**	**68.1**	**69.8**
Lower 90% Monte Carlo Probability	23.3	34.8	44.0	52.3	56.3	58.7	60.5
Upper 90% Monte Carlo Probability	46.2	62.0	76.6	93.3	104.2	113.4	122.7
Generalised Extreme Value	**36.1**	**44.5**	**51.7**	**59.9**	**65.3**	**70.2**	**75.8**
Lower 90% Monte Carlo Probability	29.3	36.7	42.4	47.9	50.9	53.1	55.1
Upper 90% Monte Carlo Probability	45.1	59.1	77.1	106.0	132.1	164.9	217.7

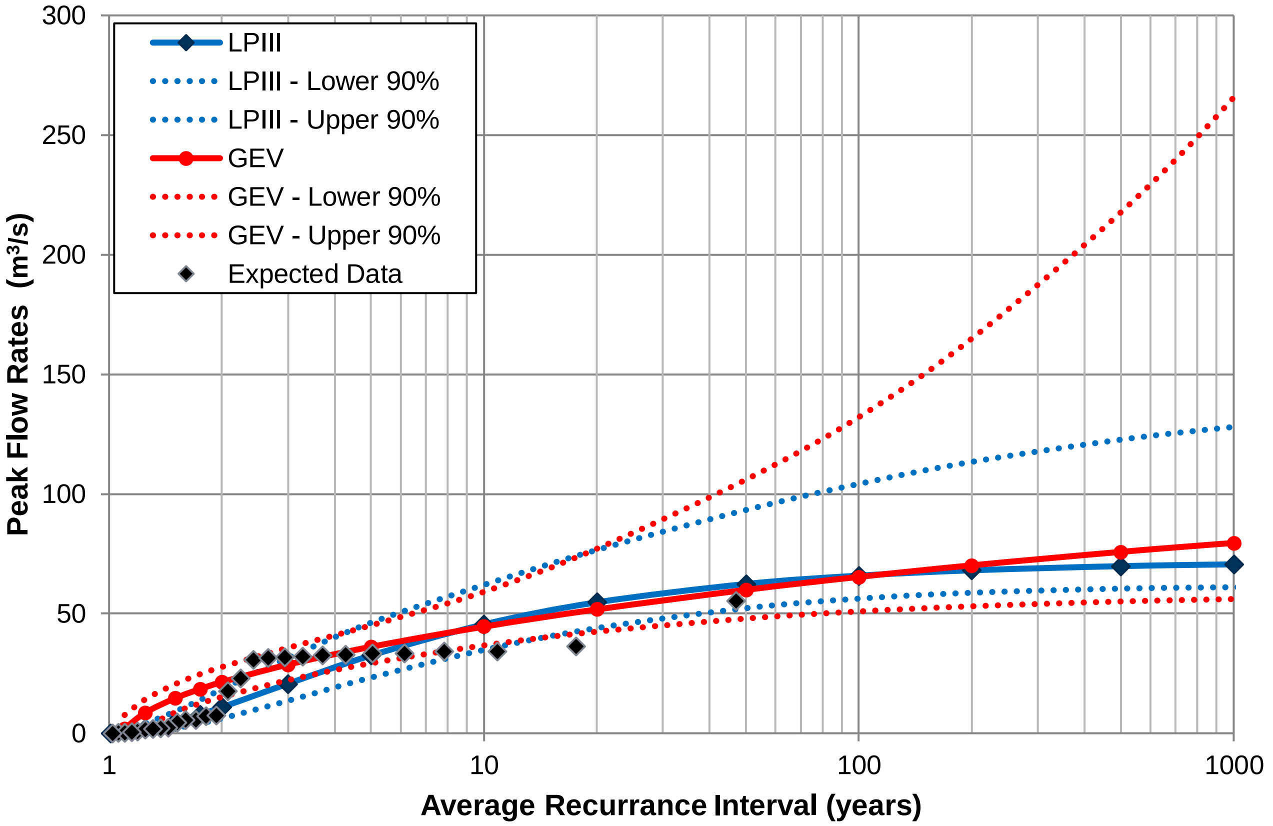


Figure 3-19 Revised peak flow FFA for Whiteheads Creek

Overall the gauge assessment shows that there is some uncertainty on the estimates of peak flow, particularly above the largest gauging. It is also worth noting that the level associated with the largest gauging was higher than the top of bank and the flow measuring via cable pull may not have included all flood waters. This may also result in an under prediction of the flow rate at this level.

Due to the uncertainty associated with predicting events such as the 100 year ARI event a regional approximation method was also tested to provide some guidance on the target design peak flow rate.

3.5.3 Regional 100 Year ARI Estimate

The regional method for estimating catchment flows was sourced from Hydrological Recipes (Grayson et al. 1996) and relates to catchments near the Great Dividing Range in Victoria. The equation is number 7.6.5 (Equation 1) and it can be used to provide guidance for the 1% AEP flood event.

$Q_{0.01} = 4.67 \times \text{Area}^{0.763}$ **Equation 1**

For the Whiteheads Creek catchment the area is 51 km^2 and this results in a 1% AEP peak flow estimate of 94 m^3/s. This estimate is higher than the peak flow rate predicted by the original and revised FFA however it does lie within the 90% confidence limits of the revised FFA estimate of 50 to 132 m^3/s.

3.5.4 Summary

Ultimately the FFA did not produce a good understanding of the expected peak flow rates for the range of statistical recurrence intervals. This was particularly the case for the rarer events above the 2% AEP. Some reasons for this were explored and included:

- There were no extreme flood events in the 28 year record which led to a high level of uncertainty for predicting the rarer event peak flow rates.
- The streamflow gauge rating table did not seem to account for the increase in floodplain width once the flows exceeded the main channel at the gauge site.
- Regional flood estimates predict that the peak flow rate for the 1% AEP should be abound 94 m^3/s which is well above the FFA predicted 1% AEP peak flow rates of 40 m^3/s for the original FFA and 66 m^3/s for the adjusted FFA.

Due to the uncertainties experienced in estimating the peak flood events additional examination of the peak predicted flow rates were explored using the RORB model and standard loss rates.

3.6 Design Events

In order to establish the design events the RORB model was used in conjunction with the calibrated k_c and m values (see section 3.4.7). For the design event the RORB model was extended to cover the full Whiteheads Creek study area as the inflows to the hydraulic model are generated within RORB. The full RORB model for the study area is shown in Figure 3-4.

In order to adjust the gauged catchment calibration model parameters to the full study area the relationship between k_c and the average flow path length was used. This relationship allows for the calibrated k_c to be adjusted based on the average reach length in the full study model. This relationship is shown in equation 2.

$$\frac{kc_1}{Dav_1} = \frac{kc_2}{Dav_2}$$ **Equation 2**

Where:

- kc_1 is the kc parameter for catchment 1
- kc_2 is the kc parameter for catchment 2
- Dav_1 is the average flow path length from each subcatchment to the outlet for catchment 1
- Dav_2 is the average flow path length from each subcatchment to the outlet for catchment 2

Using this relationship the kc was determined for the full Whiteheads Creek catchment from the calibrated model. The resulting k_c is summarised in Table 3-22.

Table 3-22 **Revised catchment parameter for the full Whiteheads Creek model**

Model	k_c	Average flow path length (km)
Calibration model to Whiteheads Creek gauge	4.0 – 6.0	7.59
Full Whiteheads Creek study area	**6.6 – 9.9**	12.52

Using the dominant k_c value of 5.0 from the calibration resulted in a k_c for the overall catchment of 8.2. This was used in the design event simulations. The initial loss was set at 25 mm and the continuing loss was set at 2.5 mm/h as per the AR&R recommended loss rates (volume 2 AR&R).

Table 3-23 **Design peak flow rates for Whiteheads Creek**

Event	Peak flow ate Whiteheads Creek gauge (m^3/s)	Peak flow downstream of Emily Street (m^3/s)	Critical Duration
20%	16.4	26.0	9 hour
10%	29.5	48.7	6 hour
5%	49.6	85.0	6 hour
2%	81.1	143.3	6 hour
1%	105.3	186.4	6 hour
0.5%	131.6	233.1	4.5 hour
0.2%	172.7	304.1	3 hour

The RORB generated 1% AEP event at the Whiteheads Creek gauge was 105.3 m^3/s with a critical duration generated by the 6 hour design storm. The peak flow rate for the 1% AEP event is similar to the regional estimated peak flow rate and is well within the confidence limits of the FFA. Given the clear uncertainties in the FFA it was decided to use the AR&R design loss rates and use the conservative flow estimated from RORB as the design simulations.

The peak flow rate for the 5% AEP flood event is generated at 29.5 m^3/s using RORB. Within the 27 year flow record events around 30 m^3/s occur 4 times. Statistically this matches the design event estimate of a 5% AEP reasonably well as you would expect to see 2-3 of the 30 m^3/s events in a 27 year record.

Overall the developed design events seem to be consistent with the events experienced within the catchment and are likely to be conservative which gives some allowance to the uncertainty associated with the streamflow gauge.

3.7 Design event 0.1% AEP

In order to derive the 0.1% AEP event the AR&R rainfall totals were required to be estimated using CRC-FORGE. This allows the rainfall totals to be extended to the 0.1% AEP event and used to simulate this event using RORB. CRC-FORGE estimates event rainfall totals between 6 to 120 hour events. The 6 to 18 hour duration events are summarised in Table 3-24. The critical event is the 6 hour duration event.

The 0.1% AEP event is approximately 67% higher than the 0.2% AEP event at the downstream end of Whiteheads Creek which seems in line with the estimated rainfall depths.

Table 3-24 **Summary of the 0.1% AEP events for Whiteheads Creek**

Duration	0.1% AEP Rainfall Total (mm)	Peak flow at Whiteheads Creek gauge (m^3/s)	Peak flow at Emily Street (m^3/s)
6 hour	121.4	284.1	509.1
9 hour	134.9	219.8	388.4
12 hour	148.6	234.9	427.1
18 hour	170.7	142.2	271.3

3.8 Probable Maximum Flood

The Probable Maximum Flood (PMF) has been estimated using the Generalised Short-Duration Method (GSDM) and the Generalised Southeast Australia Method (GSAM). The GSDM accounts for event estimated up to the 6 hour duration and the GSAM accounts for events from 24 to 96 hours.

The PMF is the flooding associated with the Probable Maximum Precipitation (PMP). The PMP is defined by the Manual for Estimation of Probable Maximum Precipitation (WMO, 2009) as:

"*…the theoretical maximum precipitation for a given duration under modern meteorological conditions."*

The methods estimate the Probable Maximum Precipitation (PMP) for the catchment based on the parameters set in Table 3-25.

Table 3-25 **PMP parameters for the GSDM and GSAM**

Parameter	
Zone	Inland Zone
Area	106 km^2
Topographical Adjustment Factor	1.0
EPW_{Annual}	59.0
EPW_{Autumn}	48.0
Moisture Adjustment Factor	0.56
Portion Smooth / Rough	80% smooth / 20% rough
Rainfall Depth	Sourced from charts supplied for each method
Rainfall Pattern	Sources from temporal pattern for 100 km^2 catchments and from RORB deign storm files

The resulting PMP estimates were used to create RORB storm files to simulate the PMF event. The temporal patterns were taken from the 100 km^2 inland zone patterns supplied. For the shorter durations the temporal pattern from the 0.2% AEP design events were applied.

The PMF peaks at Whiteheads Creek gauge and at the downstream end of the catchment near Emily Street are shown in Table 3-26. The critical PMF event is at the downstream end of the catchment is the 6 hour event.

Table 3-26 **Estimated PMP and PMFs for Whiteheads Creek**

Duration	Probable Maximum Precipitation (PMP) (mm)	Peak flow at Whiteheads Creek gauge (m^3/s)	Peak flow at Emily Street (m^3/s)
2 hour	280	1,576	2,619
3 hour	320	1,576	2,760
4.5 hour	375	1,565	2,768
6 hour	420	1,548	2,805
12 hour	430	1,088	1,939

3.9 Design Event Sensitivity

In order to assess the sensitivity of the design events the RORB model has been run to test the design parameters of k_c, initial loss and continuing loss. These are the primary influencing factors on the peak flow rates for the design events and it is important to understand the hydrological model's sensitivity to each parameter set.

The k_c parameter influences the runoff characteristics of the catchment, reducing the k_c increases the runoff response within the catchment and results in the catchment having a higher peak flow rate and an earlier peak than larger k_c parameters. The sensitivity on the kc parameter targets modifying the kc value by +/-20% from the design rate which maintaining the loss rates. The kc parameters used in this assessment were 6.56 and 9.84, this also corresponds to the extremes of the calibrated kc parameters (see Section 3.6).

The peak flow rate for the 1% AEP event under these scenarios ranged from 92.0 to 122.2 m^3/s. This shows that the model is sensitive to the kc with the differences ranging from -13% to +16% respectively. Ultimately the typical catchment behaviour has been represented using a kc of 8.2. The uncertainty associated with this estimate is +/- 16% based on the extremes of the calibrated kc parameter.

Table 3-27 and Table 3-28 shows the peak flow rates changes for Whiteheads Creek at the gauge and from Whiteheads Creek near Emily Street for the 20% to the 0.2% AEP events. It is evident that the kc change produces a similar change in peak flow rate for more frequent and rarer events.

The loss rates were then adjusted from the adopted range of 25 mm initial loss and 2.5 mm/h continuing loss to assess the full recommended ranger of losses within AR&R. The two scenarios included:

- Low loss case – 15 mm initial loss, 1.5 mm/h continuing loss
- High loss case – 35 mm initial loss, 3.5 mm/h continuing loss

The loss rates have a large impact on the peak flow rates, particularly for the smaller more frequent events. The change in the peak flow rate ranges from a 72% reduction in the peak to an increase of 225% due to the loss rates. Clearly the model is more sensitive to loss than to the changes in the kc parameter. For the 1% AEP the change in loss rate resulted in a range of peak flow from 62 to 154 m^3/s.

As the events get larger the changes in the loss rate reduce and as such the uncertainty associated with the loss rate decreases. For the 1% AEP event at Whiteheads Creek the range was +/- 46% between the low and high loss rate scenarios.

Table 3-27 **Sensitivity assessment at Whiteheads Creek gauge**

AEP	Design	k_c Parameter				Loss Rate			
k_c	8.2	6.56		9.84		8.2		8.2	
Loss rate	25 mm / 2.5 mm/h	25 mm / 2.5 mm/h		25 mm / 2.5 mm/h		15 mm / 1.5 mm/h		35 mm / 3.5 mm/h	
20%	16.4	20.0	22%	13.7	-17%	53.3	225%	4.5	-72%
10%	29.5	35.9	22%	24.8	-16%	69.0	134%	10.7	-64%
5%	49.6	59.3	20%	42.5	-14%	93.0	87%	19.4	-61%
2%	81.1	94.8	17%	70.7	-13%	126.7	56%	41.9	-48%
1%	105.3	122.2	16%	92.0	-13%	153.5	46%	61.8	-41%
0.5%	131.6	155.7	18%	114.4	-13%	184.1	40%	87.8	-33%
0.2%	172.7	206.1	19%	149.9	-13%	227.8	32%	126.2	-27%

Table 3-28 **Sensitivity assessment at Whiteheads Creek near Emily Street**

AEP	Design	k_c Parameter				Loss Rate			
k_c	8.2	6.56		9.84		8.2		8.2	
Loss rate	25 mm / 2.5 mm/h	25 mm / 2.5 mm/h		25 mm / 2.5 mm/h		15 mm / 1.5 mm/h		35 mm / 3.5 mm/h	
20%	26.0	32.4	25%	21.4	-18%	92.8	257%	6.9	-73%
10%	48.7	61.6	26%	39.6	-19%	120.4	147%	16.3	-67%
5%	85.0	104.3	23%	70.2	-17%	163.4	92%	31.3	-63%
2%	143.3	171.2	19%	120.2	-16%	221.6	55%	69.1	-52%
1%	186.4	222.6	19%	160.0	-14%	271.4	46%	105.2	-44%
0.5%	233.1	277.6	19%	198.3	-15%	323.3	39%	154.1	-34%
0.2%	304.1	362.4	19%	262.9	-14%	397.1	31%	224.7	-26%

3.10 Developed Catchment Design Events

The developed catchment was assessed by making assumptions regarding future growth areas within the town and areas where growth was likely to result in an increase in impervious area. Accordingly these areas had their fraction impervious increased which results in increased rates and volumes of runoff generated from the catchment.

The locations of predicted growth where developed in discussion with Council and other relevant stakeholders.

3.11 Climate Change Assessment

The climate change modelling was undertaken assuming a low, medium and high change in rainfall intensity. This was modelled by assuming an increase in the intensities in the IFD parameters by 10%, 20% and 30% respectively. This range of increases allows for an examination of the possible impacts of climate change in the short, medium and long term.

The intensity increases for Whiteheads Creek are summarised in Table 3-29 for the 10%, 20% and 30% climate change scenarios. The resulting changed in the peak flow rates are summarised in Table 3-30.

The low climate change scenario (10% intensity increase) increased the peak flow rates by between 21 to 70%. The medium climate change scenario (20% intensity increase) resulted in increases in peak flow rates from 42 to 115%. The high climate change scenario (30% intensity increase) resulted in increases in peak flow rates from 61 to 193%.

The more frequent events (i.e. 20% and 10% AEP) showed proportionally larger increases than the rarer events as a result of climate change for the low, moderate and high climate change scenarios.

Table 3-29 **Intensity-Frequency-Duration parameters with climate change for Whiteheads Creek**

IFD Coefficient	Value	10% CC	20% CC	30% CC
$^{2}I_{1}$	23.16	25.5	27.8	30.1
$^{2}I_{12}$	3.79	4.2	4.5	4.9
$^{2}I_{72}$	1.07	1.2	1.3	1.4
$^{50}I_{1}$	43.07	47.4	51.7	56.0
$^{50}I_{12}$	6.99	7.7	8.4	9.1
$^{50}I_{72}$	2.24	2.5	2.7	2.9
G (skew)	0.25			
F2	4.31			
F50	15.04			
Zone	2			
Location				
Latitude	37.050 S			
Longitude	145.225 E			

Table 3-30 **Climate change events generated for the Whiteheads Creek gauge**

AEP	Design Peak (m^3/s)	Climate Change Peaks (m^3/s)					
	Existing	**10% Increase**	**Difference**	**20% Increase**	**Difference**	**30% Increase**	**Difference**
20%	16.4	27.8	70%	35.3	115%	48.1	193%
10%	29.5	41.9	42%	55.8	89%	70.7	139%
5%	49.6	67.7	36%	85.1	72%	99.9	102%
2%	81.1	102.6	27%	120.7	49%	142.3	76%
1%	105.3	129.2	23%	153.6	46%	179.5	71%
0.5%	131.6	161.3	23%	189.9	44%	221.0	68%
0.2%	172.7	209.2	21%	244.7	42%	277.4	61%

Table 3-31 **Climate change events generated for peak flow at Emily Street on Whiteheads Creek**

AEP	Design Peak (m^3/s)	Climate Change Peaks (m^3/s)					
	Existing	**10% Increase**	**Difference**	**20% Increase**	**Difference**	**30% Increase**	**Difference**
20%	26.0	45.9	76%	58.7	126%	82.5	217%
10%	48.7	71.3	46%	95.3	96%	123.8	154%
5%	85.0	118.0	39%	147.6	74%	174.0	105%
2%	143.3	181.3	27%	213.6	49%	251.7	76%
1%	186.4	228.4	23%	270.3	45%	316.9	70%
0.5%	233.1	283.2	22%	335.3	44%	386.7	66%
0.2%	304.1	365.2	20%	428.0	41%	488.1	60%

4 Hydraulics

The WL|Delft 1D2D modelling system, SOBEK, was used to compute the channel (1D) and overland flow (2D) components of the study. SOBEK is a professional software package developed by WL|Delft Hydraulics Laboratory, which is one of the largest independent hydraulic institutes in Europe (situated in The Netherlands) and is world-renowned in the fields of hydraulic research and consulting (WL|Delft, 2005).

This combined package allows for the computation of channel and pipe flow (including structures such as culverts, weirs, gates and pumps, and pipe details such as inverts, obverts, pipe sizes and pipe material) by the 1D module, which is then dynamically linked to the 2D overland flow module. The 1D and 2D domains are automatically coupled at 1D-calculation points (such as manholes) whenever they overlap each other. The model commences with the 1D component operating as the inflow increases until such time as the pipe or channel is full and overflows, with the flow then moving to the 2D domain. The 1D network and the 2D grid hydrodynamics are solved simultaneously using the robust Delft scheme that handles steep fronts, wetting and drying processes and subcritical and supercritical flows (Stelling et al., 1999).

The advantages of this system are that the channel/pipe system is explicitly modelled as a sub-system within the two-dimensional overland flow computation. This means that generalised assumptions regarding the capacity of the channel/pipe system are not required. This system employs a unique implicit coupling between the one and two-dimensional hydraulic components that provides high accuracy and stability within the computation.

4.1 Hydraulic Model Development

4.1.1 Study Area

The Whiteheads Creek hydraulic model study area was required to be developed to incorporate the streamflow gauge at the upper end of the catchment for emergency planning and response arrangements. Downstream of Telegraph Road the hydraulic model is included to capture the full extent of the current and future planning zones for Seymour. This was an important component of the current Master Plan being developed for the township.

The full extent of the tributaries has been included as these have not previously been mapped. The study area focuses on Whiteheads Creek only and does not include the areas impacted by the Goulburn River aside from that influencing Whiteheads Creek.

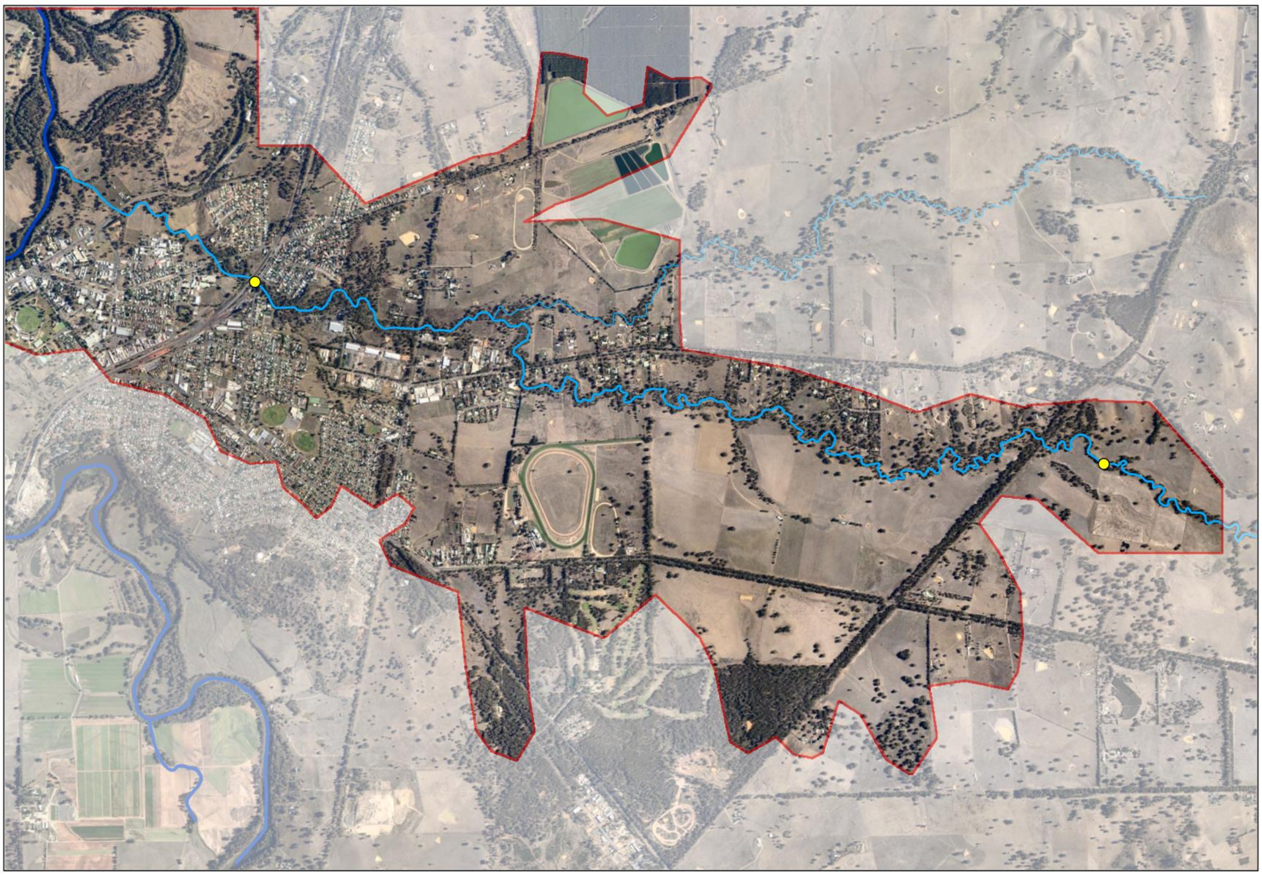


Figure 4-1 Whiteheads Creek hydraulic model study area

4.1.2 Topography

The topography was developed using the available supplied LiDAR information. The LiDAR data was flown in Nov 2013 at 1 m horizontal spacing and a +/- 10cm vertical accuracy (to one sigma). It has been supplied by the Goulburn Broken CMA. The LiDAR data covered the entire model area and is shown in Figure 4-2 with the survey comparison point locations.

Survey was captured by Cardno as part of the project and during this process levels were captured along the roads as shown in Figure 4-2. These locations have been compared to the LiDAR to verify the accuracy of the data. Overall the average difference was – 7.9 cm (i.e. the LiDAR was lower than the surveyed data), and 74.4% of the points were between +/- 10 cm. This exceeds the stated vertical accuracy of +/- 10cm to one sigma (~66%).

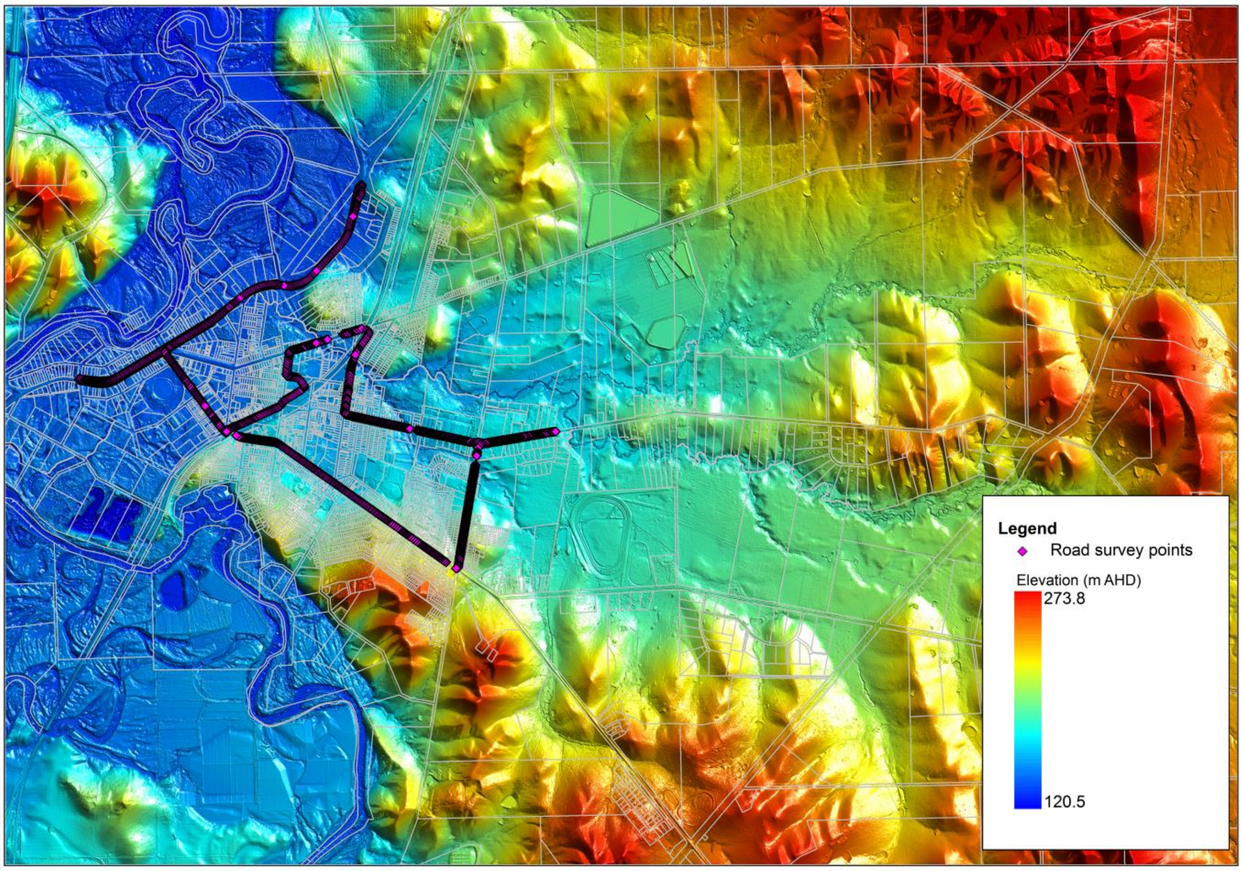


Figure 4-2 Survey comparison locations and LiDAR extent

Table 4-1 **Analysis of the difference in elevation between the LiDAR and survey data**

Difference Range (LiDAR less Survey)	Number of points	Percentage of Points
-0.3 to -0.25	1	0.1%
-0.25 to -0.2	5	0.4%
-0.2 to -0.15	54	4.7%
-0.15 to -0.1	232	20.4%
-0.1 to -0.05	611	53.6%
-0.05 to 0	223	19.6%
0 to 0.05	11	1.0%
0.05 to 0.1	2	0.2%
Total	**1139**	**100%**

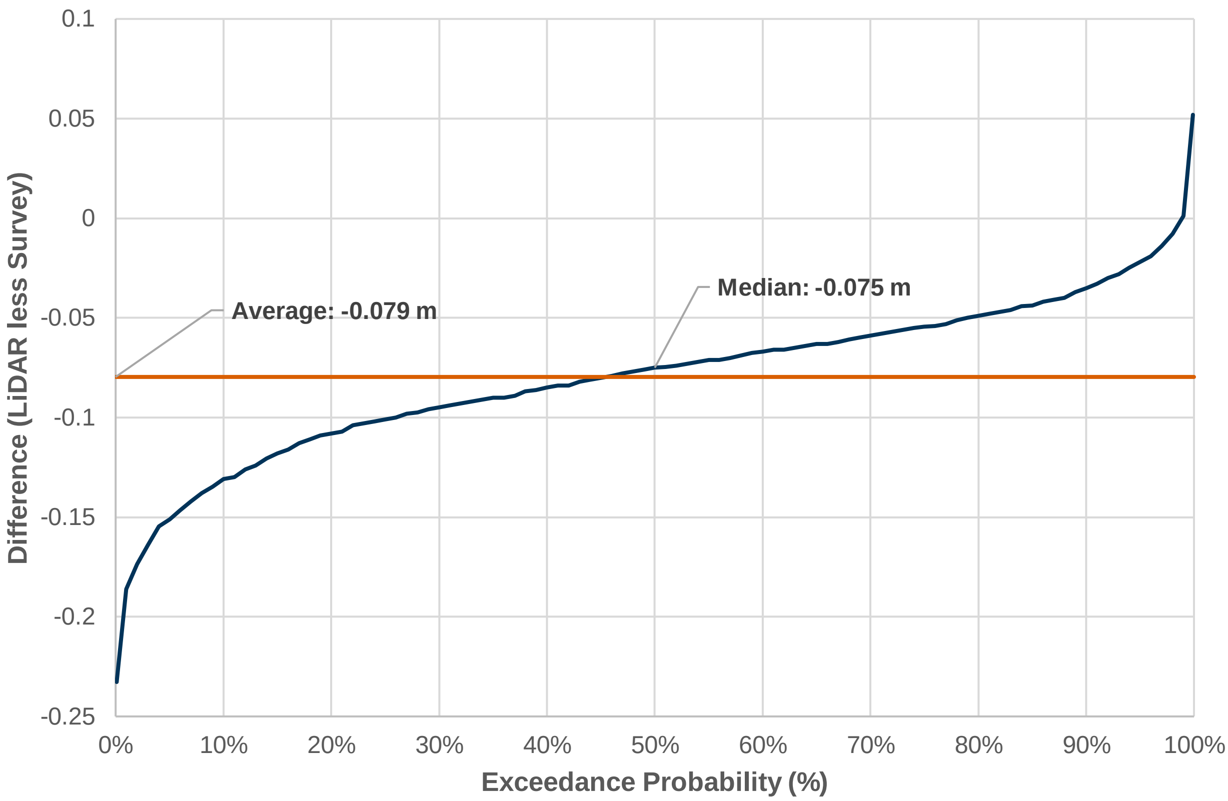


Figure 4-3 Distribution of the differences between the LiDAR and survey levels

4.1.2.2 Model Grid

The hydraulic model area is approximately 16 km^2. The grid cell resolution selected for the model was 5m x 5m as this provided sufficient accuracy to the property level but allowed for manageable model runtimes. The details of the developed hydraulic grid are summarised in Table 4-2.

Table 4-2 **Hydraulic model grid cell information**

Parameter	Grid
Cell size	5m x 5m
Grid cells (x direction)	1,792
Grid cells (y direction)	1,024
Model cells	1,835,008

4.1.3 Structures

Structures have been included within the model as 1D structures and connected via 1D/2D connections. Where information was not available for the structures from plans, survey was captured. All structures were checked within the hydraulic model to ensure suitable head loss occurred across each structure and that they were passing suitable volumes of water.

4.1.4 Roughness

The roughness for the hydraulic model has been developed using Manning's roughness based on the calibration events and site inspections. The selected roughness is shown in Figure 4-4.

The main creek has been set at a higher roughness of 0.06 as there are a large number of trees and plants growing in the system. Evidence from site inspections and from the Community suggests there is a large amount of debris in the main channel of Whiteheads Creek. Residential areas have been set at a high roughness as these areas are typically dense with buildings, fences, gardens etc. The buildings have not been blocked out of the floodplain.

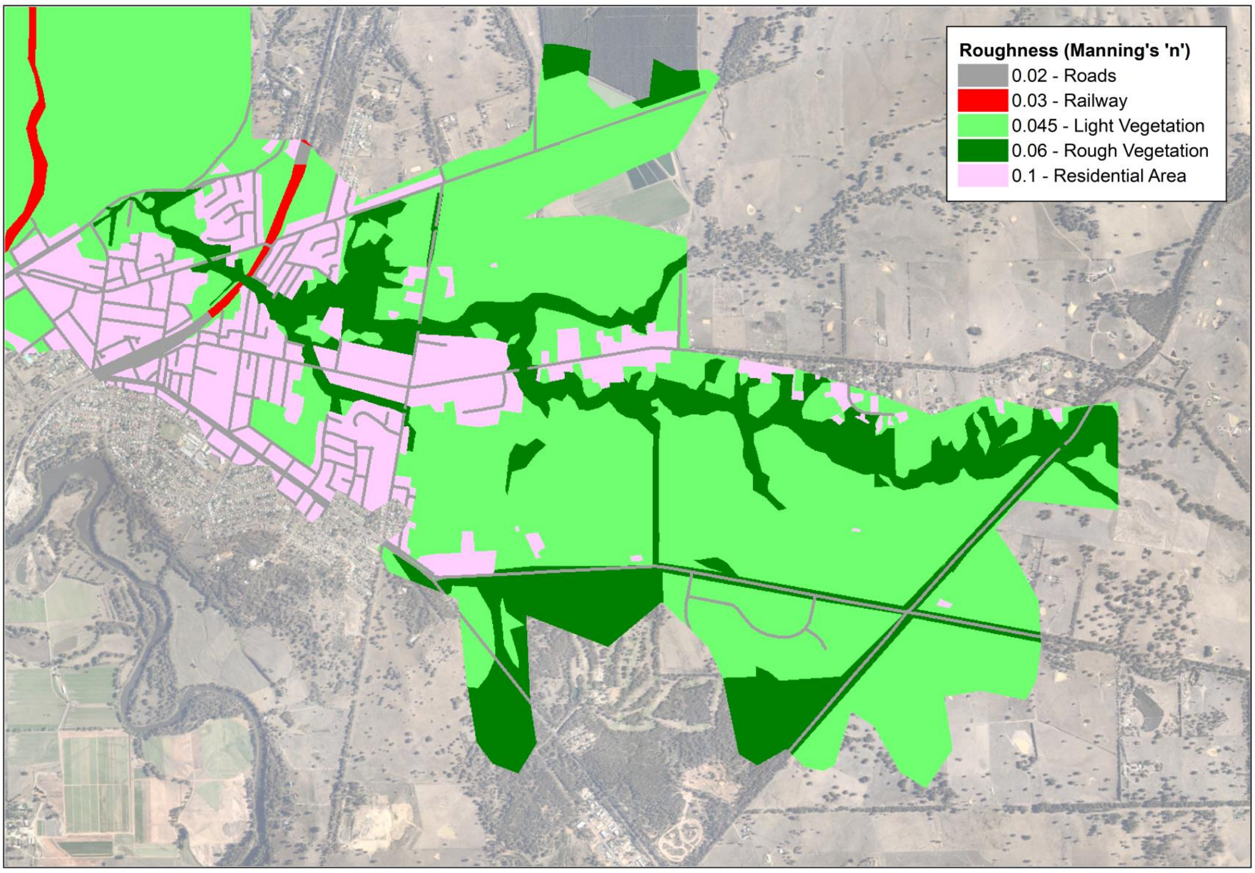


Figure 4-4 Roughness used in the Whiteheads Creek model

4.1.5 Inflows and Boundary Conditions

Inflows have been introduced to the hydraulic model using line boundaries at the upstream end of the catchment and via lateral inflows within the hydraulic model area. Line boundaries have been included for Whiteheads Creek, tributary near Whiteheads Creek, Back Creek and the Goulburn River.

All other inflows have been input into the model using small reaches with lateral inflow nodes to transfer flow into the 2D surface. These inflows correspond to the sub catchments within the RORB mode.

The Goulburn River is set as a steady state level as the Goulburn River flood events are significantly longer in duration that the Whiteheads Creek catchment. For historic events the level was set using the gauge at

Goulburn River at Seymour. For the design events the level in the Goulburn River was set at the flood level of 134.8 mAHD. This was constant for the design events as the focus for these investigations were the impacts of Whiteheads Creek rather than the impacts of the Goulburn River at the downstream end of the hydraulic model. If the level is set too high in the Goulburn this significantly impacts the Whiteheads Creek flood levels and the flooding in the main Seymour township downstream of the railway line.

The level of 134.8 mAHD is in line with the major flood level in the Goulburn River. Given the size difference of the catchments and times of concentration it is unlikely that the Whiteheads Creek flood will occur simultaneously with the Goulburn River.

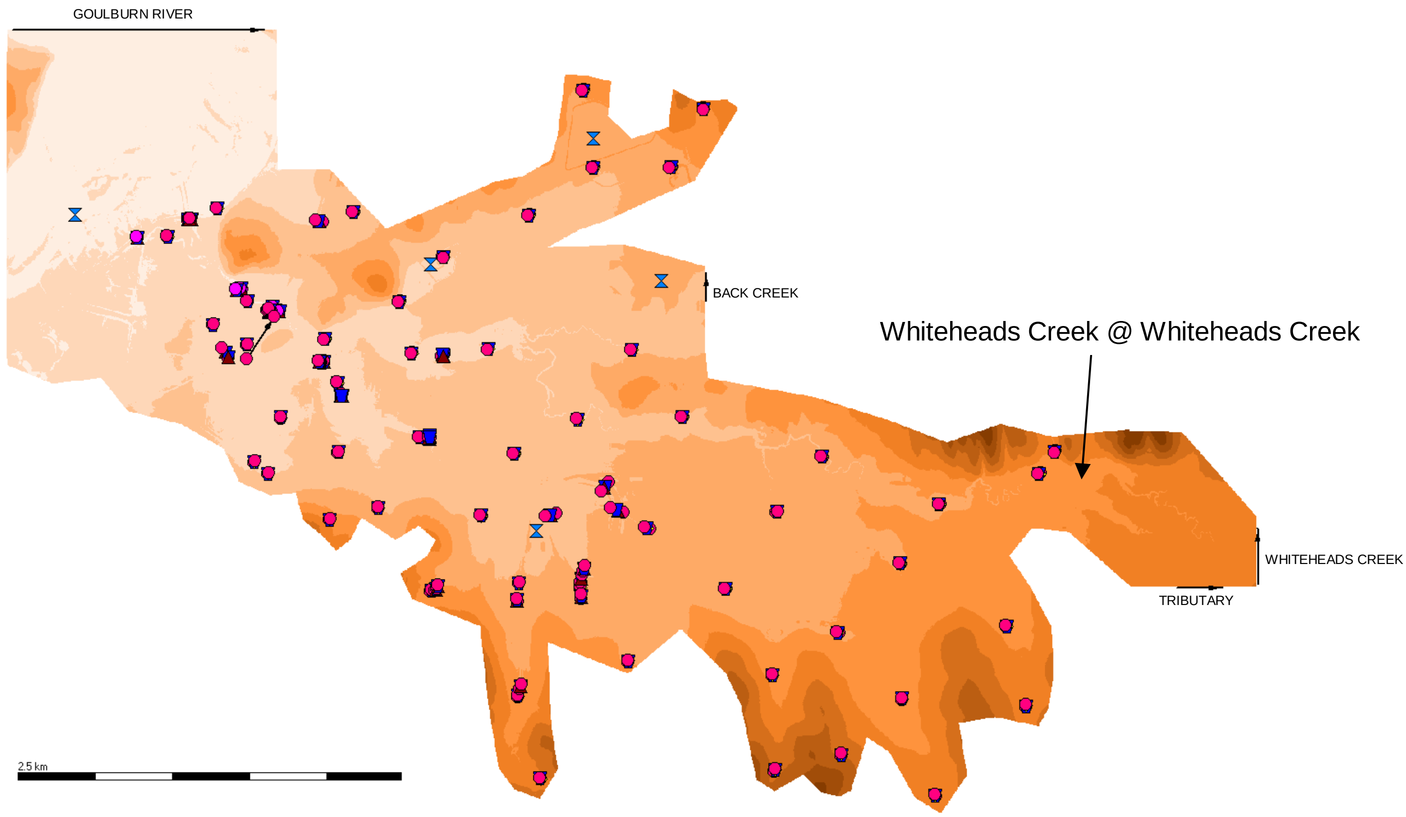


Figure 4-5 Hydraulic Model layout

4.2 Calibration and Validation

The Whiteheads Creek model has been calibrated to a number of flood events. The primary event that has emerged as being significant is the 1973 event which is the largest event on record for the Whiteheads Creek catchment. This event caused significant widespread damage through the town and overtopped the railway line. From investigation there is a range of anecdotal information that can be used to validate the model.

The other historic events simulated through the model have no calibration points and have been supplied as maps for anecdotal reference. The only historic event with recorded flood marks was the recent January 2016 event which was not a large event relative to historic events but did result in the death of a man swept off Delatite Road. This event has been included due to the recent nature of the event and the flood marks that were captured post event.

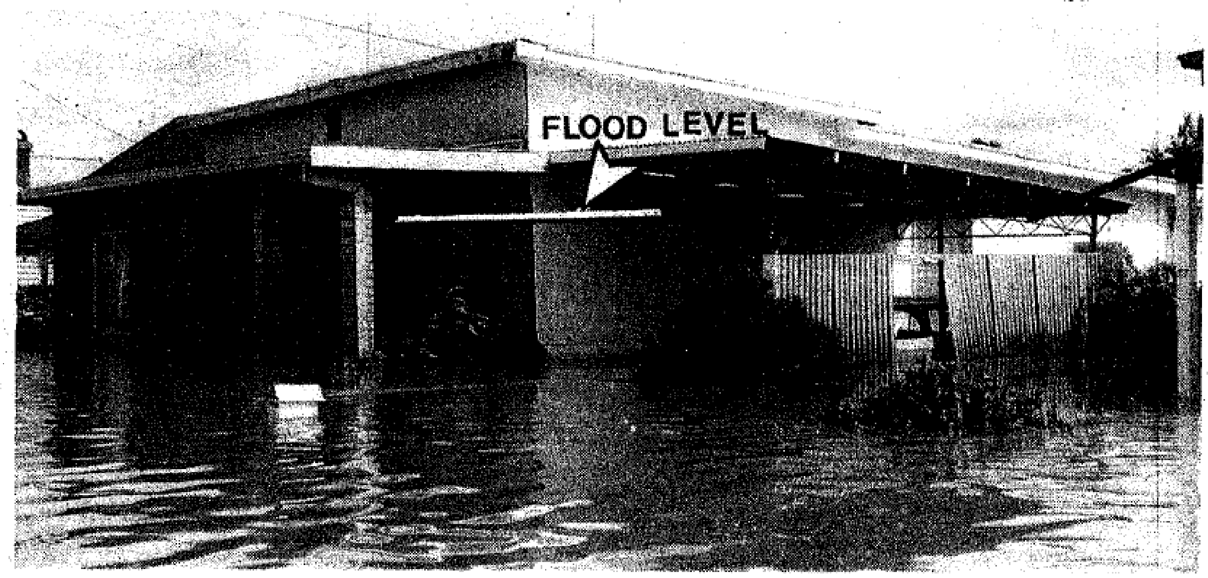
4.2.1 1973

The February 1973 flood event for Seymour was a significant flood for the township and was estimated as having a recurrence interval of much greater than 1 in 100. The township suffered significant damage with one drowning and extensive damage. The flood extent captured from the 1973 event is shown in Figure 4-7. As it can be seen there was significant flooding, particularly upstream of the railway bridge.

The hydrology from the event is relatively unknown as there was no gauge record for this event. The approach adopted in this study was to use the calibrated RORB model to develop the inflows for the event based on the recorded rainfall. The inflows can then be simulated within the hydraulic model and assessed against the past observations and estimated flood extents.

Anecdotal evidence has been supplied for properties at 31 and 55 Wimble Street which recorded peak flood heights against the structures. These levels indicated that the peak depths were over 1.5m deep within Wimble Street. Similarly, it has been noted that there was likely to have been significant blockage at the railway bridge with some images showing entire sheds pressed up against this structure. It is noted that there was at least "3 feet" of water flowing over the railway tracks based on local advice and the railway tracks were overtopped up to and past Victoria Street (adjacent to the station).

A view of Wallis St. as the water receded from Station St.



A house in Wimble St. marked to indicate height of the floodwaters.

Figure 4-6 Flood mark on Wimble Street for the 1973 event

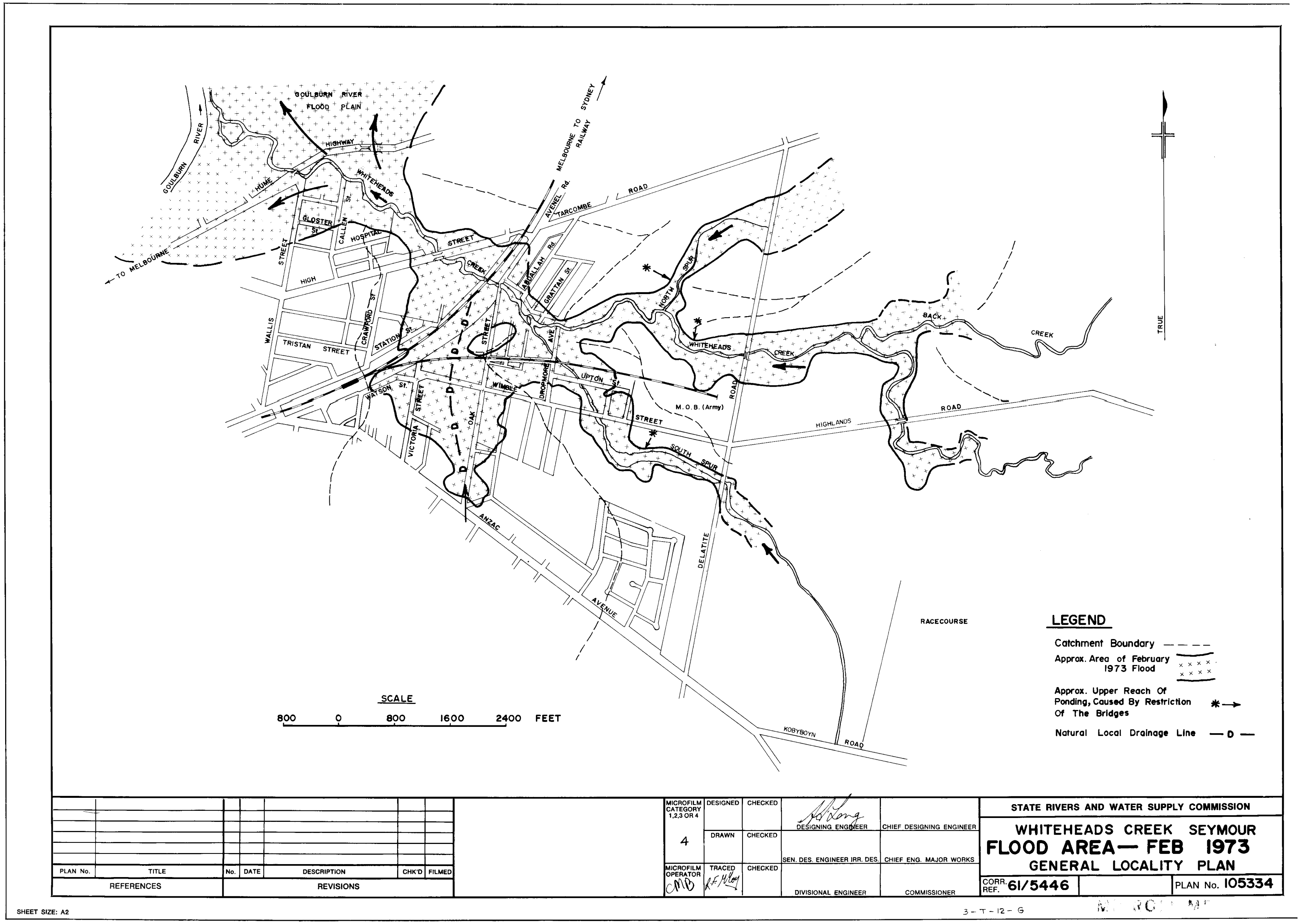


Figure 4-7 1973 flood extent – approximated taken post event

4.2.1.2 Rainfall and Pluviograph Data

The 1973 event recorded rainfall over a three day period with the largest rainfall being recorded on the 21st February. The closest gauge to the Seymour catchment is located at the Seymour Shire Depot which recorded a total of 116.4 mm over the three days. This total is not significant in the context of historic rainfall but the month of February was very wet through the region. In the lead up to this event there was a large event in January and multiple large rainfall totals through February. This preloaded the catchment and saturated the soils. When the rainfall on the 21st January fell it fell on a significantly wet and saturated catchment which resulted in significantly more runoff than typically expected from that depth of rainfall.

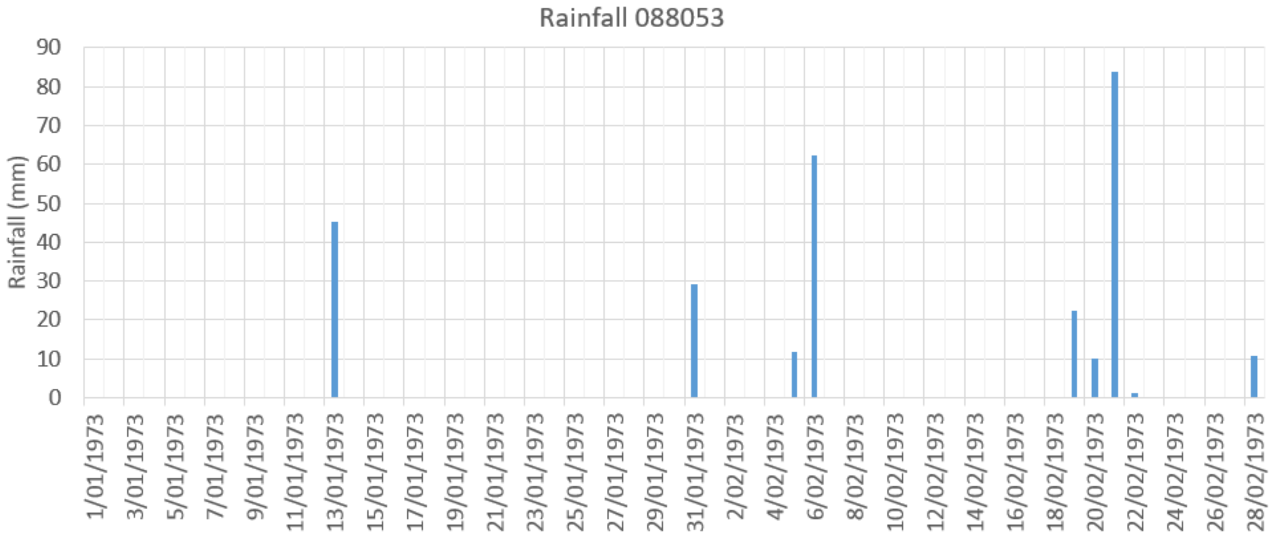


Figure 4-8 Daily rainfall recorded at 088053 in Jan/Feb 1973

This total matches that recorded at Puckapunyal, however it appears as though the rainfall depths increased as the event passed to the north and east with large totals recorded at Avenel, Mangalore Airport and at Highlands (Glentannar). For the assessment the gauged totals located at Seymour Shire depot have been used due to the close proximity to the catchments being modelled. Initial losses and continuing losses have been set low to reflect the wet antecedent conditions.

Table 4-3 **Rainfall totals for the 1973 event**

Rainfall Gauge	Gauge Name	19th Feb	20th Feb	21st Feb	Total
88053	Seymour Shire Depot	22.4	10.2	83.8	116.4
88049	Puckapunyal	22.1	9.9	69.1	101.1
88002	Avenel (Post Office)	20.8	23.4	96.8	141.0
88031	Highlands (Glentannar)	20.6	18.8	143.5	182.9
88109	Mangalore Airport	21.6	22.4	111.8	155.8
88025	Fort William	14.7	21.3	112.3	148.3

The nearest pluviograph with available data was at Puckapunyal (88049). This event captured the rainfall pattern well and had similar rainfall totals as recorded in Seymour. This was used to distribute the rainfall across the subcatchments. The pattern is shown in Figure 4-10.

Figure 4-9 Rainfall gauge locations for the 1973 event

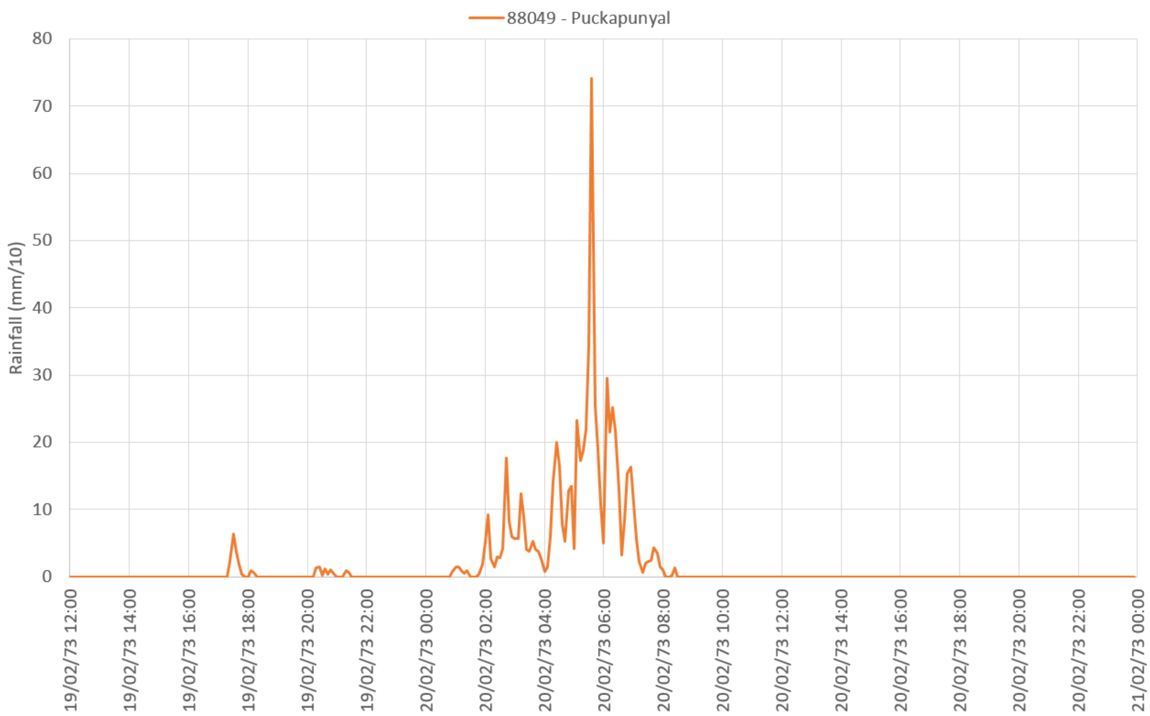


Figure 4-10 Puckapunyal pluviograph pattern for the 1973 flood event

4.2.1.3 Land form changes

Due to the time that has passed since this event it was important to determine if there were any land form changes that may impact the assessment of the 1973 event. The first land form change noted was that the three (3) x 4m diameter culverts under the railway embankment were installed as a result of the 1973 event and were not present. For the hydraulic modelling these were removed.

Similarly, it appears that a bund has been added along the western side of the railway embankment. It was established that this had been added post the 1973 event due to the flood extent supplied and from anecdotal evidence suggesting that the railway embankment was overtopped back to Victoria Street. This is not possible if the bund is present. The topography was adjusted so that this bund was removed for the 1973 event assessment.

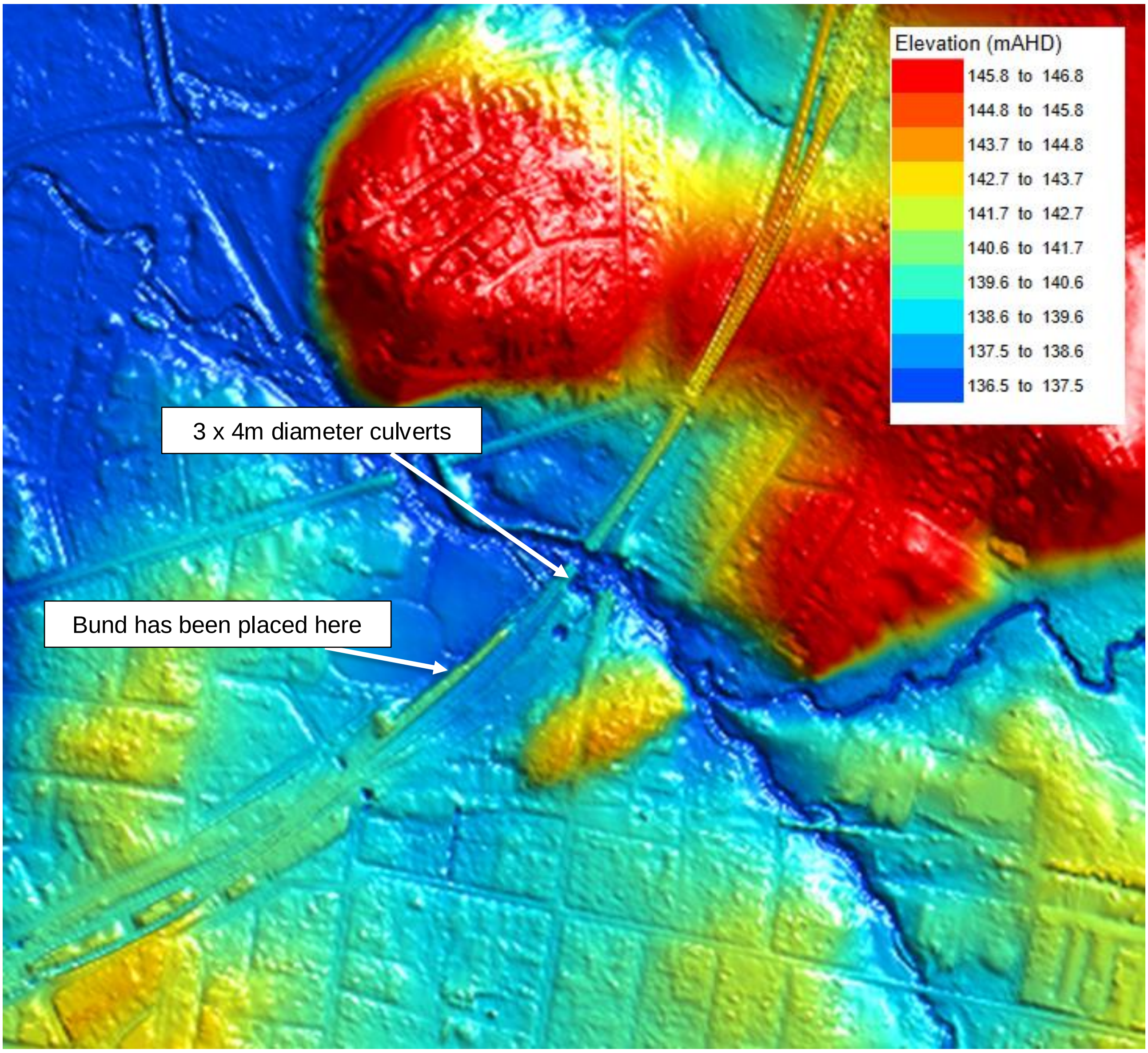


Figure 4-11 Current topography with features modified since 1973 highlighted

4.2.1.4 Event hydrograph

The 1973 event was preceded by a wet January and February and hence this results in reduced initial losses for the catchment. The calibration was run with the same k_c as the calibration at 8.2 with an 'm' value of 0.8. A wide range of initial and continuing losses were tested with the hydraulic model and it was found that a very low loss rate was required to get levels approaching those experienced around Wimble Street.

The initial loss was set low at 10 mm and the continuing loss was set at 1.5 mm/h. This set of parameters resulted in a peak flow rate at the Whiteheads Creek gauge of 279 m^3/s. When compared to the FFA and the design event estimation, this is the equivalent of a 1 in 1000 AEP. This places the event as very rare but based on the anecdotal evidence and flood extents supplied an event which was very significant must have occurred.

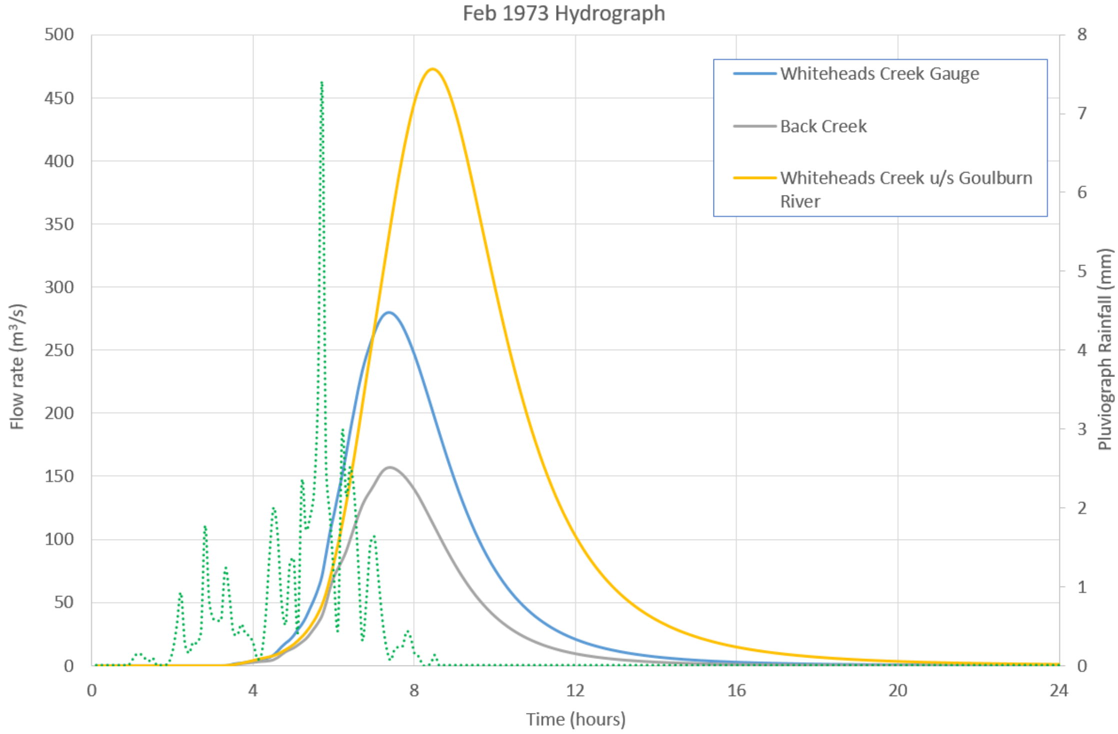


Figure 4-12 RORB hydrographs used for the 1973 flood event assessment

In addition to the low loss rates, additional blockage was introduced to the railway bridge structure. This was based on the anecdotal evidence that the railway bridge was blocked significantly by structures transported downstream from the railway yards. After testing it was decided that an 80% blockage factor was likely to have occurred due to the large shed and infrastructure that was described as being pushed up against the railway opening.

The resulting flood depths and extent are shown in Figure 4-13 with the estimated flood extent shown in the background.

The key observations from this comparison are that

> The flood extent around Wimble Street and the length of railway line being overtopped was smaller area than the estimated extent.

> Upstream of the railway bridge the flood extent exceeds the historic estimate within Whiteheads Creek.

These two observations seem to be in contradiction to each other. The estimated peak flows are significant and yet the amount of overtopping for the railway line and flood extent around Victoria Street and Wimble Street are not well matched.

Key changes that may account for these differences could include:

> It is possible that other land form changes have been introduced to the area. Perhaps the railway line has been raised (or lowered) using scree or fill. Or perhaps it was lowered post event to mitigate against the peak flood levels occurring on Wimble Street?

> Perhaps the location of the bund which was removed from the topography was set at a different level to the assumed flat level for the assessment.

> It is noted that there is also a pedestrian underpass at the end of Victoria Street which goes under the railway line. It is not known if this was in place in 1973 or not. Currently it has been assumed to not be in the hydraulic model for the calibration.

> There may have been localised greater rainfall depths, the rainfall information is only based on point source data and there can be high variability in spatial and temporal rainfall distributions.

> The rainfall pattern (based on Puckapunyal pluviography) could have been different, which may have resulted in more volume arriving quickly.

The upstream extents indicate that the peak flows passing through the model are too high for the 1973 event, however the behaviour of the flood near the rail yards and Wimble Street suggest that there is insufficient volume of water in this location to cause the flooding that is believed to have occurred historically.

Within Wimble Street the historic flood marks on the houses indicate that flood depths against the structures were at least 1.5 m deep. The peak depths reached in the hydraulic model on Wimble Street have a maximum depth of 1.3 m, this is approximately 20-30 cm lower than the newspaper article and anecdotal evidence suggests.

Downstream of the railway bridge the flood extent is well matched to the Goulburn River (Goulburn River levels were set based on recorded levels in 1973).

Overall, given the lack of recorded streamflow data and uncertainty around the flood extents and 1973 land surface characteristics the calibration was deemed to be acceptable for the purpose of the study. In particular, the overtopping of the railway line was at a depth in line with discussions with the community. There seem to be some inconsistencies with the reported levels and extents reached but it is noted that the accuracy of the predicted flood extent (Figure 4-7) is unknown.

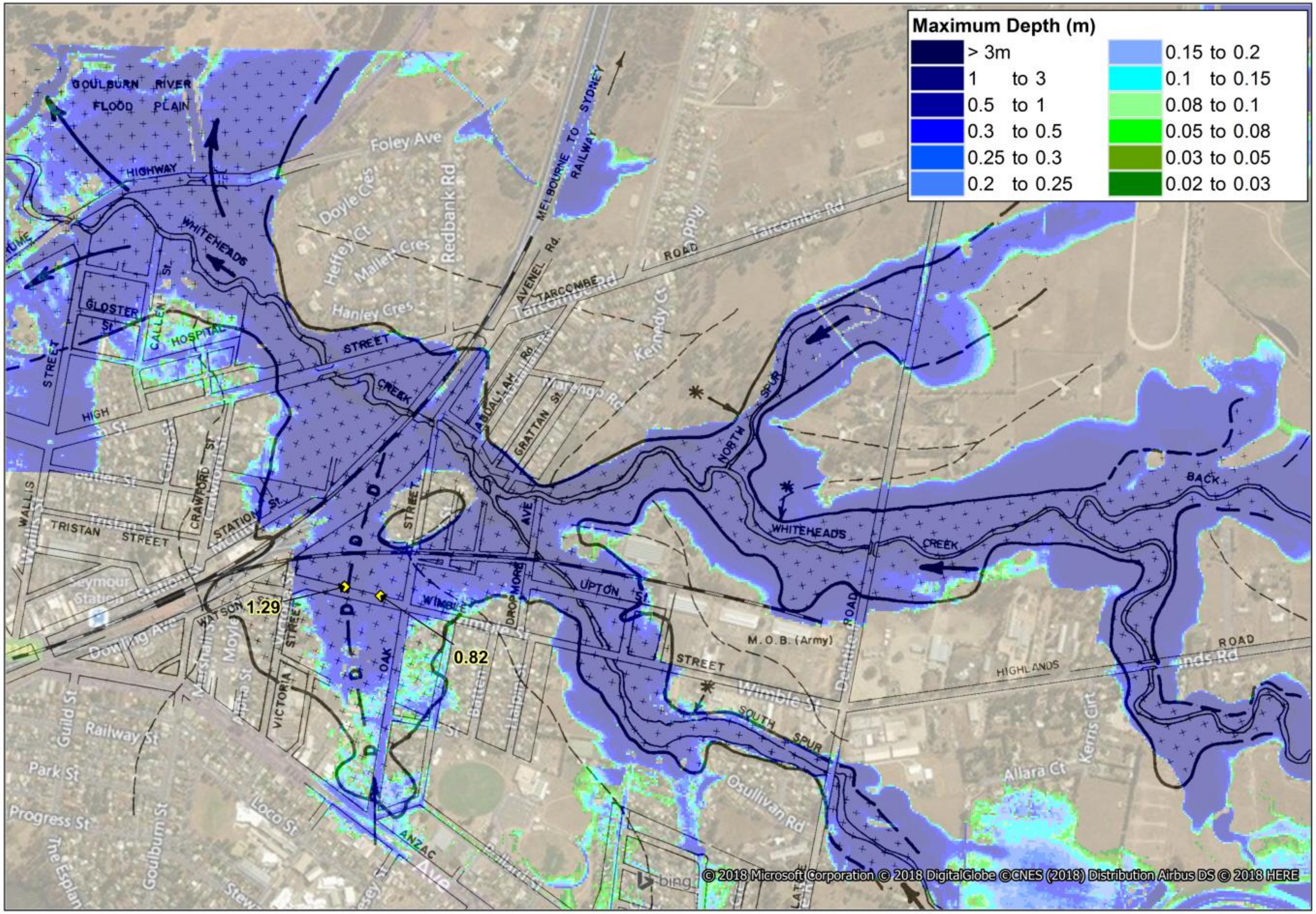


Figure 4-13 Calibration of the 1973 flood event using the hydraulic model

4.2.2 January 2016

The January 2016 flood event was significant due to the death associated with flooding at the Delatite Road crossing of Whiteheads Creek. There was also available flood level data and information for this event collected by the Goulburn Broken CMA post-event. Due to the relatively recent nature of the event there was also suitable rainfall pluviograph data and rainfall totals.

The event was not significant on a regional scale and had a recurrence interval of less than a 20% AEP flood event. The rainfall total for the event was recorded as 48.6 mm recorded at whiteheads Creek at Whiteheads Creek gauge. This was recorded as 0.2 mm increments at the gauge.

The peak recorded at Whiteheads Creek was 3.275 m on the gauge. Using the revised rating table derived in section 3.5.2.2 this results in a peak flow rate of approximately 20 m^3/s. Using the rating table as supplied via the DEWLP water monitoring website, results in a peak flow rate of 11.4 m^3/s. Due to the uncertainty associated with the rating table both hydrographs and inputs were simulated through the hydraulic model. The peak levels and hydrographs are shown in Figure 4-14.

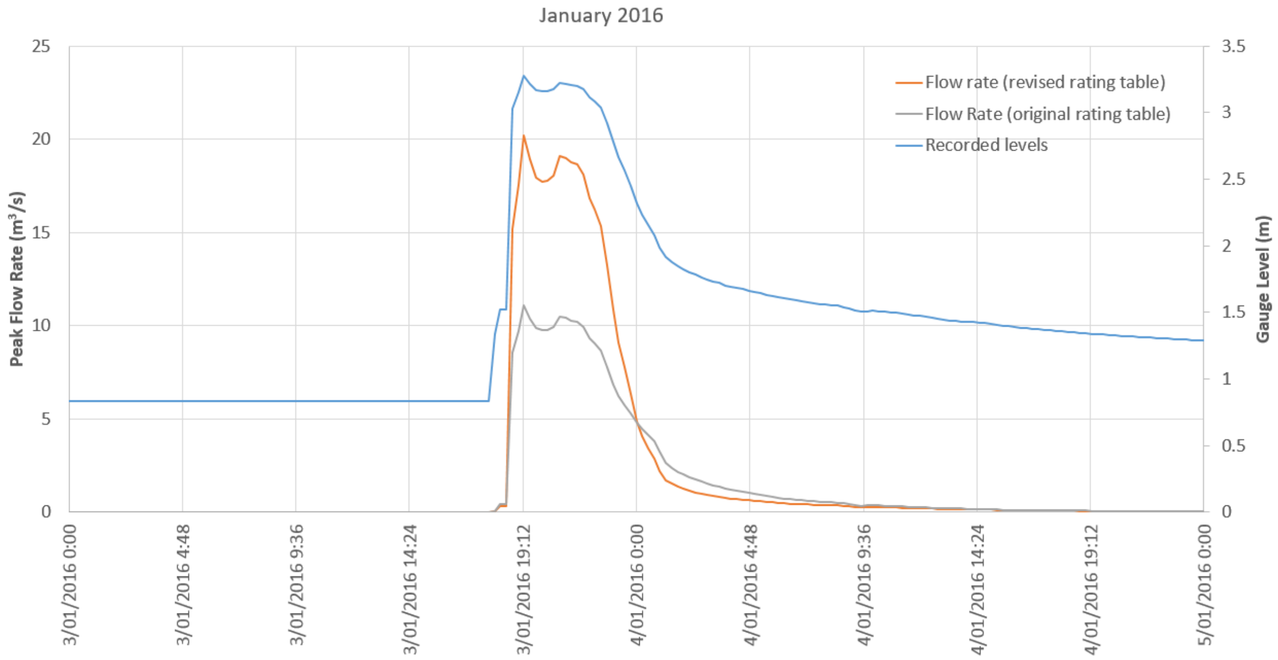


Figure 4-14 Hydrographs and levels for the January 2016 flood event

The recorded peak flood levels and extents from the 2016 flood event are summarised in Table 4-4. The first hydraulic run using the peak flow rate of 11.4 m^3/s resulted in flood extents that were below the observed peaks for the 2016 event. The average difference between the hydraulic model and the recorded extents and levels was – 27.6 cm. The flood extents were also noticeable smaller than observed in most locations along Whiteheads Creek. At the Whiteheads Creek at Whiteheads Creek gauge the peak flood level associated with the inflow rate is 66 cm lower than the recorded level.

Using the updated rating table, the peak flow rate for the event was adjusted up to 20 m^3/s. This was calibrated through RORB and used within the hydraulic model. At the Whiteheads Creek at Whiteheads Creek gauge the peak level was 160.82 mAHD which was only 5.8 cm lower than the recorded level. This suggests that some adjustment to the rating table is warranted. Overall the average difference between the hydraulic model and the recorded points is – 8 cm which is within the tolerances of the model. The differences are shown in Figure 4-15 and the overall flood extent is shown in Figure 4-16.

The key observation from the calibration was that the rating table did require some adjustment to get the peak levels at the gauge matching well. The calibration through the rest of the model was acceptable.

Table 4-4 **2016 recorded flood levels compared to hydraulic model runs**

Id	Flood record category	Recorded Flood Level (mAHD)	Model Results (11 m^3/s peak flow rate)	Difference (Model less Recorded)	Model Results (20 m^3/s peak flow rate)	Difference (Model less Recorded)
S1F	Flood level	136.33	136.22	-0.115	136.34	0.010
S2F	Flood level	137.69	137.57	-0.120	137.78	0.084
S2X	Flood Extent	137.69	137.57	-0.120	137.78	0.084
S3X	Flood Extent	138.34	137.97	-0.370	138.10	-0.242
S4F	Flood level	139.42	139.42	-0.004	139.47	0.044
S4F2	Flood level	139.43	139.42	-0.016	139.47	0.032
S5F	Flood level	141.21	140.63	-0.580	140.92	-0.293
S5F2	Flood level	141.19	140.63	-0.559	140.90	-0.293
S5F3	Flood level	141.21	140.63	-0.577	140.92	-0.288
S6X	Flood Extent	141.62	141.85	0.222	141.85	0.230
S7F	Flood level	143.63	143.32	-0.307	143.53	-0.095
S8X	Flood Extent	158.71	158.32	-0.387	158.46	-0.253
S9F	Flood level	160.88	160.22	-0.661	160.82	-0.058
			Average	**-0.276**	**Average**	**-0.080**

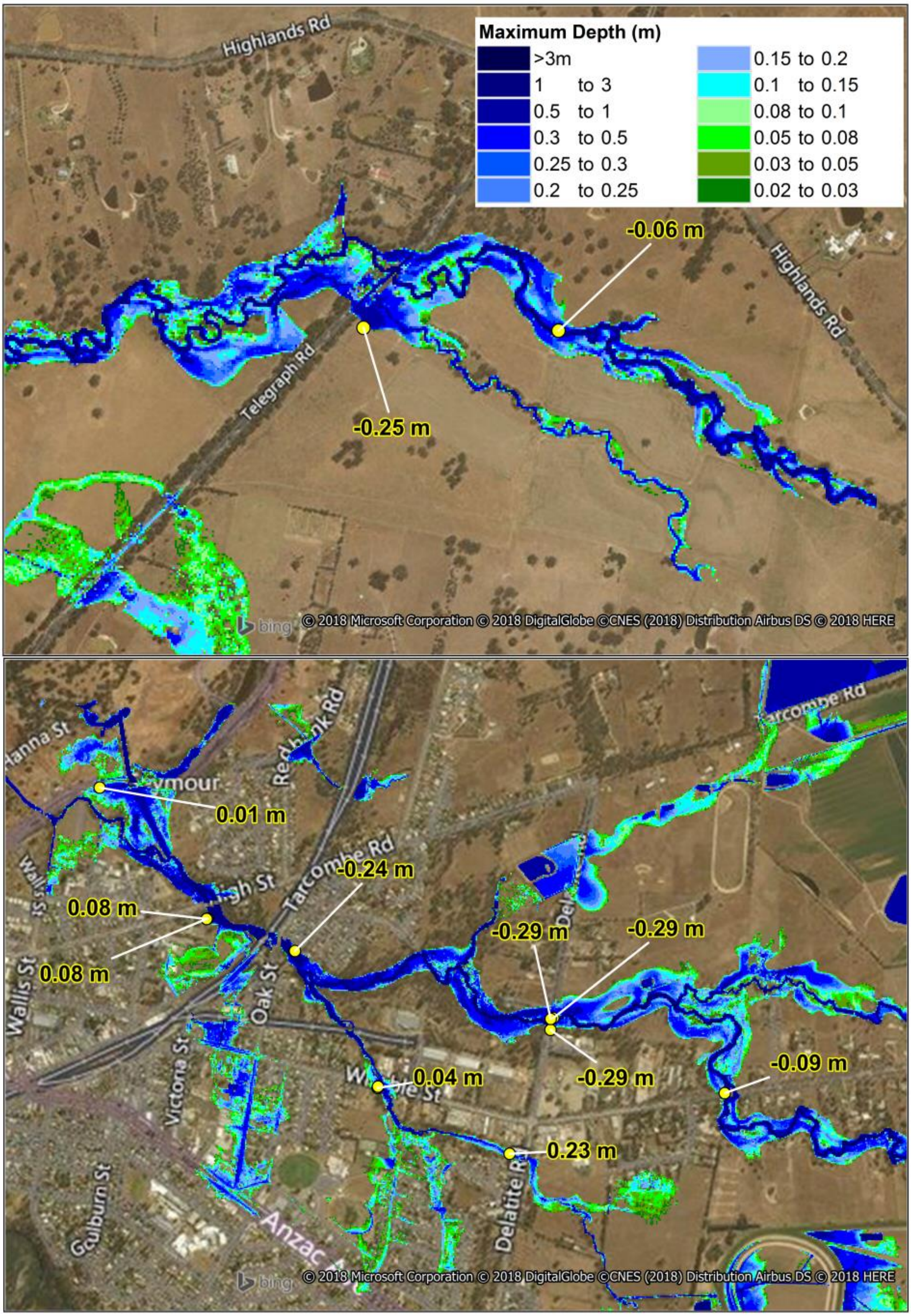


Figure 4-15 Difference between the calibrated 2016 event and recorded points

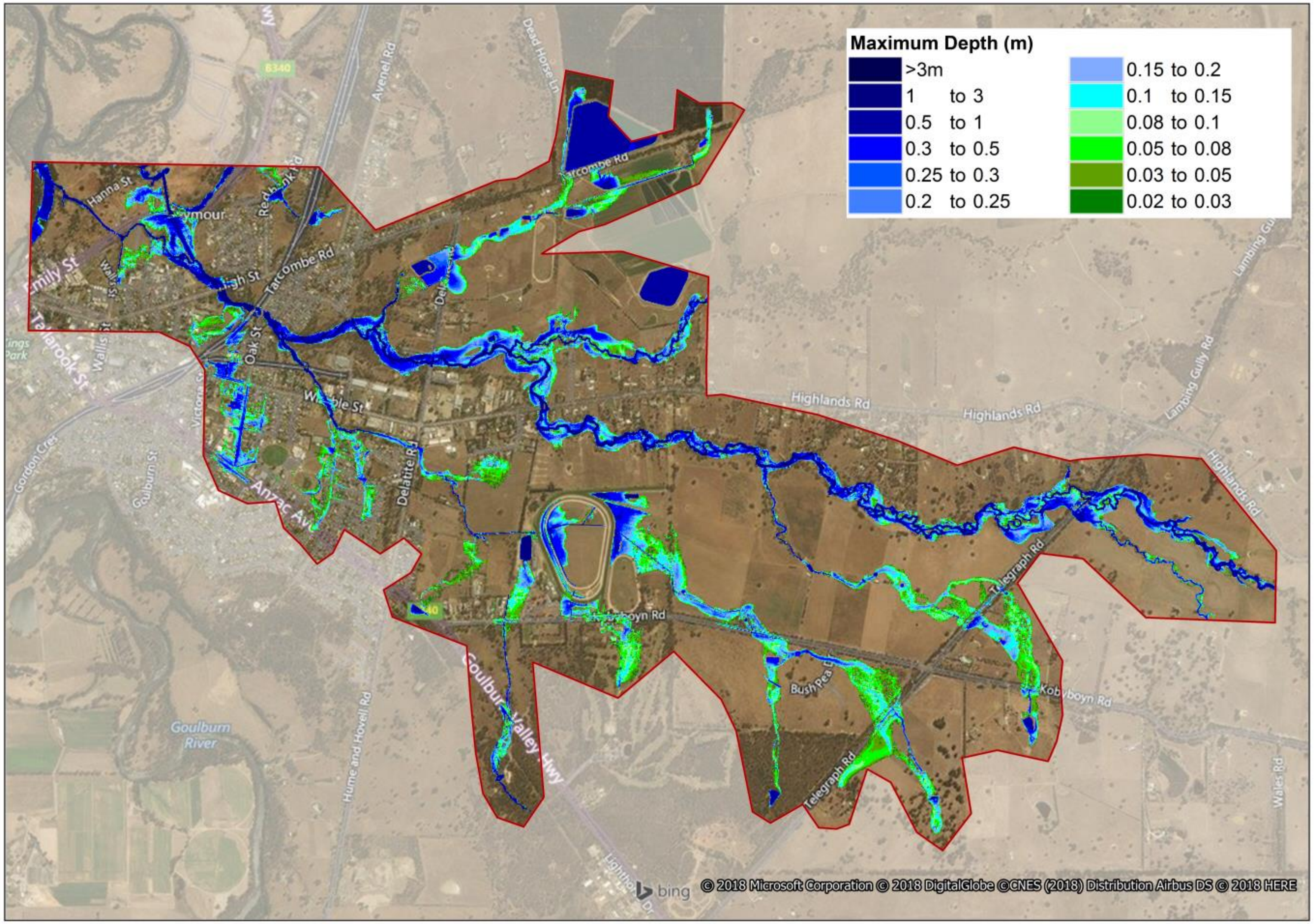


Figure 4-16 Calibrated 2016 flood event extent and maximum flood depths

4.3 Design Events

The design events have been simulated using a range of durations to ensure the critical flood event has been captured. The design events simulated were derived based on the hydrology section. Table 4-5 summarised the durations simulated.

Table 4-5 **Design events run through the hydraulic model**

AEP	2h	3h	4.5h	6h	9h	12h
20%	x	x	x	x	x	x
10%	x	x	x	x	x	x
2%	x	x	x	x	x	x
1%	x	x	x	x	x	x
0.5%	x	x	x	x	x	x
0.2%	x	x	x	x	x	x
0.1%				x	x	x
PMF	x	x	x	x		x

The hydraulic modelling results have been used to inform the flood intelligence and to assist with the update and development of planning controls. The maps showing the peak flood depths are shown in Appendix A. The 1% AEP peak flood depths are shown in Figure 4-17.

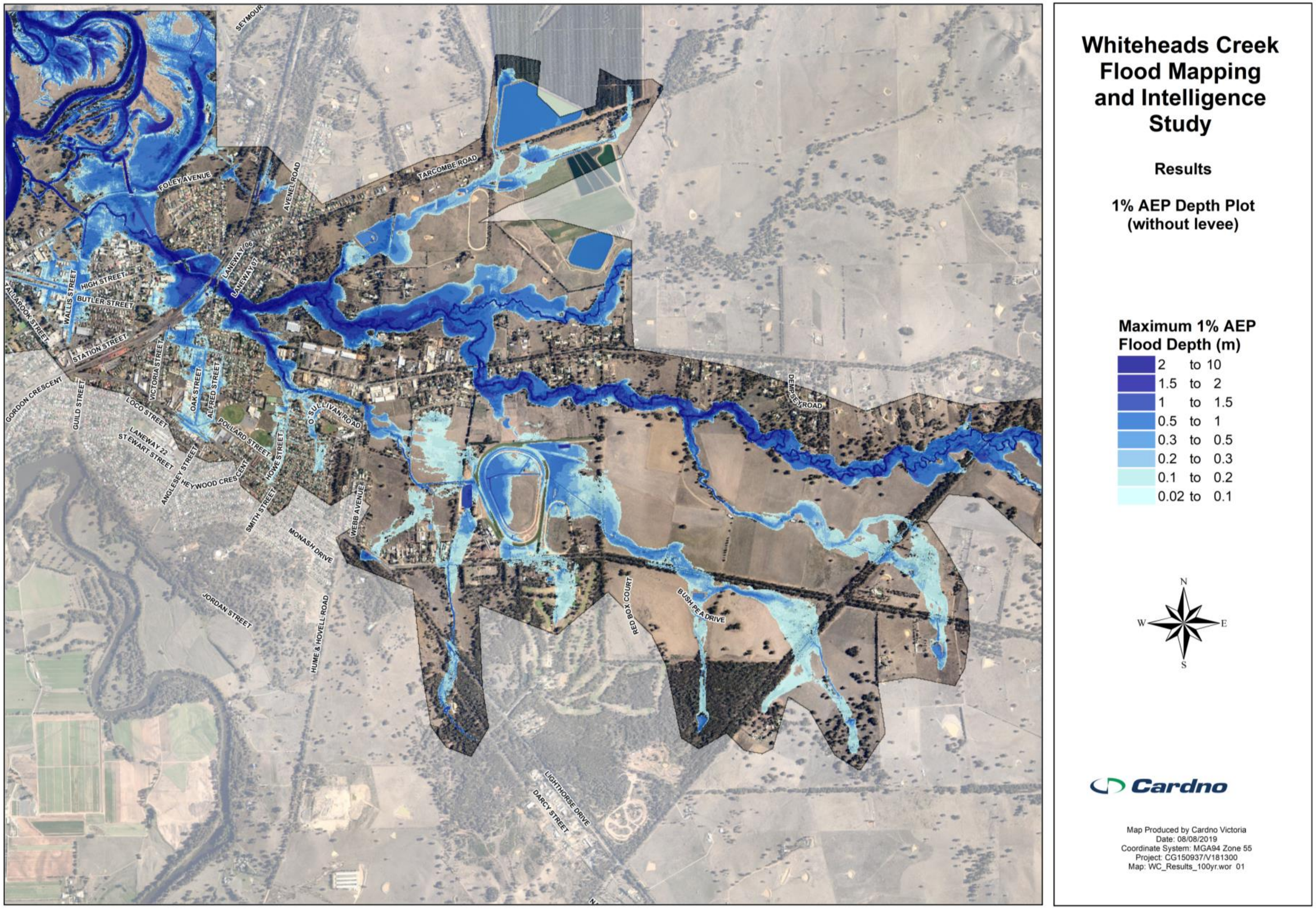


Figure 4-17 Design peak flood depths for the 1% AEP event

4.4 Levee Scenario

Currently Seymour is developing a levee system that surrounds the township. This levee has a primary focus of protecting the township from the long duration Goulburn River floods but the levee does extend up the railway line and provide some protection along Whiteheads Creek.

A complication for the levee design along Whiteheads Creek are the locations where the levee requires temporary flood barriers to be placed (like High Street). This location relies on temporary barriers to be placed to manage flooding. There is a major difference between the Whiteheads Creek warning time compared to the Goulburn River. The Goulburn River has many hours or even days to receive warning of flooding arriving from upstream whereas Whiteheads Creek is subject to faster occurring (like flash flooding) events which can rise and fall in hours. This poses some management issues for the temporary barriers for flooding arising from Whiteheads Creek.

At the time of this modelling no finalised levee design was completed and levee heights were not finalised. It is understood that the levee is expected to have a minimum of 1% AEP protection with freeboard and as such the level has been set sufficiently high within the hydraulic model to reflect this. Openings in the railway embankment and levee have been assumed to be sealed for this assessment. These locations include the Victoria Street pedestrian underpass and the levee at High Street.

The assessment of the levee has two main objectives:

- To establish the flood behaviour of the system with the levee in place
- To determine the change in peak flood levels for the 1% AEP event upstream and adjacent to the levee system.

The results of the 1% AEP flood assessment with the levee in place is shown in Appendix B. The levee causes some increases to the peak flood levels expected in the 1% AEP, as shown in Figure 4-18. These increases are less than 15 cm in the design 1% AEP flood event and do not impact any additional floors in the floodplain.

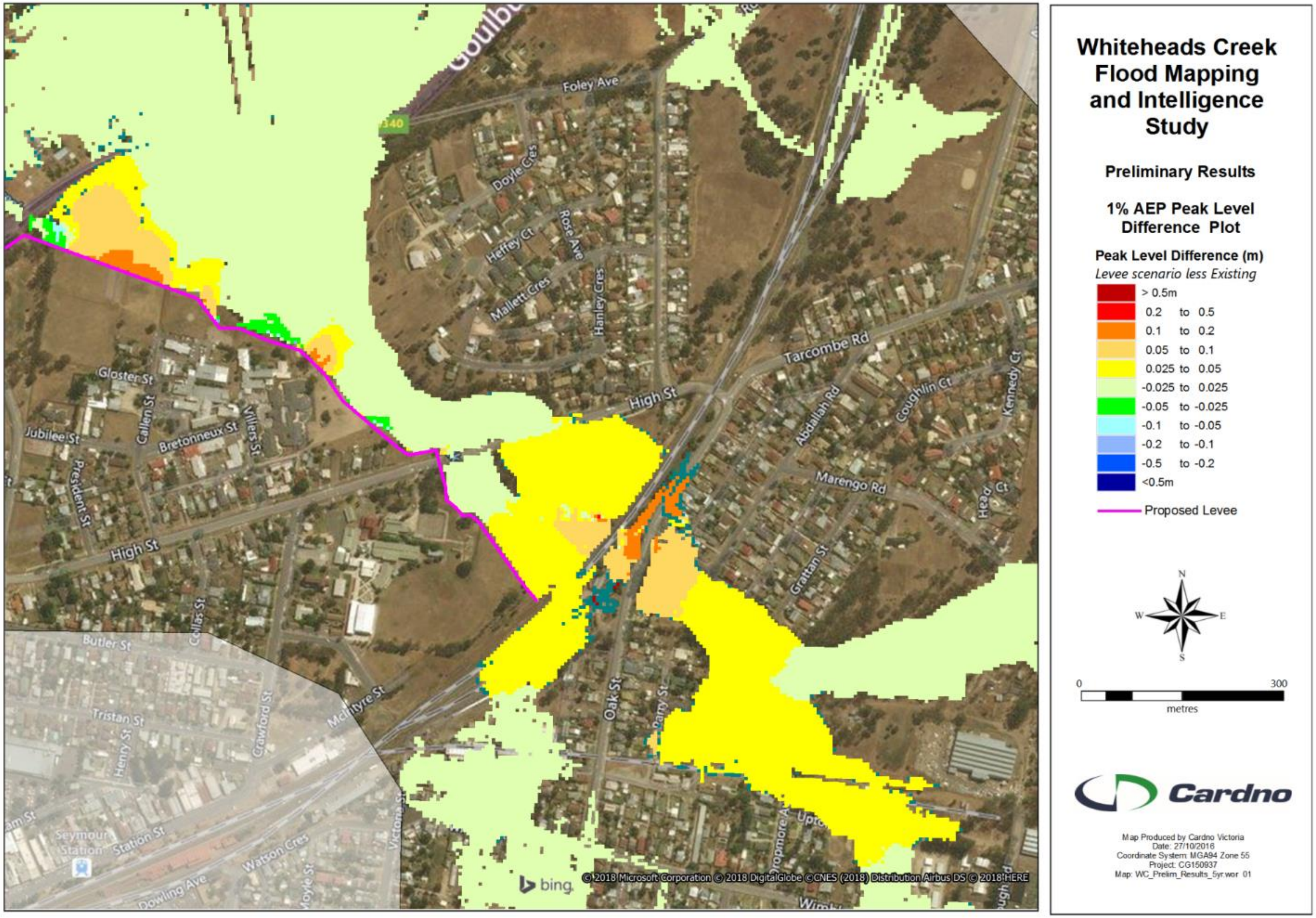


Figure 4-18 Peak flood level difference plot for the 1% AEP design event

4.5 Flood Travel Time

Definitive information on the time it takes rainfall associated with severe weather or thunderstorm activity to develop into runoff and therefore streamflow is highly dependent on antecedent conditions.

The travel time of flood waters has been calculated as the time for the peak for each of the design events and for the key calibration events from the Whiteheads Creek gauge to move to the railway bridge and the Goulburn Valley Highway. All times are in hours from the start of the rainfall event. The travel times have been calculated along the main channel of Whiteheads Creek for the critical flood event. In most design cases, this event was the 9-hour storm. Table 4-6 shows the estimated time for the peak flood to travel between Whiteheads Creek at the gauge to upstream of the railway bridge and then to Emily Street.

Table 4-6 **Estimated travel time along Whiteheads Creek**

AEP Event	Location	Start of Rise (hrs)	Peak Flood (hrs)	Travel Time of Peak from WHC Gauge (hrs)
1%	Whiteheads Creek at Gauge	3:00	4:50	0
	Upstream of Railway Bridge	3:40	7:10	2:20
	Emily Street	4:40	7:30	2:40
2%	Whiteheads Creek at Gauge	2:50	5:10	0:00
	Upstream of Railway Bridge	3:10	6:40	1:30
	Emily Street	3:20	7:00	1:50
5%	Whiteheads Creek at Gauge	2:10	4:20	0:00
	Upstream of Railway Bridge	2:30	5:50	1:30
	Emily Street	2:40	6:10	1:50

5 Assessment of Risk

5.1 Damage Assessment

The economic impact of flooding is defined by what is commonly referred to as ‘flood damages’. Flood damages are classified into direct, indirect or intangible classes, as shown in Figure 5-1.

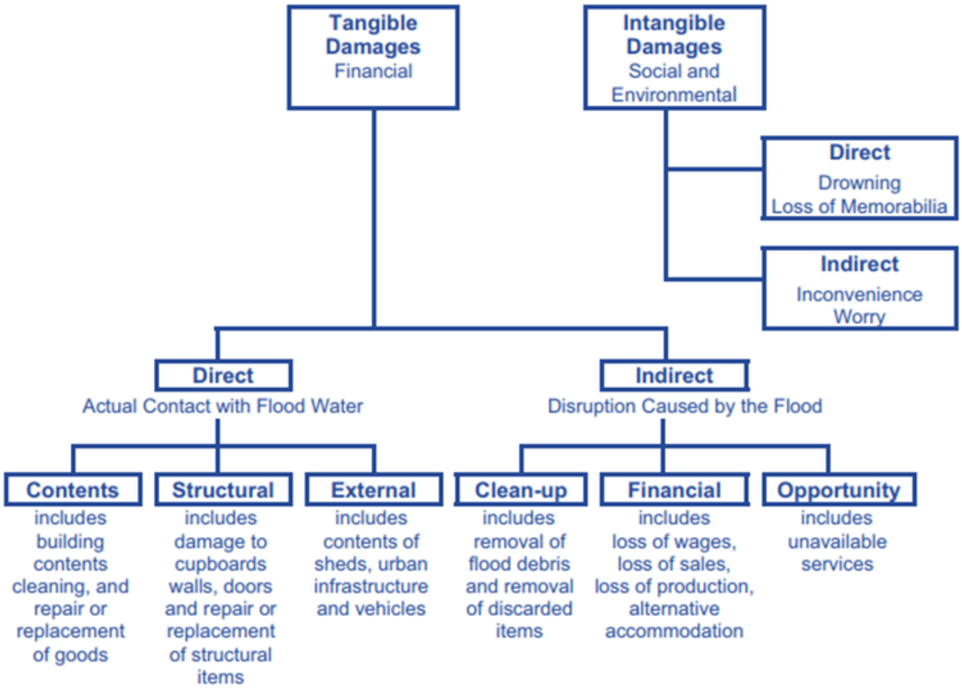


Figure 5-1 Types of flood damage (Floodplain Development Manual, NSW Gov, 2005)

The damage assessment in this report is restricted to tangible direct damages only, and does not include an estimate of the costs associated with ‘intangible’ damages, such as social distress and loss of memorabilia or tangible ‘indirect’ damages such as loss of business or income in commercial areas. Furthermore, it has been assumed that residents or commercial tenants will have little to no warning time in a flood event and hence, no allowance has been made for protecting or removing valuables. This provides a more conservative estimate of flood damages as the maximum ‘potential’ damage is assessed in this case.

Cardno have developed an automated methodology using the data manipulation software FME with integrated excel spreadsheets for calculating AAD. The assessment has been based on damage curves provided by Melbourne Water, which relate flood depth to likely property and building damage.

The Cardno approach is semi-automated, enabling faster and more accurate analysis of AAD by minimising manual data manipulation errors. In the development of this process, comparison to the MW process for several test catchments was undertaken and virtually identical results were achieved.

The following sections provide an overview of the methodology applied for the determination of damages within the Whiteheads Creek study area. Damages have been divided into three main categories:

> Building Damages (residential and commercial/industrial buildings)

> Property Damages (flooding over property with no buildings impacted)

> Road Damages (flooding occurring over roads)

AAD is calculated by a probability approach of flood damages incurred for each design storm event. The total damage, being the summation of the damage to all buildings, properties and roads within the flood extent. This has been calculated for the 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEP flood events for the existing condition. The AAD for the levee mitigation option has been calculated for the 20% and 1% AEP events.

For example, the 1 in 100 ARI design event has a 1% chance of occurring in any given year and has a total estimated damage of $7,529,745 under existing conditions. This maximum point is plotted on a probability curve, along with the estimated damage for every other modelled storm event. The AAD is then calculated as the area under the damage curve.

The AAD analysis is however, heavily weighted to the damages incurred at the lower level storm events, such as 5, 10 and 20 year ARI storms. Therefore, it can be expected that any mitigation options which are targeted to eliminate the 5, 10 or 20 year ARI flooding will perform comparatively better to any mitigation options that are specifically targeted to eliminate the flooding events greater than 50 year ARI.

The methodology employed to calculate each category of damage is detailed in the sections below.

5.1.1 Building Damages

Data Vic building footprints were mainly utilised for this analysis, with the addition of manually digitised building footprints from current Nearmap aerial imagery (August, 2019).

Floor levels were assumed to be 300 mm above the natural surface (as estimated from the average ground surface, within the building).

Damage to buildings from flooding, begins prior to any over floor flooding. The damage estimates included in this report are based on three possible scenarios:

> Where flooding overtops the property ground level but does not reach the base of the house (i.e. the slab), no building damages are incurred.

> Where flooding overtops the property ground level, residential building damages begin once the water level reaches the base of the house, defined as floor level minus 0.5m (slab on ground). Melbourne Water’s damage curves allow for a starting damage of $11,182. This value also accounts for property damages as well as some damage to cars and structures.

> Where flooding overtops the property ground level, commercial building damages begin once the water level reaches the base of the building, defined as the floor level. Melbourne Water’s damage curves allow for a starting damage of $5,500 per 100m^2 of building area.

Residential, Commercial and Industrial Building Damage Curves were implemented to conduct building damage assessments. The building damage curves have a Capital Price Index (CPI) adjustment factor of 1.06 applied to them, as calculated in Table 5-1.

Table 5-1 **Capital Price Index Adjustment for Building Damages**

	Residential	Commercial	Industrial
Base Damage Curve Year	Jul-15	Jul-15	Jul-15
Assessment Year	Jan-19	Jan-19	Jan-19
Base CPI	107.5	107.5	107.5
Assessment CPI	114.1	114.1	114.1
Adjustment Factor	1.06	1.06	1.06

Residential and Commercial Damage Curves provided by Melbourne Water are shown in Figure 5-2 and Figure 5-3. The industrial damages have been assumed as 80% of commercial damages.

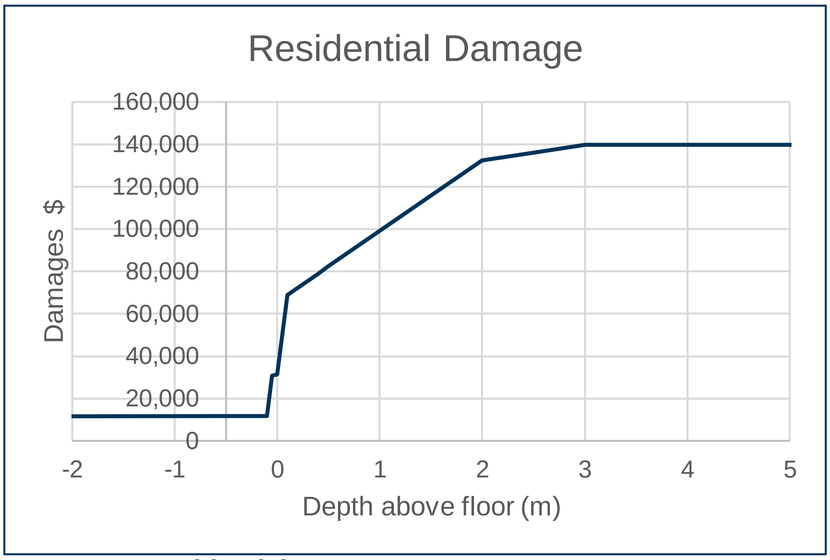


Figure 5-2 Melbourne Water Residential Damage Curve

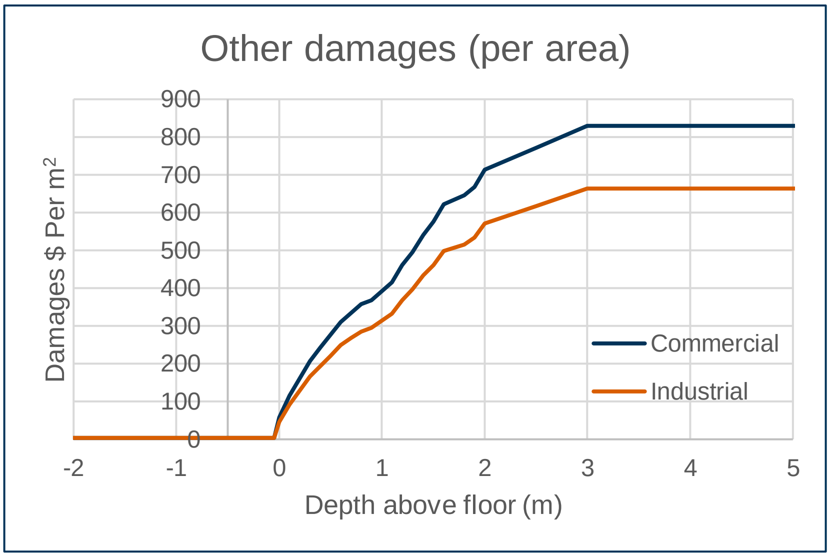


Figure 5-3 Melbourne Water Commercial Damage Curve and Assumed Industrial Damage Curve

5.1.2 Property Damages

Property damage has been defined as occurring when any property is modelled as experiencing flooding. This excludes any properties where the building is considered to be impacted by flooding (i.e. flood level is within 0.5 m of the floor level, as defined in Section 5.1.1 above) as property damage has been included in the building damage calculations.

It should be noted that property damage includes damage to gardens, fencing and extended inundation.

A property has been determined as experiencing flooding if the centroid of two or more model grid cells of the flood extent are within the property boundary.

A flat damage rate has been applied to all properties assessed as being subject to property damage. These are:

> $500 for Residential Properties

> $5000 for Commercial and Industrial Properties

5.1.3 Road Damages

Road damages have been assessed, based on the Rapid Appraisal Method (RAM), which assigns a damage value for major roads, minor roads and unsealed roads. The RAM Method was developed in May 2000, quoted in May 2000 dollars. To convert these to January 2019 dollars, the CPI was used to adjust for inflation. The adjustment factor is shown in Table 5-2.

Table 5-2 **Roads Damage Adjustment Factor**

CPI	
2000 May	69.7
2019 Jan	114.1
CPI Multiplier	1.64
Change	61.09%

The RAM uses a single estimate cost per km for all roads inundated by flood water and includes:

> Initial repairs to roads
> Subsequent additional maintenance to roads
> Initial repairs to bridges (based on 1/3 of road damages)
> Subsequent additional maintenance to bridges.

The RAM estimate of costs per m^2 of inundated road are shown in Table 5-3. These unit damages were adjusted using the CPI adjustment factor as discussed above. The RAM also states, that the damages to roads and bridges generally outweighs the costs associated with other infrastructure such as water, electricity, gas and sewerage services and is a good approximation for overall damage to regional infrastructure.

Table 5-3 **Unit Damages for Roads and Bridges (dollars per m^2 inundated)**

	Major	Minor
2000 May	$ 59,000	$ 18,500
2017 Dec	$ 96,584	$ 30,285
Road Width	20	10
Per m^2	$ 4.83	$ 3.03

5.1.4 AAD Results

The AAD for existing conditions in present value is **$629,854** per annum. The associated damage curve is shown in Figure 5-4. The combined AAD figures for the 20% AEP and 1% AEP events are shown in Appendix B.

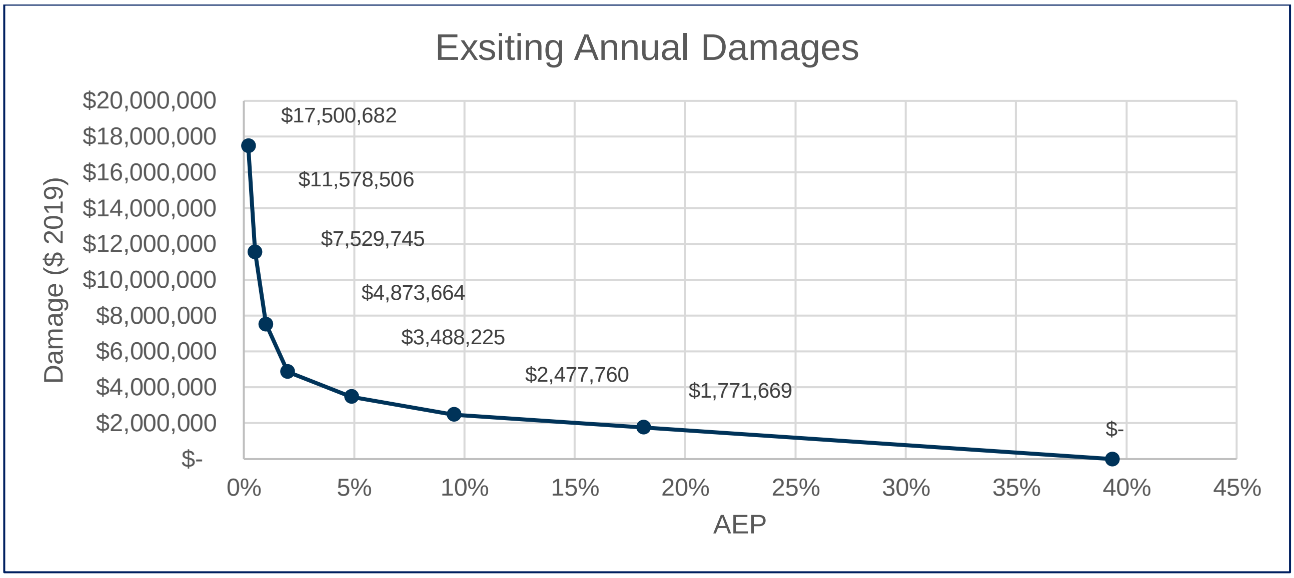


Figure 5-4 Existing Annual Damages

The AAD for the Levee Mitigation Option in present value is **$587,851** per annum. The associated damage curve is shown in Figure 5-5.

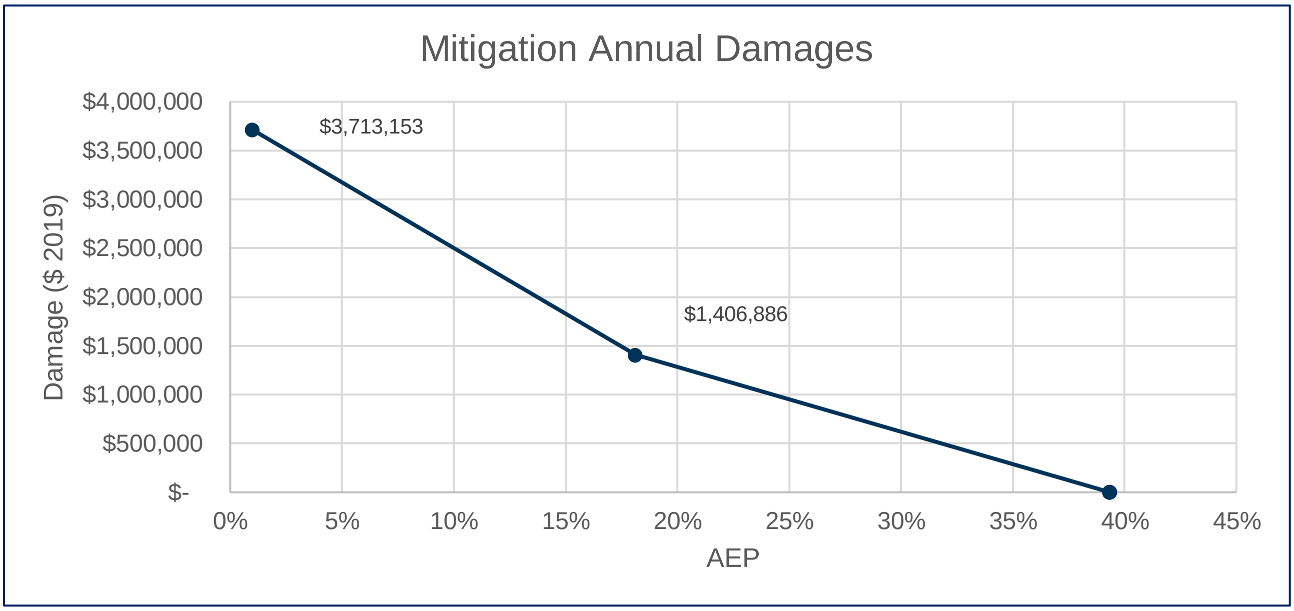


Figure 5-5 Levee Mitigation Annual Damages

Given the extensive suite of AEP flood events utilised to calculate the existing condition AAD, a comparative AAD was determined for only the 1% and 20% AEP events. The comparative AAD was determined as **$984,729**, as shown in Figure 5-6.

Therefore, based on high-level assessment, the Levee Mitigation Option provides **$396,878** average annual benefit.

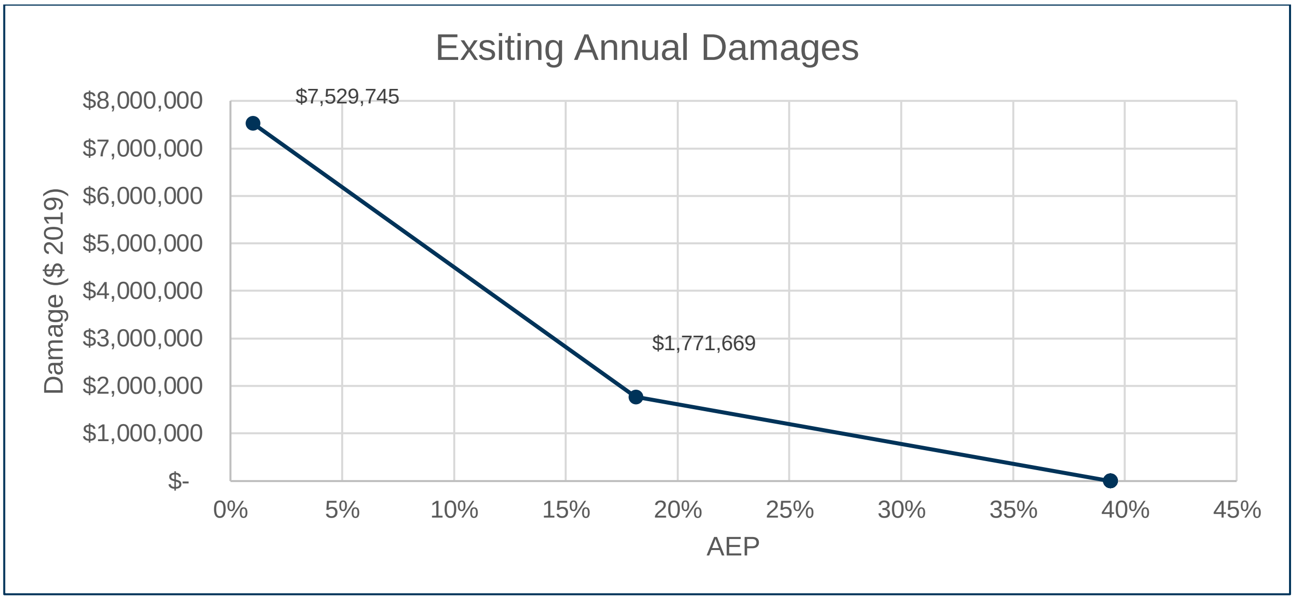


Figure 5-6 Comparative Existing Annual Damages

6 Conclusions

The flood mapping of Whitehead Creek catchment includes the full extent of the tributaries, however it only focuses on Whiteheads Creek. The outcomes of this mapping are expect to be a direct input to the Seymour Structure Plan, and will assist in developing appropriate flood planning and flood intelligence for the township.

The hydrological assessment was undertaken in RORB for the full 100 km^2 catchment of Whiteheads Creek.

The calibration of the RORB model was restricted to flows occurring at the Whiteheads Creek at Whiteheads Creek gauge as this was the only recording station with data within the catchment.

The hydraulic assessment was undertaken in SOBEK. It was calibrated to a number of flood events. The primary event that has emerged as being significant is the 1973 event which is the largest event on record for the Whiteheads Creek catchment. The 2016 event was also included due to the recent nature of the event.

The hydraulic assessment was undertaken for the 20%, 10%, 2%, 1%, 0.5%, 0.2%, 0.1%, and the PMF.

As discussed in this report, the flood model created for this study has been demonstrated to replicate flood levels well for historical events and design flood events modelled. It has been demonstrated to replicate flood levels well for historical events and design flood events modelled. Furthermore, key flood behaviours have been determined for the study area.

A damage assessment was undertaken to estimate the economic impact of flooding. The following conclusions can be made:

> Based on existing conditions, the AAD for the study area is approximately $629,854 per annum; and

> From the information available at the time of this study regarding the levee system, it has been shown that the levee causes some increases in peak flood level in the 1% AEP event.

As required by this project, a range of datasets and mapping outputs have been developed and delivered.

Mitigation options will be assessed and presented in a Mitigation Report.

7 References

World Meteorological Organization (2009) Manual for Estimation of Probable Maximum Precipitation, 3rd edition, WMO - No. 1045, Geneva, ISBN 978-92-63-11045-9

APPENDIX

A

DEPTH RESULT MAPS

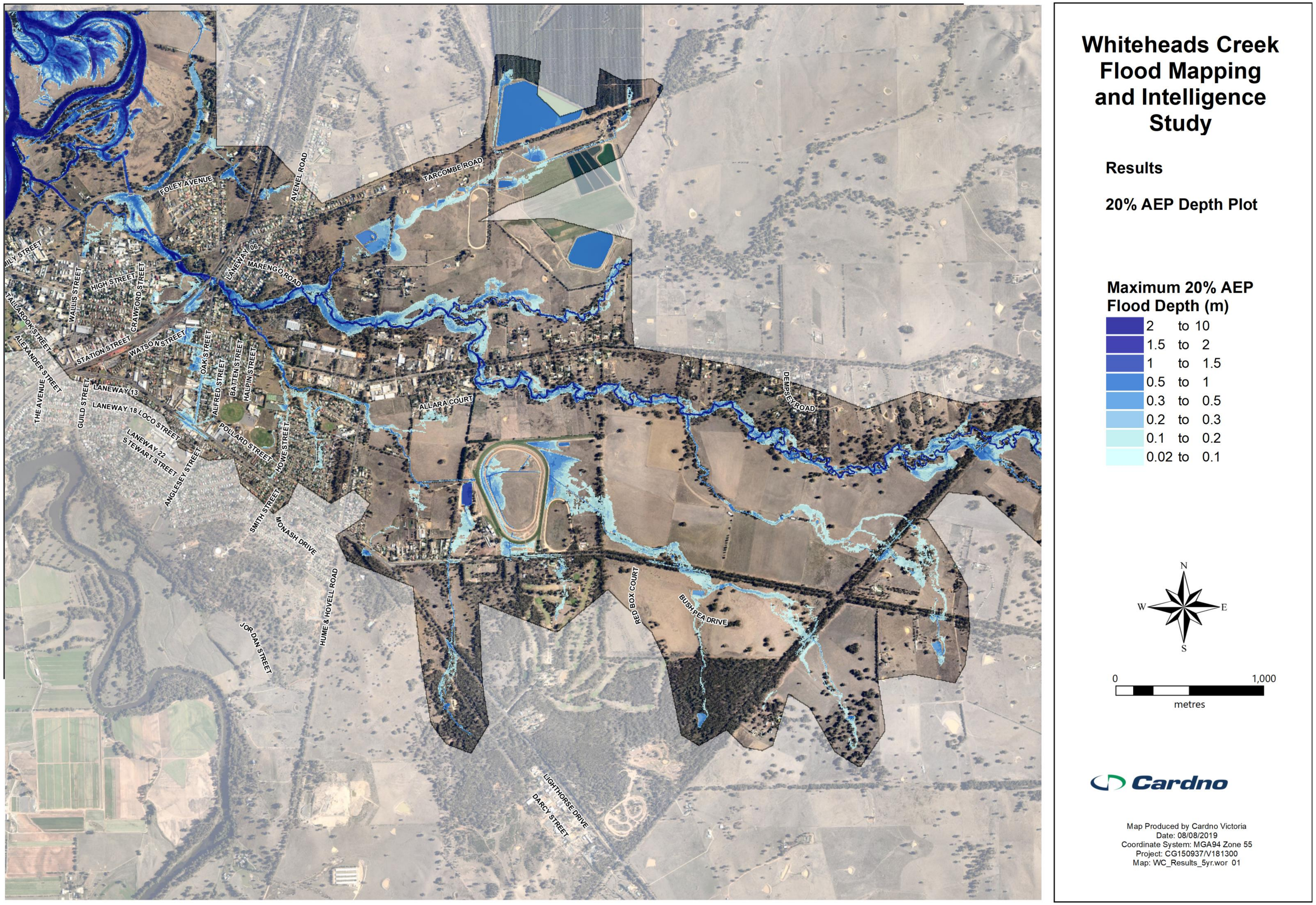


Figure 7-1 Flood depths for the 20% AEP design event

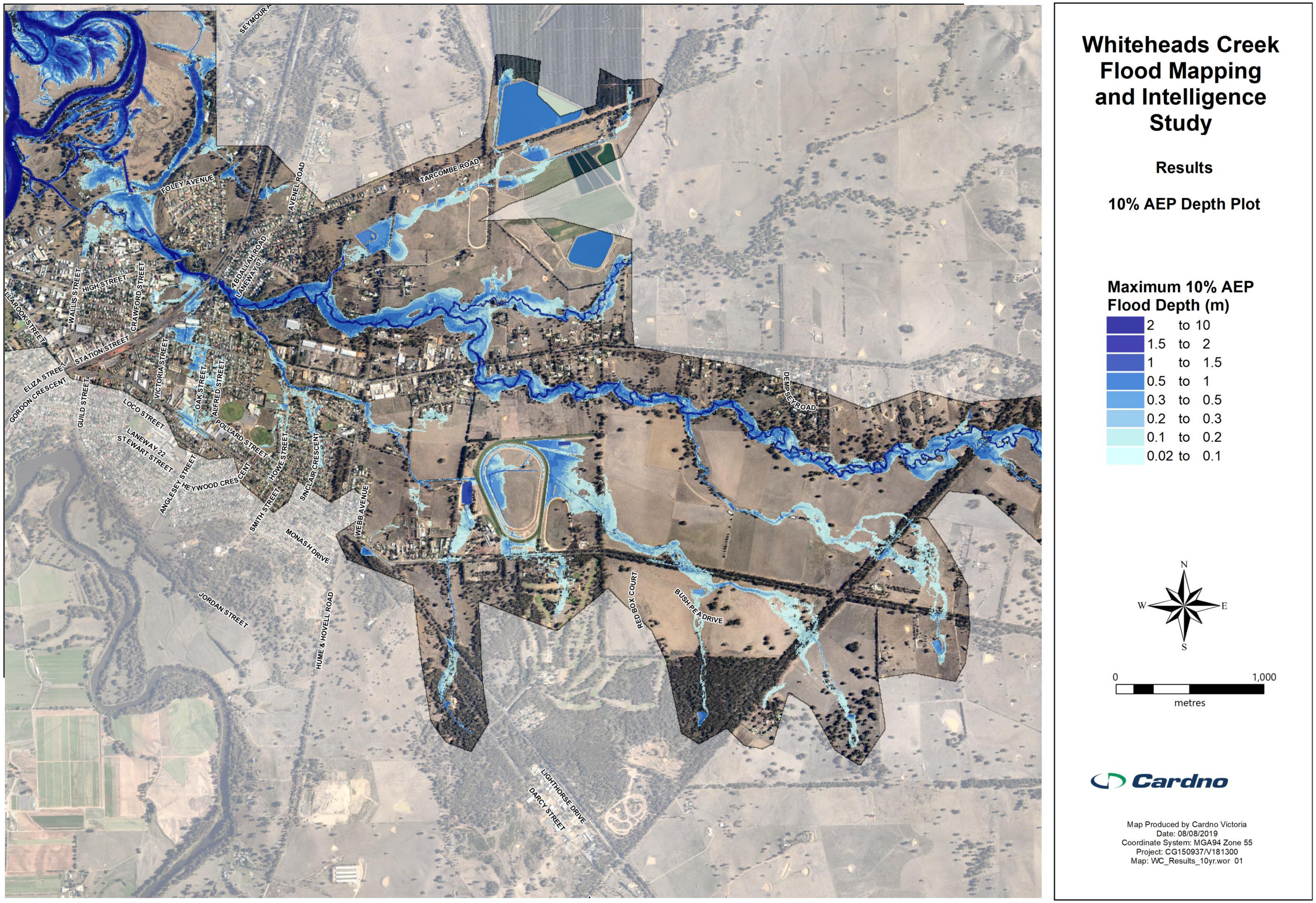


Figure 7-2 Flood depths for the 10% AEP design event

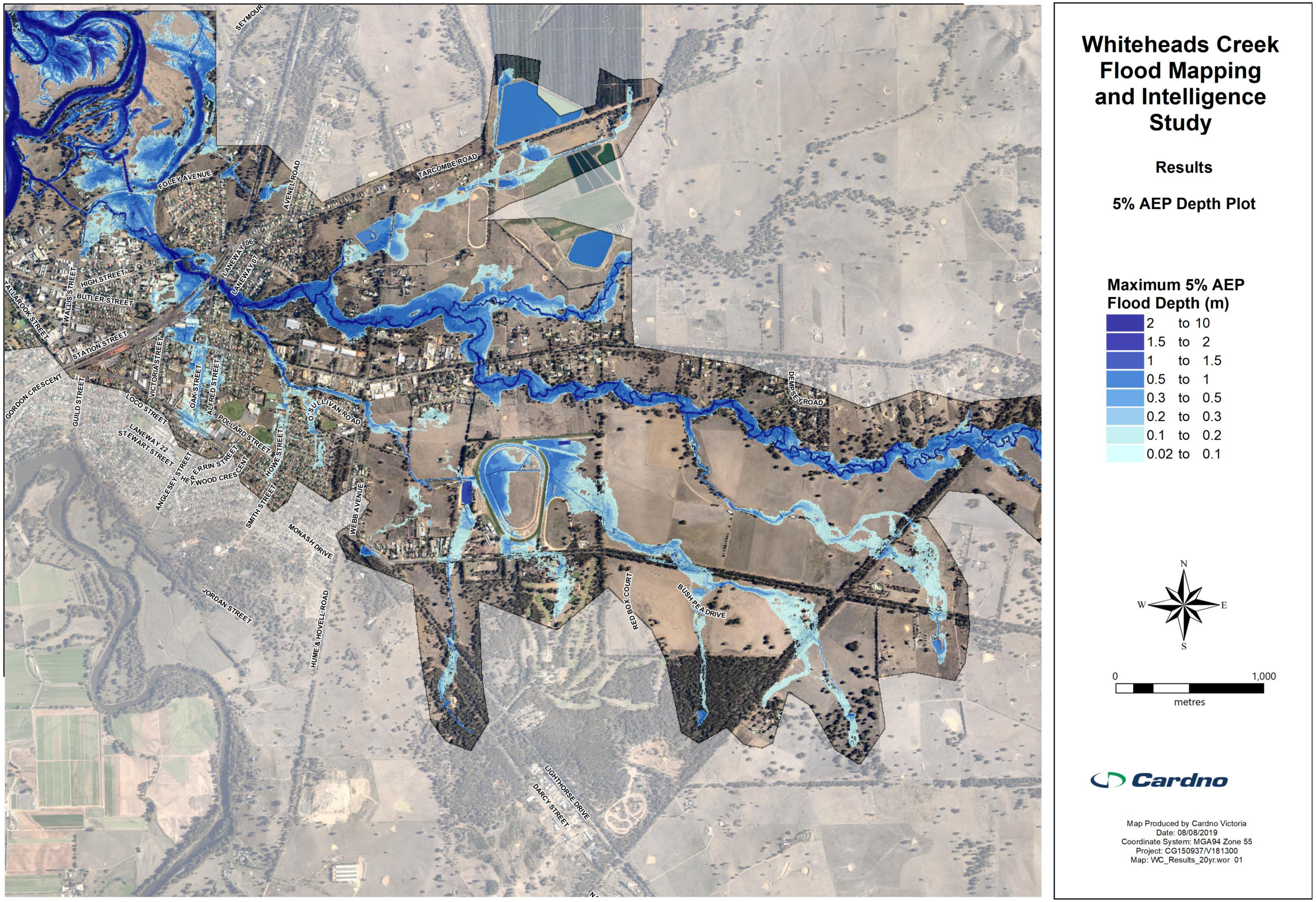


Figure 7-3 Flood depths for the 5% AEP design event

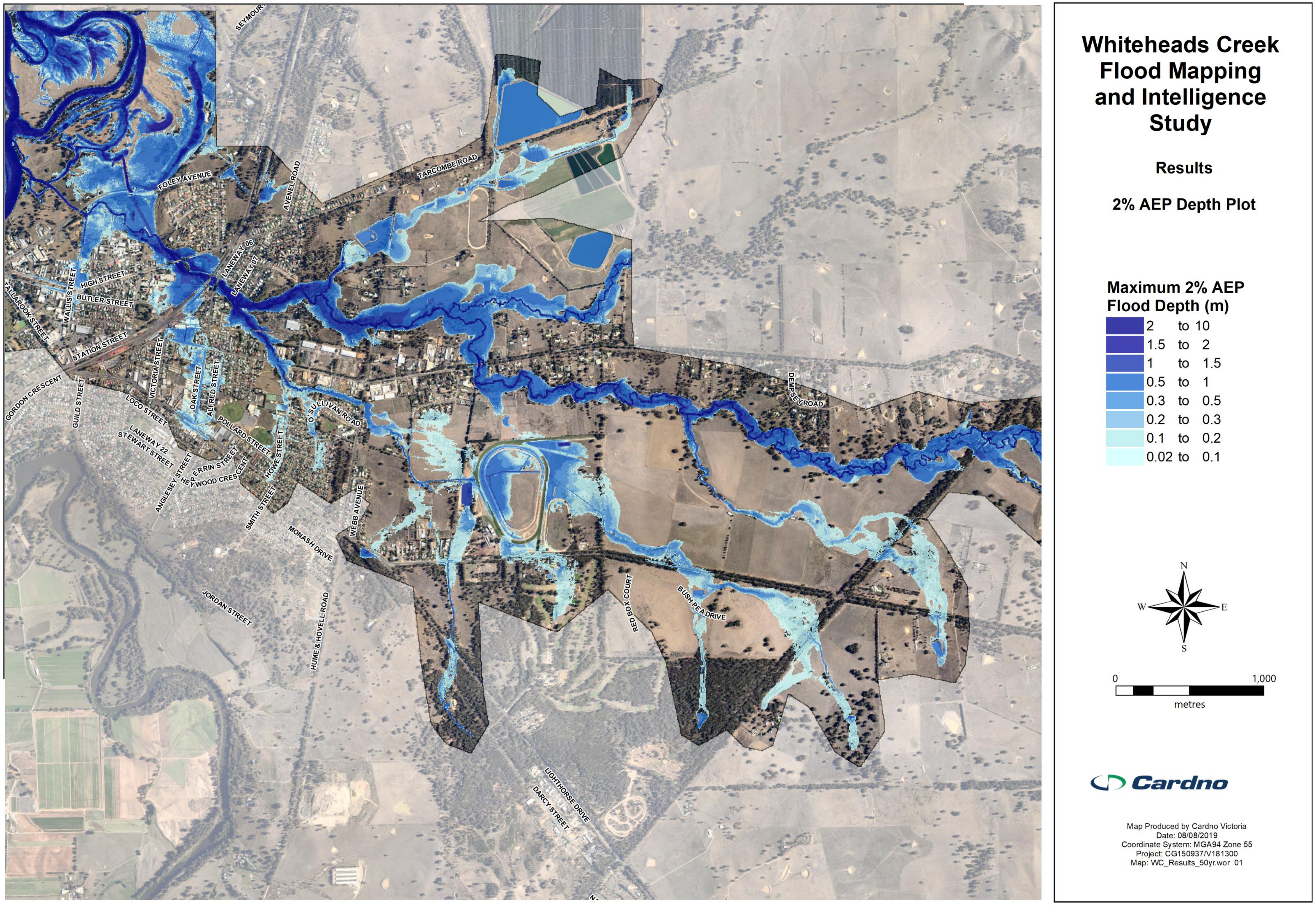


Figure 7-4 Flood depths for the 2% AEP design event

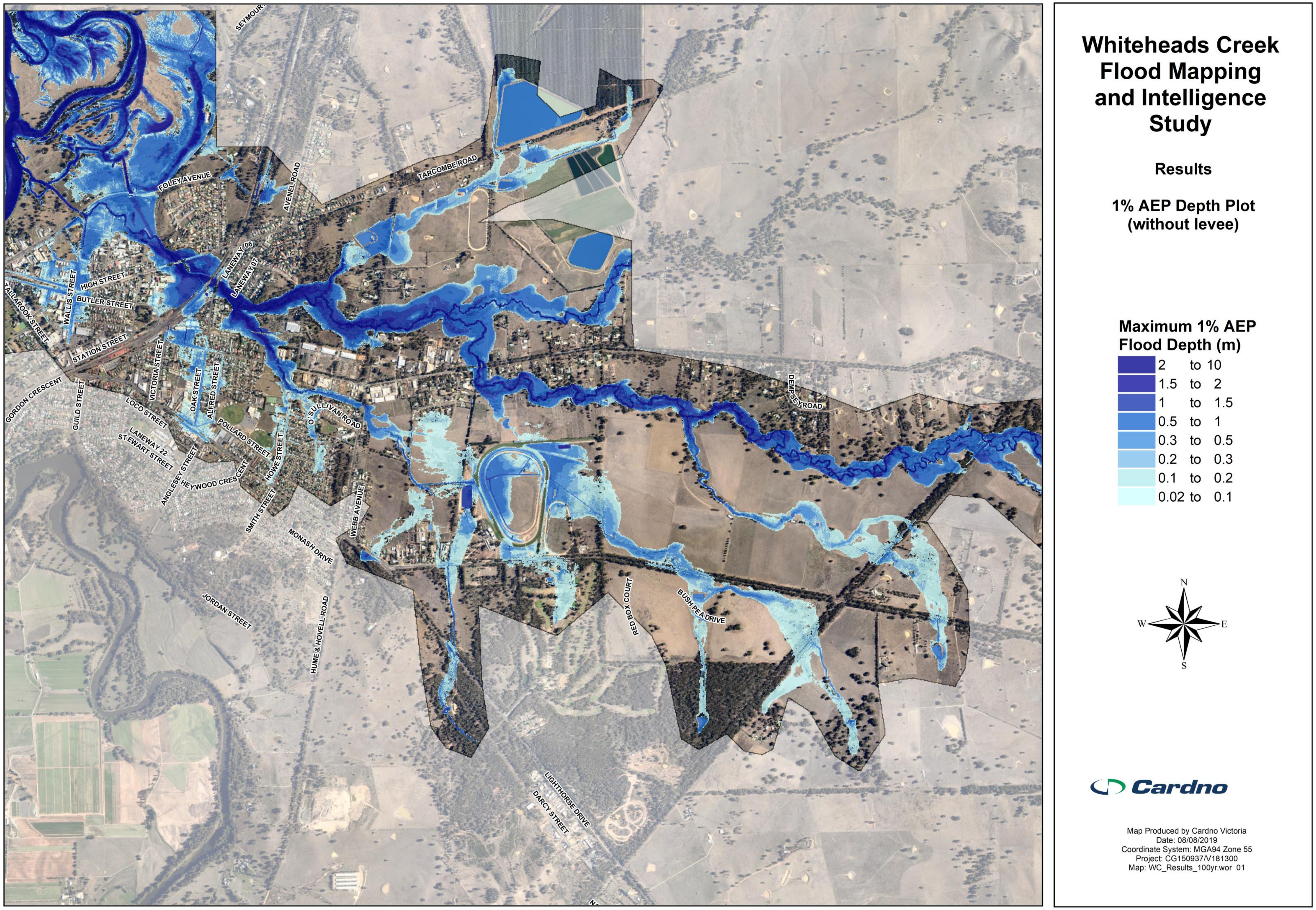


Figure 7-5 Flood depths for the 1% AEP design event

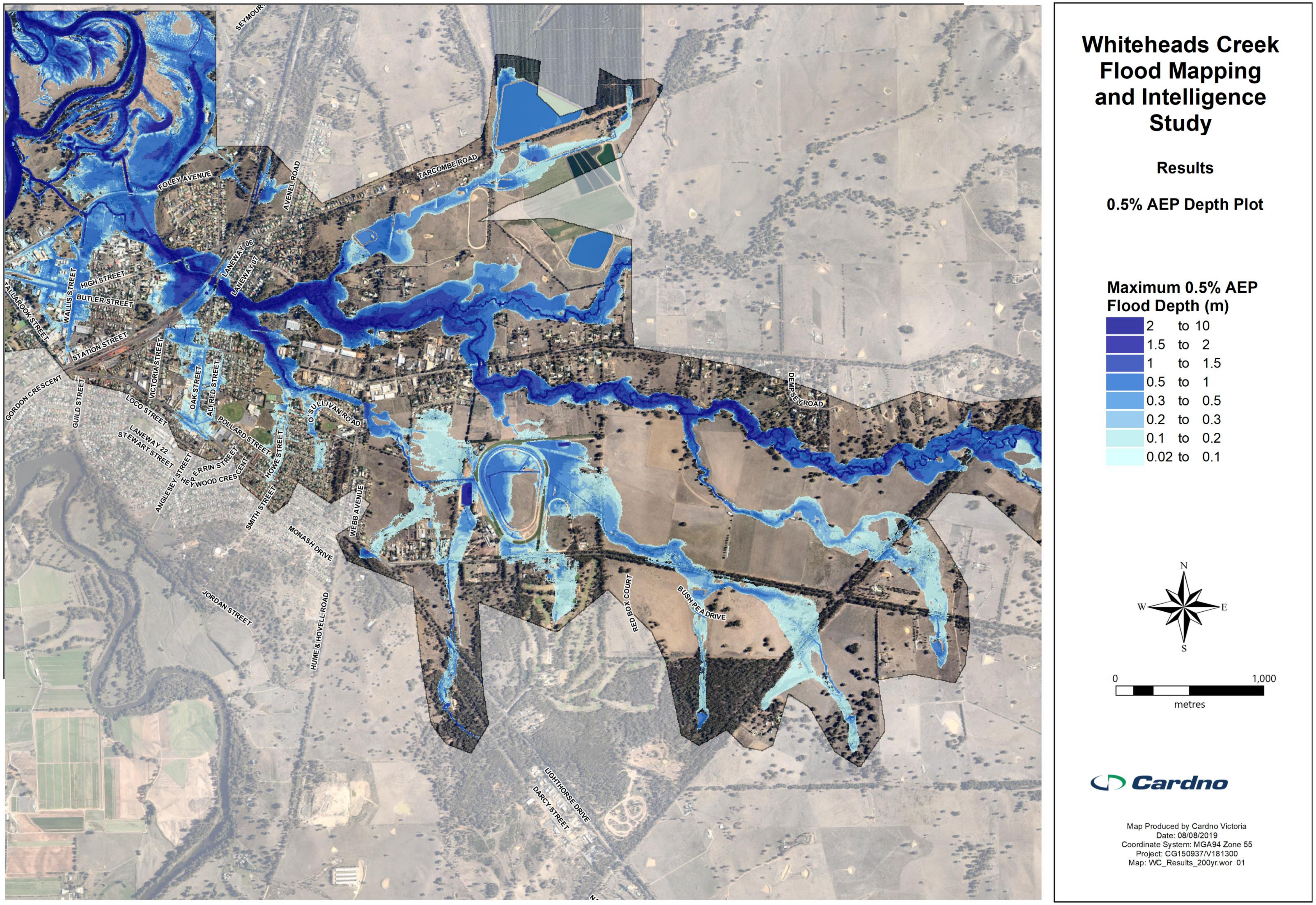


Figure 7-6 Flood depths for the 0.5% AEP design event

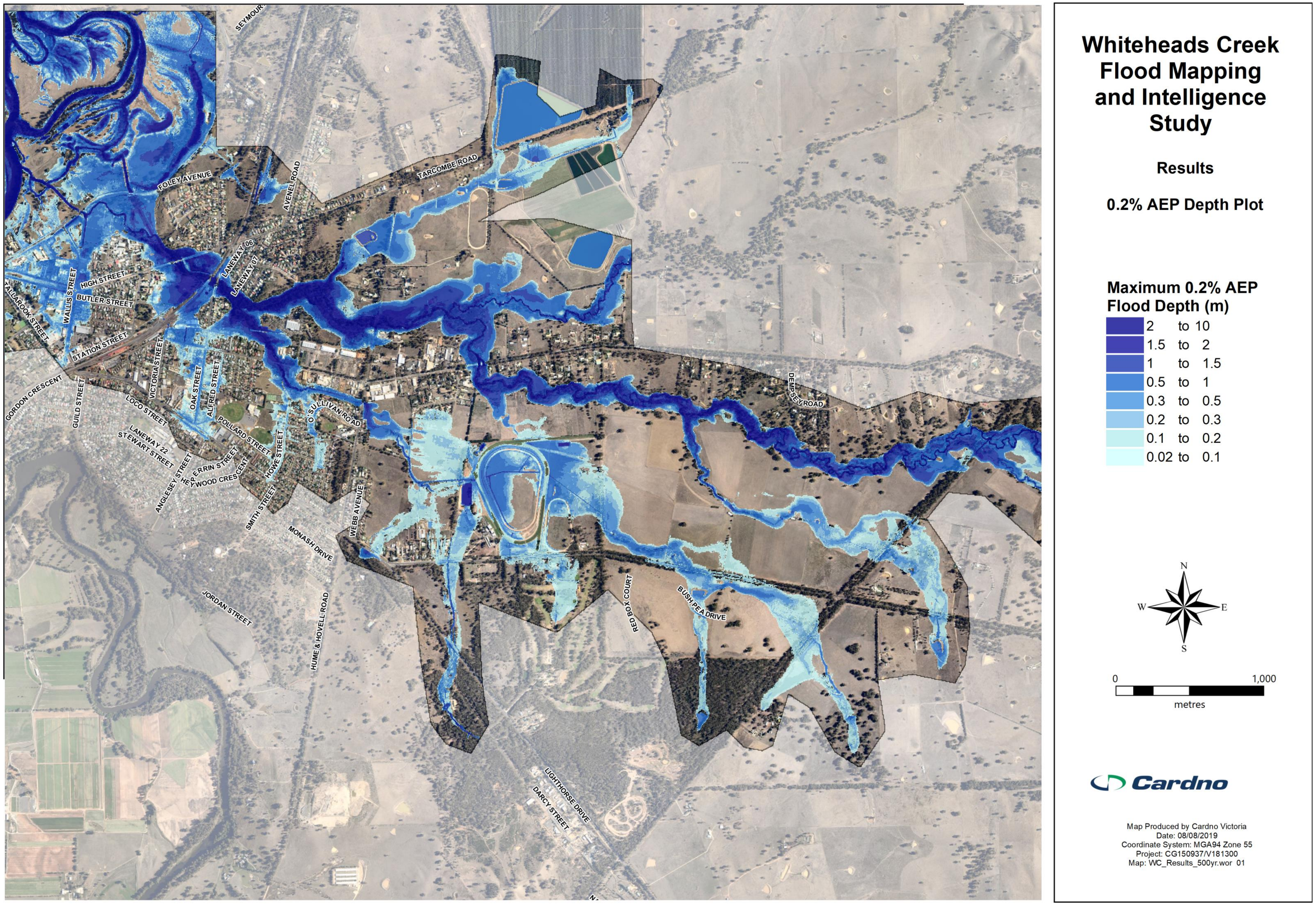


Figure 7-7 Flood depths for the 0.2% AEP design event

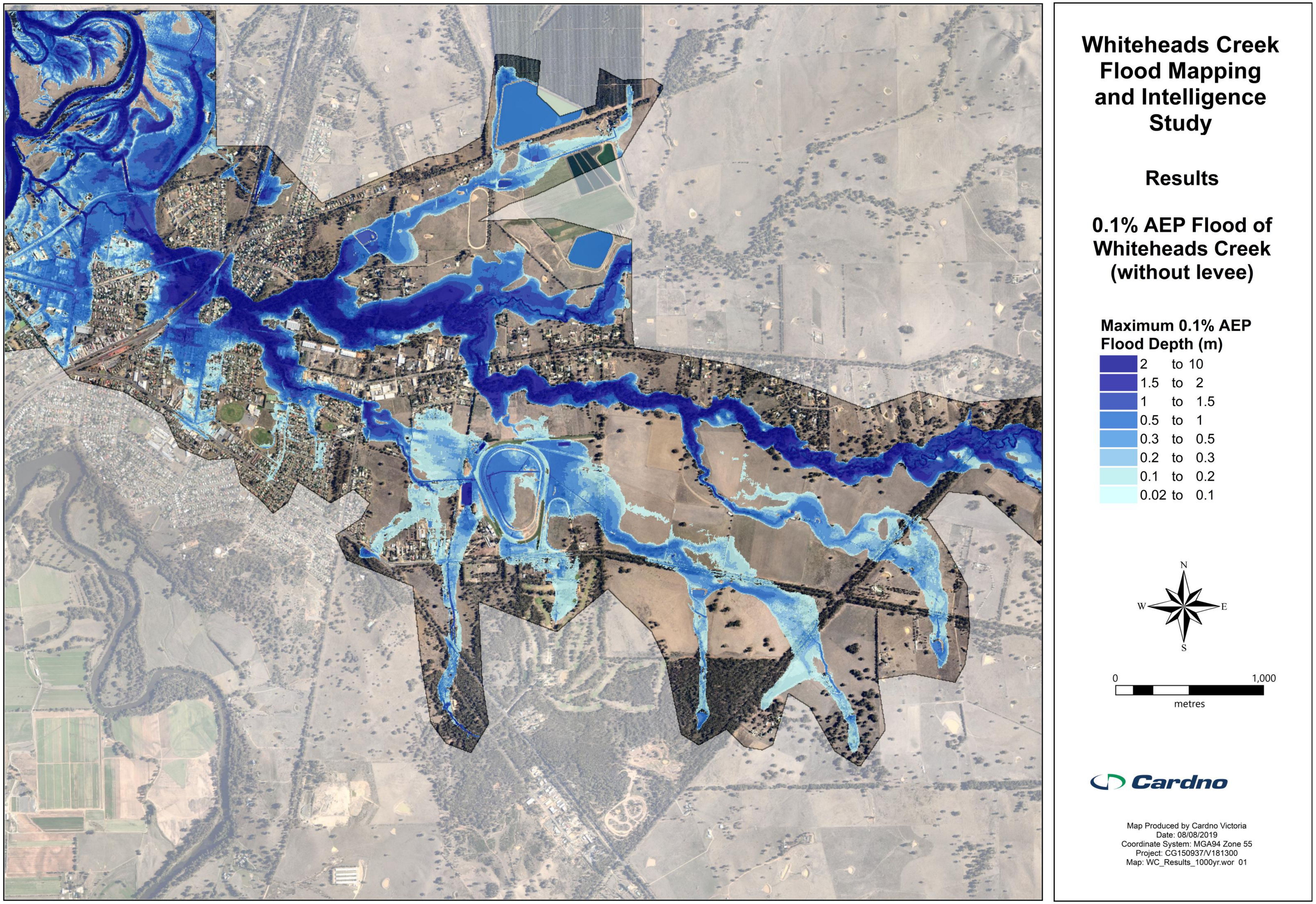
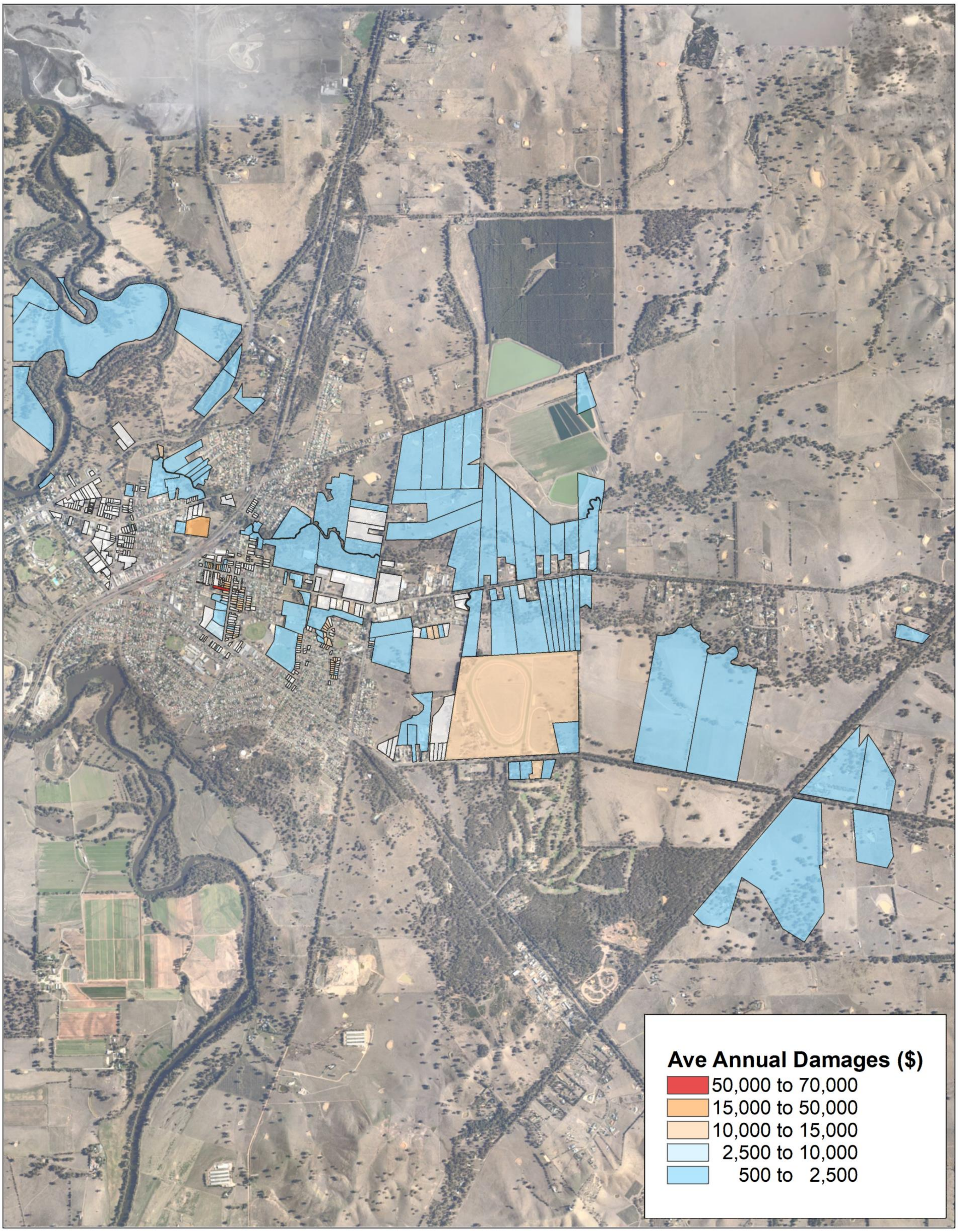


Figure 7-8 Flood depths for the 0.1% AEP design event

APPENDIX

B

DAMAGE ASSESSMENT FIGURES

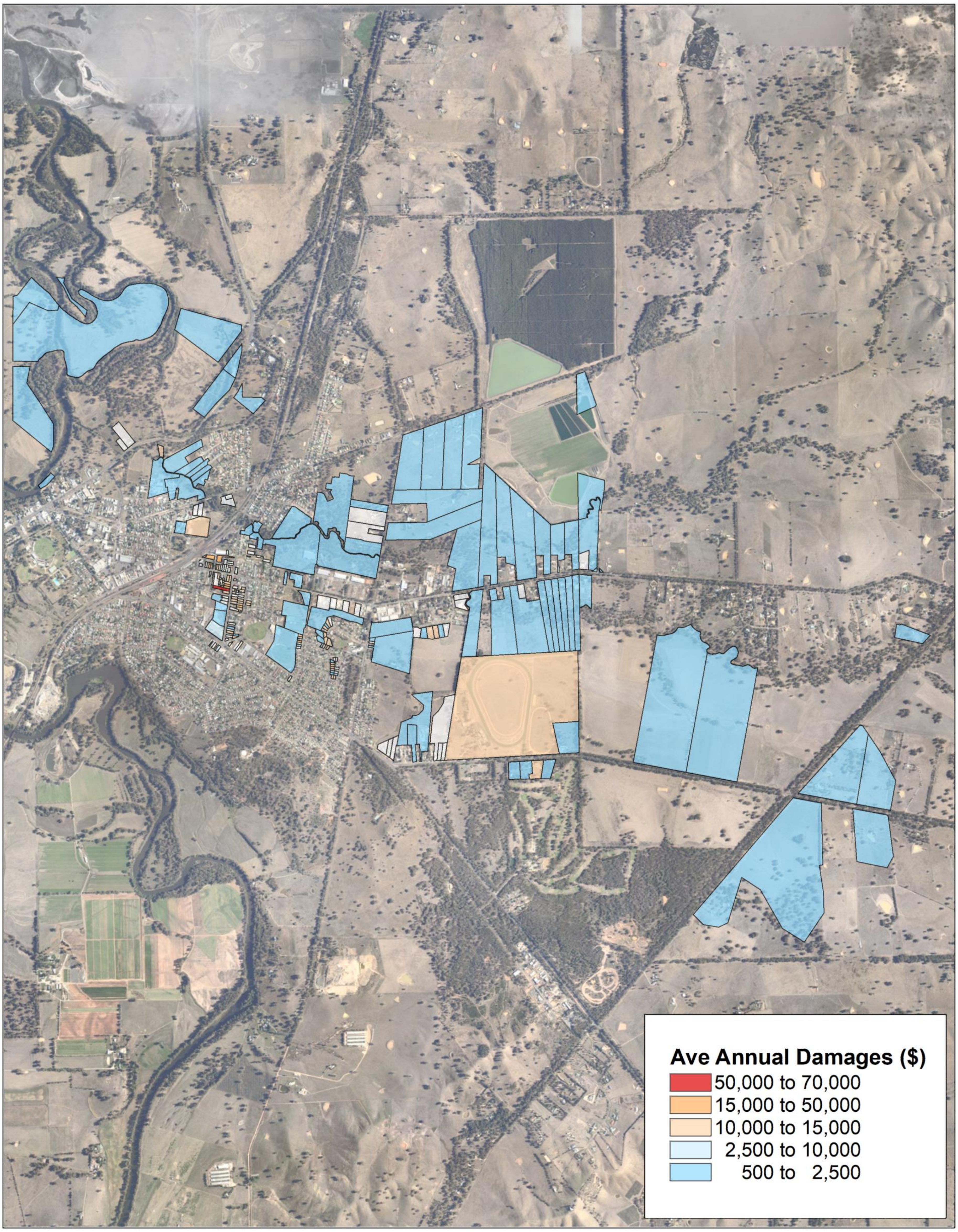


0 1,000
metres

Whiteheads Creek

5yr ARI Existing
Average Annual Damages

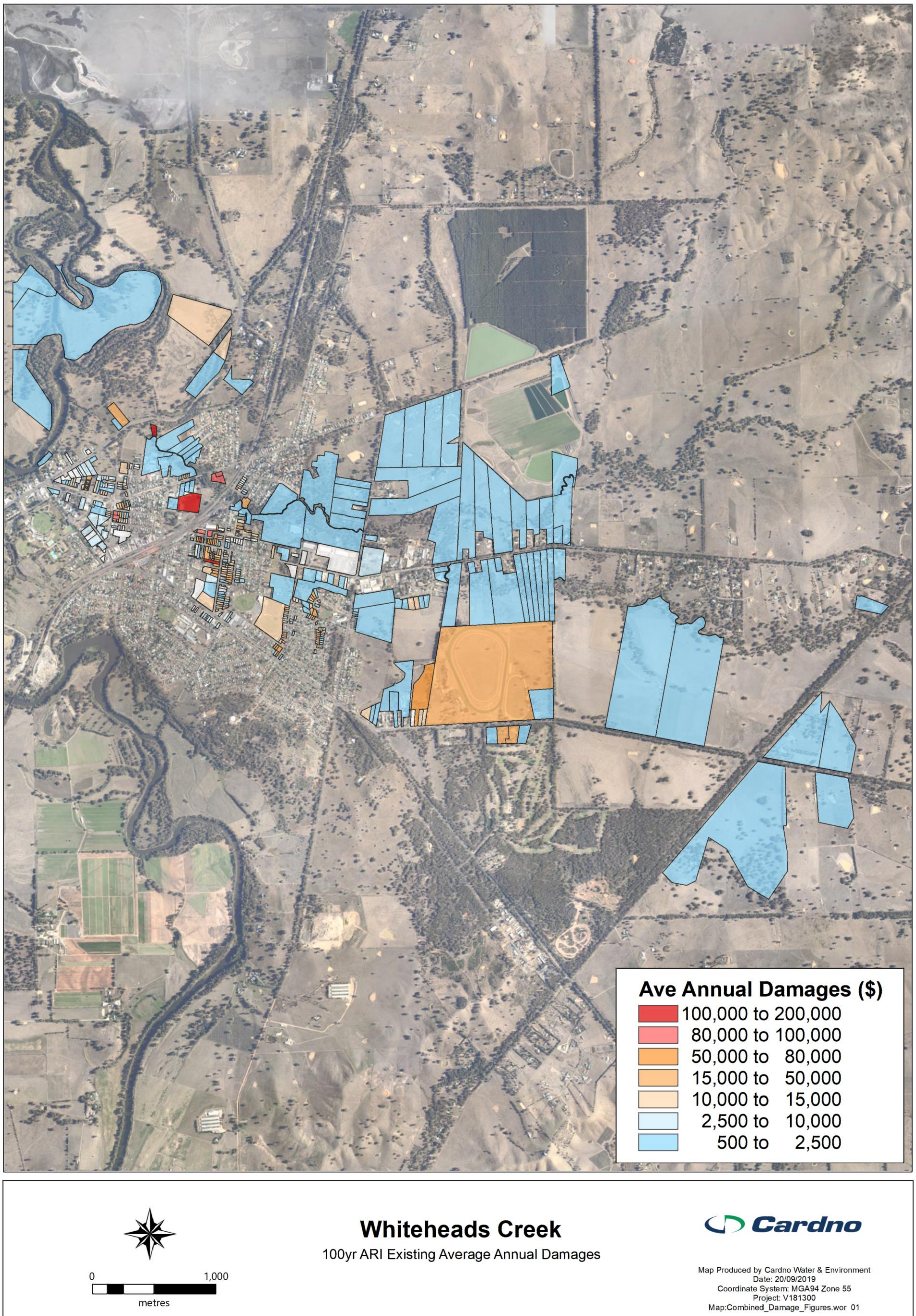
Map Produced by Cardno Water & Environment
Date: 20/09/2019
Coordinate System: MGA94 Zone 55
Project: V181300
Map:Combined_Damage_Figures.wor 01



Whiteheads Creek

5yr ARI Levee Mitigation
Average Annual Damages

Map Produced by Cardno Water & Environment
Date: 20/09/2019
Coordinate System: MGA94 Zone 55
Project: V181300
Map:Combined_Damage_Figures.wor 01

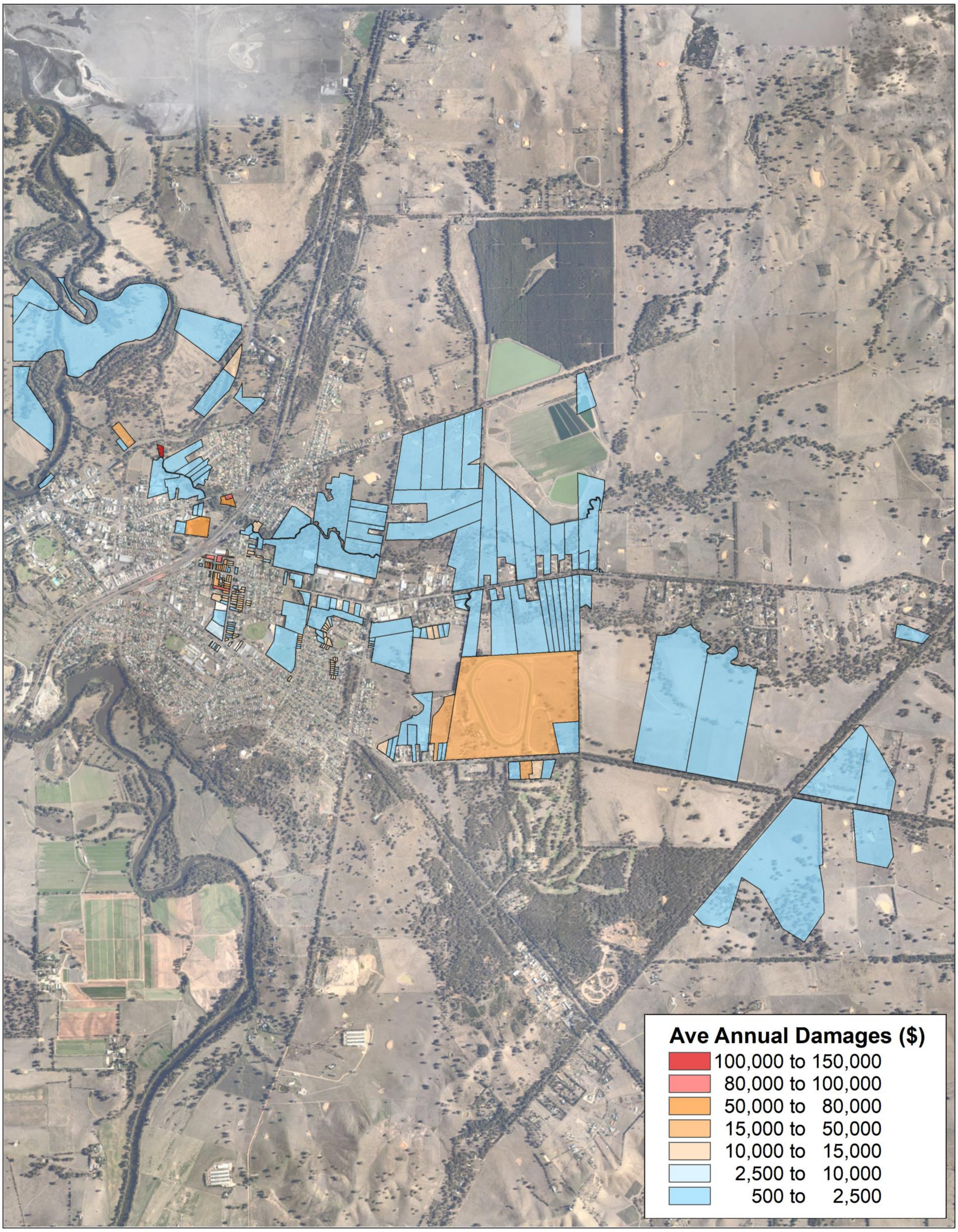


Whiteheads Creek

100yr ARI Existing Average Annual Damages

Cardno

Map Produced by Cardno Water & Environment
Date: 20/09/2019
Coordinate System: MGA94 Zone 55
Project: V181300
Map:Combined_Damage_Figures.wor 01



0 1,000
metres

Whiteheads Creek

100yr ARI Levee Mitigation
Average Annual Damages

Map Produced by Cardno Water & Environment
Date: 20/09/2019
Coordinate System: MGA94 Zone 55
Project: V181300
Map:Combined_Damage_Figures.wor 01