Kyneton Flood Study

Consultation Draft - June 2019

North Central CMA has produced this study in

Regional Floodplain Management Strategy
Everyone has a role to play in preparing for floods

NORTH CENTRAL
Catchment Management Authority
Connecting Rivers, Landscapes, People

Acknowledgement of Country

The North Central Catchment Management Authority acknowledges Aboriginal Traditional Owners within the region, their rich culture and spiritual connection to Country. We also recognise and acknowledge the contribution and interest of Aboriginal people and organisations in land and natural resource management.

North Central Catchment Management Authority acknowledges Macedon Ranges Shire Council and VicRoads for providing information regarding the hydraulic structures and previous flood studies for this report.

Front cover photo: Campaspe River at the Greenway Lane Weir during the September 2016 flood.

A copy of this report is available on www.nccma.vic.gov.au

North Central Catchment Management Authority
PO Box 18
Huntly Vic 3551
T: 03 5440 1800
F: 03 5448 7148
E: info@nccma.vic.gov.au
www.nccma.vic.gov.au

Glossary of Terms

Annual Exceedance Probability (AEP)	The likelihood of occurrence of a flood of a given size or greater occurring in any one year, usually expressed as a percentage. For example, if a peak flood flow of $500m^3/s$ has an AEP of 5%, it means that there is a 5% (one-in-20) chance of a flow of $500m^3/s$ or greater occurring in any given year.
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level.
Australian Rainfall and Runoff (ARR)	ARR is a national guideline for the estimation of design flood characteristics in Australia published by Engineers Australia. ARR aims to provide reliable estimates of flood risk to ensure that development does not occur in high risk areas and that infrastructure is appropriately designed. References in this report refer to the 2016 edition unless stated otherwise.
Average Recurrence Interval (ARI)	A statistical estimate of the average number of years between floods of a given size or larger than a selected event. For example, floods with a flow as great as or greater than the 20-year ARI (5% AEP) flood event will occur, on average, once every 20 years. ARI is another way of expressing the likelihood of occurrence of a flood event. See also Annual Exceedance Probability.
Catchment	The area of land draining to a particular site. It is related to a specific location and includes the catchment of the main waterway as well as any tributary streams.
DEM	Digital Elevation Model – a three-dimensional computer representation of terrain.
Design Flood	A hypothetical flood representing a given probability generally based on some form of statistical analysis. An average recurrence interval (ARI) or exceedance probability (AEP) is attributed to the estimate.
Flood	A natural phenomenon that occurs when water covers land that is normally dry. It may result from coastal or catchment flooding, or a combination of both.
Flood Frequency Analysis (FFA)	A statistical analysis of observed flood magnitudes to determine the probability of a given flood magnitude.
Flood Hazard	Describes the potential of flooding to cause harm or damage. Flood hazard is computed by multiplying flood depth by flood velocity.
Floodplain	An area of land that is subject to inundation by floods up to, and including, the largest probable flood event.
Flow	The volume of water which passes per unit time. Flow or discharge is measured in volume per unit time, for example, megalitres per day (ML/day) or cubic metres per second (m3/sec). Flow is different from the velocity of

	flow, which is a measure of how fast the water is moving, for example, metres per second (m/s).
Hydraulics	The study of water flow in waterways, channels or pipes; in particular, the evaluation of flow parameters such as water level, extent and velocity.
Hydrograph	A graph that shows how the discharge changes with time at a particular location.
Hydrology	The study of the rainfall and runoff process, including the evaluation of peak flows, flow volumes and the derivation of hydrographs for a range of floods.
Intensity Frequency Duration (IFD)	Statistical analysis of rainfall describing the rainfall intensity (mm/hr), frequency (probability measured by the AEP) and duration (hours). This analysis is used to generate design rainfall estimates.
LiDAR	Light Detection and Ranging – Ground survey taken from an aeroplane typically using a laser. LiDAR is used to generate a DEM.
Land Subject to Inundation Overlay	A Planning Scheme overlay to identify flood affected land. The overlay extent is based on the 1% AEP design flood event.
Manning's n	A measure of the hydraulic roughness, or resistance to flow, due to surface conditions, typically averaged over an area of relative homogeneity. For example, there is greater resistance to flow through an area of heavy brush and trees than over maintained grass.
Peak Flow	The maximum flow occurring during a flood event past a given point in the river system.
Pluviograph	A rain gauge measuring the depth of rainfall over a small period of time, typically much less than a day.
Probable Maximum Flood (PMF)	The largest flood that could conceivably occur at a particular location.
Rating Curve	The relationship defining discharge for a given water level at a particular recording location.
RORB	The hydrological modelling program used in this study to calculate the runoff generated from historic and design rainfall events.
Runoff	The amount of rainfall that becomes stream flow; also known as rainfall excess.
TUFLOW	The hydraulic modelling program used in this study to simulate the flow of floodwater through the floodplain. The model uses numerical equations to describe the movement of water.

Executive Summary

The Flood Management Plan for Macedon Ranges Shire, Melbourne Water and North Central CMA (2013) was developed collaboratively by the three named agencies. The plan outlines roles and responsibilities and documents actions to jointly advance the understanding of drainage challenges and improve flood management and coordination. A key issue identified in the plan relates to limited and outdated flood modelling and mapping. In particular, the need for flood modelling of both the Campaspe River and Post Office Creek in Kyneton was identified as a priority. Previously, the flood extents have been estimated from historical and anecdotal evidence. In order to address this issue, one of the specific actions identified in the plan is to undertake flood modelling of Kyneton Township to update the accuracy and availability of flood information.

The purpose of this study was to update flood information available for the township of Kyneton. The information produced by this study may be used to:

- **Assess the flood risk to existing and proposed development.** Kyneton is expanding particularly along the banks of the Campaspe River and hence there is a need to improve the limited flood information currently available for Kyneton in order to facilitate appropriate future development.
- **Define flood related controls in the Macedon Ranges Shire planning scheme.** Although Kyneton currently does have a Land Subject to Inundation Overlay (LSIO) applied along the Campaspe River and a section of Post Office Creek, this study will enable the LSIO mapping through Kyneton to be further refined.
- **Develop flood intelligence products and inform emergency response planning.** The flood data will assist in identifying the flood risk to existing buildings and infrastructure. This data will also facilitate a damage assessment to be undertaken for the township if complemented with a floor level survey of potentially impacted properties.
- **Assist in the preparation of community flood awareness and education products.**
- **Support the assessment of flood risk for insurance purposes.**

It should be noted that the scope of this study excludes the assessment of any mitigation options.

This report details the methodology and assumptions used to develop the design flood information. This included the creation of a hydrologic rainfall-runoff model using RORB which was calibrated to the September 2010, November 2010, and January 2011 historical floods. This model was then used to derive design flood hydrographs for 20%-0.5% annual exceedance probability (AEP) flood events. Hydrographs were also estimated for the probable maximum flood (PMF). The design flows were compared to other peak flow estimation techniques for verification and then used as inputs into a hydraulic model using TUFLOW.

Once calibrated, the TUFLOW model was used to generate flood mapping of the 20%-0.5% AEP design flood events as well as the PMF. The outputs included gridded data of the water surface elevation, depth, velocity and hazard for the range of design events modelled. Flood intelligence was then produced from this mapping by assessing the flood impacts on buildings, properties and roads.

Table of Contents

List of Figures

List of Tables

1 Introduction

This study has been undertaken to update the flood information available for the township of Kyneton. The outputs from this study may be used to:

- Assess the flood risk to existing and proposed development
- Define flood related controls in the Macedon Ranges Shire planning scheme
- Develop flood intelligence products and inform emergency response planning
- Assist in the preparation of community flood awareness and education products
- Support the assessment of flood risk for insurance purposes

The study involved detailed hydrological and hydraulic modelling of the Campaspe River and Post Office Creek through Kyneton. This report details the methodology and assumptions used to develop the design flood information. The study included the creation of a hydrologic rainfall-runoff model using RORB which was calibrated to the September 2010, November 2010, and January 2011 historical floods. This model was then used to derive design flood hydrographs for 20%-0.5% annual exceedance probability (AEP) flood events. The design flows produced from this model were compared to other peak flow estimation techniques for verification, including the probabilistic rational method.

The design hydrographs generated from RORB were then input into a hydraulic model using TUFLOW to generate the required flood data, including flood heights and depths for a range of design events.

1.1 Study Area

Kyneton is a township of 6,951 residents (2016 Census), located approximately 80km north-west of Melbourne, within the municipality of Macedon Ranges Shire. The town is primarily located on the north-eastern bank of the Campaspe River, with new residential development currently expanding on the south-western side of the river. The Campaspe River catchment for Kyneton is approximately 233km^2 and extends to the south of Woodend with headwaters in the Great Dividing Range, as shown in Figure 1-1. It consists predominately of forested land, including Wombat State Forest, and undulating open farmland. There are no significant storages within the Kyneton catchment.

As Kyneton is situated high up in the Campaspe catchment, this reach of the Campaspe River is steep with a well-defined waterway cross-section. Hence, the floodwaters are contained within the Campaspe valley and impacts on the township are relatively minor. In a 1% AEP flood event, floodwaters are generally confined to the waterway except immediately downstream of the township where water breaks out onto the floodplain, impacting the Kyneton Racecourse.

A significant tributary of the Campaspe River, Post Office Creek, is situated at the northern extent of the township and the confluence of the two waterways is located north of the Kyneton Township. Although Post Office Creek has a much smaller catchment (12km^2) than the Campaspe River, it is surrounded by existing residential and industrial development which may be impacted by flooding.

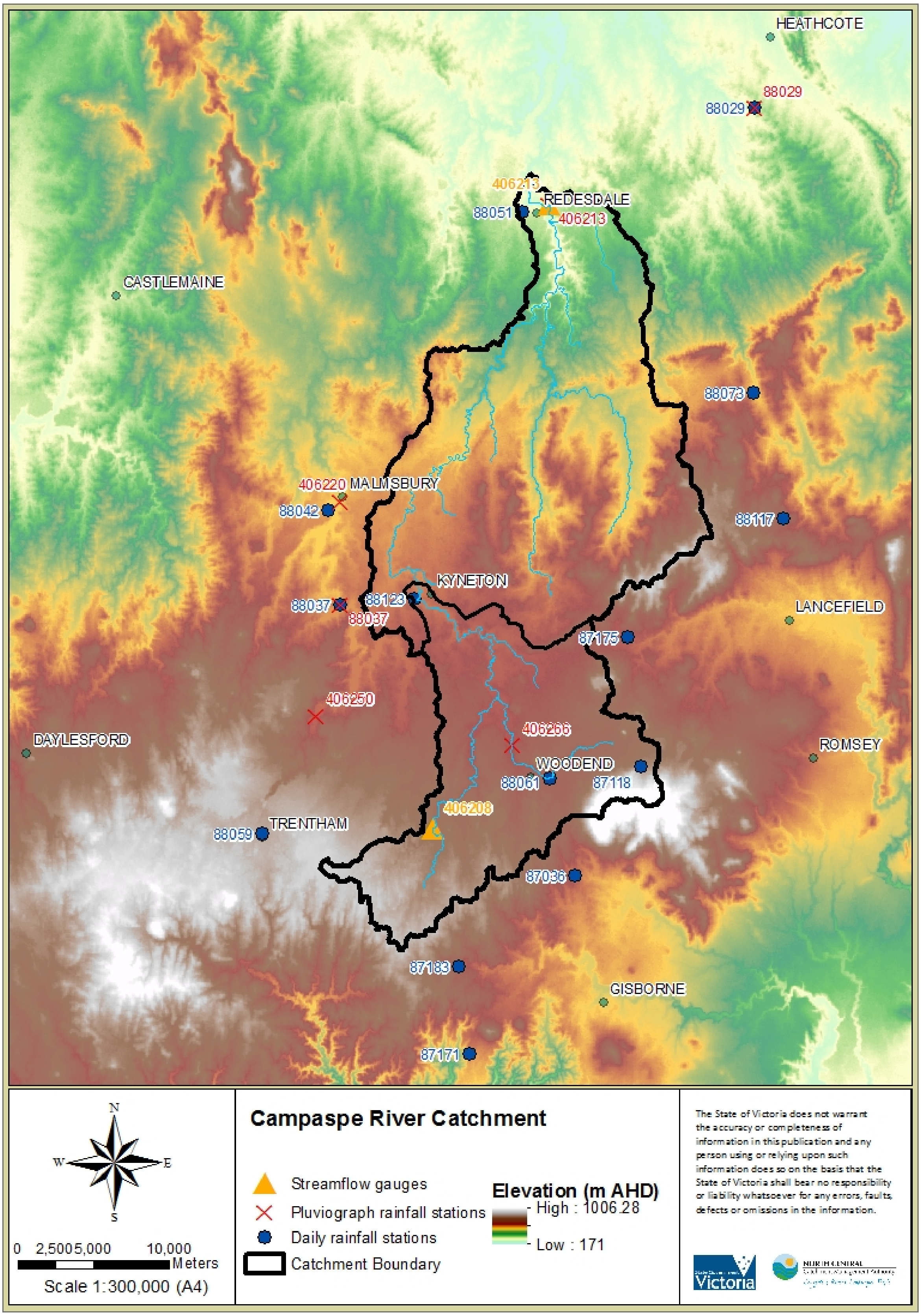


Figure 1-1 Campaspe River catchment at Kyneton and Redesdale (Statewide 25m LIDAR DEM)

1.2 Historical Flood Investigations

The existing Land Subject to Inundation Overlay (LSIO) through Kyneton currently describes the 1% AEP flood extent for area. As shown in Figure 1-2, the LSIO closely follows the Campaspe River and also includes Post Office Creek downstream of Mollison Street.

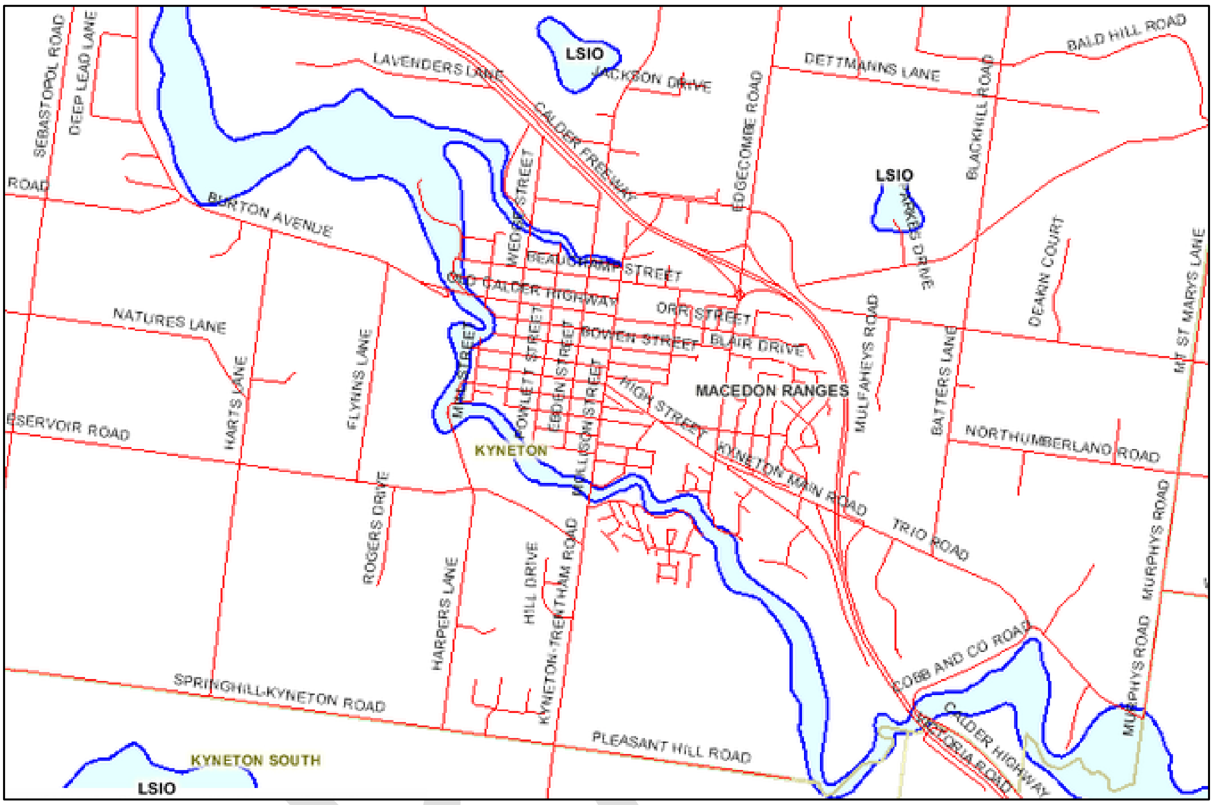


Figure 1-2 Land Subject to Inundation Overlay (DELWP Planning Scheme Online, 2017)

Flood studies previously undertaken for Kyneton include:

- **Calder Highway Carlsruhe to Kyneton – Hydrologic and Hydraulic Investigations (CMPS&F, 1995) –** VicRoads commissioned a hydrological and hydraulic investigation for the Calder Freeway crossing of the Campaspe River between Carlsruhe and Kyneton. This report is available on FloodZoom.
- **River Walk Flood Study (Earth Tech, 2005)** – In April 2005, a flood study was conducted by Earth Tech for a reach of the Campaspe River south of Kyneton Township to determine the 1% Annual Exceedance Probability (AEP). A one-dimensional HECRAS model was utilised for the study. A copy of this report is held by North Central CMA.
- **Kyneton Township Stormwater Drainage Study (Aurecon, 2011)** – Macedon Ranges Shire Council commissioned a stormwater drainage study for the township of Kyneton to identify the existing infrastructure limitations and determine the future requirements. As part of this assessment, estimated 1% AEP design flows were modelled on the Campaspe River and Post Office Creek. The Council is the custodian of this information.

1.3 Historical Flood Records

Table 1-1 displays the ten largest floods that have been recorded at the Campaspe River at Redesdale streamflow gauge which has a continuous instantaneous flow records dating back to 1966. The information provided at this gauge provides an indication of when significant Campaspe River floods occurred in Kyneton.

Table 1-1 Historical flood events measured at the Campaspe River at Redesdale gauge

Rank	Date	Peak Flow Rate (m^3/s)	Peak Level (m)
1	September 1975	422	6.697
2	May 1974	353	Level not available
3	September 2016*	348*	4.540
4	January 2011	322	6.295
5	September 2010	260	5.138
6	June 1968	231	Level not available
7	July 1990	228	4.584
8	November 2010	216	4.388
9	September 1983	189	3.362
10	June 1973	182	Level not available

*Note that there is uncertainty regarding the reliability of the peak flow rate record during the September 2016. Refer to Section 2.2 for further detail.

The Remarkable Flood Rains over South-Eastern Australia report (Bureau of Meteorology, 1909) describes the flood events during the winter of 1909. A description is given of flooding on the Campaspe River through Kyneton. In particular, the report mentions that 'flood marks have been cut on the north abutment of the Mollison Street Bridge'. A site inspection was undertaken however the flood marks referred to in the report could not be located. However, plans of the Mollison Street Bridge dated 10 March 1995 contain the following notation: 'existing northern masonry abutment and wingwalls to be dismantled, re-founded on basalt and reconstructed to the same appearance'. Hence, it is likely that the flood marks referred to in the report have been removed as a result of these works.

1.4 Site Visit

A site visit was undertaken on 13 December 2017 and 2 October 2018 with local community members. A number of locations along the Campaspe River and Post Office Creek were investigated to better understand the flood behaviour. This provided an opportunity to collect data on recent flood events, including extents and relative flood heights. Photos and measurements of key hydraulic structures were also recorded.

Additional photos, videos and anecdotal information were also obtained from the Kyneton Historical Society and several other local community members.

2 Data Review and Assessment

2.1 Topographic and Physical Data

The hydrological and hydraulic models require the input of both topographic and physical data. As described below, this study has utilised Light Detection and Ranging (LiDAR) data and information derived from survey of hydraulic structures.

2.1.1 LiDAR Data

Two sources of LiDAR data were available for this study:

- **Statewide_DEM** – covers Victoria at a grid resolution of 25 metres. Due to the low resolution, this LiDAR data was only used to define the subcatchment areas for the hydrological model where other LiDAR was not available.

- **MD_Rivers_ISC_2010** – produced by the Department of Environment and Primary Industries in 2010 for the Index of Stream Condition (ISC) analysis. The LiDAR has a quoted horizontal accuracy of ±30cm and a vertical accuracy of ±10cm. As this dataset has a grid resolution of 1 metre and covers the Campaspe River and the associated floodplain it was deemed suitable for the hydraulic model. Figure 2-1 shows the extent of the LiDAR and elevations around the Kyneton Township relative to the Australian Height Datum (AHD).

It should be noted that the method used to collect LiDAR data does not penetrate the surface of water and therefore the data generated does not represent the natural surface level of the bed of the waterway. No bathymetric survey has been undertaken for this reach of the Campaspe River and Post Office Creek, and funding was not available for this study to obtain this information. However, the MD_Rivers_ISC_2010 LiDAR data was collected during an extensive period of drought in the north central region. Consequently, the water level was low at the time the data was gathered and therefore it provides a reasonable approximation of the topography of the waterway.

Field surveys from several sources were used to validate the accuracy of the MD_Rivers_ISC_2010 LiDAR. The available survey was from three locations around the township and consisted of spot heights captured along road centre lines. A comparison between the survey information and the LiDAR data was undertaken and the results shown in Figure 2-2 to Difference between survey and LiDAR elevations along Edgecombe Street (difference between levels shown for clarity)Figure 2-4. Overall, the LiDAR was found to correspond well to the survey data, with a mean difference of 80mm and no skew evident. Consequently, only minor modifications to the LiDAR were undertaken in order to more accurately represent the low flow channels in the hydraulic model.

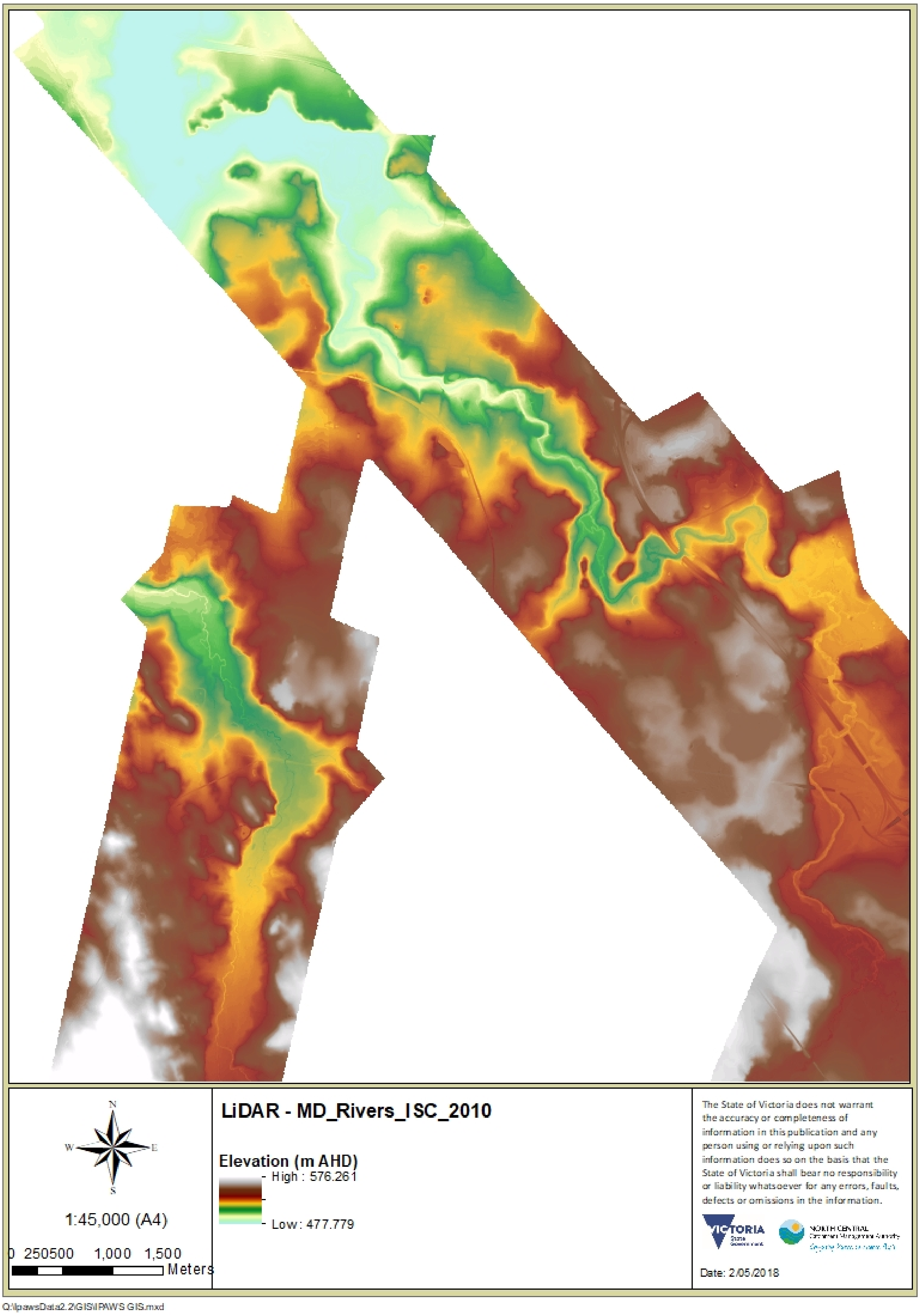


Figure 2-1 1m resolution LiDAR coverage of the study area

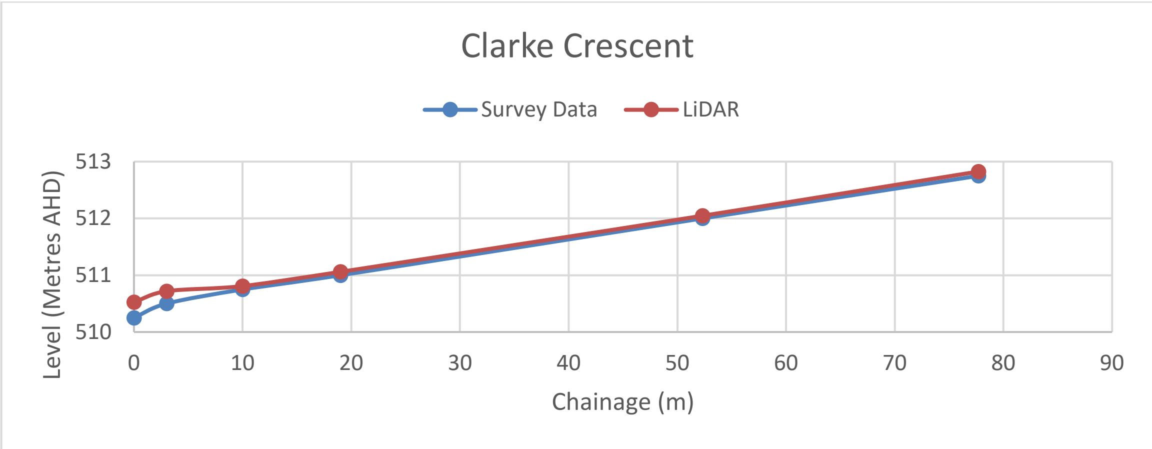


Figure 2-2 Survey and LiDAR elevations along Clarke Crescent

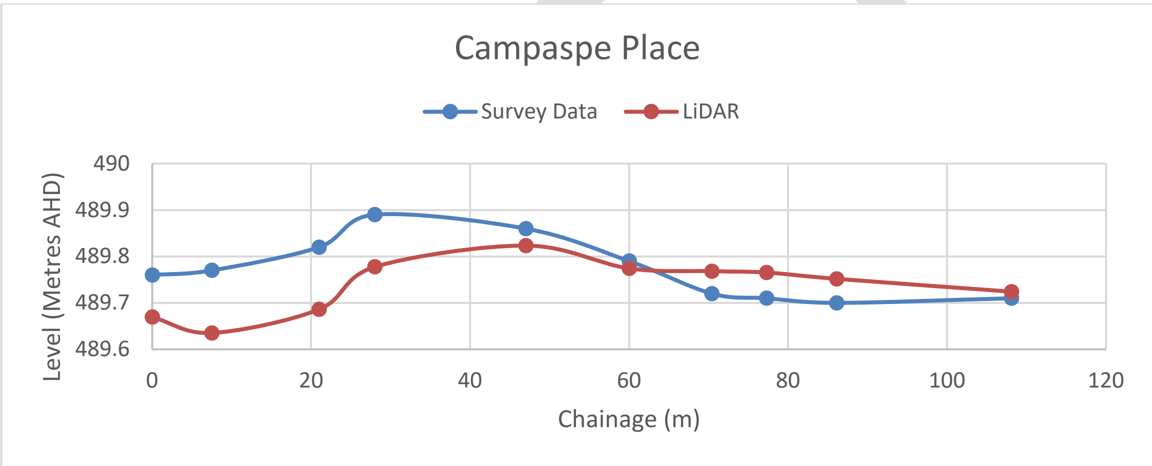


Figure 2-3 Survey and LiDAR elevations along Campaspe Place

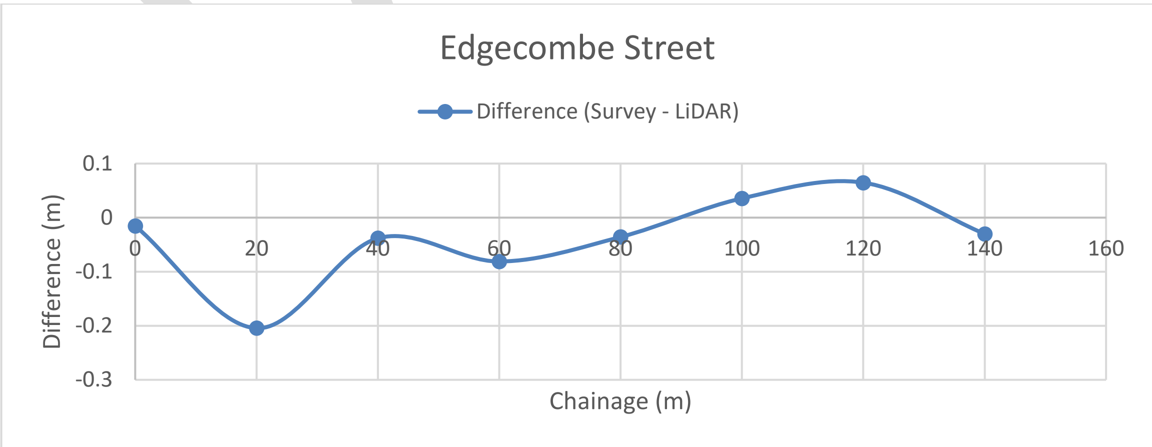


Figure 2-4 Difference between survey and LiDAR elevations along Edgecombe Street (difference between levels shown for clarity)

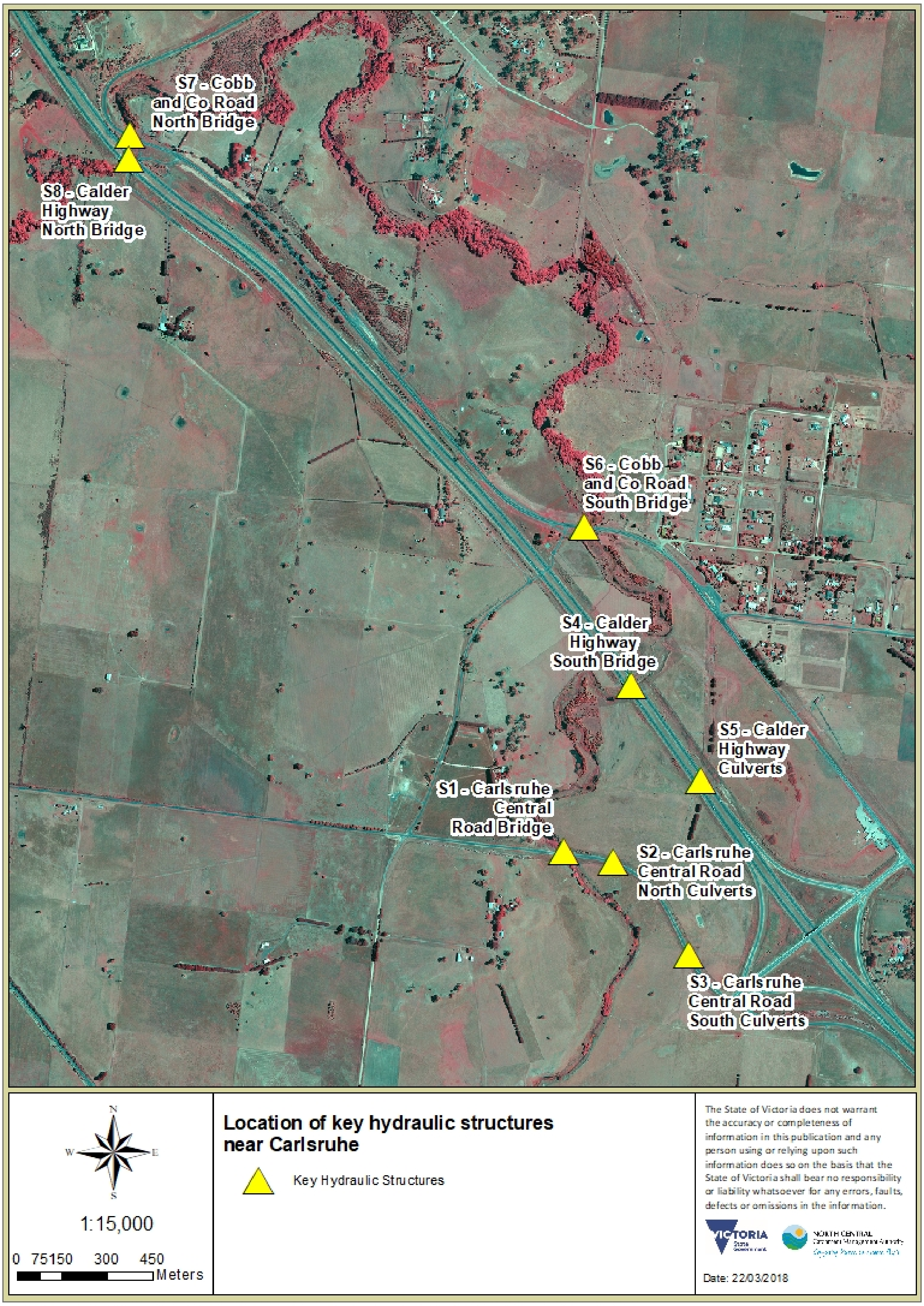
2.1.2 Structure Survey

The hydraulic model requires the input of key hydraulic structures that impact on flood behaviour. There are numerous bridges and weirs on both the Campaspe River and Post Office Creek as detailed in Table 2-1, Figure 2-5 and Figure 2-6. Table 2-2 shows photos of these structures. Plans of the bridges were supplied by VicRoads and the Macedon Ranges Shire Council. However, no details of the weirs were available from the Local Council or the applicable water authorities. The weirs are all of a historical nature and it is likely that they were originally constructed by local landowners. The crest levels of the weirs are picked up reasonably well in the available LiDAR used for the hydraulic model. These levels were also checked during the site visit to ensure they were reasonable. Therefore, no additional structure survey was required for the model construction.

Table 2-1 Details of key hydraulic structures within the study area

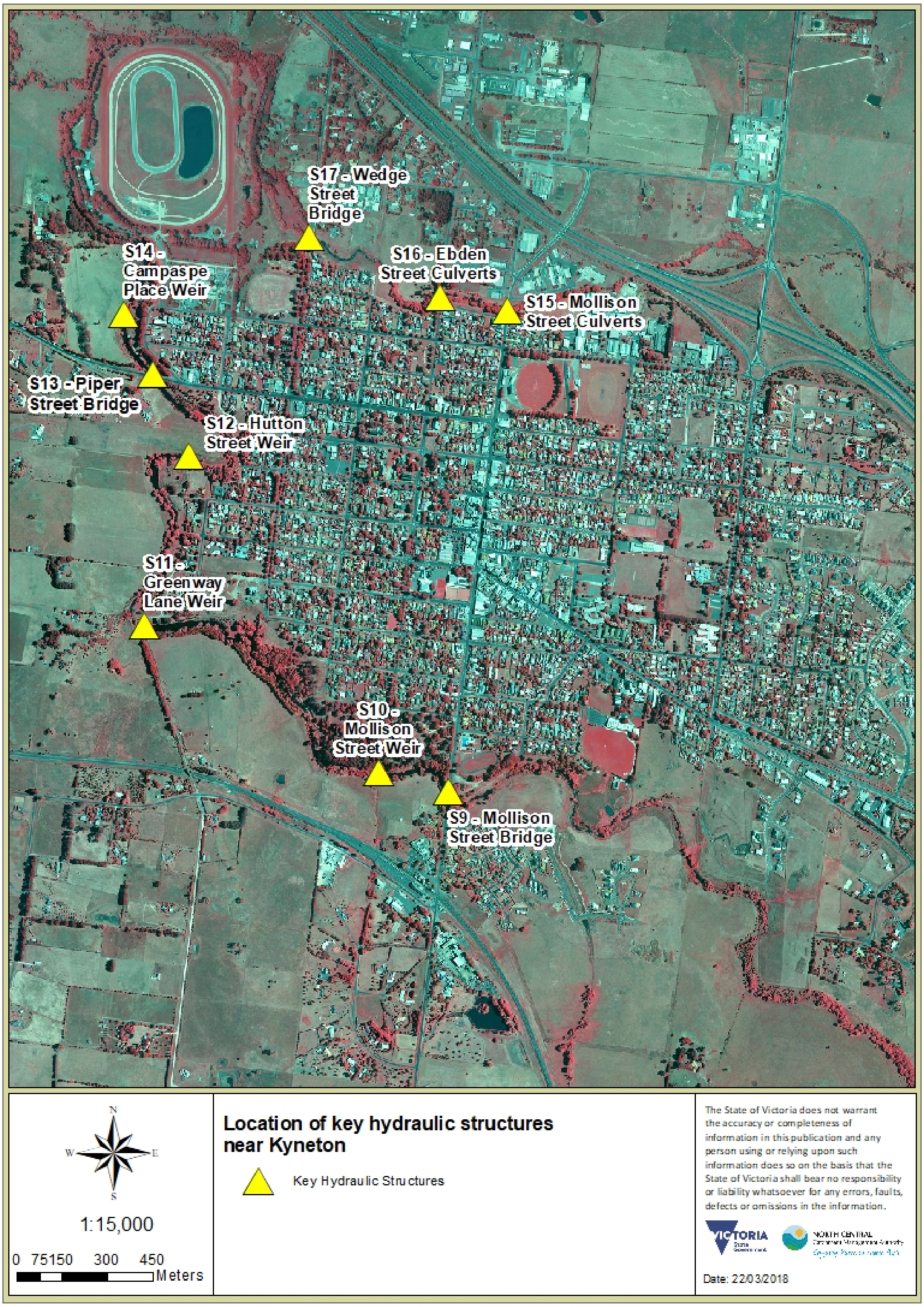
Waterway	Structure Name	Managing Authority	Structure Details	Reference Number
Campaspe River	S1 – Carlsruhe Central Road Bridge	Local Council	5-span bridge Width = 75.7m	Asset ID 220
	S2 - Carlsruhe Central Road North Culverts	Local Council	15 3.1x1.5m box culverts	Asset ID 219
	S3 - Carlsruhe Central Road South Culverts	Local Council	15 3.1x1.35m box culverts	Asset ID 218
	S4 - Calder Highway South Bridges	VicRoads	Two parallel bridges 4-span bridge Width = 69m	SN9680 & SN9681
	S5 - Calder Highway Culverts	VicRoads	Two parallel sets of culverts 11 3.1x1.8m box culverts	SN9678 & SN9679
	S6 - Cobb and Co Road South Bridge	Local Council	6-span bridge Width = 55m	Asset ID 221
	S7 - Cobb and Co Road North Bridge	Local Council	6-span bridge Width = 66.5m	Asset ID 223
	S8 - Calder Highway North Bridges	VicRoads	Two parallel bridges 2-span bridge Width = 56m	SN9682 & SN9683
	S9 – Mollison Street Bridge	VicRoads	3-spans bridge Width = 52.6m	SN4415
	S10 - Mollison Street Weir	N/A		
	S11 - Greenway Lane Weir	N/A		

	S12 - Hutton Street Weir	N/A		
	S13 - Piper Street Bridge	VicRoads	4-span bridge Width = 53.6m	SN0164
	S14 - Campaspe Place Weir	N/A		
Post Office Creek	S15 - Mollison Street Culverts	VicRoads	Four 1.8m diameter culverts	SN1255
	S16 - Ebden Street Culverts	Local Council	Four 1.8m diameter culverts	Asset ID 32
	S17 - Wedge Street Bridge	Local Council	Single span bridge Width = 5.1m	Asset ID 119



T:\GISData\FLOODPLAIN\MACEDON RANGES\KYNETON\KYNETON FLOOD STUDY 2014\CMA DATA\Kyneton_Study_(20170821_MGA55).mxd

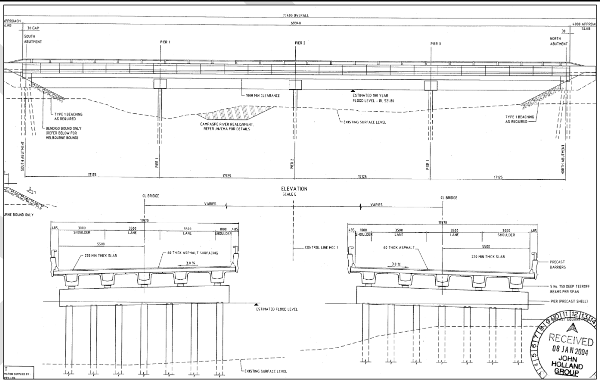
Figure 2-5 Location of key hydraulic structures near Carlsruhe



T:\GISData\FLOODPLAIN\MACEDON RANGES\KYNETON\KYNETON FLOOD STUDY 2014\CMA DATA\Kyneton_Study_(20170821_MGA55).mxd

Figure 2-6 Location of key hydraulic structures near Kyneton

Table 2-2 Images of key hydraulic structures within the study area

 **S1 – Carlsruhe Central Road Bridge**	 **S2 - Carlsruhe Central Road North Culverts**
 **S3 - Carlsruhe Central Road South Culverts**	 **S4 - Calder Highway South Bridges**
 **S5 - Calder Highway Culverts**	 **S6 - Cobb and Co Road South Bridge**

 **S7 - Cobb and Co Road North Bridge**	 **S8 - Calder Highway North Bridges**
 **S9 – Mollison Street Bridge**	 **S10 - Mollison Street Weir**
 **S11 - Greenway Lane Weir**	 **S12 - Hutton Street Weir**

S13 - Piper Street Bridge

S14 - Campaspe Place Weir

S15 - Mollison Street Culverts

S16 - Ebden Street Culverts

S17 - Wedge Street Bridge

2.1.3 Kyneton Drainage Network

The underground drainage network was not included in this hydraulic model. The purpose of this study is to investigate how large flood events are conveyed through Kyneton Township by the Campaspe River and Post Office Creek. Hence, this study does not consider the stormwater system which would have a negligible impact on the riverine flood behaviour.

2.2 Streamflow Data

Streamflow data was required for the calibration of the hydrological model. The two active streamflow gauges in the catchment are the Campaspe River at Ashbourne gauge and the Campaspe River at Redesdale gauge (see Table 2-3). However, as the Campaspe River at Ashbourne gauge is located at the top of the catchment it was not suitable for the hydrological model calibration. Instantaneous streamflow data was sourced from the Department of Environment, Land, Water and Planning (DELWP) Water Measurement Information System and the *Victorian Surface Water Information to 1987 – Volume 4* (Rural Water Commission of Victoria, 1990).

Table 2-3 Streamflow gauge details

Station Name	Station No.	Status	Data Type	Period of record
Campaspe River @ Redesdale	406213	Active	Instantaneous Flow, Mean Daily Flow, Water Level	November 1953 – Present
Campaspe River @ Ashbourne	406208	Active	Instantaneous Flow, Water Level	April 1933 – Present

A review of the Campaspe River at Redesdale streamflow data quality revealed a discrepancy in the flow records for the September 2016 flood event. In addition to the quality of this peak flow data being described as a rating extrapolation, there are also several reasons to doubt the validity of this measurement:

- **Comparison of rainfall AEP to flood AEP**
  Rain gauge 87175 was used as an indicative measure of the rainfall over the Redesdale catchment. During the September 2016 event 125mm rain fell over 7 days. Based on the IFD for the Redesdale catchment the 7 day rainfall had the rarest intensity equivalent to between a 20% and 10% AEP rainfall event. However, the peak flow at the Campaspe River at Redesdale gauge was estimated to be around a 3% AEP event (Table 1-1), indicating that there may be some discrepancy in the peak flow measurement.

- **Model Calibration**
  A hydrologic model analysis of the September 2016 rainfall using RORB required unrealistic values for the losses in order to reproduce the recorded hydrograph. A very low value of the k_c parameter was also required which was not consistent with the results determined for the

three historical events selected for calibration, namely September 2010, November 2010 and January 2011 (Section 3.4).

- **Comparison to historical events**
  It is evident when comparing the peak gauge levels and corresponding flows from previous historical events that the September 2016 event does not correlate. For example, the total rainfall during January 2011 was much higher than the September 2016 event and occurred within a shorter time. Moreover, the peak gauge height was almost two metres higher during the January 2011 flood compared to the September 2016 flood. However, the flow during the September 2016 event is stated as being higher than the January 2011 event (Table 1-1). Furthermore, the September 2010 peak gauge level was 600mm higher than the September 2016 event yet, contradictorily, the 2016 peak flow rate was recorded as almost 90m^3/s greater.

- **Anecdotal Evidence**
  Anecdotal evidence from the local community also confirmed that flooding in Kyneton was relatively minor during September 2016 and that the January 2011 flood was significantly larger. Photos taken during both events also establish this.

The anomaly with the September 2016 event appears to be due to the updated ratings table for the Campaspe River at Redesdale gauge. The significant change in the ratings table is presumably based on three gaugings that were taken during this flood event on the 14th and 15th of September 2016. These gaugings were obtained from the Bureau of Meteorology website and are listed in Table 2-4 below.

Table 2-4 Gaugings taken at the Campaspe River at Redesdale gauge during the September 2016 flood event

Time	Watercourse Level (m)	Watercourse Discharge (m^3/s)
15/09/2016 12:10	2.934	114.568
14/09/2016 13:22	3.819	234.016
14/09/2016 12:19	4.051	157.158

When compared to the original ratings curve (blue curve in Figure 2-7), which has been in operation since 2005, the flows measured at gauge levels 2.934m and 4.051m coincide reasonably well. However, the flow measured at gauge level 3.819m is substantially greater than the flow indicated on the original ratings curve. Furthermore, it can be seen that this gauging is inconsistent with the gauging taken at a gauge level of 4.051m. Although the former gauging is 0.2 metres lower, the corresponding measured flow was almost 80m³/s greater. As a result, this single gauging appears to have significantly influenced the current ratings curve at the upper end, which is displayed as the red curve in Figure 2-7. However, the lower end of the ratings curve, below a level of approximately 3m, remains essentially unchanged.

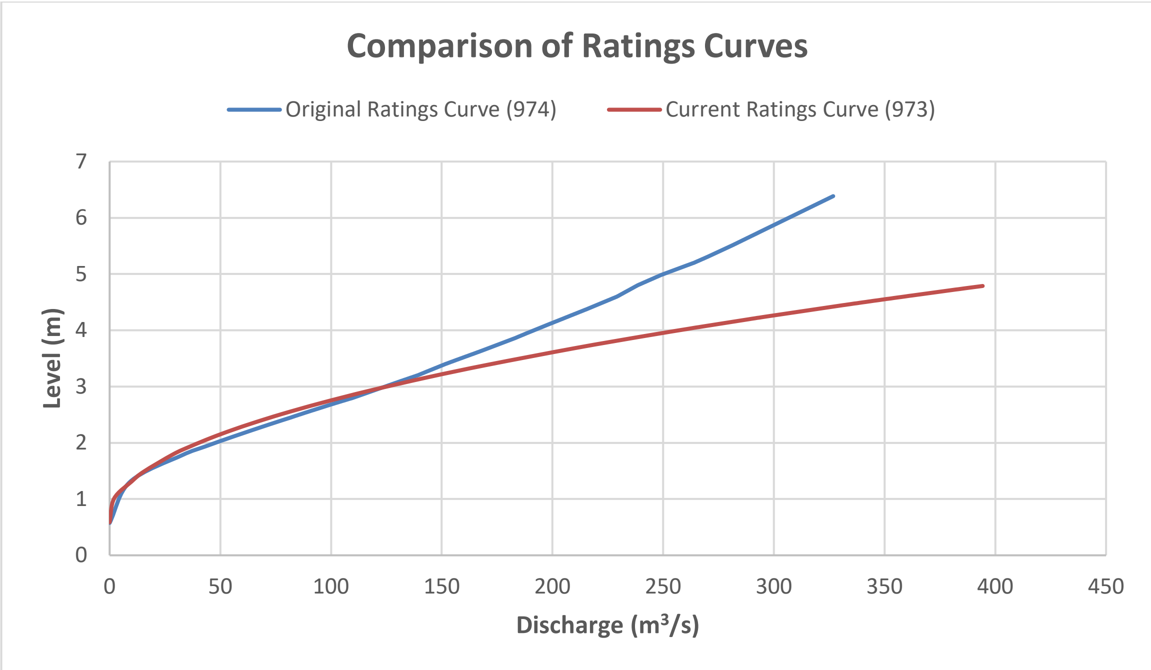


Figure 2-7 Comparison of current and original ratings curves for Campaspe River at Redesdale gauge

If the original ratings curve is applied, the September 2016 peak gauge height would correspond to a peak flow of approximately 226m³/s, rather than 348m³/s. This peak flow correlates more reasonably with the other historical flows when considering the depth and duration of rainfall and the peak gauge height recorded during each event. Furthermore, it also allows good calibration to be achieved using similar hydrologic parameters to those derived from the other historical events modelled. Hence, due to the inconsistency and the low data reliability, the September 2016 event was not included in the flood frequency analysis (Section 3.2) nor selected to calibrate the hydrologic model (Section 3.4).

2.3 Rainfall Data

Calibration of the hydrologic model requires both pluviograph and daily rainfall data which was sourced from the Bureau of Meteorology and DELWP Water Measurement Information System. Pluviograph rainfall data is used to understand the temporal distribution of a historical rainfall event while daily rainfall data provides the spatial variation in rainfall depths. This data is essential to calibrate the hydrological model.

Figure 2-8 shows the locations of daily rainfall and pluviograph stations in the region. As detailed in Table 2-5, there are seven pluviograph stations within, or in the vicinity of, the catchment. Daily rainfall records were obtained from 14 applicable daily rainfall stations which are shown in Table 2-6.

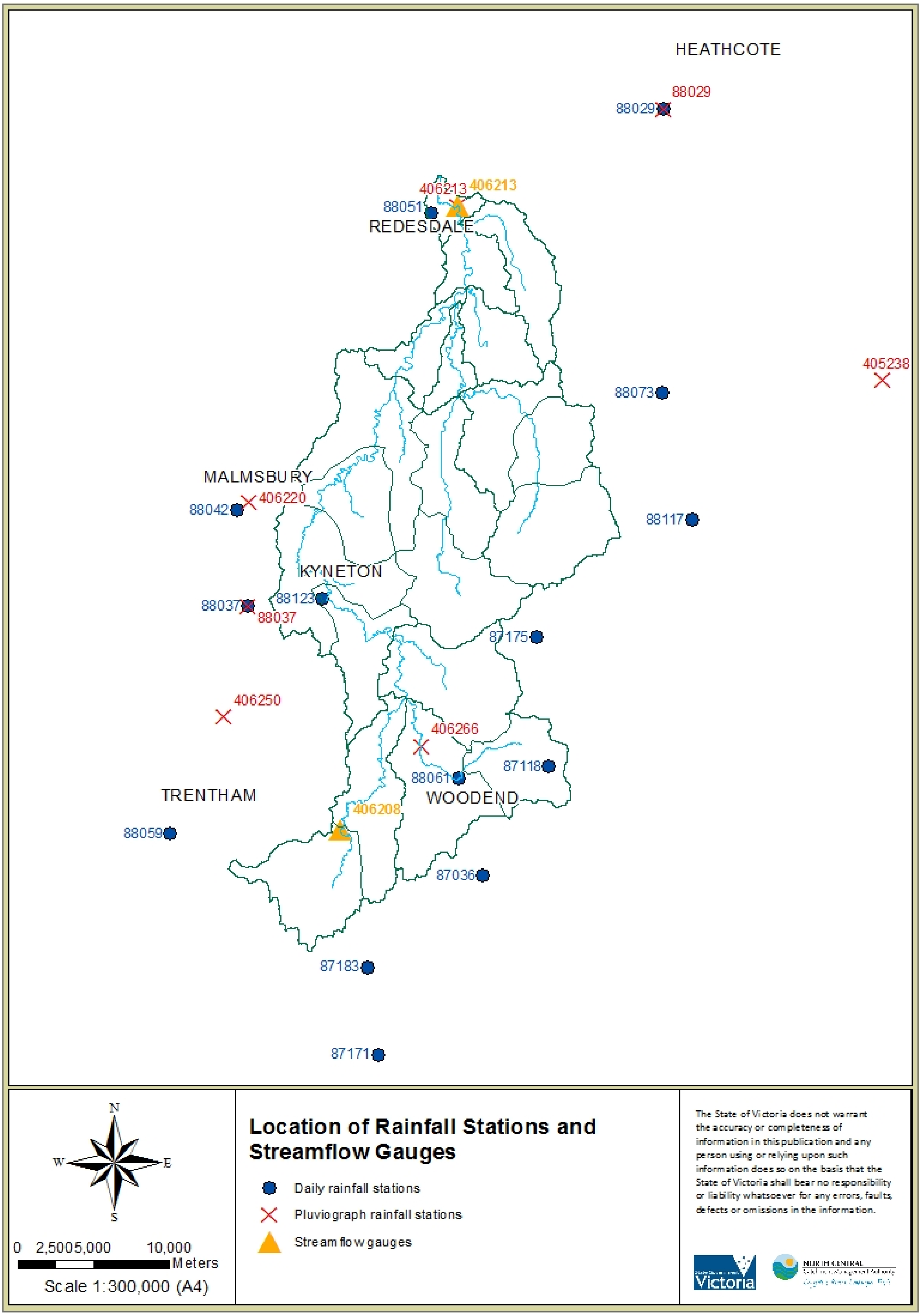


Figure 2-8 **Locations of rainfall stations and streamflow gauges**

Table 2-5 Pluviograph station details

Station Name	Station Number	Period of Record
Heathcote	88029	Jan 1882 – Present
Lauriston Reservoir	88037	Jul 1948 - Present
Mollison Creek @ Pyalong	405238	May 1966 – Present
Campaspe River @ Redesdale	406213	Feb 1989 – Present
Coliban River @ Malmsbury Reservoir (Head Gauge)	406220	April 2014 – Present
Coliban River @ Springhill-Tylden Road	406250	Oct 2010 – Present
Five Mile Creek @ Woodend Treatment Plant	406266	Oct 1998 – Present

Table 2-6 Daily rainfall station details

Station Name	Station Number	Period of Record
Baynton	88073	Mar 1953 – Present
Benloch	88117	Jan 1969 – Jun 2015
Bullengarook (North West)	87183	Oct 2010 – Jan 2012
Heathcote	88029	Jan 1882 – Present
Hesket (Straws Lane)	87118	Dec 1968 – Present
Kyneton	88123	Aug 1969 – Present
Lauriston Reservoir	88037	Jul 1948 – Present
Macedon Forestry	87036	Dec 1873 - Present
Malmsbury Reservoir	88042	Aug 1872 – Present
Newham (Cobaw)	87175	Jan 1995 – Present
Redesdale	88051	Jan 2003 - Present
Trentham (Post Office)	88059	Jan 1878 – Present
Woodend (Carlisle Street)	88061	Aug 1889 – Present
Bullengarook South	87171	Mar 1992 - Present

2.4 Storage Data

There are no significant water storages located within the study area.

3 Hydrologic Analysis

3.1 Overview

A hydrologic model of the catchment was developed using the rainfall-runoff program RORB (version 6.32). The hydrologic model was calibrated to the Campaspe River at Redesdale gauge using three historical flood events. Design event hydrographs were then derived from RORB to be input as boundary conditions in the TUFLOW hydraulic model.

RORB is a non-linear runoff and streamflow routing program used to calculate flood hydrographs from rainfall and other channel inputs. The catchment is delineated into subareas which are connected by reach storages. Specific losses are subtracted from the rainfall on each subarea to produce rainfall-excess. The rainfall-excess is then routed through the catchment storage to generate hydrographs at any location.

The following methodology was applied for the hydrologic modelling of the Campaspe River catchment up to the Campaspe River at Redesdale gauge:

- A flood frequency analysis was undertaken for the Campaspe River at Redesdale gauge to produce a flood frequency curve;
- A RORB model was prepared. Catchment subareas were initially obtained from a previous RORB model which was created for the Rochester Flood Management Plan (2013). However, the extent of the RORB model was reduced for the Kyneton Flood Study, terminating at the Campaspe River at Rochester gauge site;
- The catchment subareas were further delineated based on the topography to provide an adequate number of subareas upstream of Kyneton Township. Additional hydrograph print locations were also added to the existing RORB model to obtain the required inputs for the hydraulic model;
- The inputs adopted from the Rochester Flood Management Plan (2013)were inspected and revised reaches, slopes and subarea fraction impervious values were input where necessary;
- Storm files for the November 2010 and January 2011 events were adopted from the Rochester Flood Management Plan (2013) with minor changes, including updated pluviographs. A storm file for the September 2010 event was also created to calibrate the RORB model. All storm files were assembled using the available pluviograph and daily rainfall data for each event;
- The RORB model parameter k_c was calibrated to the observed Campaspe River at Redesdale gauge flood hydrographs for the September 2010, November 2010 and January 2011 events;
- A Monte Carlo analysis was undertaken on the RORB model to determine appropriate design losses by fitting it to the Campaspe River at Redesdale gauge flood frequency curve;
- Using the design losses and the calibrated parameter, k_c, a second RORB Monte Carlo analysis was run using the applicable design inputs for the Kyneton catchment to determine the flood frequency curve for the critical storm duration at the township.
- Individual runs from the Monte Carlo analysis, which produced peak flows approximately equal to the required design flood AEPs, were then selected.

- The inputs of the selected runs (including rainfall depth, loss factors and temporal patterns) were then used to produce complete hydrographs for the 20%, 10%, 5%, 2%, 1% and 0.5% AEP design events. These design hydrographs were then used as inflow boundary conditions for the hydraulic model.

A separate, smaller RORB model was prepared for Post Office Creek, which flows into the Campaspe River just downstream of Kyneton. No streamflow gauge exists for this waterway and therefore the RORB model could not be calibrated to historical flood events. Instead, the routing parameter (k_c) for this model was scaled from the calibrated Campaspe River RORB model. The design losses were also adopted from this model. Using these design parameters, a Monte Carlo analysis was undertaken to determine the flood frequency curve for the critical storm duration of the catchment. Individual sets of model parameters were then chosen to produce the design event hydrographs. The peak flow rates of these hydrographs were then compared to other peak flow estimates to ensure they were reasonable.

3.2 Flood Frequency Analysis

3.2.1 Data Analysis

A flood frequency analysis was conducted on the Campaspe River at Redesdale gauge to assist with the RORB model calibration and generation of design hydrographs. Flood frequency analysis (FFA) involves the fitting of a probability model to an annual series of maximum recorded flows to relate the magnitude of extreme events to their frequency of occurrence. This statistical analysis allows the estimation of design flood flows.

The annual maximum flood series for the Campaspe River at Redesdale gauge was extracted from the available 52 years of instantaneous streamflow data, from 1966 to 2017. This data was evaluated to ensure that the annual maximum flows were independent and homogenous. During the gauge streamflow record no significant storages have been constructed upstream of the gauge and there has not been a significant increase in urbanisation of the gauge catchment. Hence, the annual maximum series derived from the gauge satisfies the criterion of homogeneity. Additionally, all annual maximum flows were produced from separate storm events; therefore, the independence criterion is also achieved. However, as discussed in Section 2.2, there is some uncertainty as to the accuracy of the 2016 measurement and therefore this flow was censored from the analysis.

Most of the gauge discharge data is classified as good quality. However, the quality of the larger annual maximum flows is described by DELWP's Water Measurement Information System as 'Rating extrapolated due to insufficient gaugings'. Hence, it is necessary to analyse the waterway cross-section at the gauge site to determine whether the extrapolation is a valid assumption.

The Campaspe River is reasonably confined at the location of the Campaspe River at Redesdale gauge. The cross-section shown below in Figure 3-1 was generated from the MD_Rivers_ISC_2010 LiDAR. It should be noted that as the LiDAR cannot penetrate the water surface this cross-section does not display the natural surface levels of the river bed. However, according to the Water Measurement Information System the zero-gauge datum is 213.053 metres AHD.

The gauge rating curve has been produced by measuring the flow rate corresponding to a particular gauge level of a historical event. The maximum water level used to calibrate the rating curve was 5.714 metres corresponding to a flow of 389m^3/s. However, the maximum water level recorded at this gauge was 6.697 metres. Hence, the rating curve was extrapolated in order to estimate the corresponding flow rate. To determine whether the rating curve extrapolation is reasonable, the maximum water level is compared to the cross-section of the topography at the gauge site (Figure 3-1). The gauge level of 6.697 metres corresponds to an elevation of 219.750 metres AHD. As shown in the cross-section below, this level is still within the confined area of the waterway. As there is no significant change in the gauge cross-section up to the maximum water level recorded at the gauge the extrapolated flows can be considered reasonable.

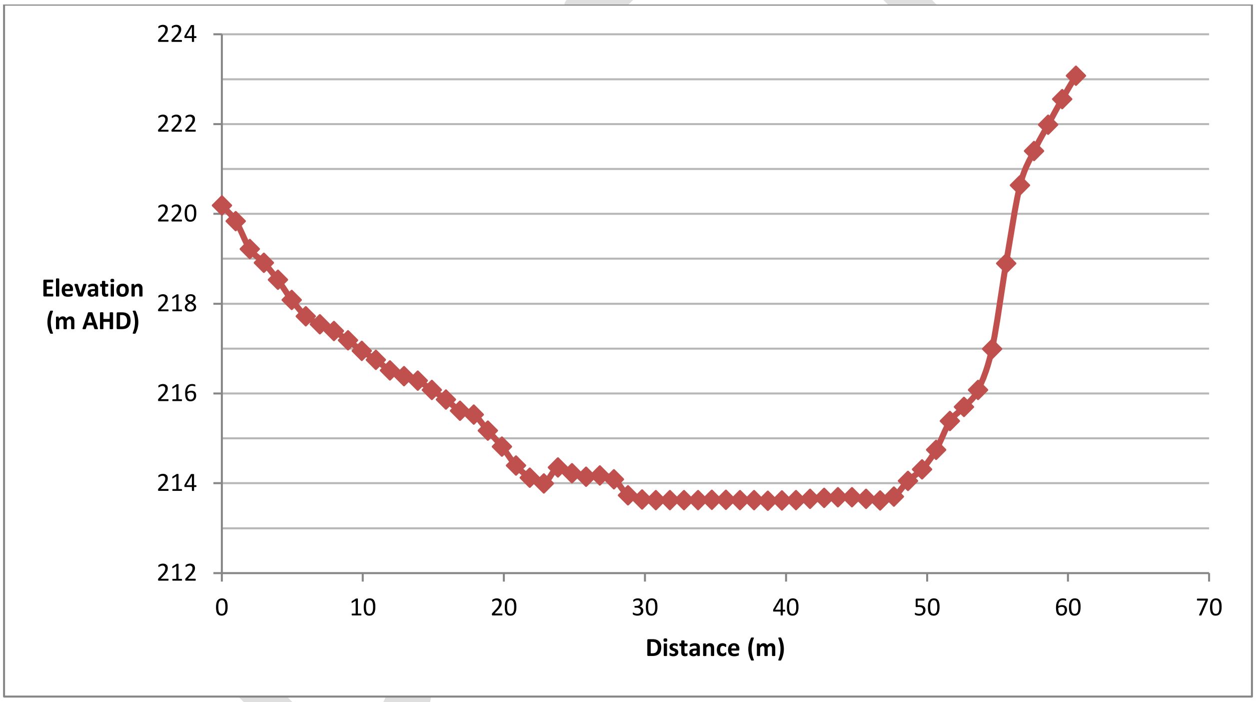


Figure 3-1 Cross-section at Campaspe River at Redesdale gauge

3.2.2 FLIKE

The program FLIKE was used to undertake a flood frequency analysis of the annual maximum series of flows at the Campaspe River at Redesdale gauge. A Log Pearson Type III distribution was fitted to the annual maximum flood series using the Bayesian Inference method consistent with the recommendations of ARR, Book 3, Chapter 2.

No prior information from the Regional Flood Frequency Estimation method was incorporated into the analysis. An initial analysis was undertaken using the regional parameters and it was determined that they were not consistent with the gauge data. This is in accordance with ARR, Book 3, Section 2.3.10 and 2.6.3.5, which states that regional prior information should be used unless there is evidence that it is not applicable to the catchment of interest.

As recommended in ARR, Book 3, Section 2.8.6, the multiple Grubbs-Beck test was used to identify potentially influential low flows. These low flows are not representative of the population of floods and it is important that they are censored so that they do not unduly influence the distribution fit. The multiple Grubbs-Beck test identified 20 low flows which were censored to achieve an improved distribution fit.

Table 3-1 and Figure 3-2 present the AEP quantile estimates and their 90% confidence limits. The results of the FLIKE flood frequency analysis indicate that the September 2010 (259.6m^3/s), November 2010 (216m^3/s) and January 2011 (322m^3/s) flood events were approximately 7%, 13% and 5% AEP flood events respectively.

Table 3-1 Campaspe River at Redesdale FFA results

AEP (%)	5% Confidence Limit (m^3/s)	Quantile Estimate (m^3/s)	95% Confidence Limit (m^3/s)
50	70	89	114
20	151	183	231
10	207	252	323
5	258	318	415
2	318	399	556
1	355	457	668

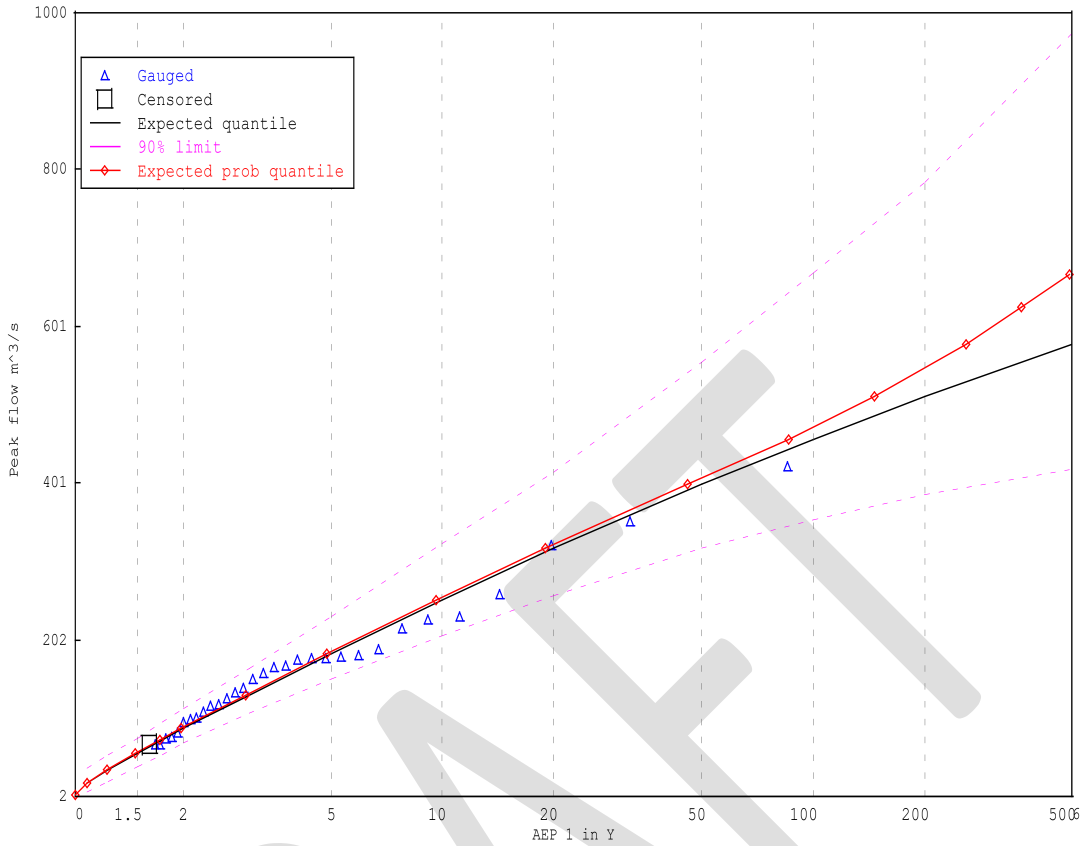


Figure 3-2 Flood frequency analysis of Campaspe River at Redesdale gauge

3.3 RORB Model Construction

3.3.1 Subarea Delineation, Reach Type and Loss Model

The catchment area for the Campaspe River RORB model is approximately 642.6km^2. The catchment has been divided into 20 subareas for the Campaspe River RORB model. Figure 3-3 shows the subarea delineation for the study area. The RORB model outlet is located at the Campaspe River at Redesdale gauge.

A RORB model was also prepared for the Post Office Creek catchment with the downstream outlet at the Mollison Street bridge crossing. The Post Office Creek catchment area, approximately 12.1km^2, was delineated into six subareas as shown in Figure 3-4. The location of the downstream outlet adopted for the RORB model was largely determined by the extent of LiDAR data available for the hydraulic model.

For both models, nodes were placed at areas of interest (including streamflow gauges and inflow locations to the hydraulic model) and the junction of any two reaches. Reaches were then used to connect the nodes, representing the length and type. As all reaches were classified as natural based on aerial photography, the reach slope was not required to be input into RORB. Additionally, as there are no significant storages on either the Campaspe River or Post Office Creek in the study area, drowned reaches were not required.

In order to determine the corresponding runoff generated by a particular rainfall event, RORB provides two alternative models of the loss processes:

- Initial loss and runoff coefficient (constant proportional rate of loss)
- Initial loss and continuing (constant) loss

The RORB manual recommends the use of the runoff coefficient method for urban or partly urban catchments but notes that either model is suitable for rural catchments. However, Australian Rainfall and Runoff 2016 (ARR) states that the continuing loss model is the most suitable for design flood estimation for both rural and urban catchments (ARR, Book 4, Section 2.6.2). For the Campaspe River RORB model, the continuing loss method was adopted as the majority of the catchment is rural. This also allowed a comparison with the Rochester Flood Management Plan RORB model which also utilised this loss model. Similarly, the continuing loss method was also adopted for the Post Office Creek RORB model to be consistent with the Campaspe River RORB model. It was critical that the model parameters and design losses could be applied from the Campaspe River RORB model to the Post Office Creek model due to the lack of historical flood data available for calibration.

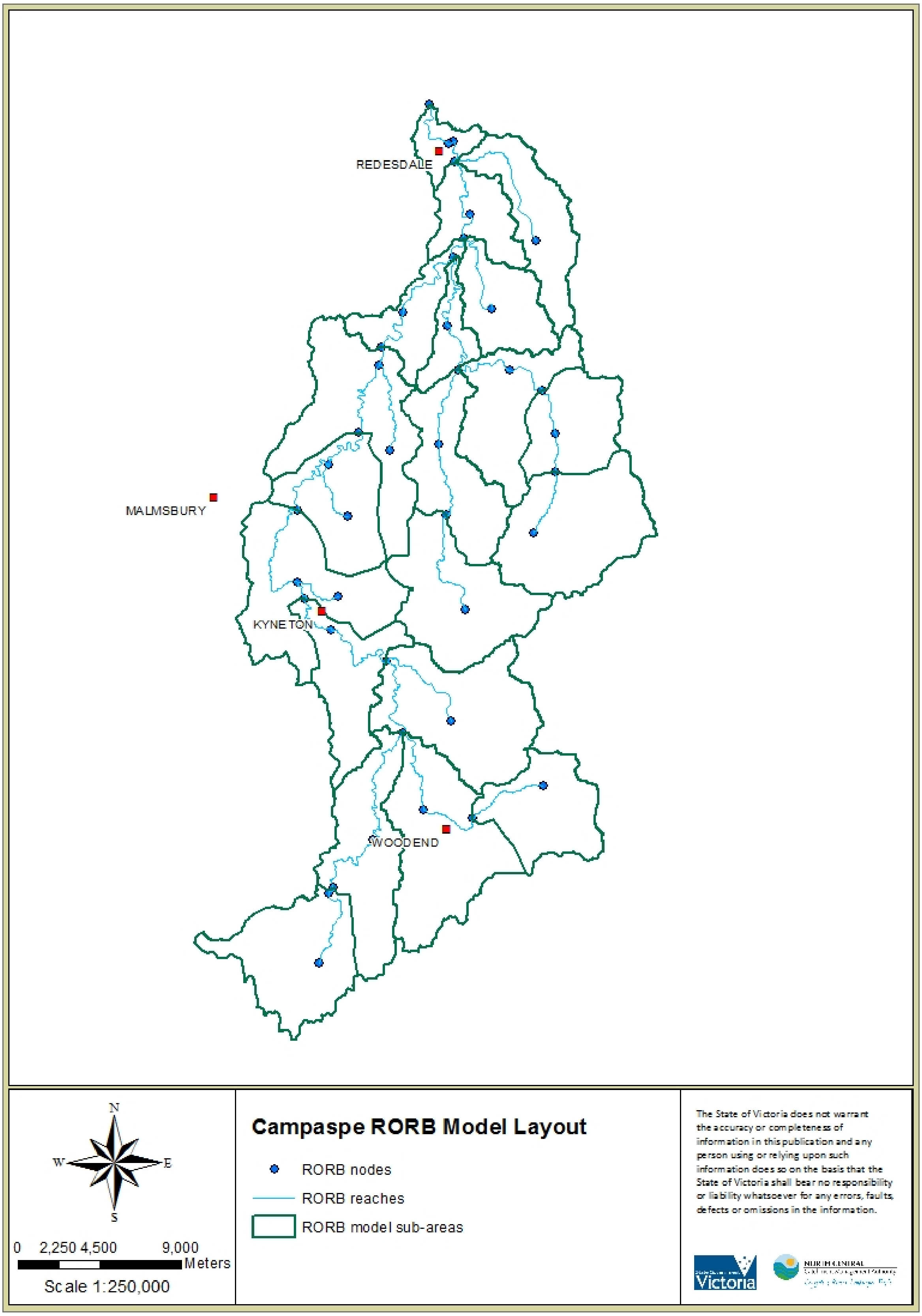


Figure 3-3 Graphical representation of the Campaspe RORB model

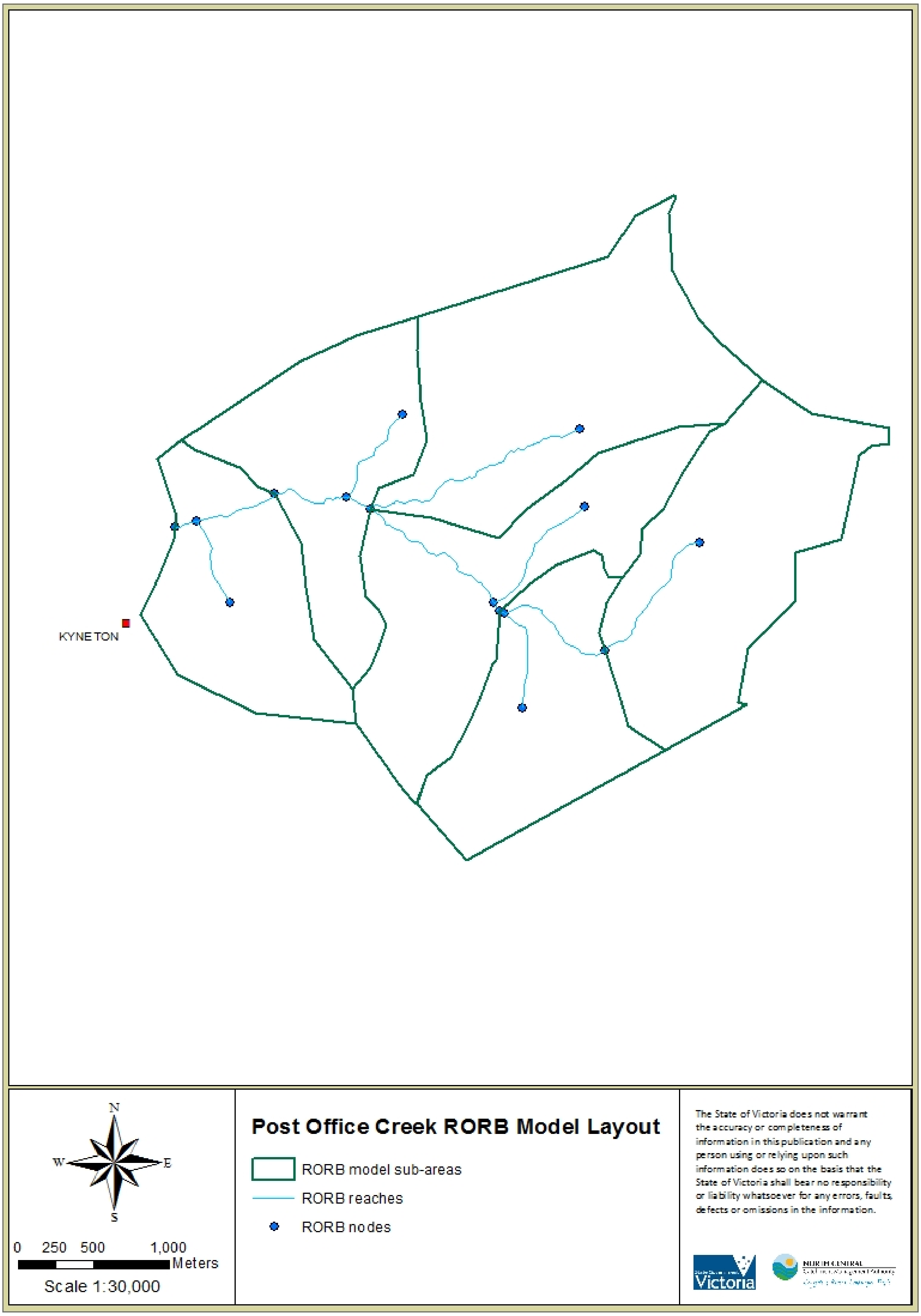


Figure 3-4 Graphical representation of the Post Office Creek RORB model

3.3.2 Fraction Impervious Data

The RORB model for both the Campaspe River and Post Office Creek require the input of fraction impervious values for the subareas. Values were assigned based on the various planning zones, as shown below in Table 3-2. The fraction impervious values adopted were derived from the Rochester Flood Management Plan (2013).

Table 3-2 Land use and fraction impervious values

Land Use Zone	Description	Fraction Impervious
Commercial Zone (B1Z & B3Z)	Commercial centres with retail, office, business and community uses	0.80
Farm Zone (FZ)	Rural areas	0.001
Industrial Zone (IN1Z, IN2Z)	Manufacturing and storage facilities	0.80
Low Density Residential (LDRZ)	0.4 Ha minimum lot size	0.20
Public Conservation and Resource Zone (PCRZ)	Natural environment with associated facilities	0
Public Park and Recreation Zone (PPRZ)	Public recreation and open space	0.01
Public Use Zone (PUZ1-7)	Public utility and community services and facilities	0.60
Residential Zone (R1Z, TZ)	Normal range of densities	0.45
Rural Conservation Zone (RCZ, RCZ1, RCZ2)	Natural environment and agricultural use	0
Road Zone (RDZ1, RDZ2)	Secondary and local roads	0.60
Rural Living Zone (RLZ1, RLZ2, RLZ5)	Rural residential and agricultural use	0
Special Use Zone (SUZ1)	Private educational uses	0.60
Special Use Zone (SUZ2)	Racecourses and associated uses	0.01
Special Use Zone (SUZ3)	Golf Courses and associated uses	0.01
Special Use Zone (SUZ4)	Private hospitals	0.60

The spatial distribution of the fraction impervious data is shown in Figure 3-5 and Figure 3-6 for the Campaspe River and Post Office Creek catchments respectively. The average fraction impervious for both catchments is 0.022 and 0.156 respectively.

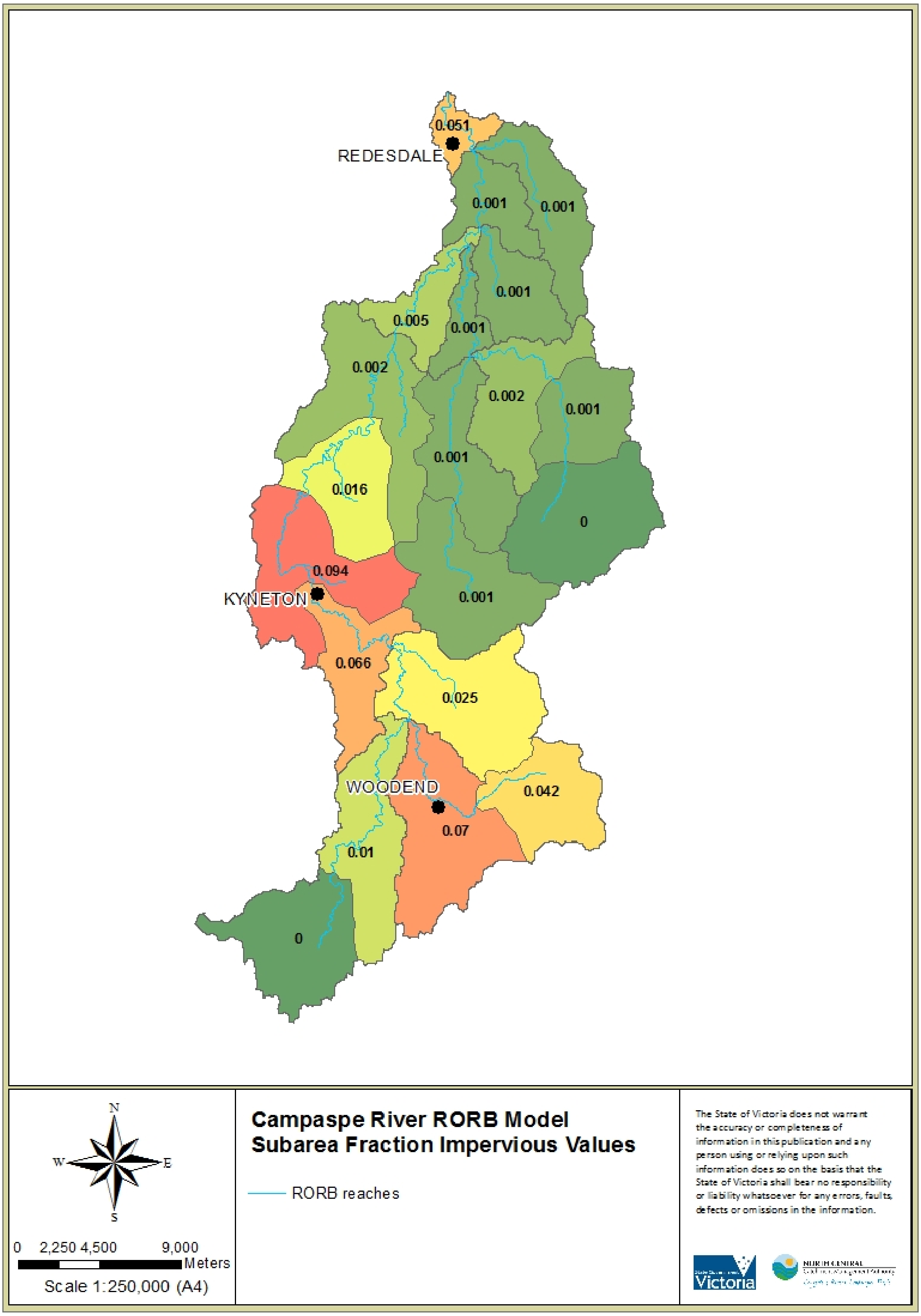


Figure 3-5 Fraction impervious values for the Campaspe RORB model

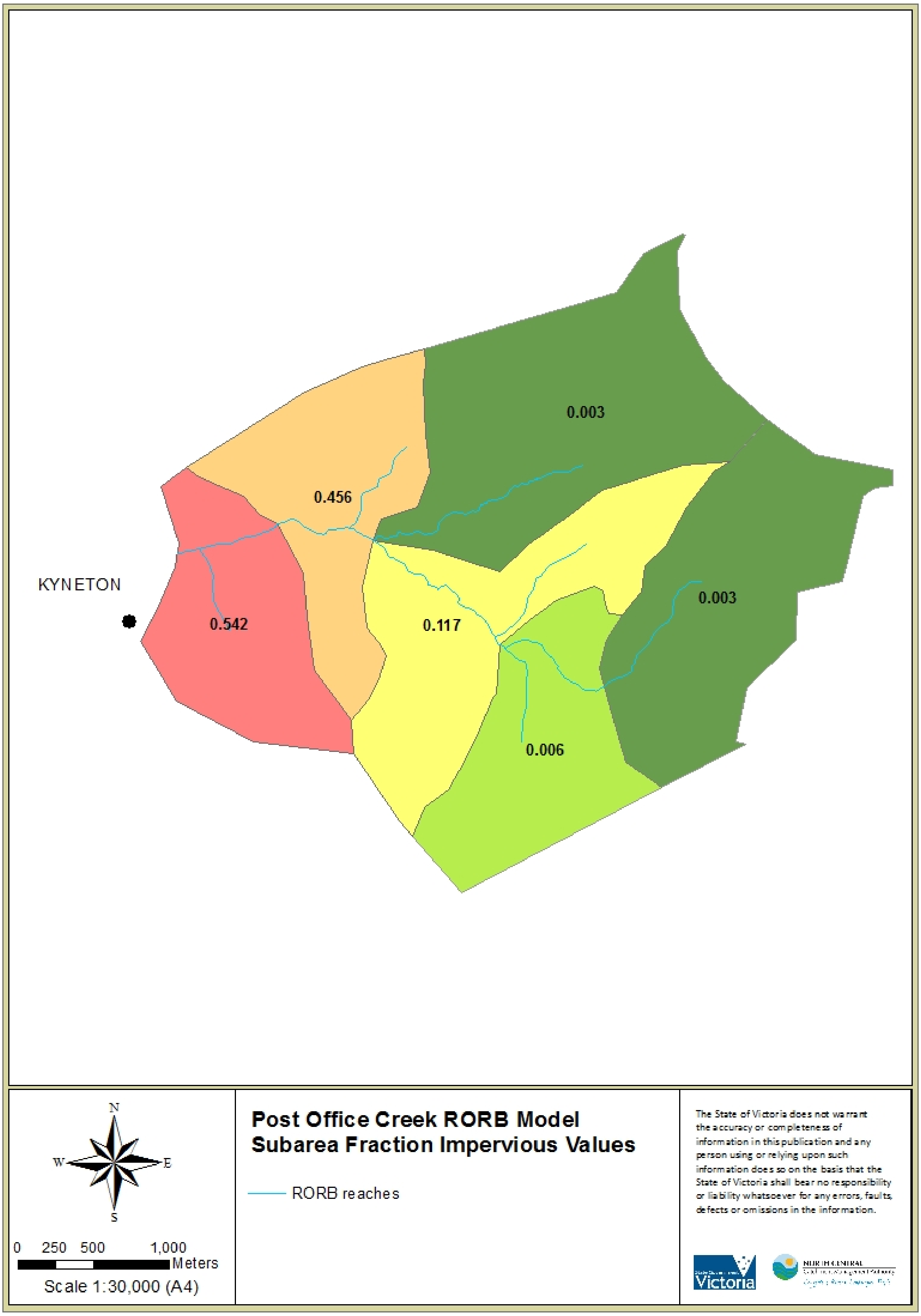


Figure 3-6 Fraction impervious values for the Post Office Creek RORB model

3.4 Campaspe River RORB Model Calibration

3.4.1 Overview

The RORB model was calibrated by fitting it to the observed rainfall and runoff data of recorded flood events to determine the routing parameter k_c. As recommended in RORB manual, and consistent with the Rochester Flood Management Plan (2013), a value of 0.80 was adopted for the nonlinearity parameter, m. A trial and error fitting procedure was utilised to reproduce the flood peak, volume and shape of the observed hydrograph. The adopted k_c value was then compared to regional estimations to assess its reasonableness.

As the catchment does not significantly vary in topography or density of vegetation cover, it is likely to have consistent storage runoff behaviour throughout the catchment. Consequently, it was not considered necessary to vary the parameters by interstation area. Moreover, the Rochester Flood Management Plan (2013) also assessed the results of varying parameters by interstation areas and determined that this does not significantly improve the calibration. Therefore, a single routing parameter has been adopted for the catchment. It should be noted that the k_c parameter determined in the Rochester Flood Management Plan (2013) could not be directly adopted for this flood study model as the catchment size and shape was not comparable.

The RORB model was calibrated to the Campaspe River at Rochester gauge located further downstream of Kyneton as this was the nearest available gauge. Three recent flood events were considered; September 2010, November 2010 and January 2011. These historical events were selected due to their large size and the fact that they are recent experiences of flooding in Kyneton. It should be noted that although the peak flows recorded in 1974 and 1975 are the highest on record (Table 1-1), pluviograph data was not available to enable these events to be used for calibration. Furthermore, there was uncertainty regarding the reliability of the September 2016 peak flow rate as discussed in Section 2.2 and therefore this event was not selected for calibration. Details of the selected calibration events are provided in Table 3-3 below.

Table 3-3 RORB model calibration events

Event	September 2010	November 2010	January 2011
Event Start Date	03/09/2010 12:00am	24/11/2010 12:00pm	09/01/2011 12:00am
Event Finish Date	14/09/2010 12:00am	12/12/2010 12:00am	23/01/2011 12:00am
Average Catchment Rainfall (mm)	91.1mm (11 day period)	166.6mm Burst 1: 122.9mm (4 day period) Burst 2: 25.4mm (4 day period) Burst 3: 18.3mm (2 day period)	182.0mm (5 day period)
Recorded Peak Flow at Campaspe River @ Redesdale gauge (m^3/s)	259.6	216.2	322.1
Recorded Water Level at Campaspe River @ Redesdale gauge (m)	5.138	4.388	6.295
Estimated AEP (based on FFA)	7%	13%	5%

3.4.2 RORB Model Calibration Event Data

3.4.2.1 Observed Streamflow Data

Instantaneous streamflow data for the Campaspe River at Redesdale gauge was obtained from for the DELWP Water Measurement Information System for the selected calibration events. Gauged streamflow data is shown in Figure 3-7 to Figure 3-9 for the September 2010, November 2010 and January 2011 flood events respectively.

The gauge data quality was reviewed for each of the historical flood events considered for the hydrological model calibration. For both the September 2010 and January 2011 flood peaks the data was estimated by extrapolating the rating curve due to the flow exceeding the maximum rated flow for the gauge. Also, the quality codes indicate that medium editing (more than 30 millimetres) was performed on the majority of the September 2010 event data. The November 2010 event data quality was mostly of good quality. However, data recorded for the second flood peak, from approximately 9:00am 8/12/2010, was classified as raw data which had not been validated.

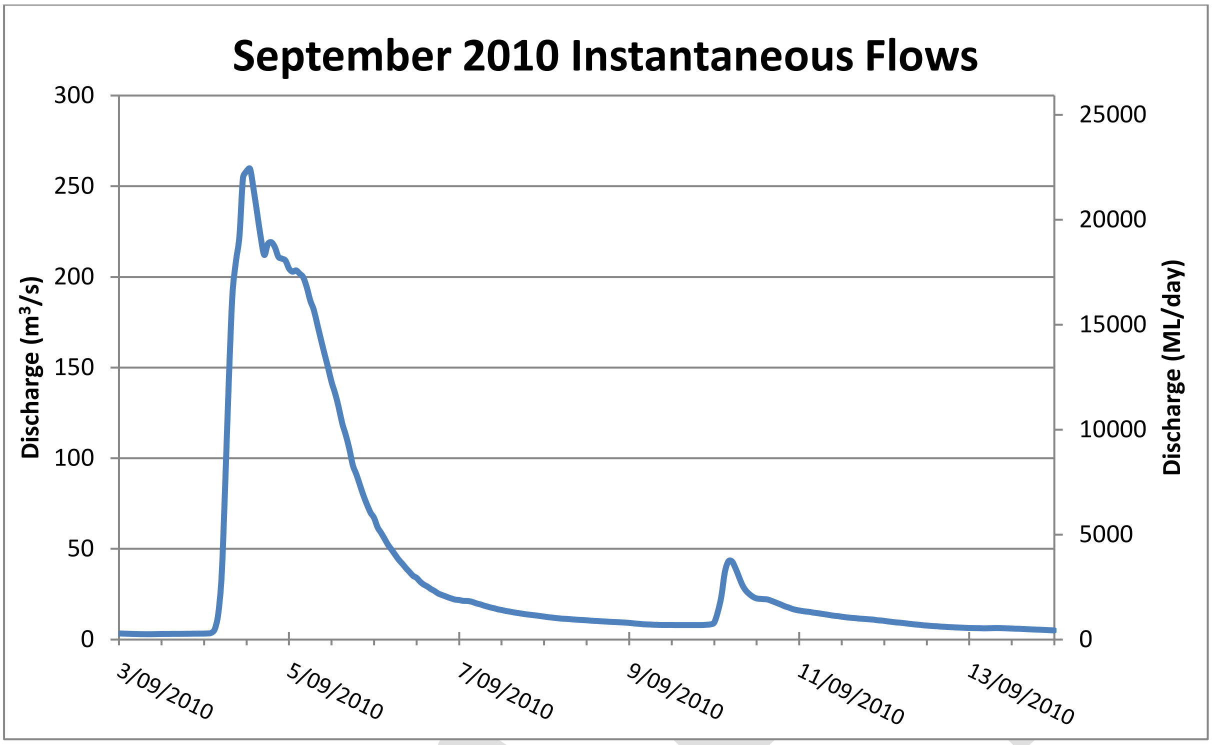


Figure 3-7 **Recorded flood hydrograph for the September 2010 event at the 406213 Campaspe River @ Redesdale gauge**

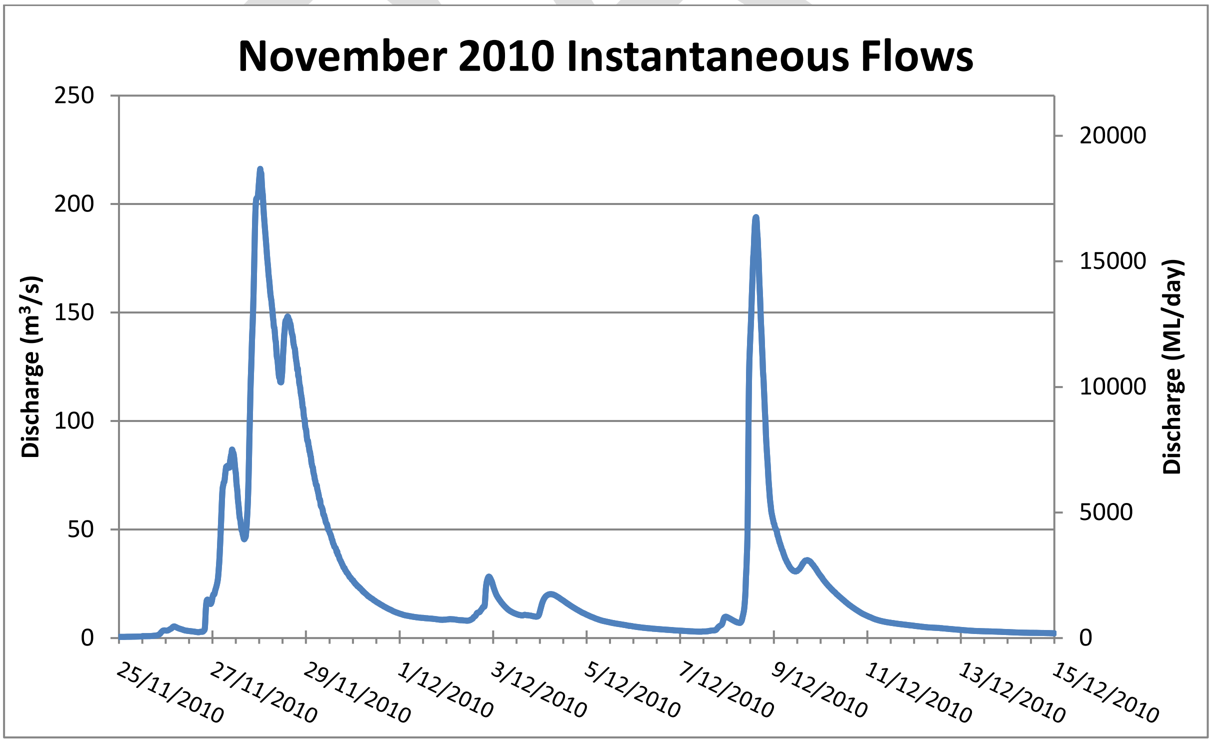


Figure 3-8 **Recorded flood hydrograph for the November 2010 event at the 406213 Campaspe River @ Redesdale gauge**

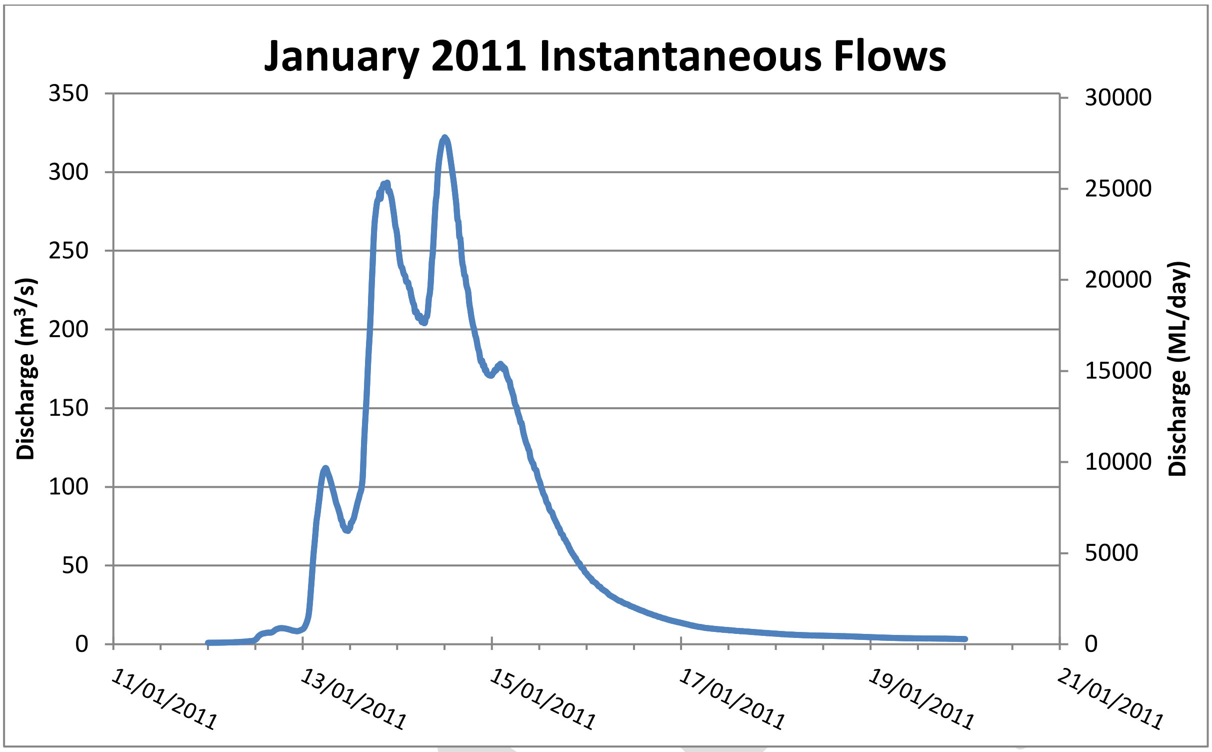


Figure 3-9 **Recorded flood hydrograph for the January 2011 event at the 406213 Campaspe River @ Redesdale gauge**

3.4.2.2 Baseflow Separation

Baseflow describes the portion of streamflow resulting primarily from groundwater discharge, as opposed to surface runoff. As RORB only models direct rainfall runoff, it is necessary to understand the different components and, if necessary, separate the total streamflow into surface runoff and baseflow.

The Rochester Flood Management Plan (2013) analysed the baseflow component of the observed flood hydrographs and determined this contribution to be very small. ARR recommends that the following be considered in order to determine whether baseflow is likely to be a significant component of the flood hydrograph:

- **Baseflow Peak Factor**
  The Baseflow Peak Factor (BPF) is defined as the relative magnitude of baseflow compared to surface runoff for a 10% AEP event. A map of the BPF is provided in ARR, Book 5, Section 4.4, to allow identification of the appropriate factor for the catchment. According to this map, the Campaspe catchment has a factor of less than 0.05. Furthermore, the AEP scaling factors for the BPF indicate that, for rarer events, the BPF reduces (ARR, Book 5, Section 4.5.2). Hence, for a 1% AEP flood event, the baseflow contribution for the Campaspe catchment is expected to be less than 3% of the surface runoff. Data Hub specifies the BPF for the Redesdale catchment as 0.02 (Section 7.3.1).

- **Streamflow data review**
  A review of the magnitude of flows between flood events relative to the peaks can be used to determine whether baseflow is likely to be an important component of the flood hydrograph. The duration curve shown in Figure 3-10 displays the percentage of time that a particular streamflow is exceeded at the Campaspe River at Redesdale gauge. It can be seen that the majority of flows are very small, with flows only exceeding 0.1m^3/s 50% of the time over the 41-year gauge record.

Hence, baseflow is considered to have a negligible impact on the flood hydrographs in this catchment and therefore baseflow has not been explicitly removed from the recorded hydrographs.

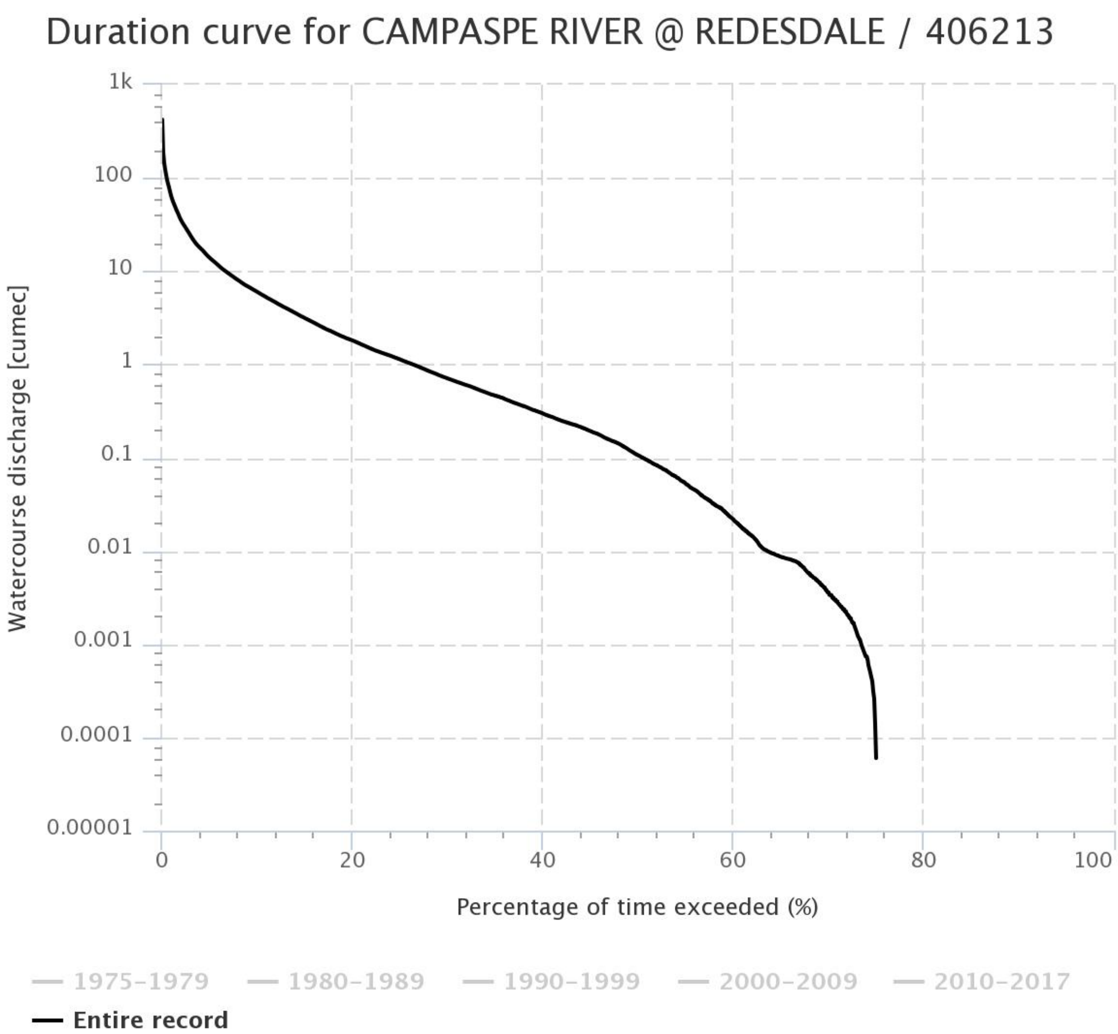


Figure 3-10 **Duration Curve (Bureau of Meteorology)**

3.4.2.3 Observed Rainfall Data

The temporal rainfall distributions for each RORB subarea were sourced from the closest available pluviograph stations for each storm event. The temporal patterns for the available pluviographs during the September 2010, November 2010 and January 2011 flood events are shown in Figure 3-11 to Figure 3-13 respectively.

The total storm rainfall depth is also required at each available daily rainfall gauge for the calibration events. This data is used to produce rainfall isohyets for each event to estimate the rainfall depth for each model subarea for the total storm duration. This process was undertaken for the September 2010 event. However, as this information was already available for the November 2010 and January 2011 events from the Rochester Flood Management Plan (2013), these previously derived rainfall totals were adopted for this study. The pluviographs applied in the calibration modelling of these two events was also reviewed and, for some subareas, more relevant pluviograph data was substituted in.

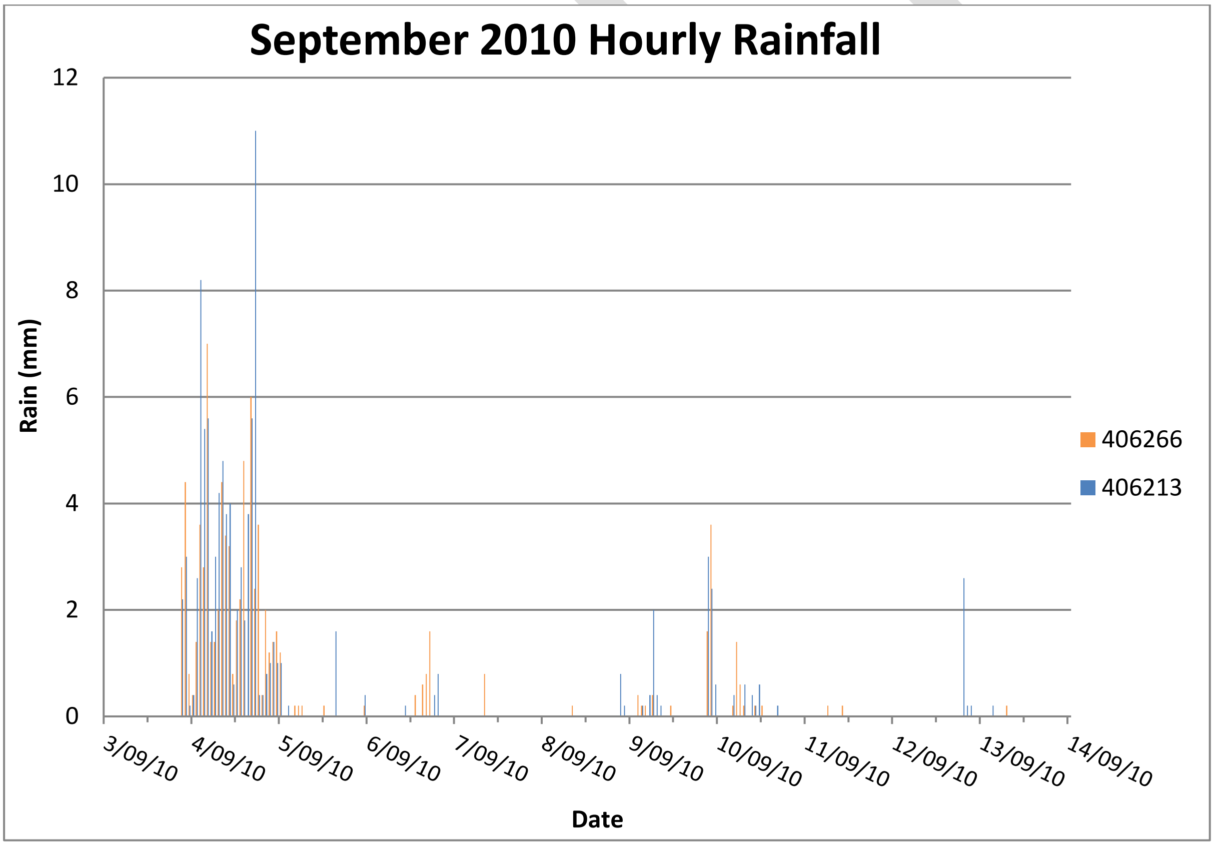


Figure 3-11 Pluviograph records for the September 2010 event

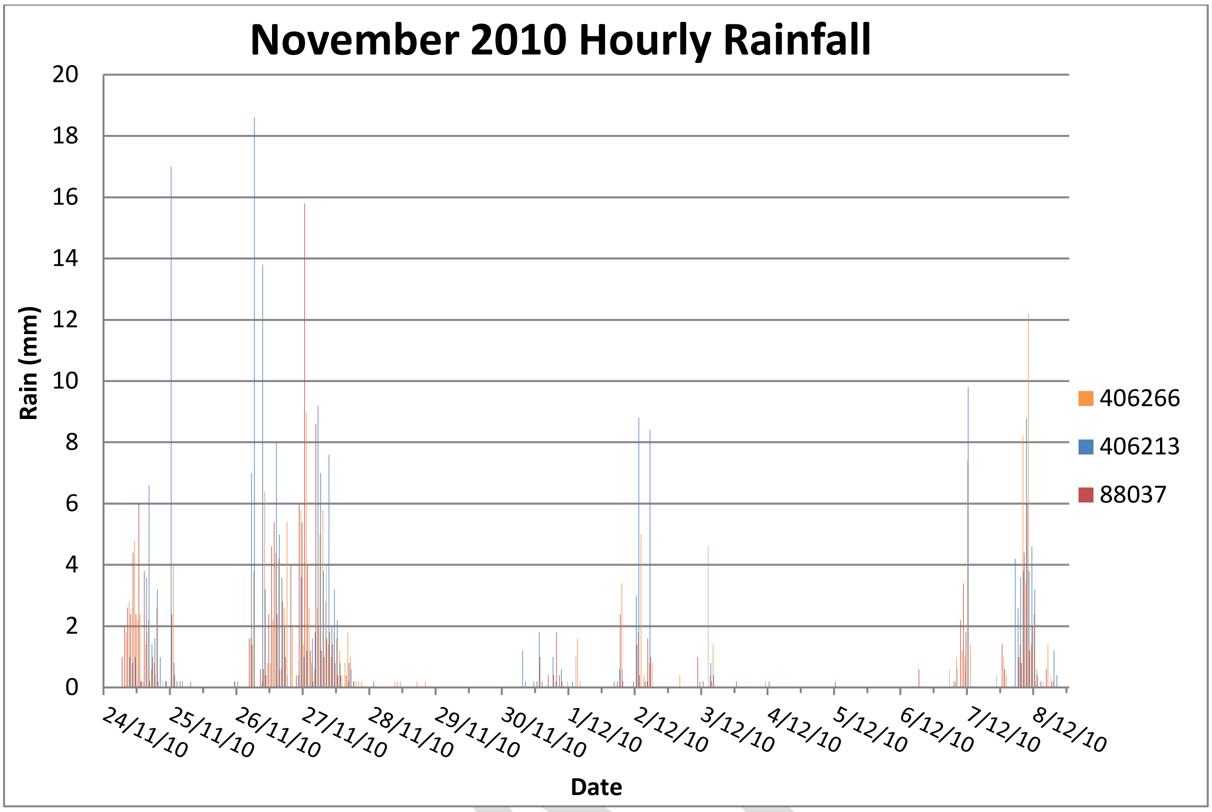


Figure 3-12 **Pluviograph records for the November 2010 event**

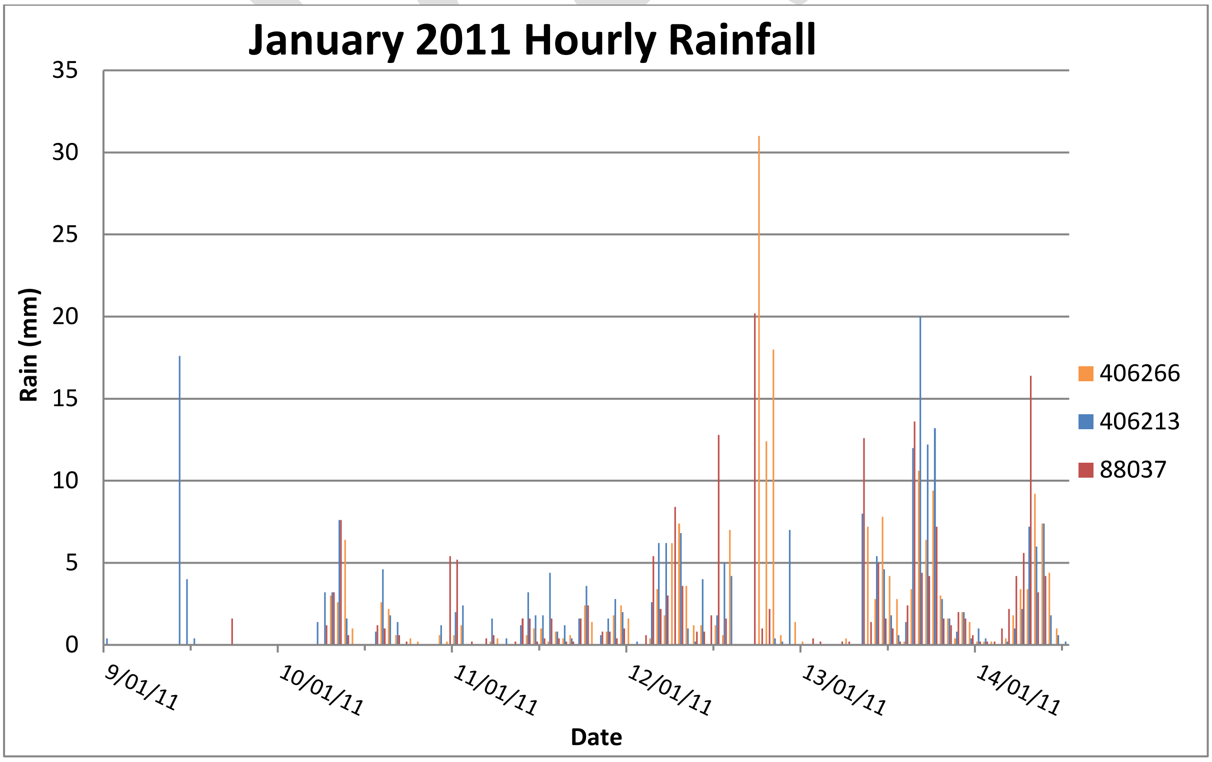


Figure 3-13 **Pluviograph records for the January 2011 event**

3.4.2.4 Losses

An initial loss/continuing loss model was adopted for the RORB model and calibration was achieved using the FIT option in RORB. The initial loss parameter was determined by trial and error to reasonably reproduce the observed rising limb of the hydrograph. Depending on the initial loss chosen, the FIT option enabled RORB to automatically select the continuing loss value that minimises the error between the calculated and observed hydrograph volume. In addition to ensuring a good model fit, the adopted calibration losses were also reviewed against those adopted in the Rochester Flood Management Plan (2013) as well as whether the values were realistic in general.

3.4.3 September 2010 Flood Event Calibration

The September 2010 event was modelled from 12:00am 3rd September 2010 to 12:00am 14th September 2010. This timing was based on an analysis of the available daily rainfall pluviograph and flow data for this flood event. Based on a flood frequency analysis of the Campaspe River at Redesdale gauge the September 2010 event was approximately a 7% AEP flood event.

The accumulated rainfall total for the entire storm duration was determined for each rainfall station. These values were then mapped spatially and interpolated to create a raster surface as shown in Figure 3-14. The interpolation was performed using the Inverse Distance Weighted technique based on a minimum of 12 points and cell size of 250 metres. The rainfall totals for each model subarea was then determined from the interpolated rainfall raster surface by averaging the rainfall totals at all grid cells that intersect with the spatial extent of the subarea.

The temporal patterns for each subarea were assigned based on the closest available pluviograph station. As the majority of rainfall fell over 4th – 5th of September, the rainfall was modelled as a single burst.

Figure 3-15 shows the observed and calculated hydrographs at the Campaspe River at Redesdale gauge. The calculated hydrograph reasonably matches the shape and peak of the observed hydrograph. However, it can be seen that the timing is off by approximately 13 hours. Also, it is evident that the rising limb of the observed hydrograph appears very steep, particularly compared to the other flood hydrographs. It should be noted that the data quality codes for this event indicate that medium editing of more than 30 millimetres was undertaken.

The adopted values of k_c, m, initial loss (IL) and continuing loss (CL) for the calibration are summarised in Table 3-4. As shown, the difference between the observed and modelled peak flow is -1.8% while the difference in flood volume is -0.2%. Overall, the model calibration for the September 2010 flood event is considered good.

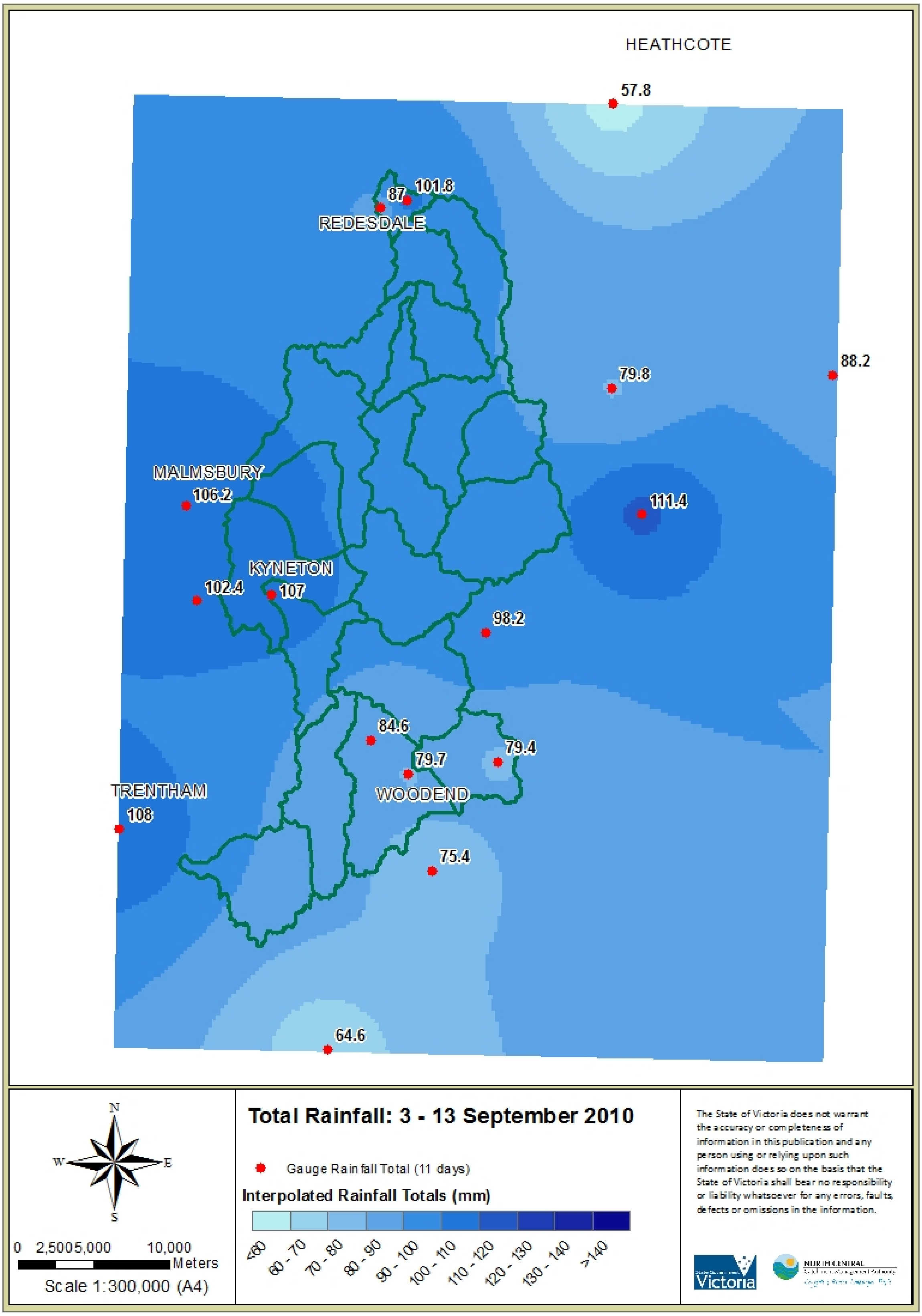


Figure 3-14 **Total rainfall accumulated over 10 days during September 2010 storm (3 – 13 September)**

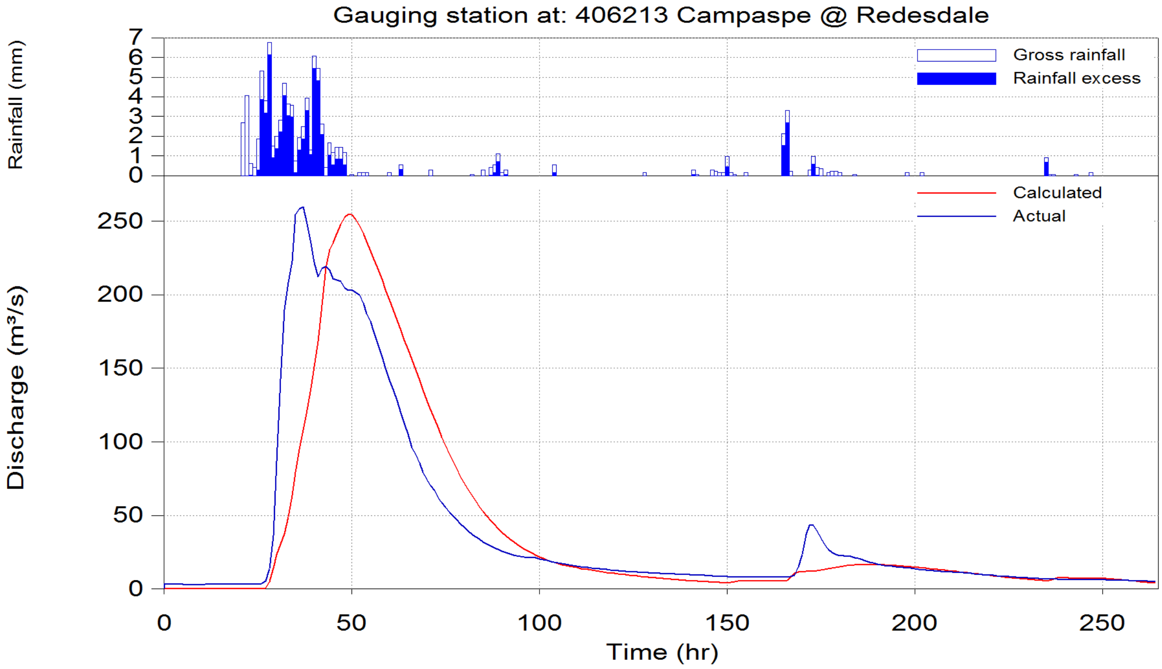


Figure 3-15 **Comparison of modelled and observed hydrographs for the September 2010 event on the Campaspe River at Redesdale gauge (406213)**

Table 3-4 **RORB calibration parameters and results for September 2010 event**

Location		**Campaspe River @ Redesdale**
Model Parameters	k_c	62
	m	0.8
	IL	10
	CL	0.63
Peak flow (m^3/s)	Observed	255.0
	Calculated	261.0
	Relative difference (%)	-1.8
Volume (ML)	Observed	37.6×10^6
	Calculated	37.5×10^6
	Relative difference (%)	-0.2

3.4.4 November 2010 Flood Event Calibration

The November 2010 event was modelled from 12:00am 24th November 2010 to 12:00am 11th December 2010. This timing was based on an analysis of the available daily rainfall pluviograph and flow data for this flood event. Based on a flood frequency analysis of the Campaspe River at Redesdale gauge the November 2010 event was approximately a 13% AEP flood event.

The accumulated rainfall total for each subarea was adopted from the Rochester Flood Management Plan (2013). The pluviographs applied in the Rochester Flood Management Plan (2013) calibration model was reviewed and the subarea temporal pattern was adjusted where more relevant pluviograph data was available.

A multiple burst approach was adopted for the November 2010 event to allow the loss parameters to vary across each burst. As discussed in the Rochester Flood Management Plan (2013), this was required since:

- The flooding event resulted from rainfall events that ran over multiple days, resulting in daily variation of rainfall totals (from daily rainfall stations) across subareas;
- The pluviographs (Figure 3-12) show separate rainfall events during the November 2010 flood event. The events were separated by a minimum of 24-hour period of no rainfall; and
- The hydrograph recorded at the gauge also shows multiple peaks (Figure 3-16). Multi-peaked hydrographs can be calibrated better if the event is treated as a multi burst event.

Figure 3-16 shows the observed and calculated hydrographs at the Campaspe River at Redesdale gauge. The calculated hydrograph closely matches the shape and maximum flow of the first and largest observed hydrograph peak. However, the peak at approximately 340 hours is significantly underestimated. Although the quality of the streamflow data was good, the final hydrograph peak was classified as raw data that had not been validated. The model calibration for the Rochester Flood Management Plan (2013) was also not able to achieve a good fit for this peak. It is considered that as the first peak is larger, it is more important to achieve a good fit to this peak.

The adopted values of k_c, m, initial loss (IL) and continuing loss (CL) for the calibration are summarised in Table 3-5. As shown, the difference between the observed and modelled peak flow is 0.8% while the difference in flood volume is 0.0%. Overall, the model calibration for the November 2010 flood event is considered good given the uncertainty of several variables.

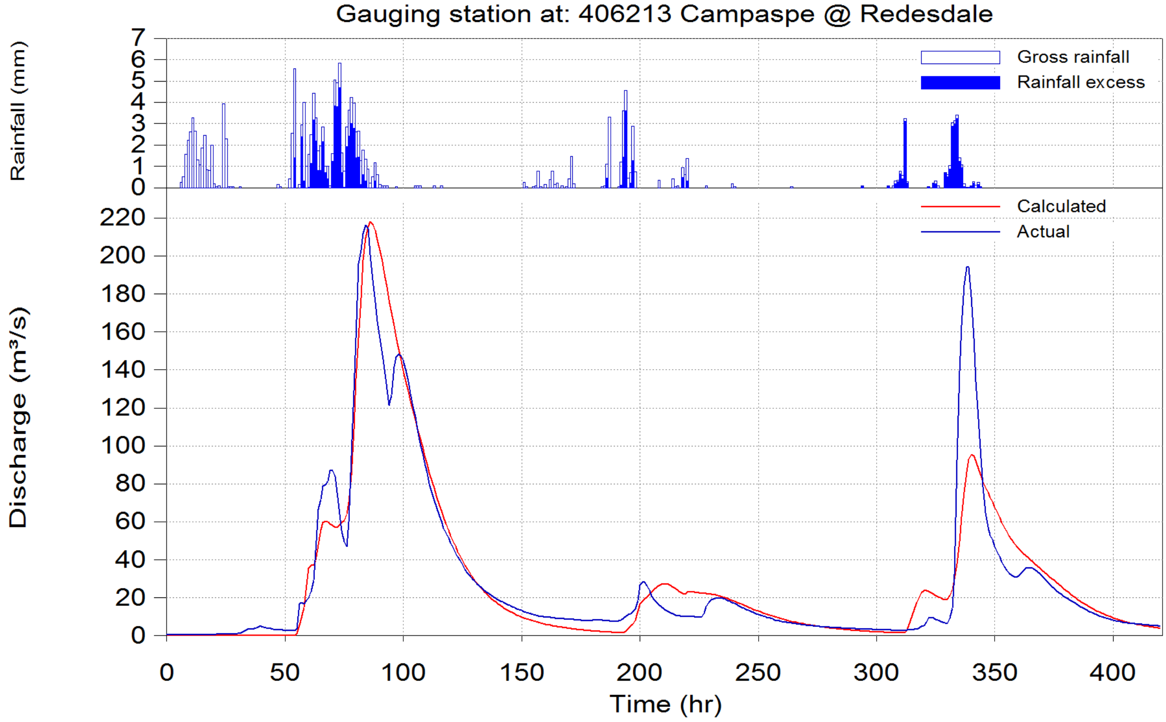


Figure 3-16 **Comparison of modelled and observed hydrographs for the November 2010 event on the Campaspe River at Redesdale gauge (406213)**

Table 3-5 **RORB calibration parameters and results for the November 2010 event**

Location		**Campaspe River @ Redesdale**
Model Parameters	k_c	62
	m	0.8
	IL1	45
	CL1	1.25
	IL2	10
	CL2	1.92
	IL3	0
	CL3	0.17
Peak flow (m^3/s)	Observed	218.0
	Calculated	216.9
	Relative difference (%)	0.8
Volume (ML)	Observed	45.2×10^7
	Calculated	45.2×10^7
	Relative difference (%)	0.0

3.4.5 January 2011 Flood Event Calibration

The January 2011 event was modelled from 12:00am 9th January 2011 to 12:00am 23rd January 2011. This timing was based on an analysis of the available daily rainfall pluviograph and flow data for this flood event. Based on a flood frequency analysis of the Campaspe River at Redesdale gauge the January 2011 event was approximately a 5% AEP flood event.

The accumulated rainfall total for each subarea was adopted from the Rochester Flood Management Plan (2013). The pluviographs applied in the Rochester Flood Management Plan (2013) calibration model was reviewed and the subarea temporal pattern was adjusted where more relevant pluviograph data was available. As the rainfall was relatively continuous for the duration of storm and only a single flood peak was observed in the streamflow data, a single burst was adopted for the calibration.

Figure 3-17 shows the observed and calculated hydrographs at the Campaspe River at Redesdale gauge. The calculated hydrograph closely matches the rising and falling limbs in addition to the peak of the observed hydrograph.

The adopted values of k_c, m, initial loss (IL) and continuing loss (CL) for the calibration are summarised in Table 3-6. As shown, the difference between the observed and modelled peak flow is 3.0% while the difference in flood volume is 0.5%. Overall, the model calibration for the January 2011 flood event is considered good.

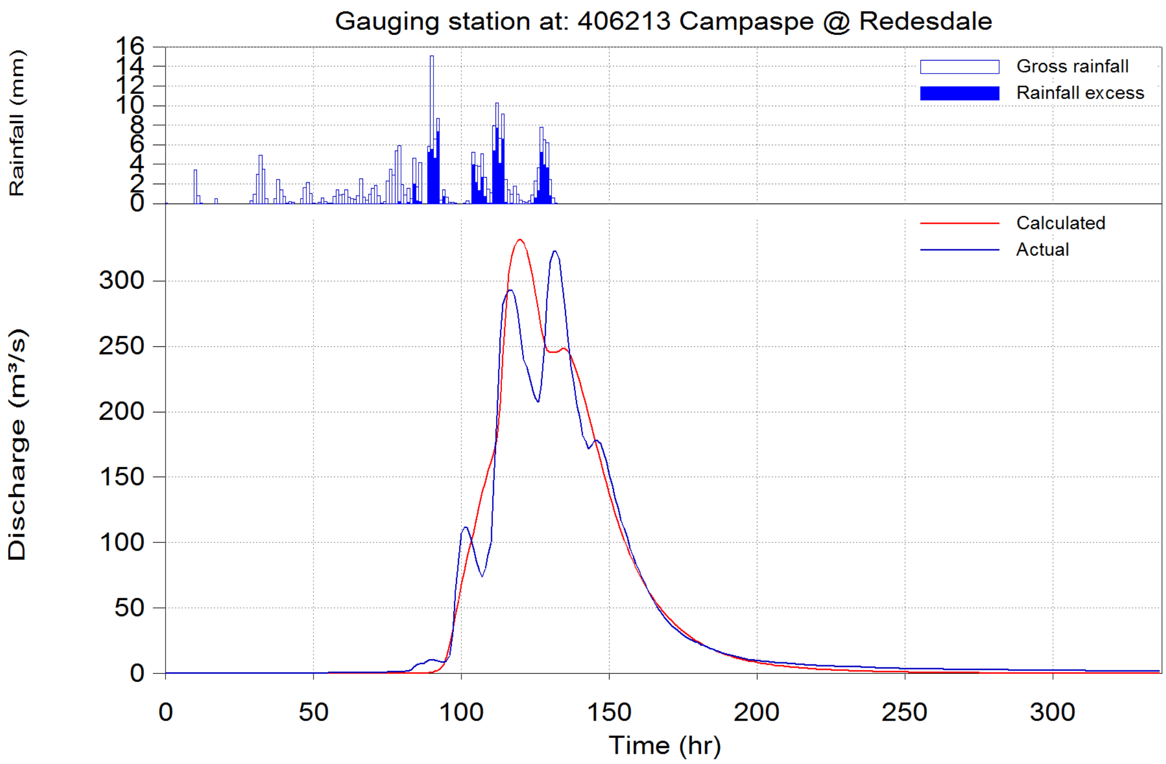


Figure 3-17 Comparison of modelled and observed hydrographs for the January 2011 event on the Campaspe River at the Redesdale gauge (406213)

Table 3-6 RORB calibration parameters and results for the January 2011 event

Location		Campaspe River @ Redesdale
Model Parameters	k_c	62
	m	0.8
	IL	85
	CL	2.61
Peak flow (m^3/s)	Observed	322.1
	Calculated	331.6
	Relative difference (%)	3.0
Volume (ML)	Observed	48.6×10^6
	Calculated	48.8×10^6
	Relative difference (%)	0.5

3.4.6 Discussion

3.4.6.1 Routing Parameter

All events were calibrated with a nonlinearity parameter, *m,* set to 0.8, which is the value commonly adopted for RORB models. A value of 0.8 was also used for the Rochester Flood Management Plan (2013). There appears to be no reason to vary this value for the Campaspe River catchment and thus 0.8 was used for the calibration and also adopted for the design runs.

The model was calibrated to three large historical events, with estimated magnitudes of 5%, 7% and 13% AEP. This ensured that the derived parameters are suitable for design flood estimation and are AEP neutral. That is, the AEP of the design flow corresponds to the same AEP as the causative rainfall that generates the flow.

Each event was calibrated independently to determine the most appropriate routing parameter, k_c. The values for each event were very similar and an average value of 62 was adopted which provided the best fit when applied to all three historical events.

Although the routing parameter was calibrated to three separate events, there still remains some uncertainty in the value adopted. In particular, due to the limited available data, the largest event the parameter could be calibrated to has an annual exceedance probability of 5%. Hence, extrapolation of the parameter is required to produce design flood estimates exceeding this value, for example the 1% AEP event. Therefore, the calibrated value of k_c was compared to a range of recommended prediction equations as shown in Table 3-7. This included the regional equations for Victoria as recommended in ARR. Note that the catchment area (A) referred to in the estimation equations is 642.6km^2. Furthermore, d_{av} provides an indication of the travel distance to the outlet of the RORB model, and is given by the weighted average flow distance from all nodes to the catchment outlet. The value of d_{av} obtained for the Campaspe River RORB model was 48.6km.

Table 3-7 Comparison to additional routing parameter estimates

Method	Applicable Region	Equation	Predicted k_c
RORB Default Equation (RORB Manual, Equation 2-5)	Australia wide	$k_c = 2.2*A^{0.5}*(Q_p/2)^{0.8-m}$	55.74
Regional Equation (ARR, Book 5, Equation 3.22)	For areas where annual rainfall <800mm	$k_c = 0.49*A^{0.65}$	32.76
Regional Equation (ARR, Book 5, Equation 3.21)	For areas where annual rainfall >800mm	$k_c = 2.57*A^{0.45}$	47.15
Victorian Data (Pearse et al. 2002)	Victoria	$k_c = 1.25*d_{av}$	60.75
Australian wide Dyer (1994) Data (Pearse et al. 2002)	Australia wide	$k_c = 1.14*d_{av}$	55.40
Australian wide Yu (1989) Data (Pearse et al. 2002)	Australia wide	$k_c = 0.96*d_{av}$	46.66
Comparison to Rochester Flood Management Plan (2013) ($k_c/d_{av} = 1.278$)	Campaspe catchment	$k_c = 1.278*d_{av}$	62.11

As shown in Table 3-7, a comparison with the routing parameter adopted for the Rochester Flood Management Plan (2013) was also undertaken by considering the ratio between the routing parameter, k_c, and the weighted average flow distance from all nodes to the catchment outlet, d_{av}. The weighted average flow distance for the Rochester RORB model was 126.35km and a value of 161.5 was adopted for k_c, resulting in a k_c/d_{av} ratio of 1.278. Applying this same ratio to the weighted average flow distance of the Campaspe River RORB model, 48.6km, gives a k_c value of 62.11.

A review of the routing parameter estimates determined from alternative methods indicated that the parameters used in calibration were reasonable (Table 3-8). Therefore, the k_c value determined from the calibration was considered to be suitable for the design runs.

Table 3-8 Adopted RORB model parameters

k_c	m
62	0.8

3.4.6.2 Losses

The losses used in the calibration of each historical event are shown in Table 3-9 to Table 3-11. The initial loss (IL) parameter was determined by trial and error to reasonably reproduce the observed rising limb of the hydrograph. Then, using the FIT option in RORB, a corresponding continuing loss (CL) was automatically determined in RORB to minimise the error between the calculated and observed hydrograph volume. It can be seen that in some cases significant losses were required to achieve a reasonable fit. These values were compared to the losses adopted in the Rochester Flood Management Plan (2013) to assess their reasonableness (shown in Table 3-12).

It should be noted that the design losses are not derived from the losses used for calibration. This is due to the fact that the losses applied to these historical events depend on the antecedent conditions of the catchment.

Table 3-9 RORB calibration loss parameters – September 2010

Location	Burst 1	
	IL (mm)	CL (mm/hr)
Campaspe River @ Redesdale	10	0.63

Table 3-10 RORB calibration loss parameters – November 2010

Location	Burst 1		Burst 2		Burst 3	
	IL (mm)	CL (mm/hr)	IL (mm)	CL (mm/hr)	IL (mm)	CL (mm/hr)
Campaspe River @ Redesdale	45	1.25	10	1.97	0	0.17

Table 3-11 RORB calibration loss parameters – January 2011

Location	Burst 1	
	IL (mm)	CL (mm/hr)
Campaspe River @ Redesdale	85	2.61

Table 3-12 Rochester Flood Management Plan (2013) calibration losses

	November 2010	January 2011
IL1 (mm)	50	82
CL1 (mm/hr)	2.40	2.60
IL2	10	-
CL2	1.00	-
IL3	0	-
CL3	0	-

3.5 Campaspe River Design Event Modelling

This section details the process used to determine appropriate design parameters and flows for the 20%, 10%, 5%, 2%, 1% and 0.5% AEP events for the Campaspe River at Kyneton.

3.5.1 Calibrate Design Losses

Initially, a Monte Carlo analysis was run for the Campaspe River RORB model to determine the applicable design losses. The critical design parameters for the Campaspe River catchment to Redesdale were used in this model. That is, the Intensity-Frequency-Duration (IFD) design rainfalls, temporal patterns, spatial patterns and Areal Reduction Factors (ARF) relative to the Redesdale catchment centroid were used. These values were used to calibrate the design losses by fitting the Monte Carlo peak flow estimates for the 50-1% AEP events at the Campaspe River at Redesdale gauge to the values determined in the flood frequency analysis for this same gauge. The relevant inputs are described below.

3.5.1.1 IFD

The relevant IFD was obtained from the Bureau of Meteorology website for the entire Redesdale catchment. Rainfall depth units were selected instead of intensity for the RORB input.

Additional durations were added to the IFD table to match the durations for which temporal patterns were available. The table was also expanded by adding the rainfall depths for rare events. At the time of writing, rainfall depths for events from 1 in 200 to 1 in 2000 AEP were not available on the Bureau of Meteorology's website for durations less than 24 hours. Hence, the method recommended in ARR, Book 8, Section 3.6.3 for estimating very rare sub-daily rainfalls was used. Sub-daily rainfall depths are determined by multiplying the relevant 1% AEP design rainfall depth for each duration by specific growth curve factors. ARR notes that due to the method used to derive these growth curve factors there may be the potential for significant discontinuity when compared to the values provided for durations of 24 hours and longer. As a result, it was necessary to smooth the growth factors to ensure the depths varied in a consistent manner across storm durations and exceedance probability. The growth curve factors were applied to the shortest durations and intermediary depths were smoothed between these values and those provided on the Bureau of Meteorology's website for 24-hour storms. A log graph displaying the smoothed results is shown in Figure 3-18.

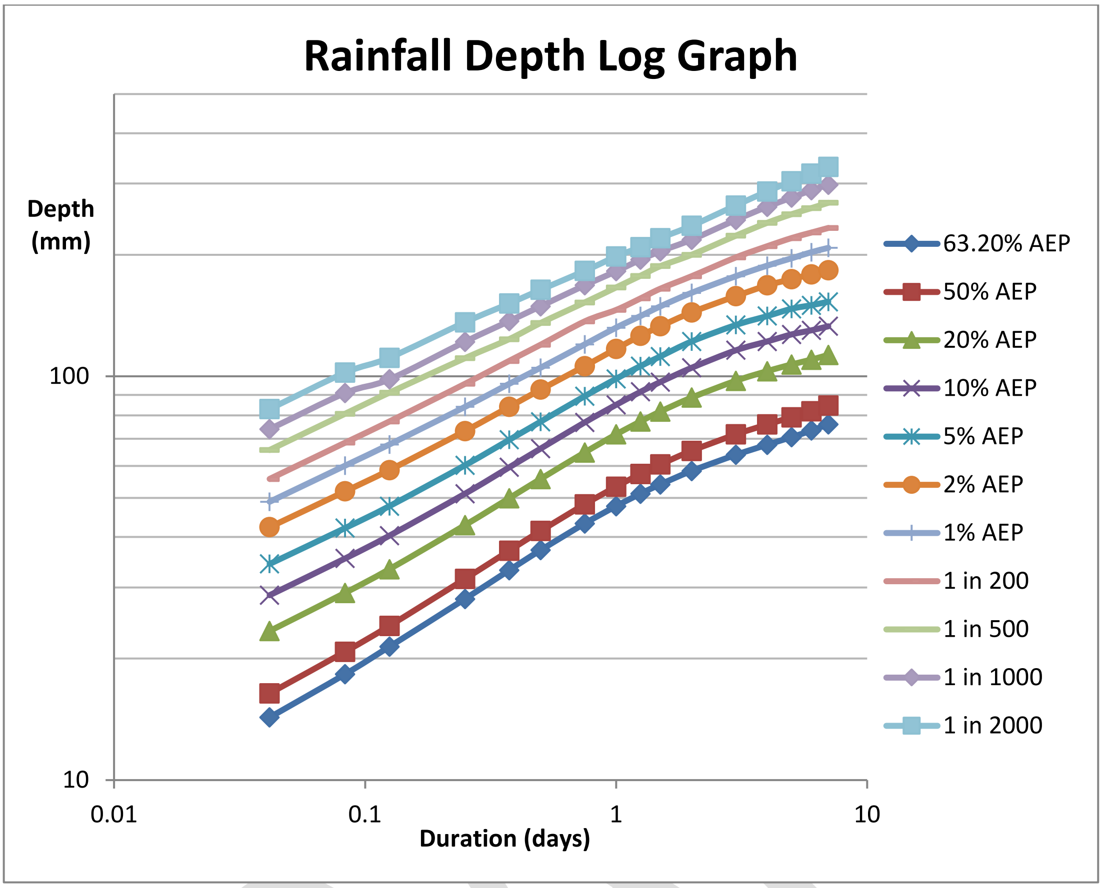


Figure 3-18 Log graph showing smoothing of depth-duration relationship for very rare rainfall events (1 in 200 to 1 in 2000) for Redesdale IFD

3.5.1.2 Areal Reduction Factor

The Areal Reduction Factor (ARF) is the ratio between the design values of areal average rainfall and point rainfall. It is used to account for the fact that larger catchments are less likely than smaller catchments to experience high intensity storms simultaneously over the whole of the catchment area. The values applied were read into the RORB model from the Data Hub file. The Data Hub parameters are shown in the Appendix (Section 7.3.1).

3.5.1.3 Design Temporal Pattern

The applicable design temporal patterns were obtained from Data Hub (Section 7.3.1). Areal patterns were applied as the catchment is greater than 75km^2.

The temporal pattern sample is selected based on the closest area, in this case 500km^2. Areal temporal patterns were available for following storm durations: 12, 18, 24, 36, 48, 72, 96, 120, 144, and 168 hours. For each duration there are ten different temporal patterns, resulting in a total of 100 patterns available for modelling.

The temporal patterns have been assessed to determine if any contain embedded bursts which would cause the RORB model to overestimate the peak flows. This was done by comparing the sub-period rainfall totals of a particular temporal pattern against the IFD to determine whether it is rarer than the AEP of the entire burst. The analysis revealed that two temporal patterns contained embedded bursts; pattern 8 from the 36-hour duration storm, and pattern 5 from the 48-hour duration storm. For example, Figure 3-19 shows pattern 8 from the 36-hour duration which contains an embedded rainfall burst rarer than a 1 in 1000 AEP event.

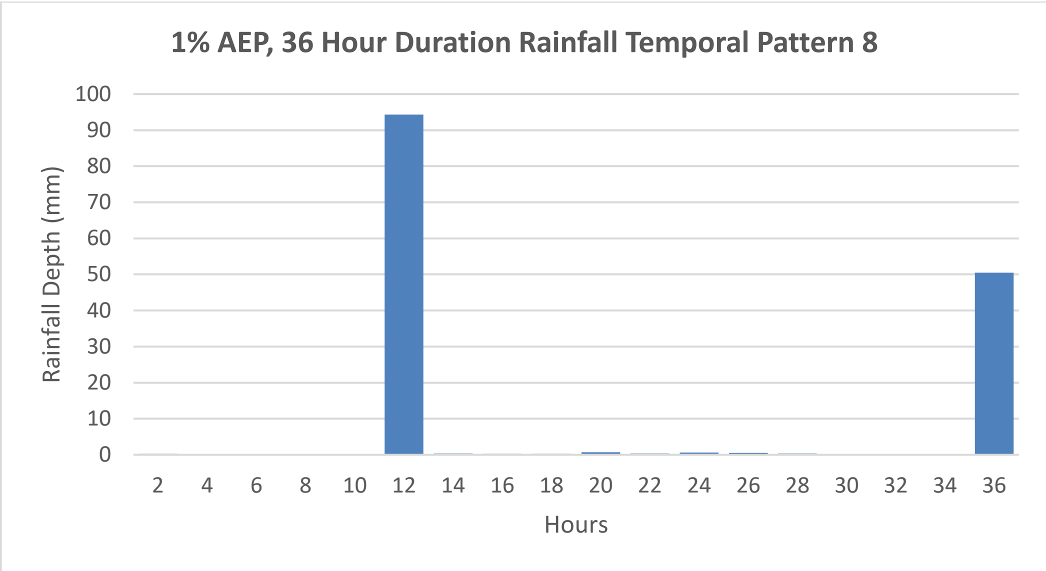


Figure 3-19 Rainfall temporal pattern 8 for the 36 hour duration, 1% AEP storm

As stated in *Addressing embedded bursts in design storms for flood hydrology* (Scorah et. al., 2016), "Censoring of temporal patterns which contain embedded bursts may be appropriate if the number of afflicted patterns is small." As the patterns with embedded bursts represent a small proportion of the total number of patterns available these embedded patterns were simply excluded from the modelling.

3.5.1.4 Design Spatial Pattern

As the catchment area was greater than $20km^2$ and the AEPs modelled were not rarer than the 1% AEP event, the method recommended in ARR, Book 2, Section 6.3 was used to determine the design spatial pattern. The IFDs at each subarea centroid were extracted. Based on a preliminary model run with a uniform spatial pattern, the critical duration for the entire Redesdale catchment was estimated to be 48 hours. Hence, the rainfall depth for each subarea corresponding to the 48-hour duration, 1% AEP storm was collated and used to determine the weighted average rainfall depth. The rainfall depths at each of the subareas were then divided by the weighted average rainfall depth to derive the non-dimensional spatial pattern. The spatial pattern used is shown in Table 3-13.

Table 3-13 Design spatial pattern for Campaspe River catchment to Redesdale

Subarea	Area (km^2)	Rainfall (48hr, 1% AEP) (mm)	Rainfall x Area	Pattern
A	39.2453	200	7849.1	120.81%
B	38.2727	183	7003.9	110.54%
C	29.4384	186	5475.5	112.35%
D	48.5321	182	8832.8	109.94%
E	49.0837	168	8246.1	101.48%
F	28.0527	166	4656.7	100.27%
G	46.0809	157	7234.7	94.84%
H	30.4072	158	4804.3	95.44%
I	47.6609	158	7530.4	95.44%
J	17.965	156	2802.5	94.23%
K	42.308	160	6769.3	96.65%
L	32.7072	156	5102.3	94.23%
M	46.0337	164	7549.5	99.06%
N	26.1505	156	4079.5	94.23%
O	31.0766	155	4816.9	93.63%
P	10.8315	155	1678.9	93.63%
Q	22.8799	153	3500.6	92.42%
R	18.6747	151	2819.9	91.21%
S	28.7242	153	4394.8	92.42%
T	8.48315	146	1238.5	88.19%
		Weighted Average Rainfall	**165.6**	

3.5.1.5 Simulation Parameters

The default stratified sample was used with 50 rainfall divisions and 20 samples per division. 70 time increments were modelled for each simulation. In accordance with the calibration, the model parameters used were k_c =62, m=0.80.

3.5.1.6 Design Losses

As recommended in ARR an Initial Loss/Continuing Loss model was applied to the RORB Monte Carlo analysis. To determine appropriate design loss values, a number of values were trialled and compared to the Campaspe River at Redesdale gauge flood frequency curve (see Section 3.2for the flood frequency analysis for the gauge). The losses that achieved peak flow values close to the gauge flood frequency curve were selected for use in the design flow modelling.

For all trials loss factors were constant and not varied. That is, the initial loss (IL) and continuing loss (CL) were not factored depending on AEP or duration of the event. However, the initial losses were selected stochastically. The default initial loss distribution in RORB is shown in Table 3-14 and shows the initial loss factors exceeded a given proportion of the time (ARR, Book 5, Chapter 3, Table 5.3.13).

Table 3-14 Initial loss distribution

Proportion of time value is exceeded	IL Factor
0%	3.190
10%	2.260
20%	1.710
30%	1.400
40%	1.200
50%	1.000
60%	0.850
70%	0.680
80%	0.530
90%	0.390
100%	0.140

The values trialled include the Data Hub recommended regional losses, the design losses applied in the Rochester Flood Management Plan (2013), and losses specifically fitted to the gauge flood frequency curve. The loss values are presented in the following sections and the model results are compared in Table 3-16.

- **<u>Data Hub Loss Values</u>**

The regional loss values obtained from Data Hub (Section 7.3.1) are shown below:

Storm Initial Loss = 28.0mm
Continuing Loss = 4.0mm/hr

It should be noted that the initial loss is relative to the complete storm and not only the critical design burst that is used in the RORB model. Hence, the storm initial loss must be converted to a burst initial loss as recommended in ARR, Book 2, Section 5.9.9, using the following equation:

Burst Initial Loss = Storm Initial Loss – Preburst

The median preburst depths for different AEPs and durations were obtained from Data Hub (Section 7.3.1) and are shown in Table 3-15 below.

Table 3-15 Median preburst depths (mm) for various flood AEPs and durations

Duration (hrs)	AEP %					
	50	20	10	5	2	1
1	2.2	2.0	1.9	1.8	1.5	1.3
1.5	3.1	2.8	2.6	2.5	1.9	1.5
2	3.0	2.6	2.3	2.0	1.9	1.9
3	3.5	3.2	3.1	2.9	5.1	6.7
6	1.4	1.6	1.8	1.9	5.0	7.3
12	0.3	2.8	4.4	6.0	7.5	8.7
18	0.3	1.4	2.1	2.8	4.6	6.0
24	0.0	1.1	1.8	2.6	3.1	3.4
36	0.0	0.1	0.1	0.2	0.4	0.6
48	0.0	0.0	0.0	0.1	0.0	0.0
72	0.0	0.0	0.0	0.0	0.0	0.0

The expected critical duration of the catchment (both Kyneton and Redesdale) is between 12-48 hours. Hence, a representative preburst depth of 5mm is selected, and the resulting applicable burst initial loss is 23mm.

- **<u>Rochester Flood Management Plan (2013) Loss Values</u>**

For comparison, the losses adopted in the Rochester Flood Management Plan (2013) are 20mm for the initial loss and 0.6mm/hr for the continuing loss.

- **<u>Loss Values Fitted to the Gauge Flood Frequency Curve</u>**

The results in Table 3-16 indicate that the continuing loss from Data Hub is too high. Furthermore, the design losses for the Rochester Flood Management Plan (2013) appear to be too low. Instead the design losses were derived using Monte Carlo analysis by adopting an initial loss of 23mm in accordance with Data Hub (stochastically sampled using the default RORB distribution) and adjusting the continuing loss to match the 50%-1% AEP flows to the flood frequency curve at the Campaspe River at Redesdale gauge.

Table 3-16 Comparison of flows at Campaspe River at Redesdale gauge for various design loss combinations

AEP (%)	Gauge Flood Frequency Curve (m^3/s)	Data Hub Losses IL = 23mm CL = 4.0mm/hr	Rochester Flood Management Plan (2013) IL = 20mm CL = 0.6mm/hr	Fitted Design Losses IL = 23mm CL = 0.6mm/hr	Fitted Design Losses IL = 23mm CL = 0.7mm/hr	Fitted Design Losses IL = 23mm CL = 1.0mm/hr	Difference*
50	89 (70-114)#	0.05	98.61	84.65	70.47	55.98	-4.9%
20	183 (151-231)	15.30	195.94	186.62	174.52	137.91	2.0%
10	252 (207-323)	39.70	271.14	260.25	251.78	215.98	-0.1%
5	318 (258-415)	74.94	345.95	337.89	324.46	285.71	2.0%
2	399 (318-556)	148.18	448.91	443.72	428.78	391.63	-1.8%
1	457 (355-668)	207.84	535.27	528.65	510.61	468.46	2.5%

#90% confidence interval shown in parentheses

*Note the percentage difference relates to the values highlighted in the table.

Using the results derived in Table 3-16 above, a final Monte Carlo analysis was run using the fitted continuing loss values which vary with AEP. The results are displayed in Table 3-17 below. The appropriate design losses to be applied for the different AEP events are as shown in Table 3-18.

Table 3-17 Comparison of Campaspe River at Redesdale gauge flood frequency curve to Monte Carlo analysis of design initial and continuing losses

AEP (%)	Gauge Flood Frequency Curve (m^3/s)	Fitted Design Losses IL = 23mm CL = 1.0mm/hr (2%-1%) CL = 0.7mm/hr (10%-5%) CL = 0.6mm/hr (50%-20%)	Difference
50	89 (70-114)#	71.91	-19.2%
20	183 (151-231)	181	-1.1%
10	252 (207-323)	247.73	-1.7%
5	318 (258-415)	316.85	-0.4%
2	399 (318-556)	404.28	1.3%
1	457 (355-668)	461.54	1.0%

#90% confidence interval shown in parentheses

Table 3-18 Adopted design initial and continuing losses

AEP	Initial Loss (mm)	Continuing Loss (mm/hr)
50% - 20%	23	0.6
10% - 5%	23	0.7
2% - 1%	23	1.0

3.5.2 Design Model Parameters

The RORB model has been calibrated to the downstream Campaspe River at Redesdale gauge using the September 2010, November 2010 and January 2011 historical events to determine the model parameters k_c and m. The design losses were calibrated by fitting the RORB Monte Carlo analysis results to the flood frequency curve at the same gauge. Using these values, a RORB Monte Carlo analysis was rerun with parameters specific to the Kyneton catchment including the applicable IFD rainfall data, spatial patterns and temporal patterns. The adopted design parameters are detailed below.

3.5.2.1 IFD

The IFD was obtained from the Bureau of Meteorology website for the Kyneton catchment centroid as opposed to the entire Redesdale catchment. Additional durations were added to the table to match the durations for which temporal patterns were available. A chart of the data downloaded from the Bureau of Meteorology displaying the IFD for events ranging from 63.2% AEP to 1% AEP is shown in Figure 3-20.

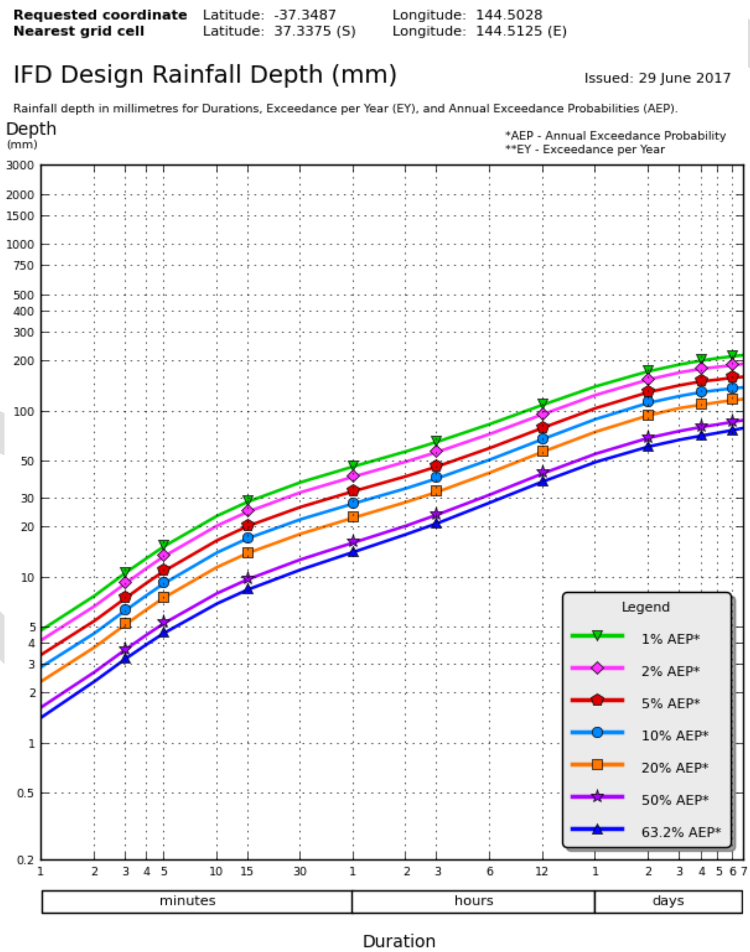


Figure 3-20 IFD graph for Kyneton catchment

Similar to the Redesdale IFD, the table was expanded by adding the depths of rare events. At the time of writing, rainfall depths for events from 1 in 200 to 1 in 2000 AEP were not available on the Bureau of Meteorology's website for durations less than 24 hours. Hence, the method recommended in ARR, Book 8, Section 3.6.3 for estimating very rare sub-daily rainfalls was used. Rainfall depths are determined by multiplying the relevant 1% AEP design rainfall depth by specific growth curve factors. ARR notes that due to the method used to derive these growth curve factors there may be the potential for significant discontinuity when compared to the values provided for durations of 24 hours and longer. As a result, it was necessary to smooth the growth factors to ensure the depths varied in a consistent manner across storm durations and exceedance probability. The growth curve factors were applied to the shortest durations and intermediary depths were smoothed between these values and those provided on the Bureau of Meteorology's website for 24-hour storms. A log graph displaying the smoothed results is shown in Figure 3-21.

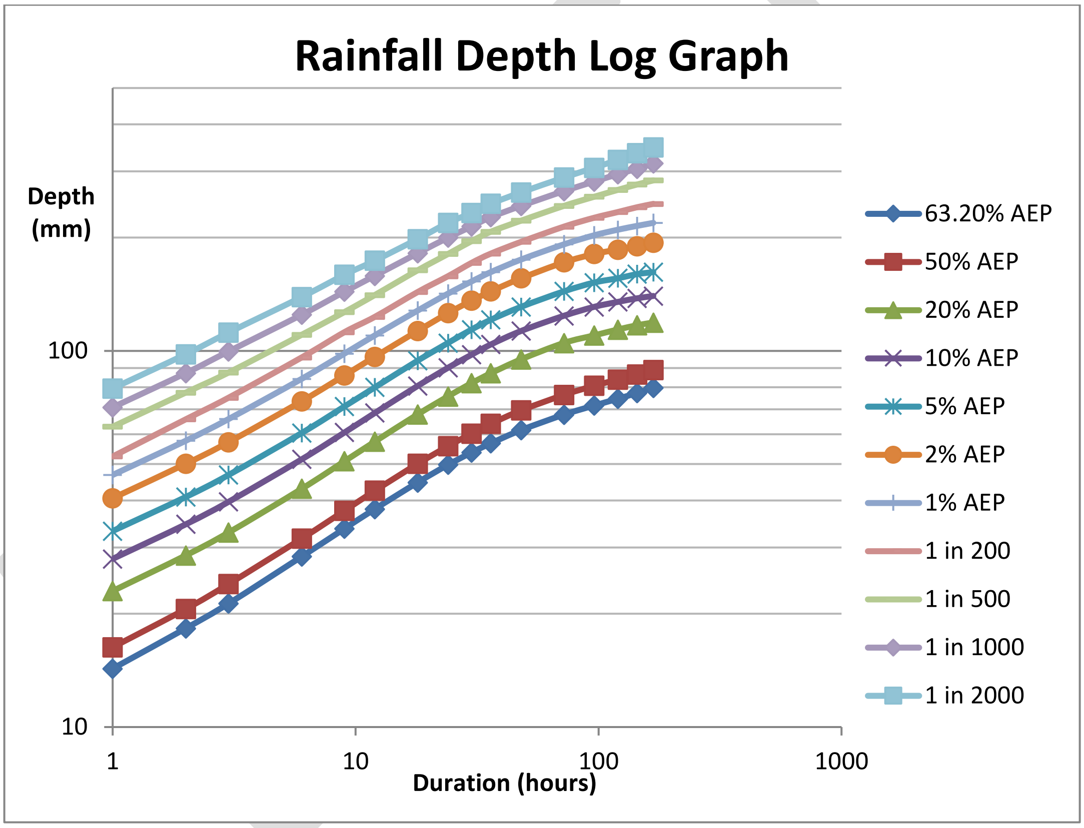


Figure 3-21 Log graph showing smoothing of depth-duration relationship for very rare rainfall events (1 in 200 to 1 in 2000) for Kyneton IFD

3.5.2.2 Areal Reduction Factor

The applicable ARF values were read into RORB directly from the Data Hub file. However, as the Kyneton catchment is smaller than the Rochester catchment the replacement option in RORB was used so that the ARF values were based on the Kyneton catchment (232.78km^2) rather than the Redesdale catchment. The Data Hub parameters are shown in Section 7.3.2.

3.5.2.3 Design Temporal Pattern

The applicable design temporal patterns were obtained from Data Hub (Section 7.3.2). Areal patterns were applied as the catchment is greater than $75km^2$.

The temporal pattern sample is selected based on the closest area. It should be noted that the temporal patterns applied will be different to those used in the model of the entire Redesdale catchment due to the difference in catchment size. The applicable catchment area selected for Kyneton was $200km^2$.

Areal temporal patterns were available for following storm durations: 12, 18, 24, 36, 48, 72, 96, 120, 144, and 168. For each duration there are ten different temporal patterns, resulting in a total of 100 patterns available for modelling.

The temporal patterns have been assessed to determine if any contain embedded bursts which would cause the RORB model to overestimate the peak flows. This was done by comparing the sub-period rainfall totals of a particular temporal pattern against the IFD to determine whether it is rarer than the AEP of the entire burst. The analysis revealed that four temporal patterns contained embedded bursts; pattern 4 from the 72-hour duration storm, pattern 2 and 5 from the 96-hour duration storm, and pattern 3 from the 120-hour storm. For example, Figure 3-22 shows pattern 5 from the 96-hour duration which contains an embedded rainfall burst rarer than a 1 in 1000 AEP event.

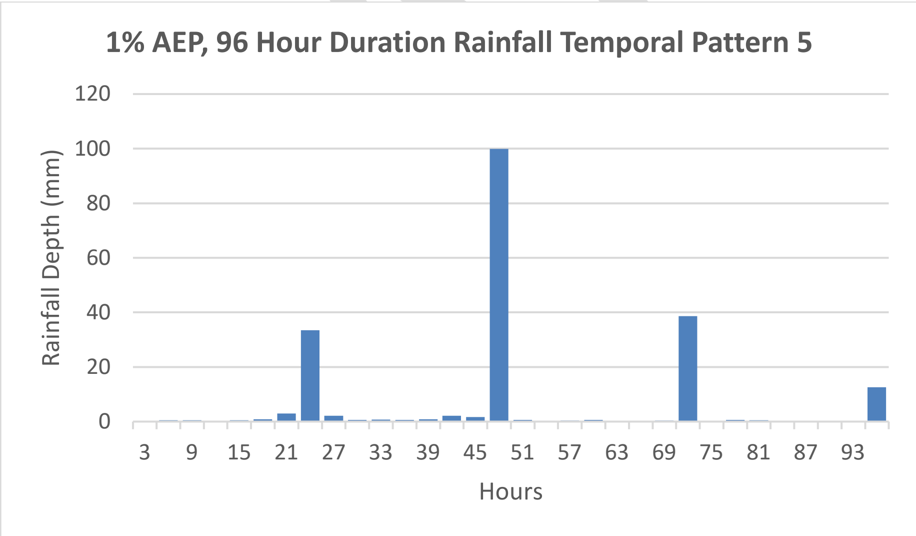


Figure 3-22 Rainfall temporal pattern 5 for the 96-hour duration, 1% AEP storm

As stated in *Addressing embedded bursts in design storms for flood hydrology* (Scorah et. al., 2016), "Censoring of temporal patterns which contain embedded bursts may be appropriate if the number of afflicted patterns is small." As the patterns with embedded bursts represent a small proportion of the total number of patterns available these embedded patterns were simply excluded from the modelling.

3.5.2.4 Design Spatial Pattern

The design spatial pattern for the Kyneton catchment was derived in a similar method to that used for the Redesdale catchment in Section 3.5.1.4. The catchment area was greater than $20km^2$ and the AEPs modelled were not rarer than the 1% AEP event, hence the method recommended in ARR, Book 2, Section 6.3 was applicable. The IFDs at each subarea centroid in the Kyneton catchment were extracted. Based on a preliminary model run with a uniform spatial pattern, the critical duration for the Kyneton catchment was estimated to be 18 hours. Hence, the rainfall depth for each subarea corresponding to the 18-hour duration, 1% AEP storm was collated and used to determine the weighted average rainfall depth. The rainfall depths at each of the subareas were then divided by the weighted average rainfall depth to derive the non-dimensional spatial pattern. The spatial pattern used is shown in Table 3-19.

Table 3-19 Design spatial pattern for Campaspe River catchment to Kyneton

Subarea	Area (km^2)	Rainfall (18hr, 1%AEP) (mm)	Rainfall x Area	Pattern
A	39.2453	138	5415.9	106.16%
B	38.2727	131	5013.7	100.78%
C	29.4384	132	3885.9	101.55%
D	48.5321	131	6357.7	100.78%
E	49.0837	124	6086.4	95.39%
F	28.0527	124	3478.5	95.39%
		Weighted Average Rainfall	**130.0**	

3.5.2.5 Simulation Parameters

The default stratified sample was used with 50 rainfall divisions and 20 samples per division. 70 time increments were modelled for each simulation.

In accordance with the calibration, the parameters used were k_c =62, m=0.80.

3.5.2.6 Design Losses

The design losses used were as determined in the analysis in Section 3.5.1. Hence, an initial loss of 23mm was selected. The Monte Carlo analysis included stochastic selection of the initial loss, with a mean value of 23mm, using the default RORB distribution shown in Table 3-20.

Table 3-20 Initial loss distribution

Proportion of time value is exceeded	IL Factor
0%	3.190
10%	2.260
20%	1.710
30%	1.400
40%	1.200
50%	1.000
60%	0.850
70%	0.680
80%	0.530
90%	0.390
100%	0.140

The design continuing loss varied with AEP. The continuing loss was set as 1.0 mm/hr and varied with AEP in accordance with the factors shown in Table 3-21.

Table 3-21 Continuing loss AEP factors

AEP	Continuing Loss Factor
63.2%	0.6
50%	0.6
20%	0.6
10%	0.7
5%	0.7
2%	1.0
1%	1.0
1 in 200	1.0
1 in 500	1.0
1 in 1000	1.0
1 in 2000	1.0

3.5.2.7 Baseflow

As discussed in Section 3.4.2.2, baseflow in this catchment is insignificant. Hence, no allowance for baseflow has been added to the design hydrographs due to there being a negligible impact on the design flood hydrograph.

3.5.3 Design Flow Results

3.5.3.1 Monte Carlo Analysis

The design parameters detailed above were used to undertake a Monte Carlo simulation for the Kyneton catchment. The critical storm duration for the Kyneton catchment was determined to be 24 hours. The results of the Monte Carlo flood frequency analysis (FFA) are shown in column 2 of Table 3-22. These flows were generated just downstream of Piper Street, Kyneton, and labelled 'Kyneton Downstream'. The individual design runs used for the Monte Carlo analysis were then assessed to determine which provided the most similar peak flow to the Monte Carlo FFA while still utilising reasonable parameters. The design parameters adopted for these particular runs are displayed in columns 3 to 9 of Table 3-22.

These run parameters were used to generate the complete hydrographs for the design floods ranging from the 50% - 0.5% AEP events. The design hydrographs are shown in Figure 3-23 below. It should be noted that the areal reduction factor (ARF) was not input into the individual design hydrograph runs as this factor is already incorporate into the rainfall depth parameter for each simulation run in the Monte Carlo analysis.

Table 3-22 Individual design runs from the Monte Carlo analysis

AEP	Peak Flow from MC FFA at Kyneton Downstream (m^3/s)	Run	Rainfall ARI	Rainfall Depth (mm)	Temporal Pattern	IL Stochastic Factor	CL AEP Factor	Run Peak Flow (m^3/s)
50%	55.52	24hr, Div 4, Run 8	1.9	51	5	1.06	0.6	55.65
20%	119.23	24hr, Div 14, Run 5	3.8	65.8	2	0.99	0.6	119.29
10%	162.45	24hr, Div 23, Run 17	9.6	84.3	2	1.29	0.69	162.72
5%	211.37	24hr, Div 29, Run 2	23.3	102	8	1.15	0.73	211.17
2%	261.16	24hr, Div 32, Run 8	37.7	112.2	6	0.56	0.88	261.50
1%	299.00	24hr, Div 37, Run 5	97.6	132.5	7	1.37	1	296.31
0.5%	363.64	24hr, Div 41, Run 6	216.6	149.5	10	2	1	364.05

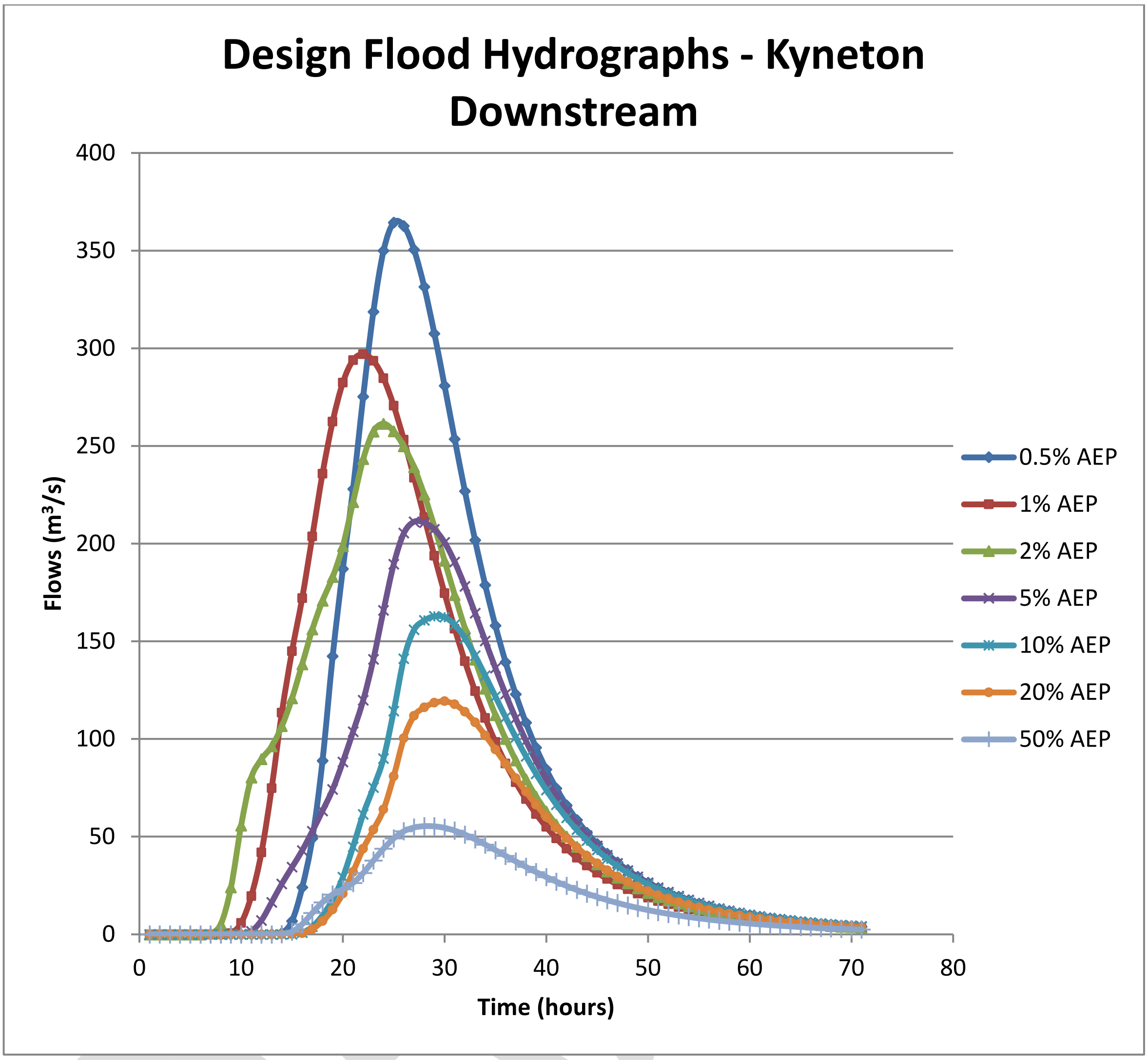


Figure 3-23 **Design flood hydrographs at Kyneton Downstream**

3.5.3.2 Ensemble Analysis

An ensemble assessment of the temporal patterns for the 1% AEP event was also undertaken for comparison with the Monte Carlo analysis. Ensemble analysis is generally used to determine the applicable temporal pattern to be applied to generate the design hydrographs. Ten areal temporal patterns for each storm duration were assessed. The results are presented in the box plot shown in Figure 3-24. The box plot shows that the 24-hour duration is critical as it has the highest mean flow. The temporal pattern that yielded the peak flow closest to the mean 24-hour storm peak flow of 309.2m^3/s was adopted as the design temporal pattern and is used to generate the design hydrograph for the ensemble analysis. The applicable temporal pattern was '6' which produced a peak flow of 305.4m^3/s.

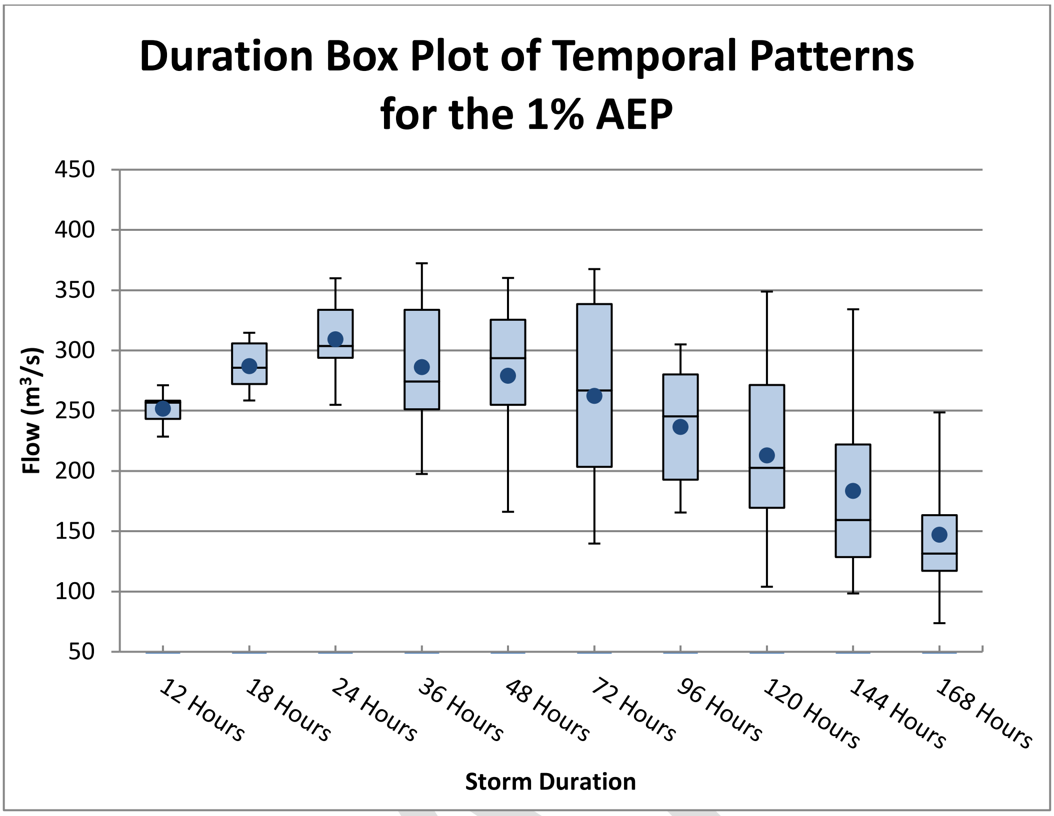


Figure 3-24 Duration box plot of temporal patterns for the 1% AEP design event. Note the mean peak flow for each duration is displayed as a blue dot.

Figure 3-25 below compares the 1% AEP design hydrographs produced from the ensemble analysis and the Monte Carlo analysis. The ensemble hydrograph peak flow is 2.8% higher than the Monte Carlo hydrograph peak flow and has a volume 9.5% greater than the Monte Carlo volume. The time to peak of the ensemble hydrograph is 2 hours behind the Monte Carlo hydrograph. Overall, the similarity between the results of the two types of analysis improves the confidence in the Monte Carlo analysis results.

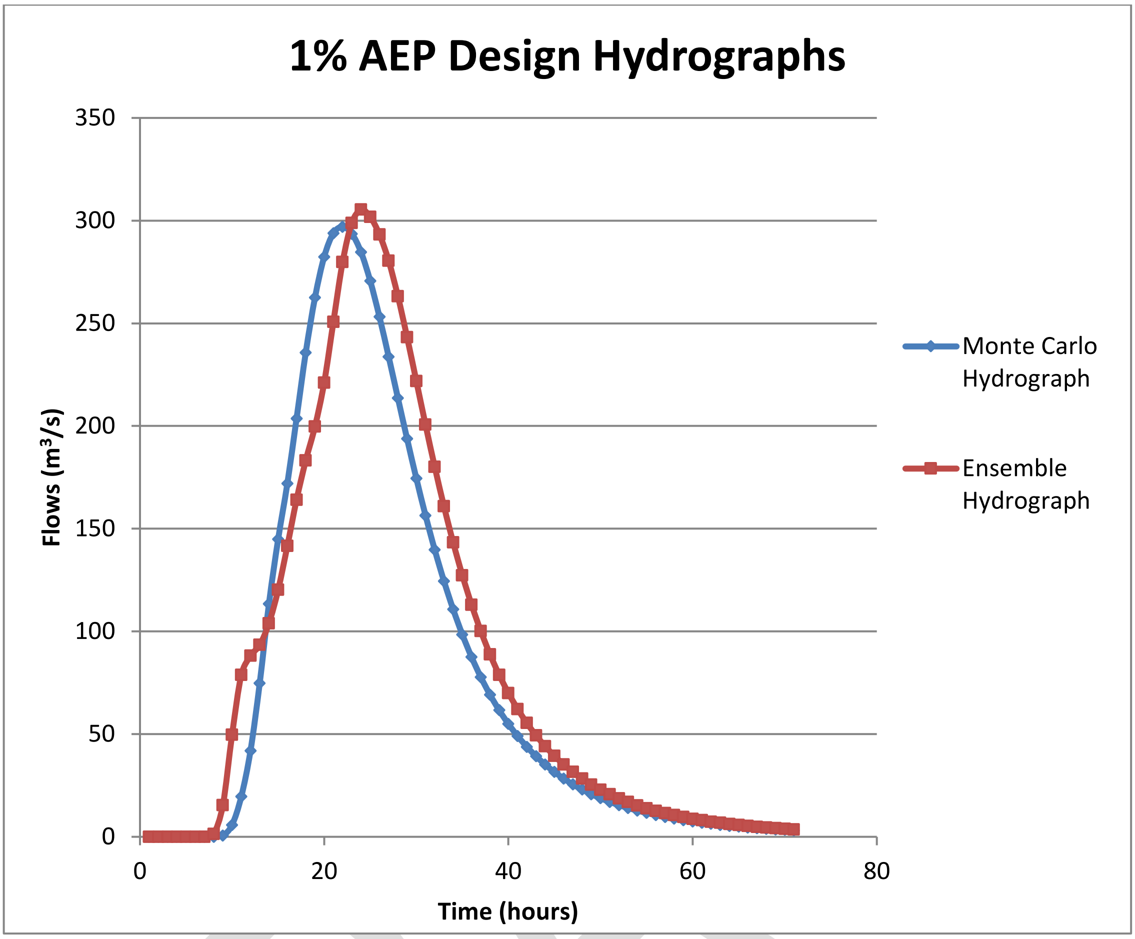


Figure 3-25 Comparison of 1% AEP design hydrographs using ensemble and Monte Carlo analysis

3.5.4 Summary

From the Monte Carlo analysis, the critical storm duration was determined to be 24 hours for the Kyneton catchment. The parameters used to generate the individual design hydrographs for the 50% - 0.5% AEP flood events are shown in Table 3-22 above. The corresponding hydrographs are shown in Figure 3-23 above.

The hydrographs for the hydraulic model were required to be input further upstream of Kyneton Township. Therefore, additional hydrograph print-out locations, labelled Campaspe at Carlsruhe, Carlsruhe Tributary and Subarea F, were added to the RORB model as shown in Figure 3-26 below. The design parameters used to produce these hydrographs were the same as those adopted to generate the design hydrographs at Kyneton as this produces critical flows through township. The design hydrographs for these three locations are shown in Figure 3-27 to Figure 3-29 below.

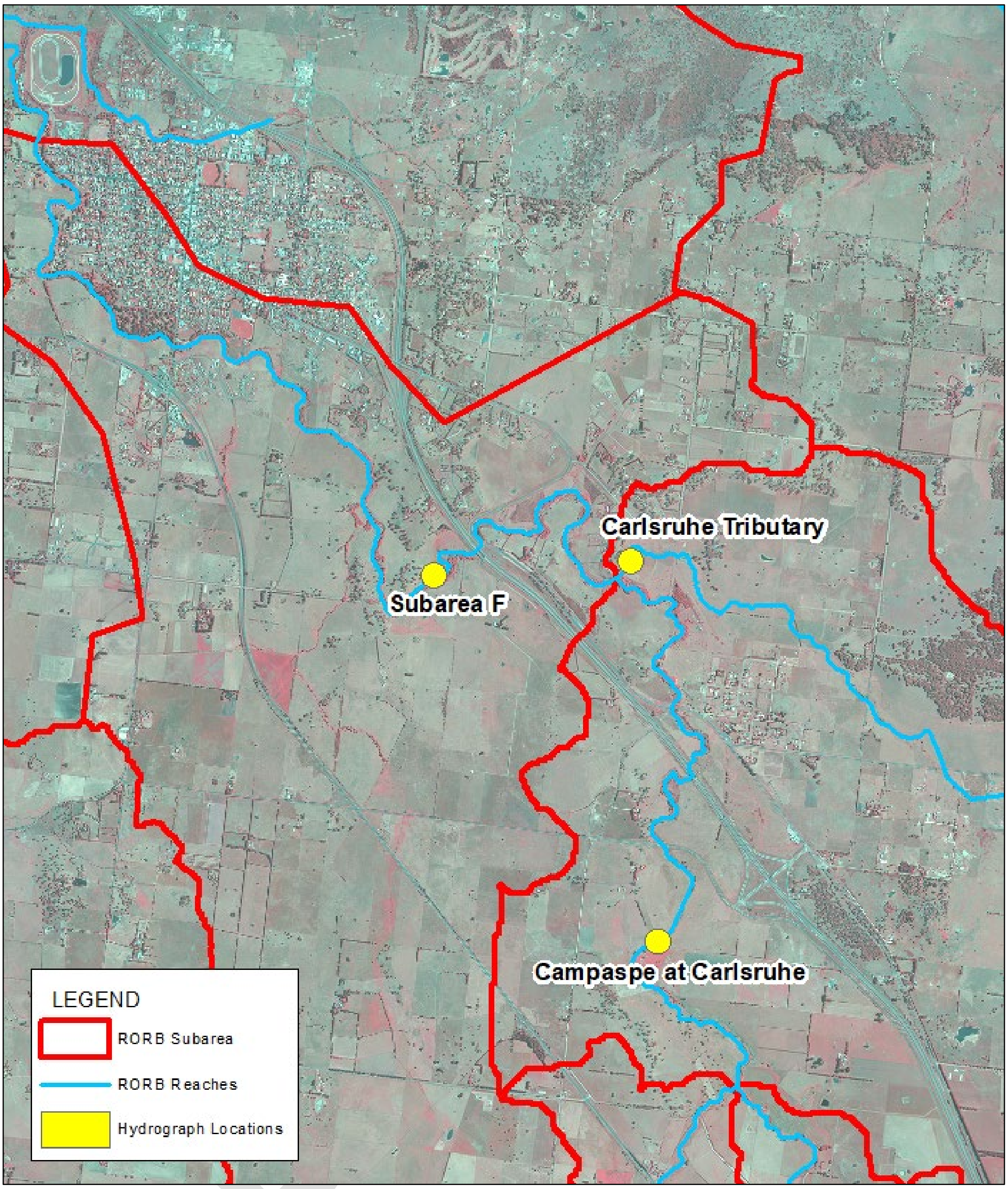


Figure 3-26 Design hydrograph locations for the hydraulic model

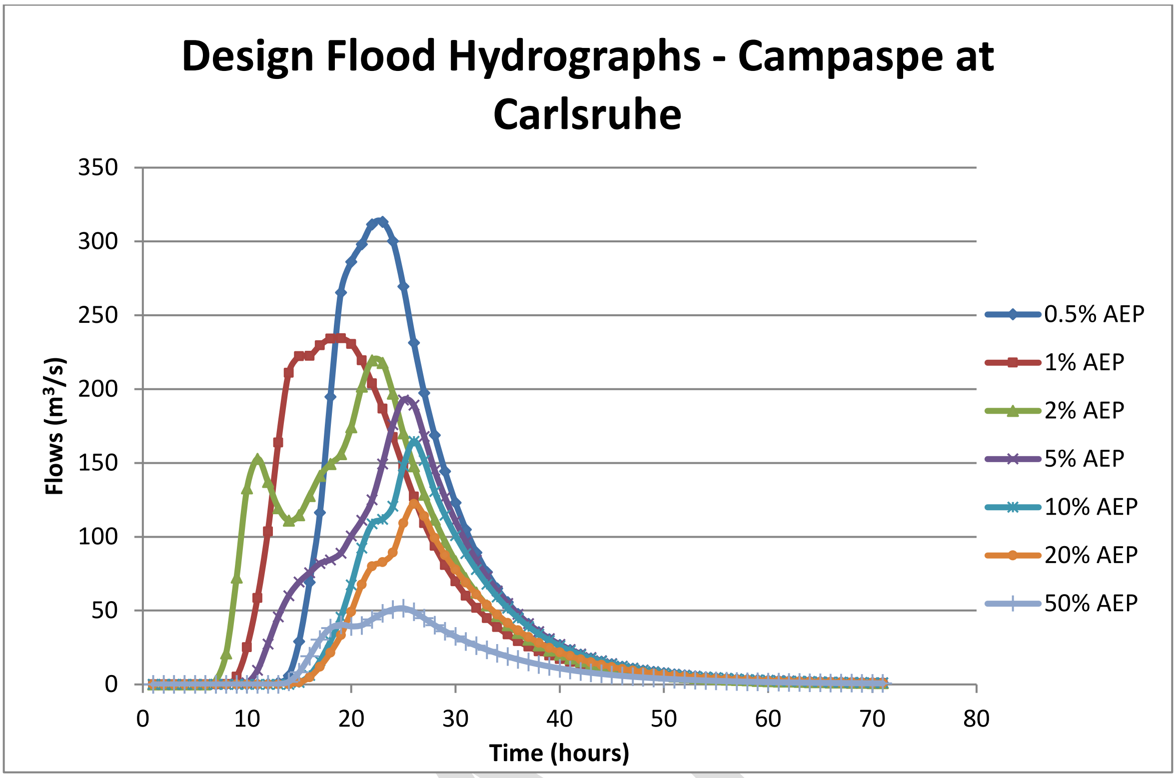


Figure 3-27 Design flood hydrographs at Campaspe at Carlsruhe

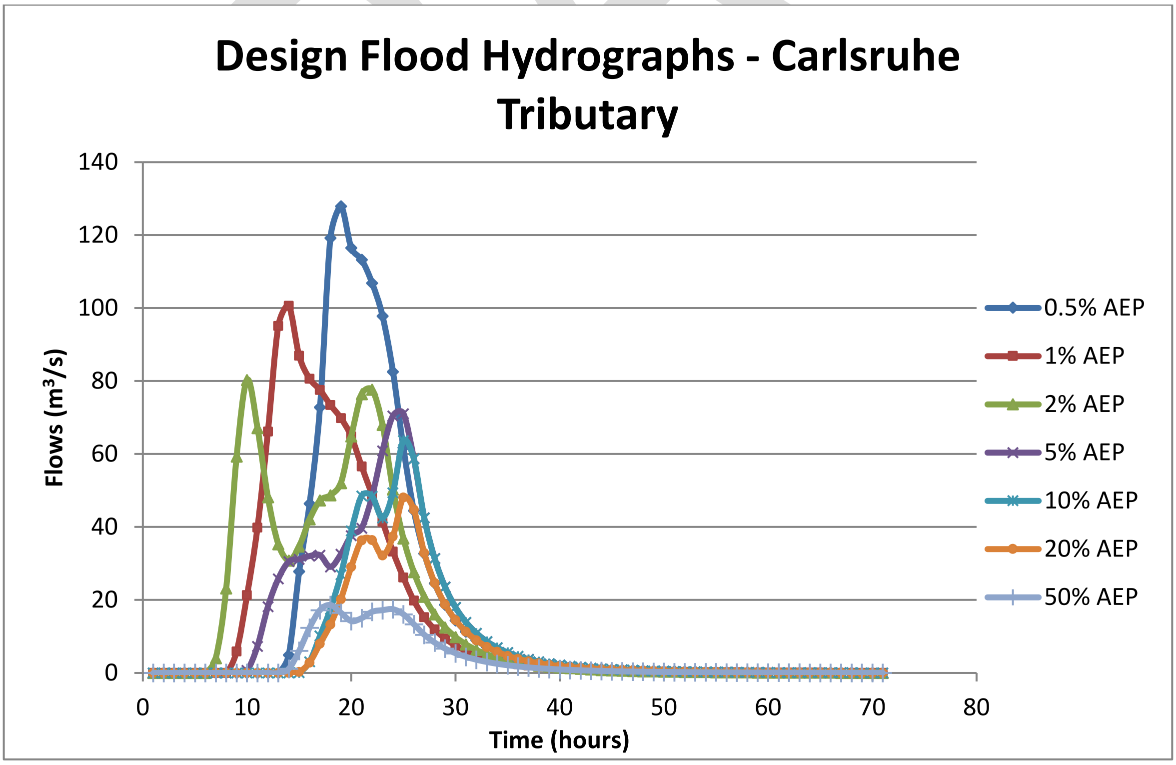


Figure 3-28 Design flood hydrographs at Carlsruhe Tributary

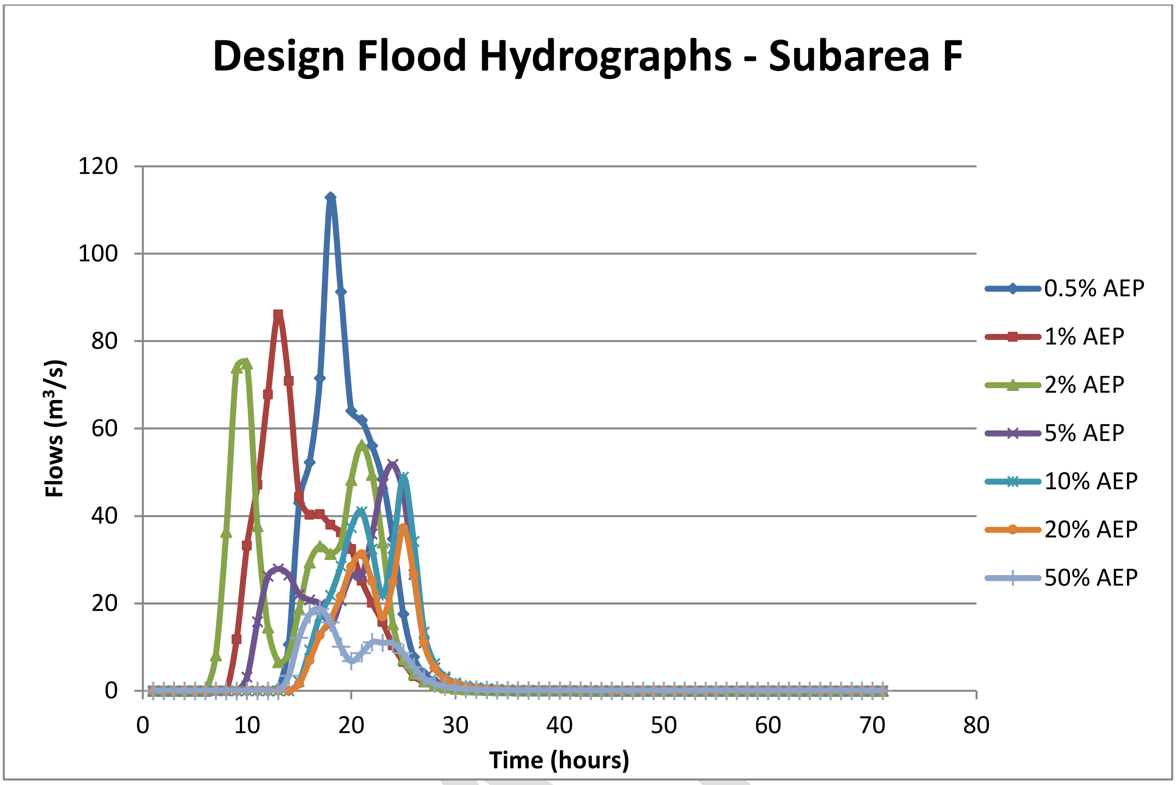


Figure 3-29 Design flood hydrographs from Subarea F

3.6 Post Office Creek RORB Model Calibration

3.6.1 Overview

Due to the limited data available, the RORB model for Post Office Creek was unable to be calibrated to observed streamflow data. However, although there is no gauge to calibrate the model to, a flood level was recorded on Post Office Creek during the January 2011 flood event. Furthermore, there were numerous photographs and videos taken during recent floods in September 2010, January 2011 and September 2016. Hence, the RORB model was utilised to produce hydrographs based on the historical rainfall. The hydrographs were then input into the TUFLOW model and the results were compared to the recorded flood mark and available photographs to calibrate the hydraulic model. This section describes the hydrological parameters selected for the RORB calibration model in order to generate hydrographs on Post Office Creek for the following historical events: September 2010, January 2011 and September 2016. Section 4.3 describes the hydraulic calibration using these historical hydrographs.

3.6.2 Routing Parameter

The nonlinearity parameter, *m,* was set to 0.8 in accordance with the Campaspe River RORB model. The routing parameter, k_c, was estimated for the ungauged Post Office Creek catchment by comparison with regional relationships and relating it to previous models undertaken within the area.

Similar to the Campaspe River RORB model, the parameters were not varied by interstation areas, and therefore a single routing parameter was adopted for the catchment. It should be noted that the k_c parameter determined in the Campaspe River RORB model and the Rochester Flood Management Plan (2013) RORB models could not be directly adopted for this flood study model as the catchment size and shape was not comparable. However, the parameter was scaled in order to make this comparison.

The various k_c estimation techniques are detailed in Table 3-23. This included the regional equations for Victoria as recommended in ARR. Note that the catchment area (A) referred to in the estimation equations is 12.07km^2. Furthermore, d_{av} provides an indication of the travel distance to the outlet of the RORB model, and is given by the weighted average flow distance from all nodes to the catchment outlet. The value of d_{av} obtained from the RORB model was 3.02km.

Table 3-23 Comparison of routing parameter estimates

Method	Applicable Region	Equation	Predicted k_c
RORB Default Equation (RORB Manual, Equation 2-5)	Australia wide	$k_c = 2.2*A^{0.5}*(Q_p/2)^{0.8-m}$	7.64
Regional Equation (ARR, Book 5, Equation 3.22)	For areas where annual rainfall <800mm	$k_c = 0.49*A^{0.65}$	2.47
Regional Equation (ARR, Book 5, Equation 3.21)	For areas where annual rainfall >800mm	$k_c = 2.57*A^{0.45}$	7.88
Victorian Data (Pearse et al. 2002)	Victoria	$k_c = 1.25*d_{av}$	3.78
Australian wide Dyer (1994) Data (Pearse et al. 2002)	Australia wide	$k_c = 1.14*d_{av}$	3.45
Australian wide Yu (1989) Data (Pearse et al. 2002)	Australia wide	$k_c = 0.96*d_{av}$	2.90
Comparison to Campaspe River @ Redesdale RORB model ($k_c/d_{av} = 1.276$)	Campaspe catchment	$k_c = 1.276*d_{av}$	3.86
Comparison to Rochester Flood Management Plan (2013) ($k_c/d_{av} = 1.278$)	Campaspe catchment	$k_c = 1.278*d_{av}$	3.87

As shown in Table 3-23, a comparison with the routing parameters adopted for the Campaspe River and Rochester Flood Management Plan (2013) RORB models was also undertaken by considering the ratio between the routing parameter, k_c, and the weighted average flow distance from all nodes to the catchment outlet, d_{av}. For example, the weighted average flow distance for the Campaspe River RORB model was 48.6km and a value of 62.0 was adopted for k_c, resulting in a k_c/d_{av} ratio of 1.276. Applying this same ratio to the weighted average flow distance of the Kyneton RORB model, 3.02km, indicates a k_c value of 3.86.

Based on a review of the routing parameter estimates, the RORB model parameters shown in Table 3-24 were adopted.

Table 3-24 Adopted RORB model parameters

k_c	m
3.9	0.8

3.6.3 Fraction Impervious

The fraction impervious values for the RORB subareas were based on the planning zones as described in Section 3.3.2. For the calibration, these values were further refined based on the aerial photography of the area taken in 2010.

3.6.4 Observed Rainfall Data

Due to the small catchment size a sub-hourly pluviograph was required. The temporal rainfall distribution for the September 2010 and September 2016 events were sourced from the closest pluviograph station, 406266, which provided data in 15-minute intervals. This station was also used to derive hourly rainfall data for the calibration of the Campaspe River RORB model as described in Section 3.4.2.3.

Data from pluviograph station 406266 was also initially used for the January 2011 event. However, a concentrated high intensity rainfall in the sub-hourly temporal pattern produced an unrealistic peak flow, significantly larger than the estimated 1 in 200 AEP design event (Section 3.7.2), which was not experienced based on the available anecdotal, photographic and recorded flood level information. Hence, the historical temporal pattern from the next closest pluviograph station, 406250, was applied. This station was located a similar distance from the Post Office Creek catchment and produced flows that were more aligned with the evidence available from this event. A comparison of the two pluviograph records is shown in Figure 3-30.

The accumulated rainfall totals for each event were adopted from the data used for subarea G of the Kyneton RORB model as the Post Office Creek catchment covers a significant portion of this subarea.

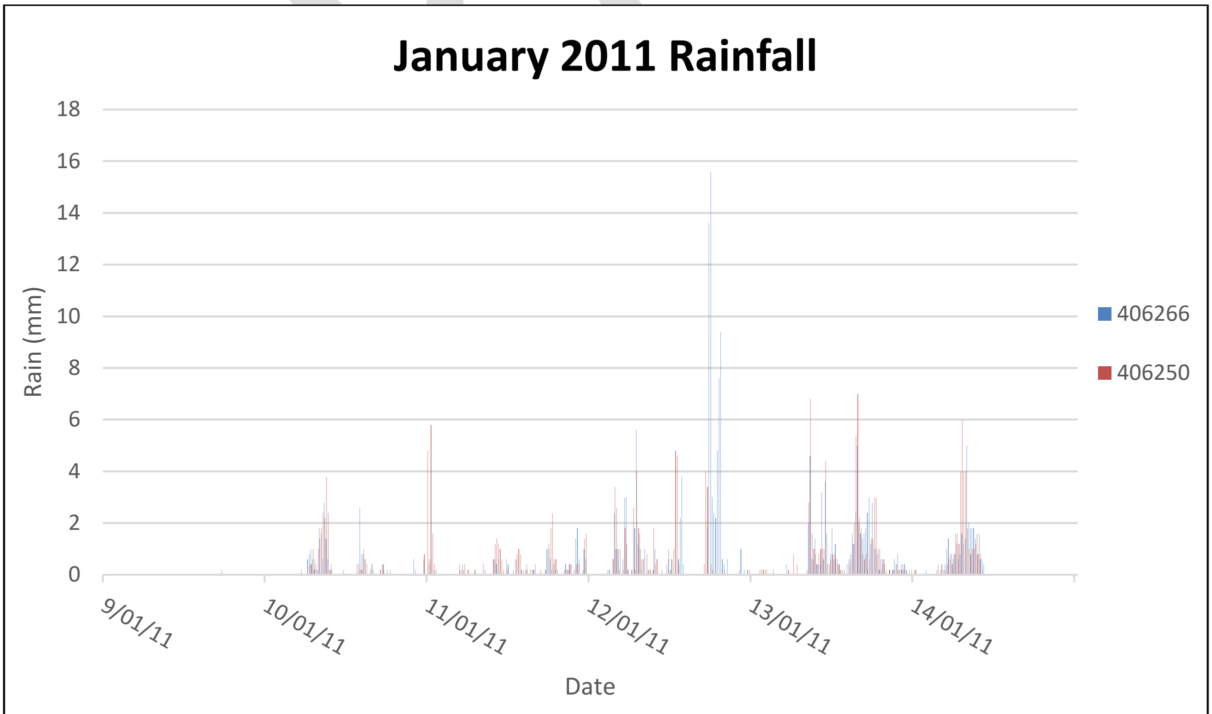


Figure 3-30 Pluviograph record (15-minute intervals) for the January 2011 event

3.6.5 Loss Model

An initial loss/continuing loss model was adopted for the Post Office Creek RORB model. The initial loss and continuing loss values were adopted based on the Campaspe River RORB calibration for the historical events as described in Section 3.4.6.2. As the September 2016 event could not be calibrated to the Campaspe River at Redesdale gauge the losses were selected based on the design losses determined in Section 3.5.1.6. An initial loss of 23mm and a continuing loss of 0.6mm/hour was adopted as the September 2016 event was estimated to be close to a 20% AEP event in Kyneton (Section 3.7.2.1). It should be noted that the peak flow for this event is not sensitive to the initial loss adopted due to the significant volume of rain occurring prior to the peak. The losses adopted for each historical event are shown in Table 3-25 to Table 3-27.

Table 3-25 Adopted losses for Post Office Creek RORB model – September 2010

IL (mm)	CL (mm/hr)
10	0.63

Table 3-26 Adopted losses for Post Office Creek RORB model – January 2011

IL (mm)	CL (mm/hr)
85	2.61

Table 3-27 Adopted losses for Post Office Creek RORB model – September 2016

IL (mm)	CL (mm/hr)
23	0.60

3.6.6 Hydrographs for Calibration

Using the parameters described in the preceding sections, hydrographs were generated for the following historical events: September 2010, January 2011 and September 2016. These Post Office Creek hydrographs are shown in Section 4.3 along with the Campaspe River historical hydrographs generated at the inflow locations to the hydraulic model. The hydrographs were then input into the TUFLOW model to replicate these four historical flood events and the model results were compared to the available historical data as described in Section 4.3.

3.7 Post Office Creek Design Event Modelling

The section details the process used to determine appropriate design parameters and flows for the 20%, 10%, 5%, 2%, 1% and 0.5% AEP events for Post Office Creek at Kyneton.

3.7.1 Design Model Parameters

In order to produce appropriate design hydrographs a Monte Carlo analysis was run with model parameters specific to the Post Office Creek catchment including temporal patterns, IFD and ARF values. The parameters adopted are detailed below.

3.7.1.1 Fraction Impervious

The fraction impervious values for the RORB subareas were based on the planning zones as described in Section 3.3.2. Unlike the calibration, these values were not refined using aerial photography as it represents the runoff potential based on future development in accordance with the planning scheme.

3.7.1.2 IFD

The IFD was obtained from the Bureau of Meteorology website for the Post Office Creek catchment. Additional durations were added to the table to match the durations for which temporal patterns were available.

The IFD table was expanded by adding the depths of rare events. At the time of writing, rainfall depths for events from 1 in 200 to 1 in 2000 AEP were not available on the Bureau of Meteorology's website for durations less than 24 hours. Hence, the method recommended in ARR, Book 8, Section 3.6.3 for estimating very rare sub-daily rainfalls was used. Rainfall depths were determined by multiplying the relevant 1% AEP design rainfall depth by specific growth curve factors. ARR notes that due to the method used to derive these growth curve factors there may be the potential for significant discontinuity when compared to the values provided for durations of 24 hours and longer. As a result, it was necessary to smooth the growth factors to ensure the depths varied in a consistent manner across storm durations and exceedance probability. The growth curve factors were applied to the shortest durations and intermediary depths were smoothed between these values and those provided on the Bureau of Meteorology's website for 24-hour storms. A log graph displaying the smoothed results is shown in Figure 3-31.

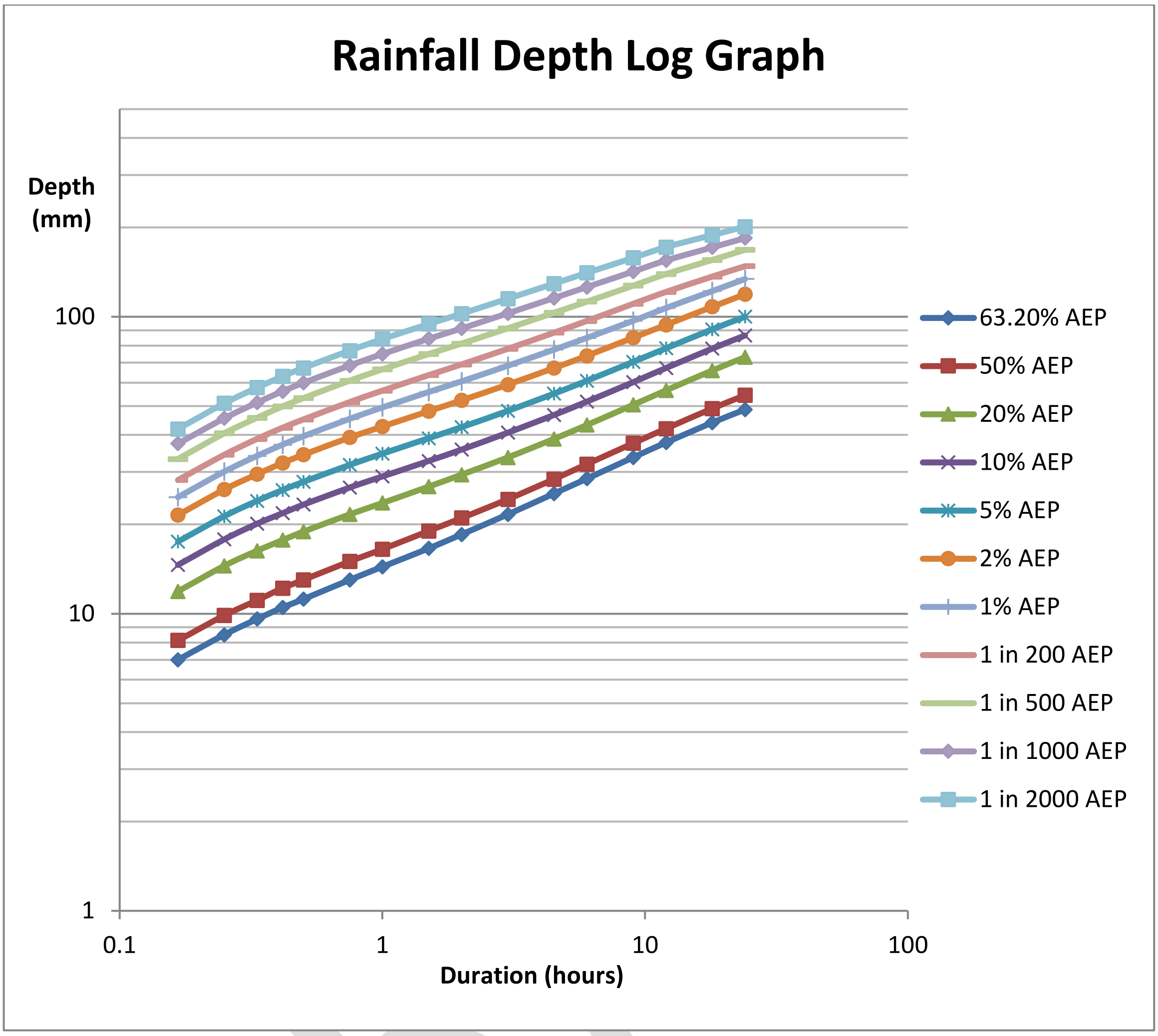


Figure 3-31 **Log graph showing smoothing of depth-duration relationship for very rare rainfall events (1 in 200 to 1in 2000) for Post Office Creek IFD**

3.7.1.3 Areal Reduction Factor

The applicable ARF values were read into RORB from the Data Hub file for Post Office Creek. The Data Hub parameters are shown in Section 7.3.3.

3.7.1.4 Design Temporal Pattern

The design temporal patterns were obtained from Data Hub (Section 7.3.3). As the catchment is less than 75km^2 in size (12.07km^2), point temporal patterns were applied, as recommended in ARR, Book 2, Section 5.9.1. Point temporal patterns are available for the following storm durations: 10, 15, 20, 25, 30, 45, 60 mins, 1.5, 2, 3, 4.5, 6, 9, 12, 18, 24, 30, 36, 48, 72, 96, 120, 144, 168 hours. For each duration there are 30 different temporal patterns: 10 each for frequent, intermediate and rare events. Hence, in total there are 720 patterns available for modelling.

The temporal patterns have been assessed to determine if any contain embedded burst which would cause the RORB model to overestimate the peak flows. This was done by comparing the sub-period rainfall totals of a particular temporal pattern against the IFD to determine whether it is rarer than the AEP of the entire burst. The analysis revealed that two temporal patterns contained embedded bursts;

- pattern 5 from the rare, 24-hour duration storms.
- pattern 7 from the intermediate, 24-hour duration storms.

For example, pattern 5 from the rare, 24-hour duration temporal patterns for the 1% AEP event contained an embedded rainfall burst which was between a 1 in 200 AEP and 1 in 500 event.

As stated in *Addressing embedded bursts in design storms for flood hydrology* (Scorah et. al., 2016), "Censoring of temporal patterns which contain embedded bursts may be appropriate if the number of afflicted patterns is small." As the patterns with embedded bursts represent a small proportion of the total number of patterns available these embedded patterns were simply excluded from the modelling.

3.7.1.5 Design Spatial Pattern

As the catchment area is less than 20km^2 a uniform spatial pattern was applied as recommended in ARR, Book 2, Section 6.3.1.

3.7.1.6 Simulation Parameters

The default stratified sample was used with 50 rainfall divisions and 20 samples per division. 70 time increments were modelled for each simulation.

In accordance with the calibration, the parameters used were k_c=3.9, m=0.80.

3.7.1.7 Design Losses

An initial loss/continuing loss model was used for the Monte Carlo analysis. The design initial and continuing losses determined for the Campaspe River RORB model were also adopted for Post Office Creek. The values are shown in Table 3-28 below. Additionally, the initial loss was stochastically selected from the default RORB distribution shown in Table 3-29.

Table 3-28 Adopted design initial and continuing losses

AEP	Initial Loss (mm)	Continuing Loss (mm/hr)
50% - 20%	23	0.6
10% - 5%	23	0.7
2% - 1%	23	1.0

Table 3-29 Initial loss distribution

Proportion of time value is exceeded	IL Factor
0%	3.190
10%	2.260
20%	1.710
30%	1.400
40%	1.200
50%	1.000
60%	0.850
70%	0.680
80%	0.530
90%	0.390
100%	0.140

3.7.1.8 Baseflow

As discussed in Section 3.4.2.2, baseflow in this catchment is insignificant. Hence, no allowance for baseflow has been added to the design hydrographs due to there being a negligible impact on the design flood hydrograph.

3.7.2 Design Flow Results

3.7.2.1 Monte Carlo Analysis

The design parameters detailed above were used to undertake a Monte Carlo simulation for the Post Office Creek catchment. The critical storm duration for the Post Office Creek catchment was determined to be 12 hours. The results of the Monte Carlo flood frequency analysis (FFA) are shown in column 2 of Table 3-30. The individual design runs used for the Monte Carlo analysis were then assessed to determine which provided the most similar peak flow to the Monte Carlo FFA while still utilising reasonable parameters. The design parameters adopted for these particular runs are shown in columns 3 to 9 of Table 3-30. These run parameters were used to generate the complete hydrographs for the design floods ranging from 50% - 0.5% AEP events. The design hydrographs are shown in Figure 3-32 below.

Table 3-30 Individual design runs from Monte Carlo analysis

AEP	Peak Flow from MC FFA (m^3/s)	Run	Rainfall ARI	Rainfall Depth (mm)	Temporal Pattern	IL Stochastic Factor	CL AEP Factor	Run Peak Flow (m^3/s)
50%	7.43	12hr, Div 1, Run 16	1.6	36.8	2	0.76	0.6	7.41
20%	17.84	12hr, Div 16, Run 6	4.6	54.1	10	1.25	0.6	17.84
10%	23.49	12hr, Div 23, Run 16	8.2	62.7	12	0.64	0.66	23.70
5%	29.72	12hr, Div 27, Run 4	18.4	74.9	19	1.7	0.70	30.16
2%	38.41	12hr, Div 33, Run 18	48.0	90.4	29	0.47	0.98	38.47
1%	44.07	12hr, Div 36, Run 12	80.1	99.5	26	1.17	1.0	44.14
0.5%	52.03	12hr, Div 40, Run 20	185.7	115.4	26	1.03	1.0	52.35

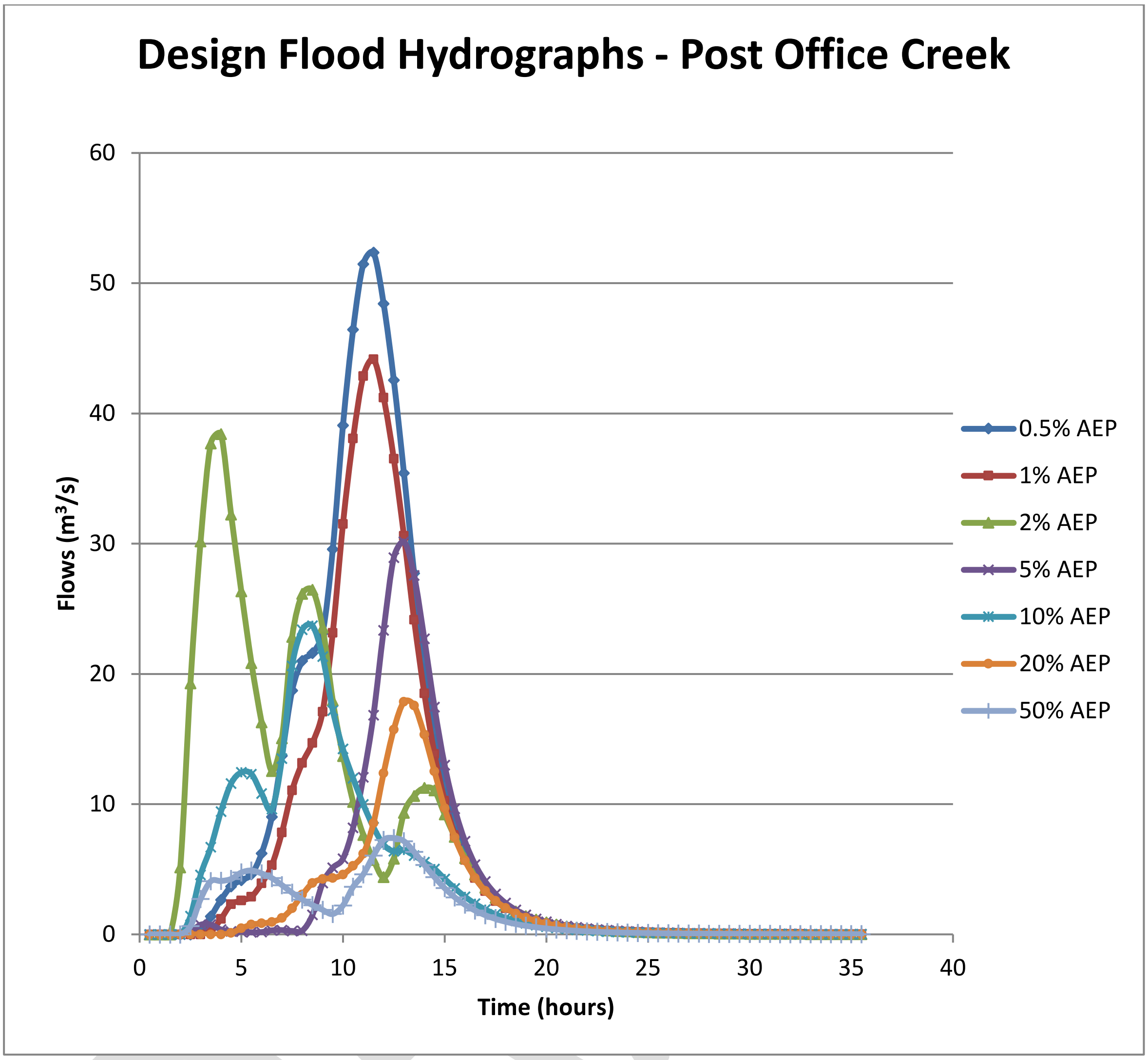


Figure 3-32 **Design flood hydrographs for Post Office Creek**

3.7.3 Discussion

The Calder Highway crosses Post Office Creek approximately 450 metres upstream of the location of the RORB model outlet where the flow is determined. The RORB model outlet location was selected based on the extent of LiDAR available for the TUFLOW model. It is therefore necessary to check the capacity of the Calder Highway Bridge to ensure it is sufficient to convey the calculated flows from Post Office Creek.

The Calder Highway Bridge consists of three 2.4 x 3.0 metre box culverts as shown in Figure 3-33 below which was received from VicRoads.

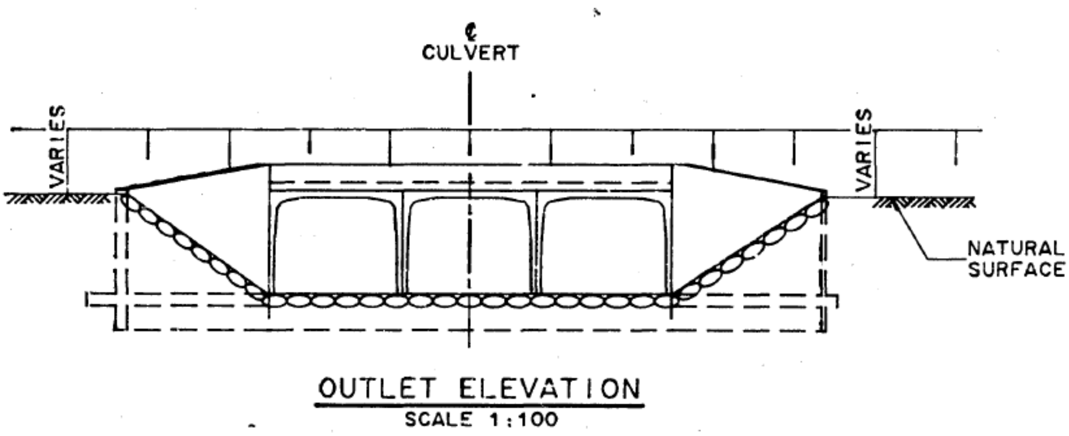


Figure 3-33 Calder Highway Bridge over Post Office Creek

The capacity of the bridge was determined using the design charts in the CPAA Design Manual – Hydraulics of Precast Concrete Conduits. Both inlet and outlet analyses were undertaken to establish what flow regime the culverts operate under. The culverts are 78 metres long with wingwall flares within 30^{O} to 75^{O}. A Manning's n value of 0.013 was adopted for the concrete pipes and the headwater depth was assumed to be the same as the culvert height of 2.4 metres. The total flow capacity of the culverts under inlet control was calculated to be $57m^3/s$. Under outlet control, the culvert capacity was estimated at $120m^3/s$. Therefore, the culverts operate under inlet control with sufficient capacity to convey the 0.5% AEP flow, hence the design flows determined in the RORB model will not be restricted by the Calder Highway Bridge.

3.8 Design Flow Verification

The design flows are largely dependent on the adopted RORB model design parameters. Therefore, these flows were compared to several other peak flow estimates for verification. The methods used to verify the design flows generated from RORB included:

- Regional Flood Frequency Estimation
- Probabilistic Rational Method
- Deterministic Rational Method
- DCNR Regional Method
- Comparison to previous studies

These methods are discussed in the following sections. A summary of the results is shown in Table 3-32 and Figure 3-34 to Figure 3-36.

3.8.1 Regional Flood Frequency Estimation

ARR recommends the use of the Regional Flood Frequency Estimation (RFFE) tool for estimating peak design flows. The RFFE tool was developed as part of the revision of ARR and is available on the ARR website. The tool requires the following inputs: catchment area, outlet location and catchment centroid location. Essentially, the RFFE approach transfers flood frequency characteristics from a group of gauged catchments to the location of interest. This estimation technique is limited to catchments that meet the following criteria:

- Catchment area is greater than 100km^2;
- Urban areas account for less than 10% of total catchment area;
- Catchment does not contain large storages. Small farm dams do not significantly impact on the estimate; and,
- Land use has not changed significantly.

The RFFE tool was used to estimate peak flows at the Campaspe River at Redesdale gauge, Kyneton Township and on Post Office Creek at Mollison Street and the results are summarised in Table 3-32 and Figure 3-34 to Figure 3-36.

3.8.2 Probabilistic Rational Method

Although no longer recommended by ARR, the Probabilistic Rational Method was used to estimate 1% AEP peak flows for comparison. The calculations were undertaken in accordance with the technique described in ARR 1987, using the 1987 IFD values that apply to this method.

Additionally, the VicRoads Probabilistic Rational Method was also calculated. This method is identical to the Probabilistic Rational Method except that it applies an additional factor to account for catchment area. For large catchments, such as that of Kyneton, the VicRoads method yields the same flow estimate as the Probabilistic Rational Method. However, for the smaller Post Office Creek catchment, the estimated VicRoads method flow is 57% greater.

The results of both methods are shown in Table 3-32 and Figure 3-34 to Figure 3-36.

3.8.3 Deterministic Rational Method

The Deterministic Rational Method is also no longer recommended by ARR as it has been replaced by the Regional Flood Frequency Estimation tool. However, this method was used to provide a rapid estimate of the 1% AEP peak flow on Post Office Creek for comparison. The Deterministic Rational Method involves assigning a runoff coefficient to all land within the catchment to specify the rainfall runoff. Standard runoff coefficients were applied to land based on the current zoning. Similar to the Probabilistic Rational Method, the rainfall intensity applied for this method was derived from the 1987 IFD as this is the dataset from which the method was created. The results of this methods are shown in Table 3-32 and Figure 3-36.

3.8.4 DCNR Regional Method

The *Hydrological Recipes – Estimation Techniques in Australian Hydrology* (Grayson et al., 1996) recommends the use of the regional method developed by the Department of Conservation and Natural Resources. This technique is based solely on the correlation between peak flow rate and catchment area. Two regional equations are provided for use depending on whether the catchment is mostly rural or urban. These equations are only applicable to small to medium sized catchments in the region of the Great Dividing Range. Furthermore, these equations do not apply to catchments affected by artificial or natural storages such as floodplains, reservoirs or breakaway channels. Hence, the use of this estimation method is appropriate in this case. The results of these equations are shown in Table 3-32 and Figure 3-34 to Figure 3-36, with the urban regional method shown simply for comparison.

3.8.5 Previous Flood Studies

3.8.5.1 Calder Highway Carlsruhe to Kyneton – Hydrologic and Hydraulic Investigations (CMPS&F, 1995)

VicRoads commissioned a hydrologic and hydraulic investigation for the Calder Highway crossings of the Campaspe River between Carlsruhe and Kyneton. In determining appropriate design peak flows two methods were considered:

1. A flood frequency analysis was undertaken for both the Campaspe River at Redesdale gauge and the Campaspe River at Ashbourne based on 26 years and 22 years of records respectively. The design flows at Carlsruhe were estimated using the results of this analysis on the basis of a linear relationship between catchment area and flow per unit area.

2. A rainfall-runoff model was created and calibrated to the September 1993 event and the peak 5 year ARI flow at the Ashbourne gauge. The design flows generated by this model were considered more reliable since the flood frequency analysis was based on a relatively short record. Hence, the rainfall-runoff model results were adopted for the study's hydraulic model. The results are shown in Table 3-31 below. It should be noted that the flows are determined at Carlsruhe Bridge, located approximately 9km upstream of Kyneton Township.

Table 3-31 Study design peak flows at Carlsruhe Bridge

ARI	Flood Frequency Analysis Peak Flows (m^3/s)	Rainfall-Runoff Modelling Peak Flows (m^3/s)
5 year	134	85
20 year	188	175
100 year	222	315

A comparison of the flows are provided below in Table 3-32 and Figure 3-34 and Figure 3-35.

3.8.5.2 River Walk Flood Study (Earth Tech, 2005)

In April 2005, a flood study was conducted by Earth Tech for a reach of the Campaspe River south of Kyneton Township to determine the 1% AEP flood levels for a residential development. The study utilised information determined from a 2002 report, prepared by Egis Consulting Australia, which had determined a 1% AEP flow of 275m^3/s at the Calder Freeway bridge in Carlsruhe. This flow was then linearly scaled in accordance with the additional catchment area of the downstream site to derive a flow at Sanctuary Drive, Kyneton. Hence, the peak flow adopted for this study was 297m^3/s. This is shown in Table 3-32 and Figure 3-35.

3.8.5.3 Kyneton Township Stormwater Drainage Study (Aurecon, 2011)

Macedon Ranges Shire Council commissioned this stormwater drainage study for the township of Kyneton to identify the existing infrastructure limitations and determine the future requirements. As part of this assessment, a one-dimensional hydraulic model was prepared for Post Office Creek. The study report does not describe how the 1% AEP design flow applied in the hydraulic model was determined. However, based on other calculations detailed in the report, the flows appear to have been estimated using the Deterministic Rational Method. This flow is displayed in Table 3-32 and Figure 3-36 for comparison with the other methods.

3.8.6 Summary

Table 3-32 below shows the 1% AEP peak flow estimations for the three locations of interest: the Campaspe River at Redesdale gauge, Campaspe River at Kyneton Township, and Post Office Creek. The estimated design peak flows are graphed for each of these locations in Figure 3-34 to Figure 3-36 respectively.

It can be seen that the RORB model results for the Redesdale catchment correlates well to the gauge flood frequency curve as it was calibrated to this. The RORB model results are significantly less than that estimated by the RFFE; however, the 1% AEP peak flow is similar to the Probabilistic Rational Method estimate.

For the Campaspe River peak flow estimates at Kyneton, the RORB model results correlated well with the two previous flood studies undertaken for the catchment. It is significantly higher than flows estimated by the RFFE and Probabilistic Rational Method.

The Post Office Creek RORB model yields a similar 1% AEP flow to that utilised in the Kyneton Township Stormwater Drainage Study and also estimated by the Deterministic Rational Method. It exceeds the RFFE and Probabilistic Rational Method due to the substantial proportion of urban development within the catchment. Hence, these estimation techniques are not directly applicable to this catchment. As expected the RORB model 1% AEP peak flow lies between the Regional Method estimates for fully rural and urban catchments.

Table 3-32 Comparison of various estimates of the 1% AEP peak flow

	Campaspe River @ Redesdale Gauge (m^3/s)	Campaspe River at Kyneton Township (m^3/s)	Post Office Creek (at Mollison Street Bridge) (m^3/s)
Probabilistic Rational Method	452	188	20.5
VicRoads Probabilistic Rational Method	452	188	32.2
Deterministic Rational Method	-	-	42.4
Regional Method (Rural)	645	299	31.2
Regional Method (Urban)	1009	493	60.3
Flood Frequency Analysis	457 (355 – 668)#	-	-
Flood Frequency Analysis (Bureau of Meteorology)	430	-	-
Regional Flood Frequency Estimation (AR&R)	861 (266 – 2800)	236 (74.5 – 754)	20.9 (6.6 – 66.7)
Kyneton Township Stormwater Drainage Study (Aurecon, 2011)	-	-	46.7
River Walk Flood Study Report (Earth Tech, 2005)	-	297	-
Calder Highway Carlsruhe to Kyneton – Hydrological and Hydraulic Investigations (CMPS&F, 1995)	355 (108 – 1164)	315	-
RORB Model	462	299	44.1

90% confidence interval shown in parentheses

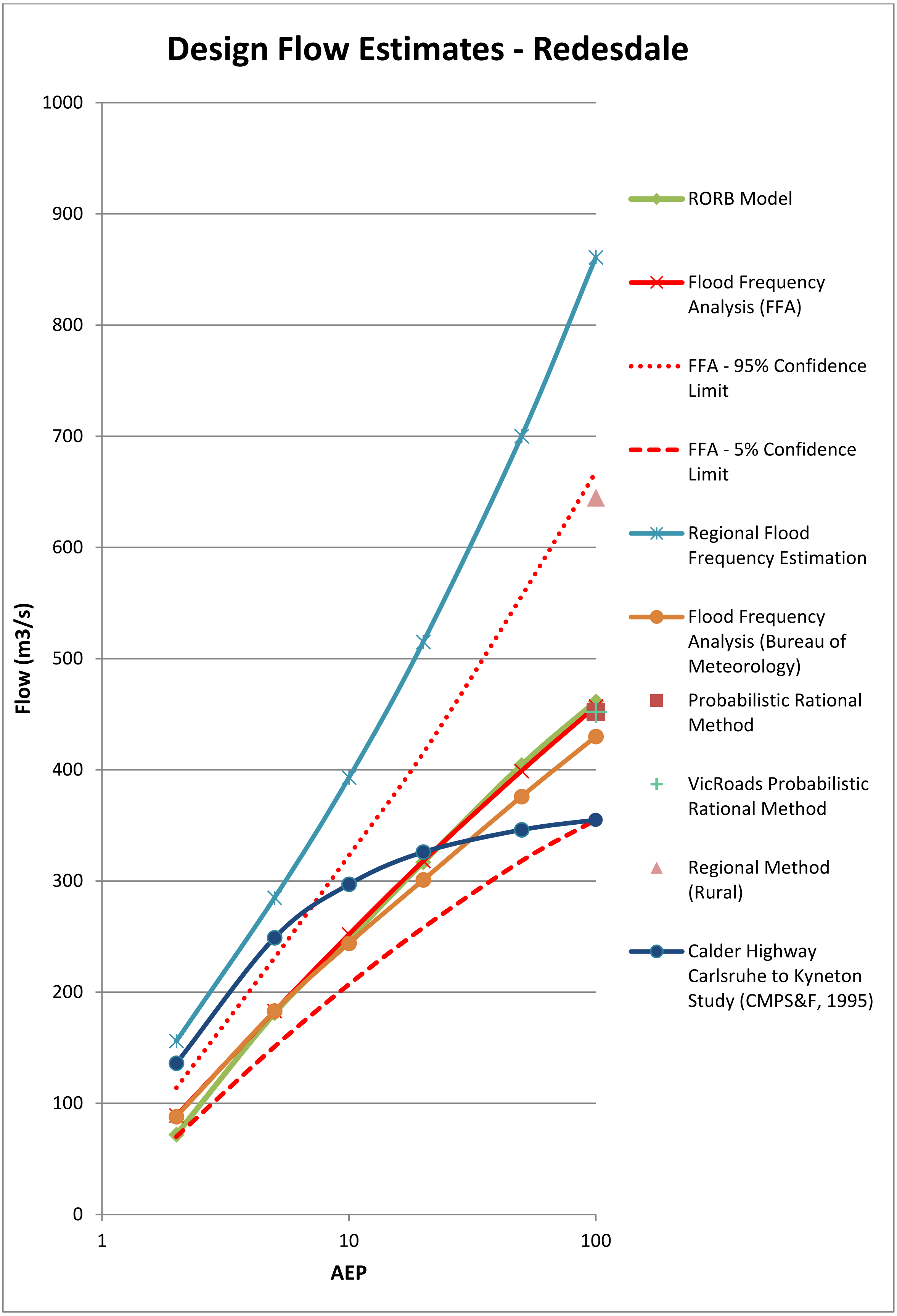


Figure 3-34 **Comparison of design peak flow estimates for Campaspe River at Redesdale gauge**

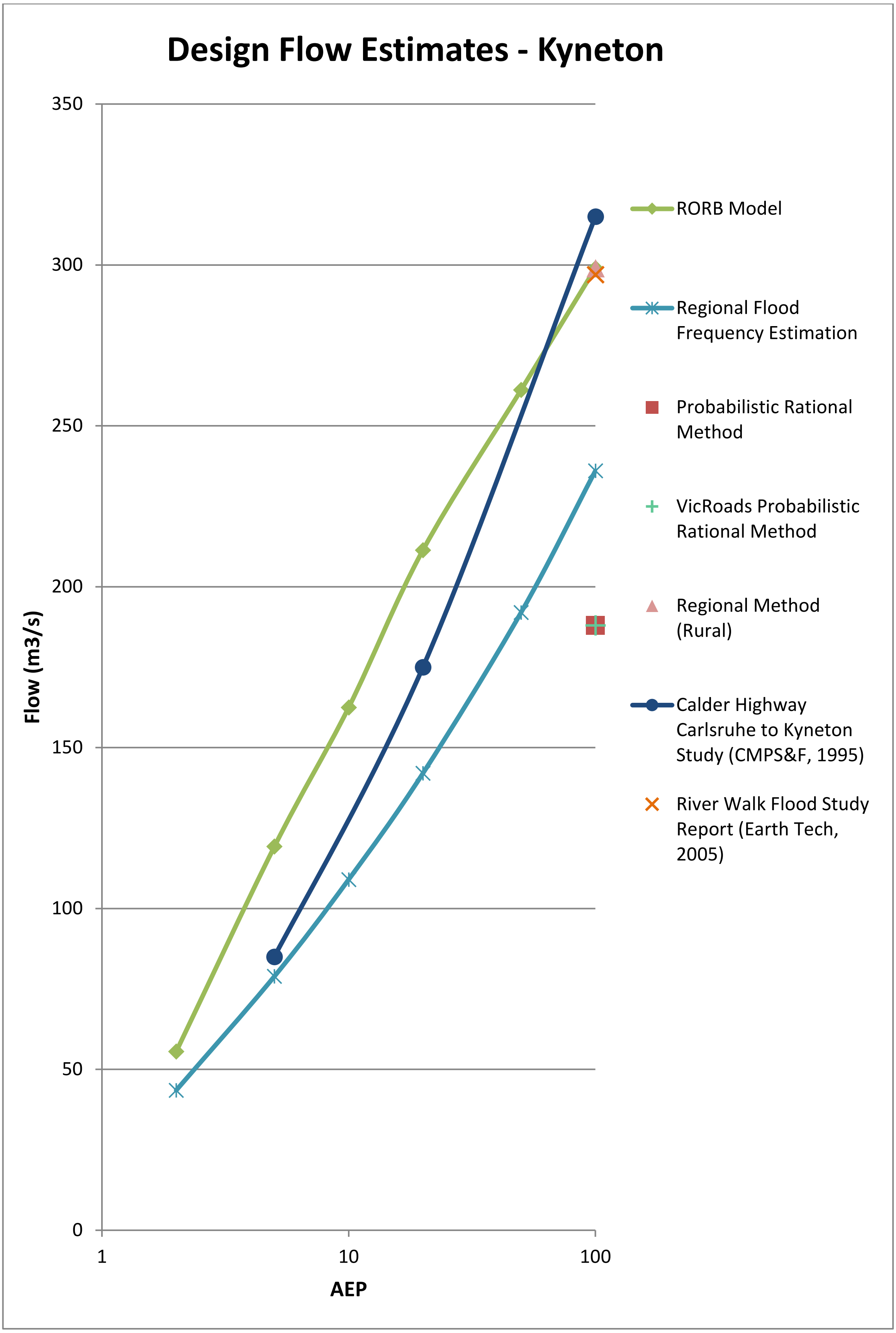


Figure 3-35 Comparison of design peak flow estimates for Kyneton Township

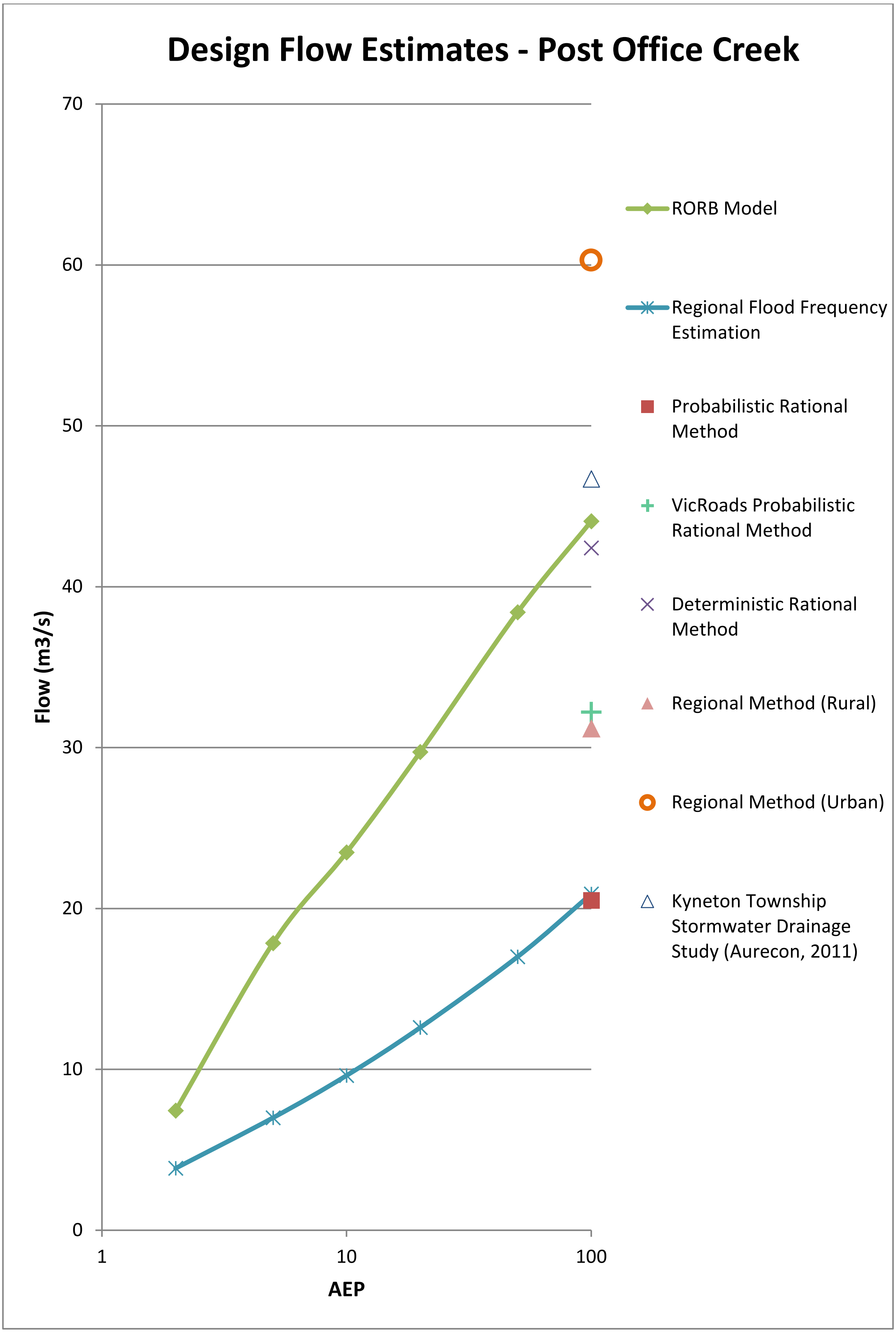


Figure 3-36 Comparison of design peak flow estimates for Post Office Creek

3.9 Sensitivity Analysis

The RORB Monte Carlo analysis used to determine the design flows inherently accounts for variation in the temporal pattern, losses and rainfall depth by stochastic sampling. Hence, further sensitivity analysis on these parameters is not required.

However, ARR recommends that the potential impacts of various climate change projections be considered. This involves adjusting the IFD rainfall data to future climates by using the method recommended in ARR, Book 1, Section 6.3.5. This method is based on temperature scaling using temperature projections from the CSIRO and is preferred as climate models produce temperature estimates more reliably than individual storm events.

The Data Hub file (Section 7.3.2) includes the interim climate change factors to apply based on the different climate scenarios modelled and the planning horizon (shown in Table 3-33). The climate scenarios are based on Representative Concentration Pathways (RCPs) which describe the different concentrations of greenhouse gases and aerosols. These factors are applicable to both the Campaspe River and Post Office Creek catchments at Kyneton.

Table 3-33 Interim climate change factors for Kyneton

Planning Horizon	RCP4.5		RCP6		RCP8.5	
	Temp. Increase (°C)	Increase in Rainfall	Temp. Increase (°C)	Increase in Rainfall	Temp. Increase (°C)	Increase in Rainfall
2030	0.85	4.3%	0.845	4.2%	0.974	4.9%
2040	1.086	5.4%	1.05	5.3%	1.341	6.7%
2050	1.303	6.5%	1.283	6.4%	1.734	8.7%
2060	1.478	7.4%	1.539	7.7%	2.212	11.1%
2070	1.629	8.1%	1.775	8.9%	2.753	13.8%
2080	1.741	8.7%	2.036	10.2%	3.26	16.3%
2090	1.793	9.0%	2.316	11.6%	3.748	18.7%

For the sensitivity analysis, the planning horizon of 2090 was adopted. ARR, Book 1, Section 6.2 recommends the use of both RCP 4.5 and 8.5 to consider the impacts of low and high concentrations. Hence, based on these assumptions, the table above indicates 9.0% and 18.7% increase in rainfall for scenarios RCP 4.5 and 8.5 respectively.

3.9.1 Campaspe River

Figure 3-37 below compares the resulting design flood hydrographs for the different climate change scenarios to the standard design hydrograph for the 10% and 1% AEP events on the Campaspe River. Table 3-34 displays the increase in peak flow for each of the climate change scenarios, which are greater than the corresponding increases in rainfall depths. For example, under scenario RCP 8.5 the rainfall is increased by 18.7% however the 1% AEP peak flow has increased by 30.6% and exceeds the 0.5% AEP peak flow under current climate conditions. Similarly, the 10% AEP peak flow is increased to the equivalent of the 5% AEP peak flow under climate scenario RCP 8.5.

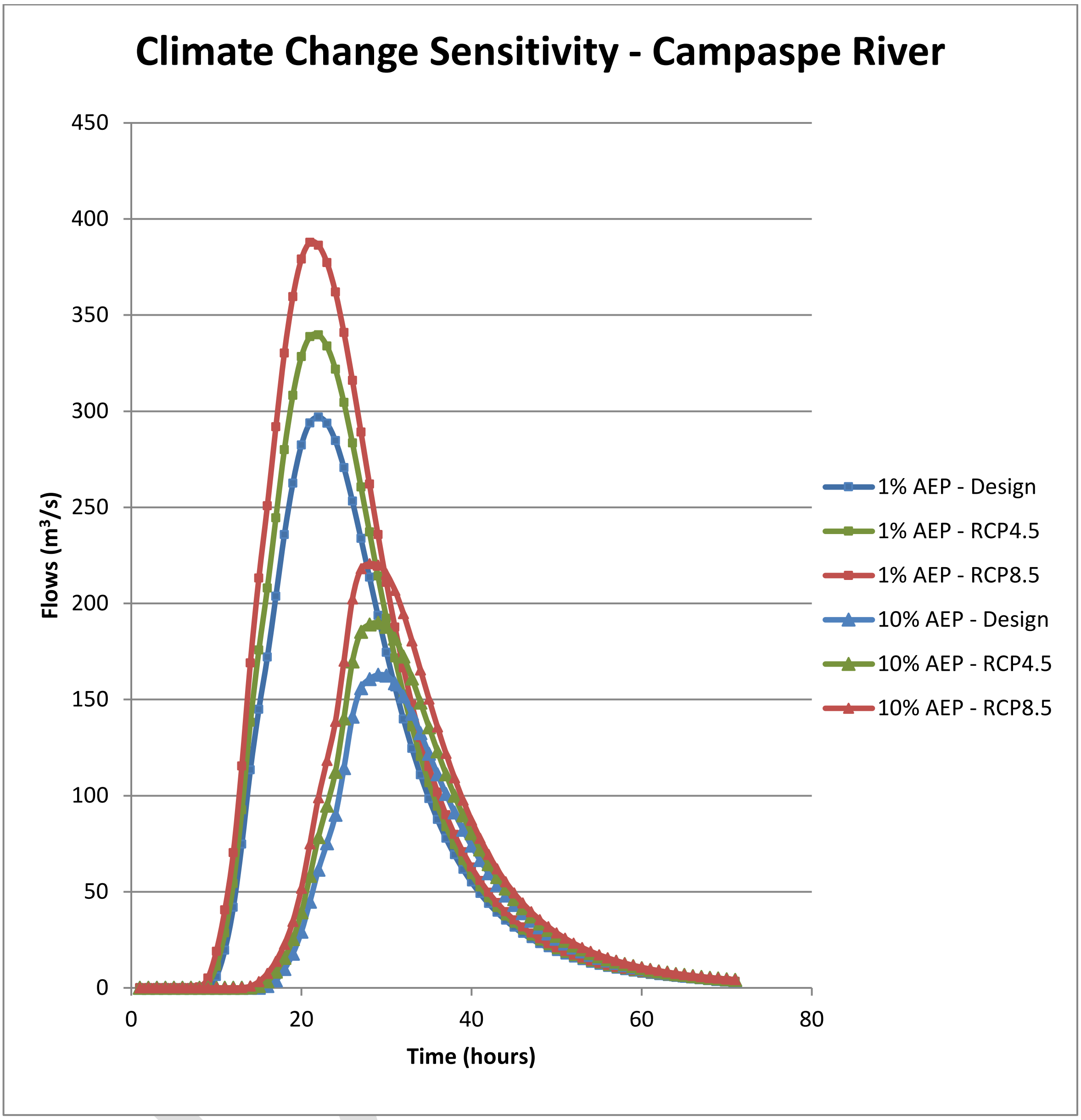


Figure 3-37 **Impacts of climate change on Campaspe River design hydrographs**

Table 3-34 **Comparison of climate change scenario peak flows for Campaspe River**

	Design Peak Flow (m^3/s)	RCP4.5 Peak Flow (m^3/s)	Difference	RCP8.5 Peak Flow (m^3/s)	Difference
1% AEP Event	297.0	339.7	14.4%	387.8	30.6%
10% AEP Event	162.8	190.0	16.7%	220.6	35.5%

3.9.2 Post Office Creek

Figure 3-38 below compares the resulting design flood hydrographs for the different climate change scenarios to the standard design hydrograph for the 10% and 1% AEP events on Post Office Creek. Table 3-35 displays the increase in peak flow for each of the climate change scenarios. Similar to the Campaspe River analysis, the percentage increase in flows for the climate change scenarios are greater than the corresponding increases in rainfall depths. For example, under scenario RCP 8.5 the rainfall is increased by 18.7% however the 1% AEP peak flow has increased by 21.5% and is equivalent to the 0.5% AEP peak flow under current climate conditions. Similarly, the 10% AEP peak flow is increased to almost the current 5% AEP peak flow under climate scenario RCP 8.5.

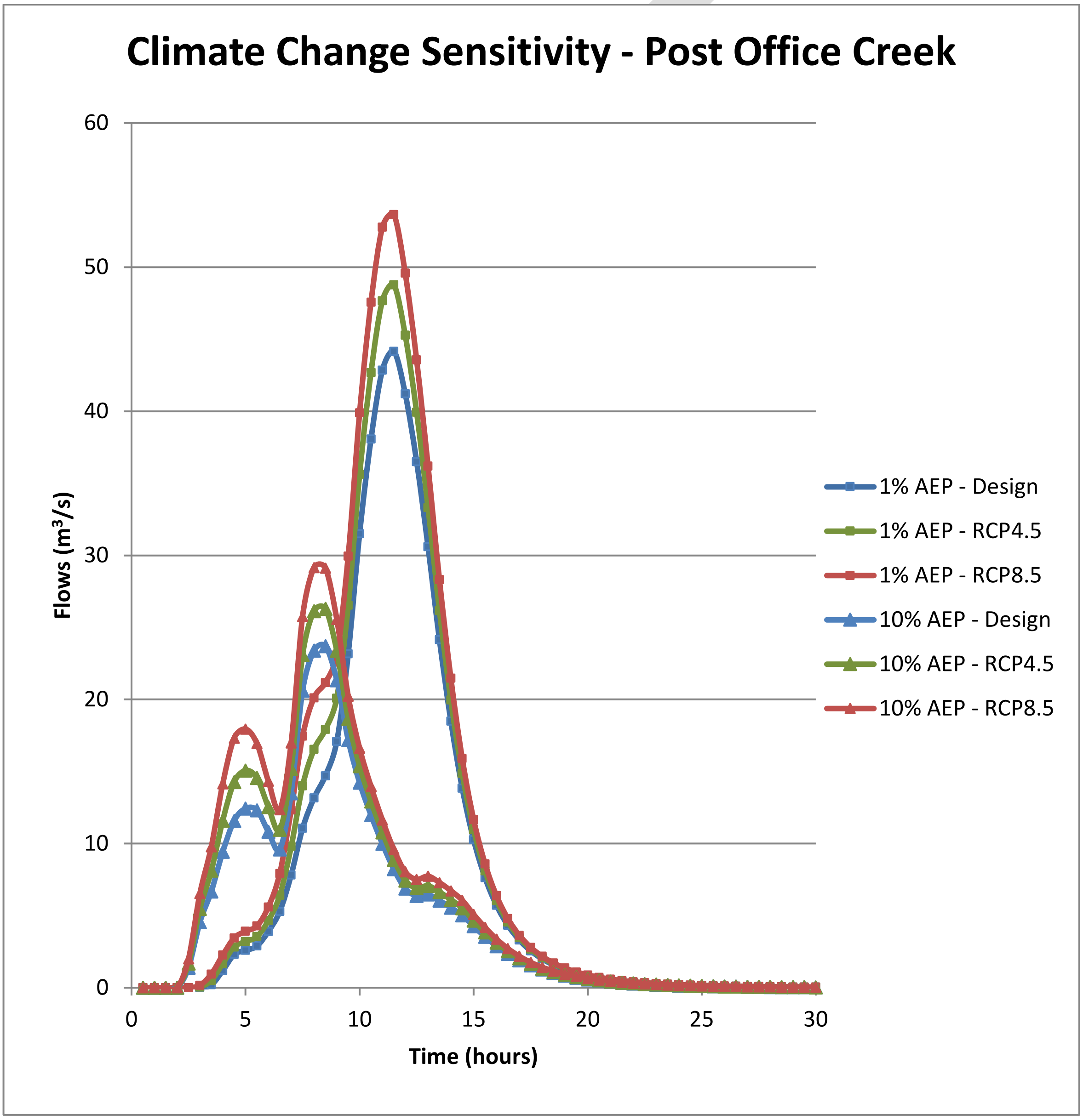


Figure 3-38 Impacts of climate change on Post Office Creek design hydrographs

Table 3-35 Comparison of climate change scenario peak flows for Post Office Creek

	Design Peak Flow (m^3/s)	RCP4.5 Peak Flow (m^3/s)	Difference	RCP8.5 Peak Flow (m^3/s)	Difference
1% AEP Event	44.2	48.8	10.4%	53.7	21.5%
10% AEP Event	23.7	26.3	11.0%	29.2	23.2%

3.10 Probable Maximum Flood

Estimates of the Probable Maximum Flood (PMF) were determined using the regression equations recommended in *Hydrological Recipes* (Grayson et al., 1996). These equations allow the computation of a triangular PMF hydrograph based on the catchment area. This estimation method was derived from analysis of PMF estimates from 56 catchments in South Eastern Australia ranging in size from 1 - 10,000km^2. As both the Campaspe River and Post Office Creek catchments investigated in this study have catchment areas within this range and do not have any significant storages, this method is directly applicable. For the South East Australia method, the PMF peak flow rates were calculated using the following equation where Q is the peak flow rate in m^3/s and A is the catchment area in km^2:

$$Q = 500A^{0.43}$$

The peak flows estimated by this method were also compared to regression equations based on empirical analysis of global flood observations as recommended in ARR, Book 1, Section 3.4.4. The following relationships are proposed by Herschy (2003) based on a data set of worldwide flood maxima:

$Q = 500A^{0.43}$ for values of A greater than 90km^2

$Q = 100A^{0.8}$ for values of A less than 90km^2

The PMF peak flow estimates for the two methods at each of the hydraulic model input locations are shown in Table 3-36 below. A comparison of the results shows that the global regression method Herschy (2003) yields significantly higher peak flow estimates than the South East Australia method (Grayson et al., 1996). As the South East Australia method has been derived from local data which is directly applicable to the study catchment, these results have been adopted to represent the PMF flow conditions. The corresponding hydrographs are shown in Figure 3-43. The timing of each hydrograph relative to the other hydrographs was approximated based on the timings determined for the 1% AEP design hydrographs.

Table 3-36 PMF peak flow estimates

Location	Peak Flow (m^3/s)	Global Equation (m^3/s)	Difference (%)
Campaspe at Carlsruhe	2891	4379	51%
Carlsruhe Tributary	1421	2253	59%
Subarea F	1007	1440	43%
Post Office Creek	599	733	22%

3.11 Summary

The design hydrographs adopted for the hydraulic model inputs are shown in Figure 3-39 to Figure 3-43 below.

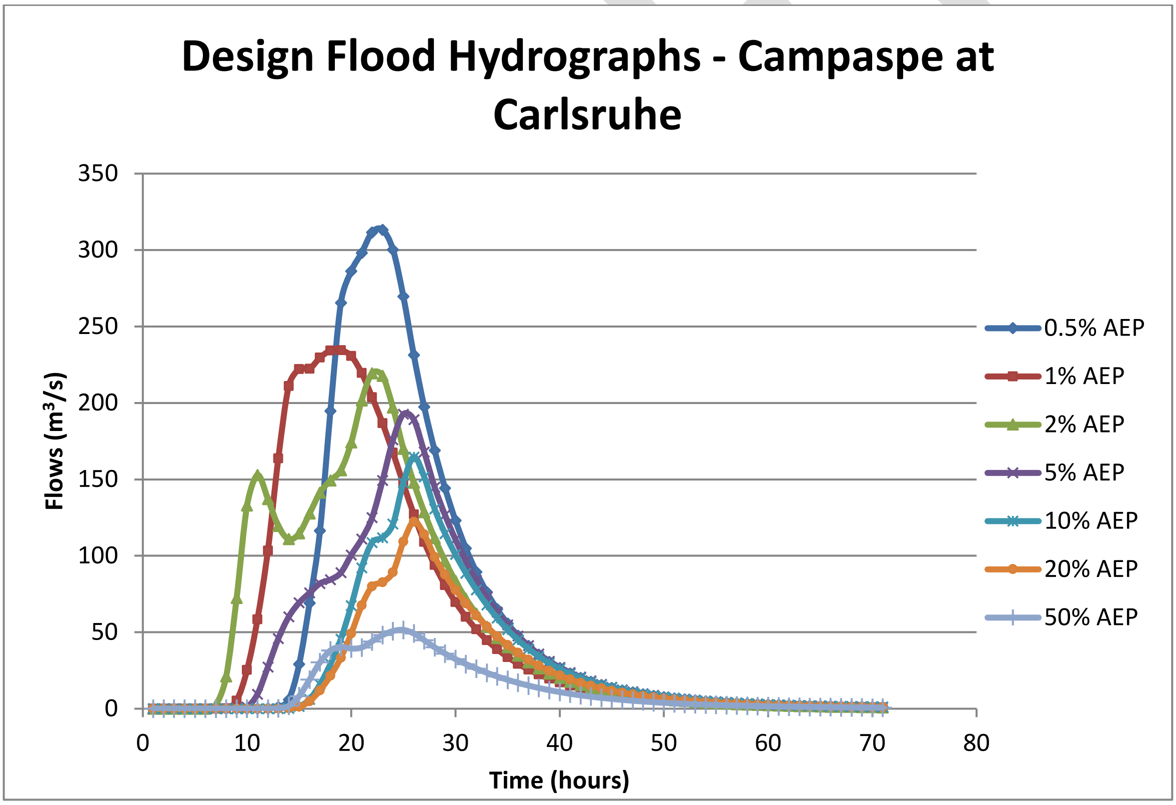


Figure 3-39 Design flood hydrographs at Campaspe at Carlsruhe

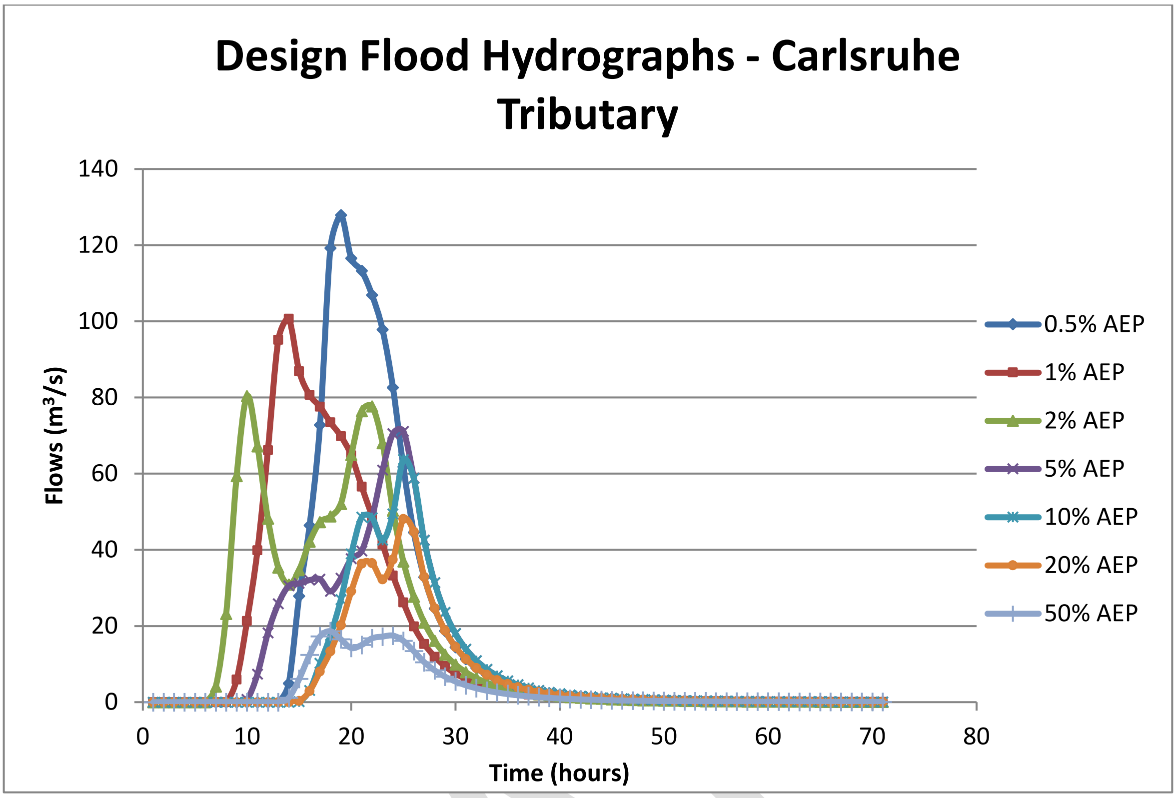


Figure 3-40 Design flood hydrographs at Carlsruhe Tributary

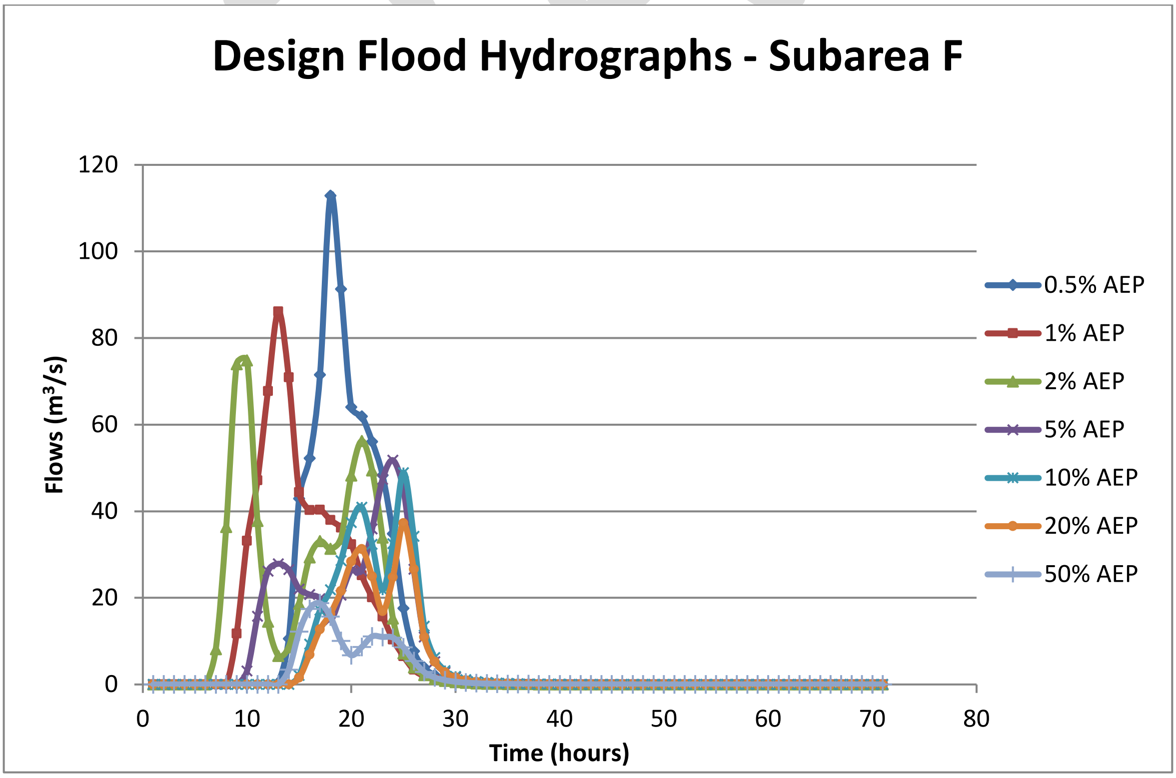


Figure 3-41 Design flood hydrographs from Subarea F

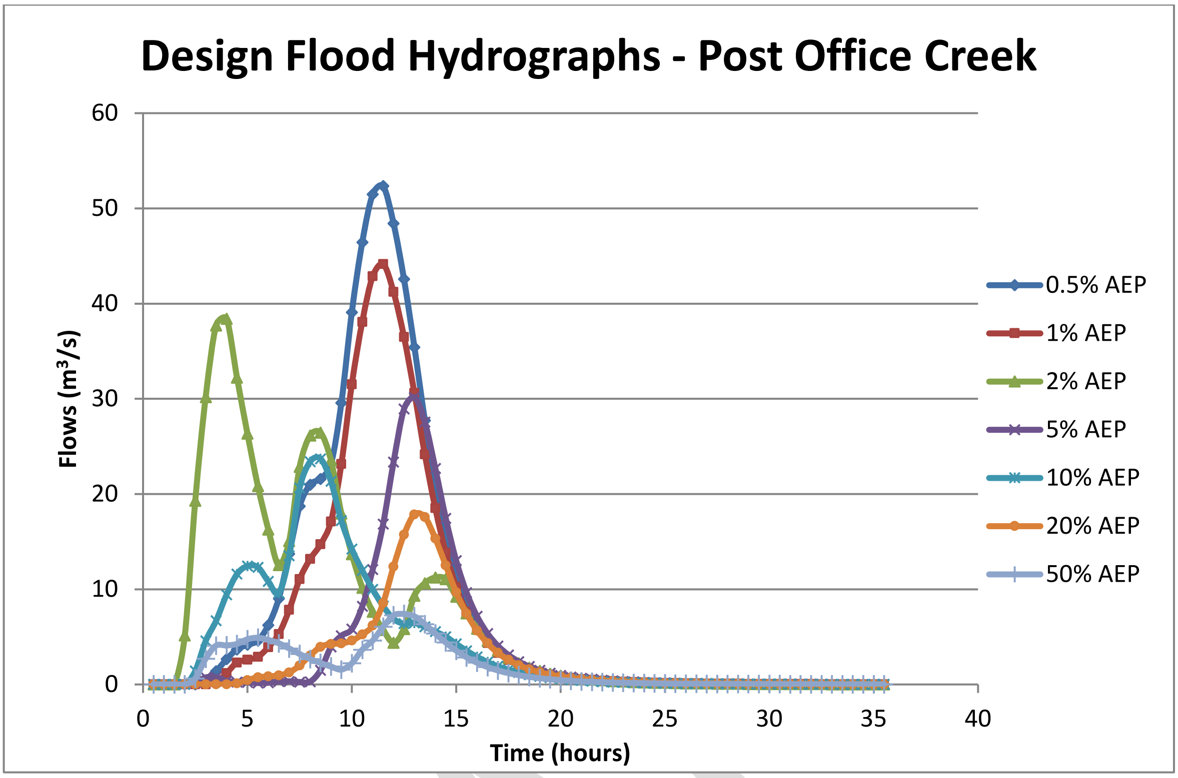


Figure 3-42 Design flood hydrographs for Post Office Creek

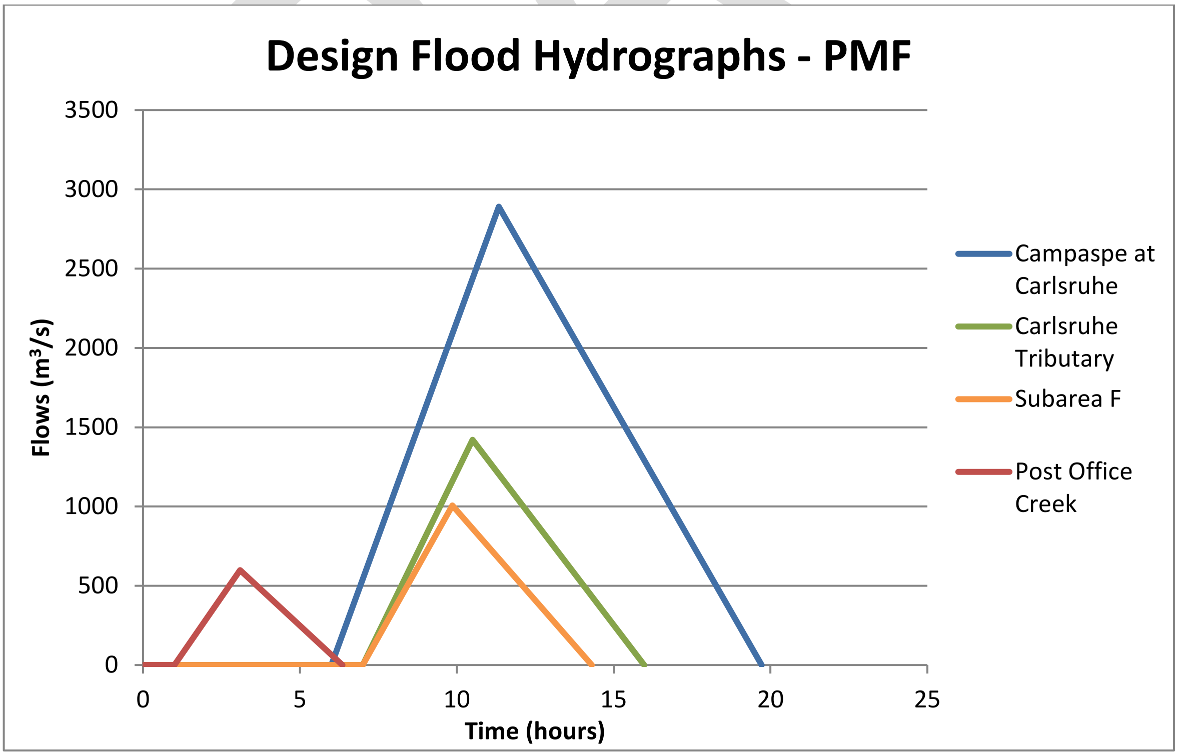


Figure 3-43 Probable Maximum Flood (PMF) hydrographs for all hydraulic model input locations

4 Hydraulic Modelling

4.1 Overview

A detailed combined 1D-2D hydraulic model of Kyneton Township was developed to produce flood mapping for the calibration and design flood events. The calibrated hydraulic model simulates flood flow behaviour of both the Campaspe River and Post Office Creek. The following sections detail the hydraulic model setup, calibration and generation of design flood mapping.

4.2 Hydraulic Model Construction and Parameters

4.2.1 Model Overview

The hydraulic modelling software TUFLOW was used for this study. The model was run with the most recent TUFLOW build 2017-09-AB-iDP-w64.

TUFLOW is a floodplain modelling tool developed by BMT WBM which can model both 1D and 2D systems. The hydraulic modelling approach consisted of the following components:

- One dimensional (1D) hydraulic model of the culverts;
- Two dimensional (2D) hydraulic model of the waterways, broader floodplain and large multi-span bridges; and
- Links between the 1D and 2D hydraulic models to integrate the 1D hydraulic structures with the broader floodplain flow.

The major waterways, Campaspe River and Post Office Creek, were modelled in the 2D domain rather than as 1D elements due to the following advantages:

- Accounts for form, bend, contraction and expansion losses are explicitly.
- Velocity is calculated for each individual cell rather than averaged horizontally across the channel.

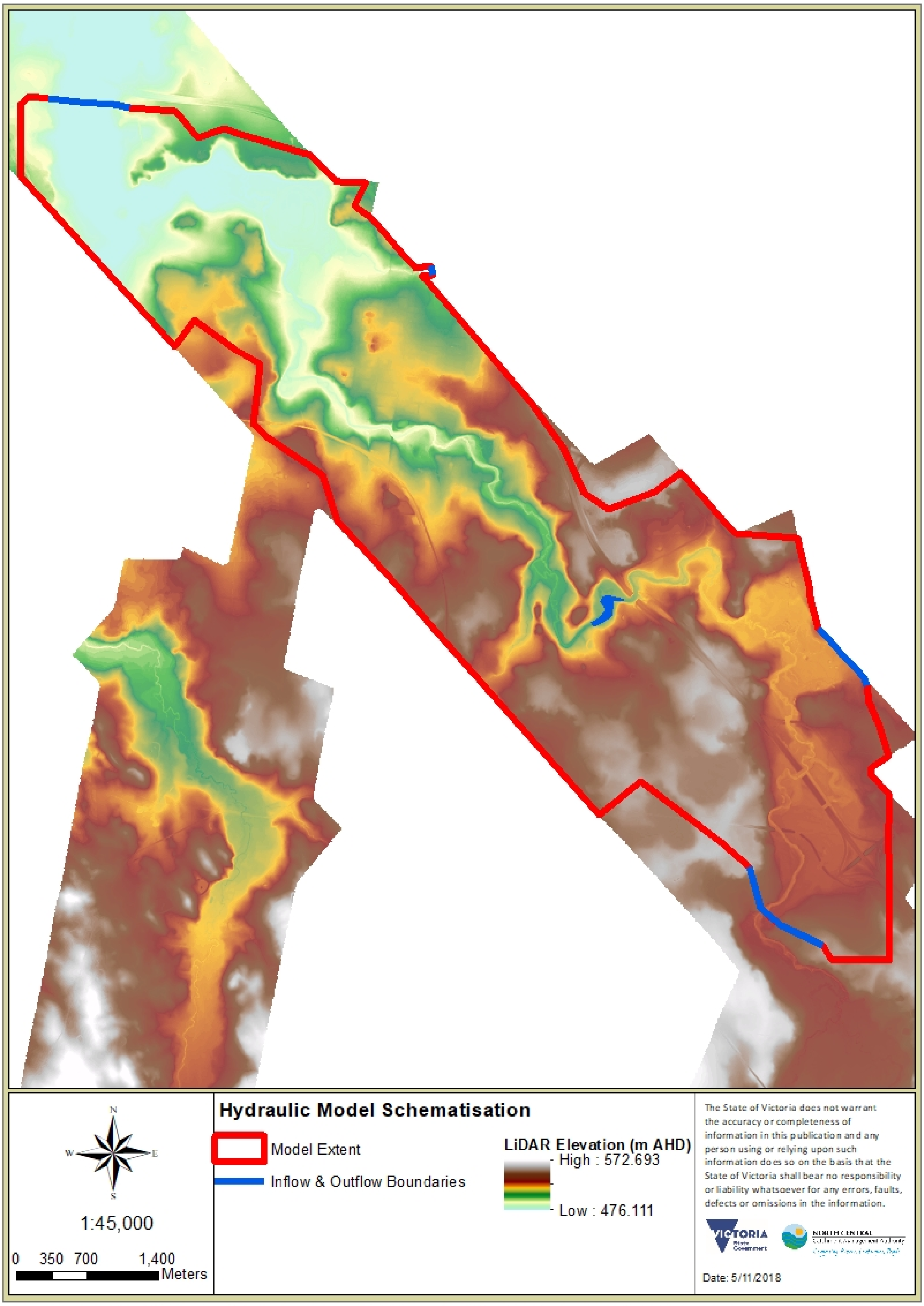
4.2.2 Modelling Parameters

4.2.2.1 Projection

The TUFLOW model was created in GDA94/MGA Zone 55.

4.2.2.2 Extent

The hydraulic model extends along the Campaspe River from upstream of Cheveley Road to immediately upstream of the Calder Freeway bridge north of Kyneton as shown in Figure 4-1. Along Post Office Creek the model was limited by the extent of the available LiDAR for this area and therefor extend to immediately upstream of the Mollison Street. The model encompasses an area of approximately 12km^2. This extent ensures that the flood behaviour within the study area is reliably represented without undue influence of boundary effects.



T:\GISData\FLOODPLAIN\MACEDON RANGES\KYNETON\KYNETON FLOOD STUDY 2014\CMA DATA\Kyneton_Study_(20181102_MGA55)_Depth Maps.mxd

Figure 4-1 Hydraulic model schematisation

4.2.2.3 Topography

The floodplain topography for the model was generated from the MD_Rivers_ISC_2010 LiDAR. This dataset was produced in 2010 and has a resolution of 1m (Figure 4-1). In order to accurately represent the study area while still allowing a reasonable run time, the 2D model domain was based on a 4m grid resolution.

The LiDAR data was collected during an extensive period of drought in the north central region. The method used to collect the data does not penetrate the surface of water and therefore the data generated does not represent the natural surface level of the bed of the waterway. However, the water level was low at the time the data was gathered and hence, provides a reasonable approximation of the topography of the waterway.

Additionally, the water that was in the waterway at the time the LiDAR was created, including the level of weir pools, is an approximate representation of the baseflow prior to a flood event. Consequently, no initial water level (IWL) was set for the model.

Minor modifications were made to the model DEM to ensure that it accurately represented the floodplain topography. This included adjusting areas of the LiDAR that were clearly obscured by vegetation, in addition to enforcing levels of the waterway thalweg, road crests and weirs using breaklines to ensure they were incorporated into the model.

Furthermore, the Mollison Street culverts were located near the boundary of the available LiDAR along Post Office Creek. It was important to model this hydraulic structure since it was likely to influence the downstream flood behaviour. Therefore, due to the limited extent of LiDAR upstream it was necessary to duplicate the LiDAR cross-section in order to extend the model DEM. This achieved a sufficient distance from the model inflow boundary to enable a smooth flow transition between the inflow boundary and the hydraulic structure. However, due to the localised uncertainty of this extended section of the DEM the outputs generated in this area were clipped out of the final datasets.

4.2.2.4 Timestep

The timestep selected is critically important for the stability and accuracy of the model. The Courant Number is a measure of the model stability and, for a 2D square grid, is defined as:

$$C_r = \frac{\Delta t\sqrt{2gH}}{\Delta x}$$

where, *Δt = timestep (s)*
Δx = cell size (m)
g = acceleration due to gravity m/s^2
H = depth of water (m)

For most real-world applications, the Courant Number generally needs to be less than 10 and is typically around 5 for a 2D scheme. In order to achieve this criterion, the computational timestep is typically set to between one half and one quarter of the cell size (TUFLOW Manual 2010, pp. 3-8 – 3-

9). For this model, a 4-metre cell size was chosen and the 2D timestep used was 1 second. The 1D timestep was set to half the 2D timestep as recommended in the TUFLOW Manual, that is 0.5 seconds.

4.2.2.5 Runtime

The model was run long enough for the input hydrograph to peak and for the peak to be conveyed through the model to the outlet. The entire hydrograph was not required to be completely run through the model as the primary cause of flooding for the study area is due to conveyance of the peak flow rather than due to the volume of floodwater conveyed. The typical runtime for the hydraulic model was 20 hours.

4.2.2.6 Hydraulic Roughness

Roughness was initially assigned to each cell as a Manning's n value based on the current zoning of the land. These values were further refined based on aerial photography, site inspections and knowledge of the area. For example, areas zoned for residential have been developed with significant portions of the residential land adjacent to the creek changed to reserve. Hence the reserve area of the residential zone has been altered to reflect the nature of the land use.

For calibration and validation modelling, the roughness values selected were based on the existing conditions at the time. However, for the design events, the roughness values adopted were based on the zoning, regardless of whether the land had been developed already or not. This is to account for the future development of the township. Furthermore, since the January 2011 flood event, significant willow removal was undertaken along the Campaspe River. Consequently, the waterway roughness has changed. A separate roughness layer was therefore prepared for the January 2011 model calibration (Figure 4-2) and the design events (Figure 4-3) to reflect the change in catchment roughness. The Manning's roughness coefficients adopted were based on standard industry values and are shown in Table 4-1 below.

Table 4-1 Roughness values

Model Material No.	Land Use	Manning's Roughness
1	Open pervious areas, minimal vegetation (grassed, pasture)	0.05
2	Open pervious areas, moderate vegetation (shrubs)	0.06
3	Open pervious areas, thick vegetation (trees)	0.1
4	Residential – low density	0.1
5	Residential – high density	0.2
6	Industrial/Commercial	0.3
7	Paved road	0.02
8	Unpaved road, tennis court	0.03
9	Carpark	0.025
10	Railway	0.04
11	Concrete lined channels	0.02
12	Waterway with minimal vegetation	0.04
13	Waterway with moderate vegetation	0.08
14	Waterway with heavy vegetation	0.1
15	Waterway with very dense vegetation	0.12
16	Lakes/Ponds (no emergent vegetation)	0.03
17	Wetlands (emergent vegetation)	0.05

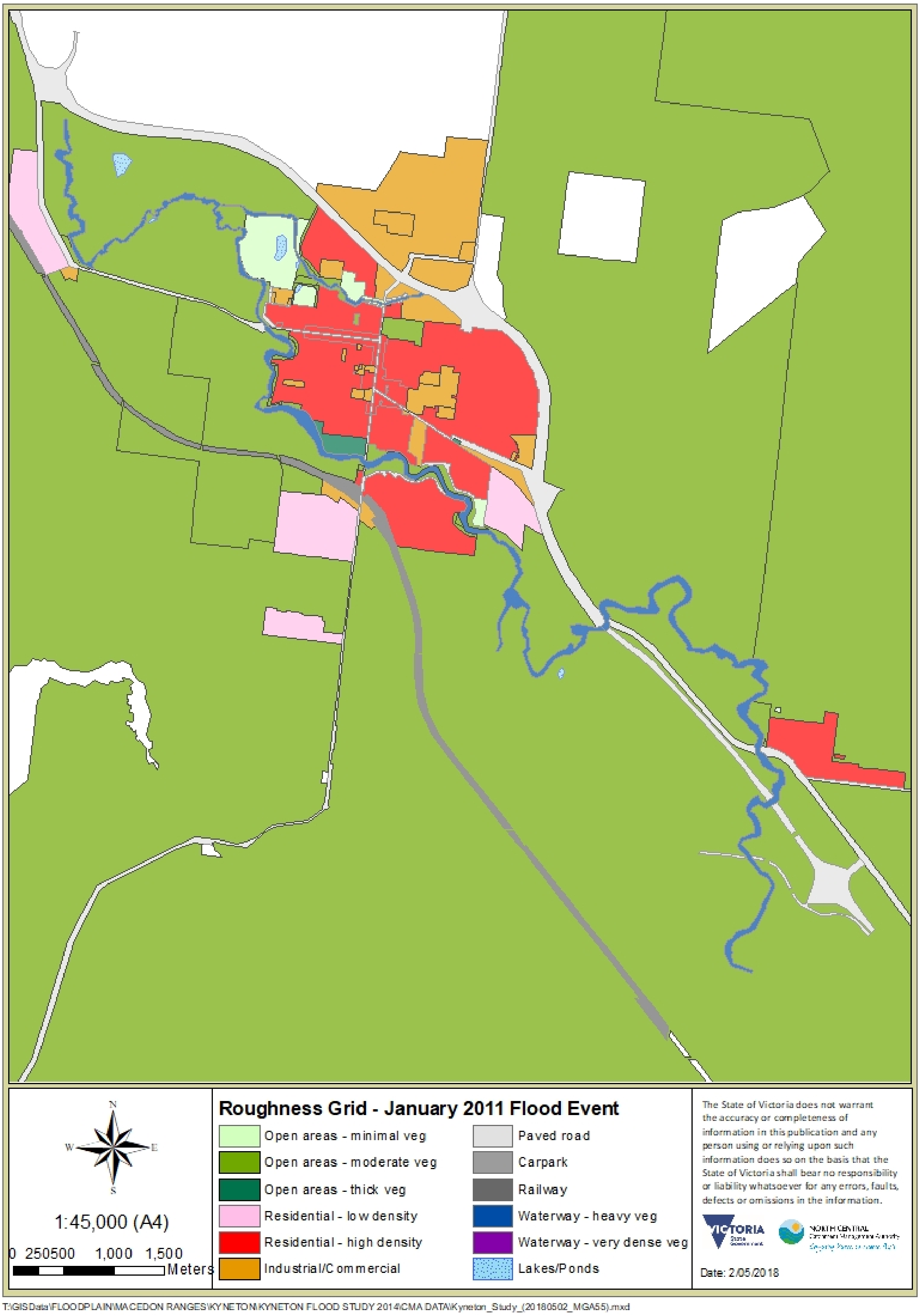


Figure 4-2 Hydraulic model roughness grid (Manning's roughness) for January 2011 flood event

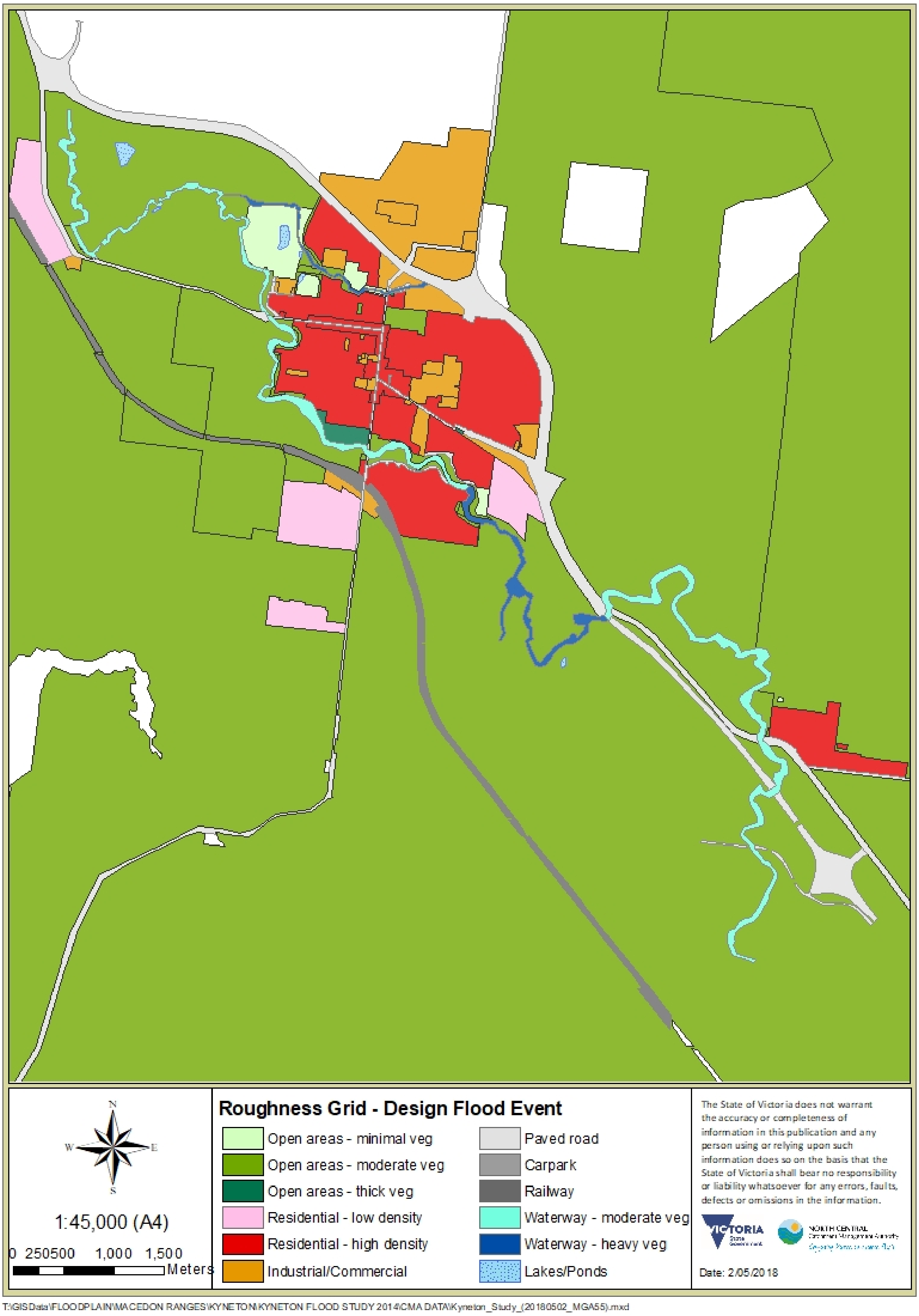


Figure 4-3 Hydraulic model roughness grid (Manning's roughness) for design flood events

4.2.3 Boundary Conditions

Inflow boundaries were applied to represent flow from the Campaspe River, a tributary in Carlsruhe and Post Office Creek. An internal inflow boundary, termed Subarea F from the RORB model schematisation, was also included to incorporate local runoff from the township catchment. The corresponding historical and design hydrographs derived from the RORB model (see Section 3) were input at these boundaries. An automatically generated stage-discharge relationship, derived from the topography and an estimated water surface slope of 0.01, was applied at the outlet boundary. The outlet boundary was positioned sufficiently downstream of the township so that the estimated flow conditions at this location would have no impact on flood behaviour at the area of interest. The location of these boundaries is shown in Figure 4-1.

4.2.4 Structures

The model included a number of hydraulic structures that impact on flood behaviour. The height of weirs was determined from the LiDAR and incorporated into the model topography using breaklines. Culverts were input as 1D elements coupled to the 2D model domain; however, flow over the top of the culverts is simulated in the 2D model domain. Plans of these structures were received from the asset owners and invert levels were estimated based on site inspections and comparisons with the LiDAR data. Large bridges were modelled as 2D layered flow constrictions with the appropriate losses adopted from *Hydraulics of Bridge Waterways (1978).*

The head loss across each of the bridges modelled in 2D was assessed to ensure the adopted loss factors were reasonable. The head losses for each bridge for both the 10% and 1% AEP design events are shown in Table 4-2 below.

Table 4-2 Head loss across bridges

Structure	Head Loss in 10% AEP Event (m)	Head Loss in 1% AEP Event (m)
S1 – Carlsruhe Central Road Bridge	0.20	0.25
S4 - Calder Highway South Bridges	0.25	0.35
S6 - Cobb and Co Road South Bridge	0.20	0.35
S7 - Cobb and Co Road North Bridge	0.30	0.50
S8 - Calder Highway North Bridges	0.20	0.30
S9 – Mollison Street Bridge	0.20	0.40
S13 - Piper Street Bridge	0.40	0.60

4.3 Hydraulic Model Calibration and Validation

4.3.1 Overview

The hydraulic model was calibrated to the January 2011 flood event as this was the largest event which had sufficient evidence with which to calibrate the model. The September 2010 and September 2016 were then modelled to validate the model parameters determined through calibration. This also enabled a range of flows to be simulated to ensure the hydraulic model could reasonably reproduce both frequent and rare flood events. It should be noted that while the September 2016 event could not be used to calibrate the hydrologic model to the Campaspe River at Redesdale gauge due to uncertainties with the recorded gauge flow rates, this does not impact on the applicability of this event for validating the hydraulic model. Additionally, although there were some photos available of the November 2010 event, these were taken well after the flood peak had passed and therefore were of limited value in validating the model. A summary of the historical flood events is displayed in Table 4-3 and the input hydrographs for the September 2010, January 2011 and September 2016 events are shown in Figure 4-5, Figure 4-16 and Figure 4-23 respectively.

Table 4-3 Summary details for the historical events selected for calibration and validation

Event	Campaspe River at Redesdale Gauge		Campaspe River at Kyneton		Post Office Creek Inflow	
	Peak Flow (m^3/s)	AEP (%)	Peak Flow (m^3/s)	AEP (%)	Peak Flow (m^3/s)	AEP (%)
September 2010	259.6	7%	123.2	20%	13.3	50%-20%
January 2011	322.1	5%	129.1	20%-10%	25.9	10%-5%
September 2016	347.9*	3%*	89.4	50%-20%	13.2	50%-20%

*Note that there is uncertainty regarding the reliability of the peak flow rate recorded at the Campaspe River at Redesdale gauge during the September 2016. Refer to Section 2.2 for further detail.

Calibration and validation was based on photographs and anecdotal observations gathered from the community. A site inspection along the Campaspe River through Kyneton was also conducted with a local community member who provided valuable information regarding the flood behaviour during recent flood events. Recorded flood marks within the study area were very limited with only one surveyed flood mark on Post Office Creek available. Figure 4-4 displays the location of the flood observations for each of the events. The following sections describe the hydraulic calibration and validation for each event by comparing the modelled results to the historical observations.

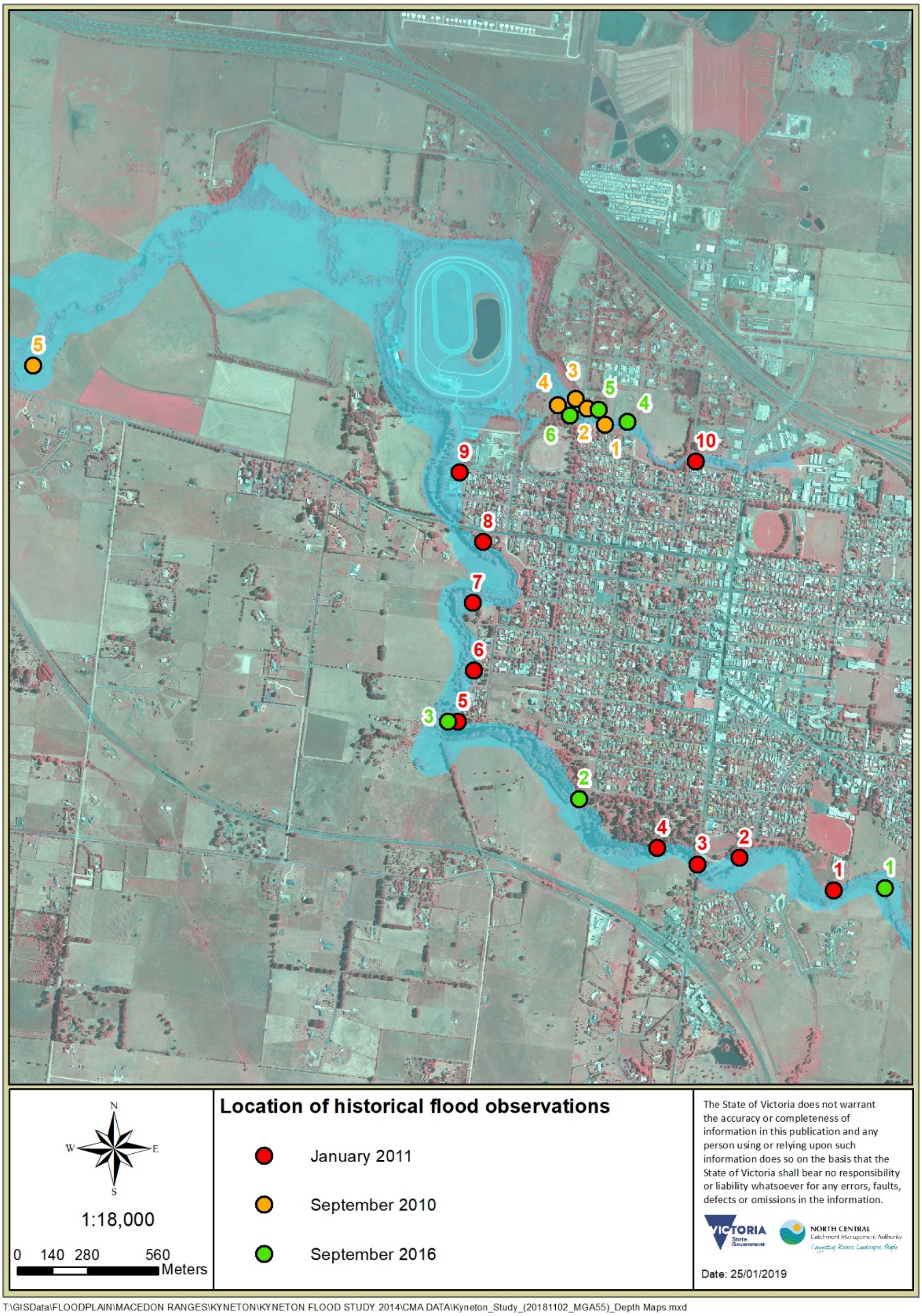


Figure 4-4 Location of historical flood observations

4.3.2 January 2011 Calibration

The RORB hydrologic model was used to generate hydrographs for the January 2011 flood event which are shown in Figure 4-5. The hydrographs were then input into the TUFLOW hydraulic model at the corresponding inflow boundaries. The mapping outputs were compared to the historical observations to calibrate the hydraulic model. The location of the available calibration data is shown in Figure 4-4. The calibration at each of these locations is detailed below.

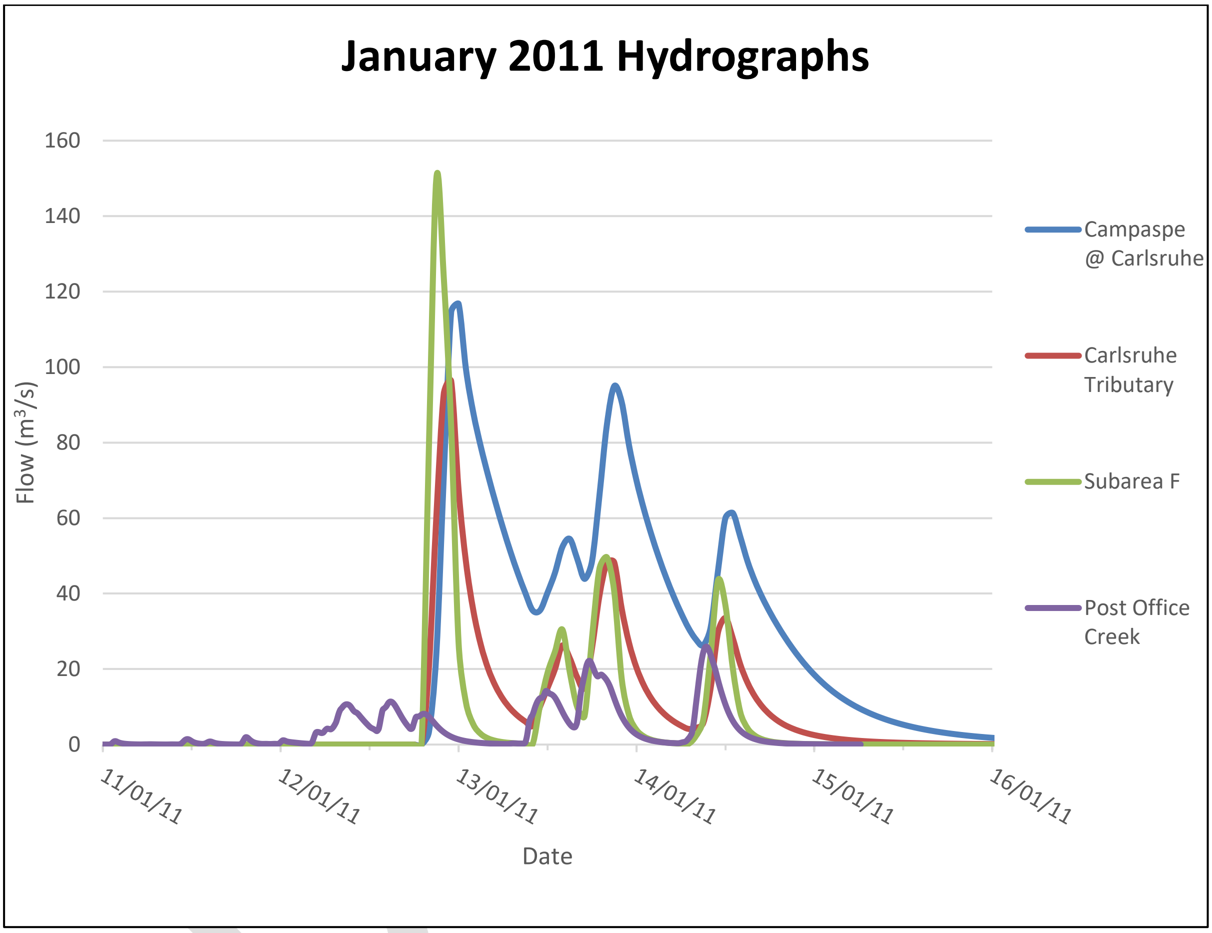


Figure 4-5 Inflow hydrographs for the January 2011 flood event

1. Floodwater was observed up against but not overtopping the dam bank at the rear of Sacred Heart College. The location of the floodwater in relation to this dam is shown in Figure 4-6 below. As seen in Figure 4-7 the hydraulic model of the January 2011 event accurately replicates this.

Figure 4-6 Extent of flooding at the rear of Sacred Heart College looking south-east during the January 2011 flood event

Figure 4-7 Modelled flood extent of the January 2011 event at the rear of Sacred Heart College. Approximate direction of photo in Figure 4.4 indicated by yellow arrow

2. Immediately upstream of the Mollison Street Bridge the walking path along the northern bank was completely inundated and the floodwater extended almost to the line of trees adjacent to the gravel road, as shown in Figure 4-8 below.

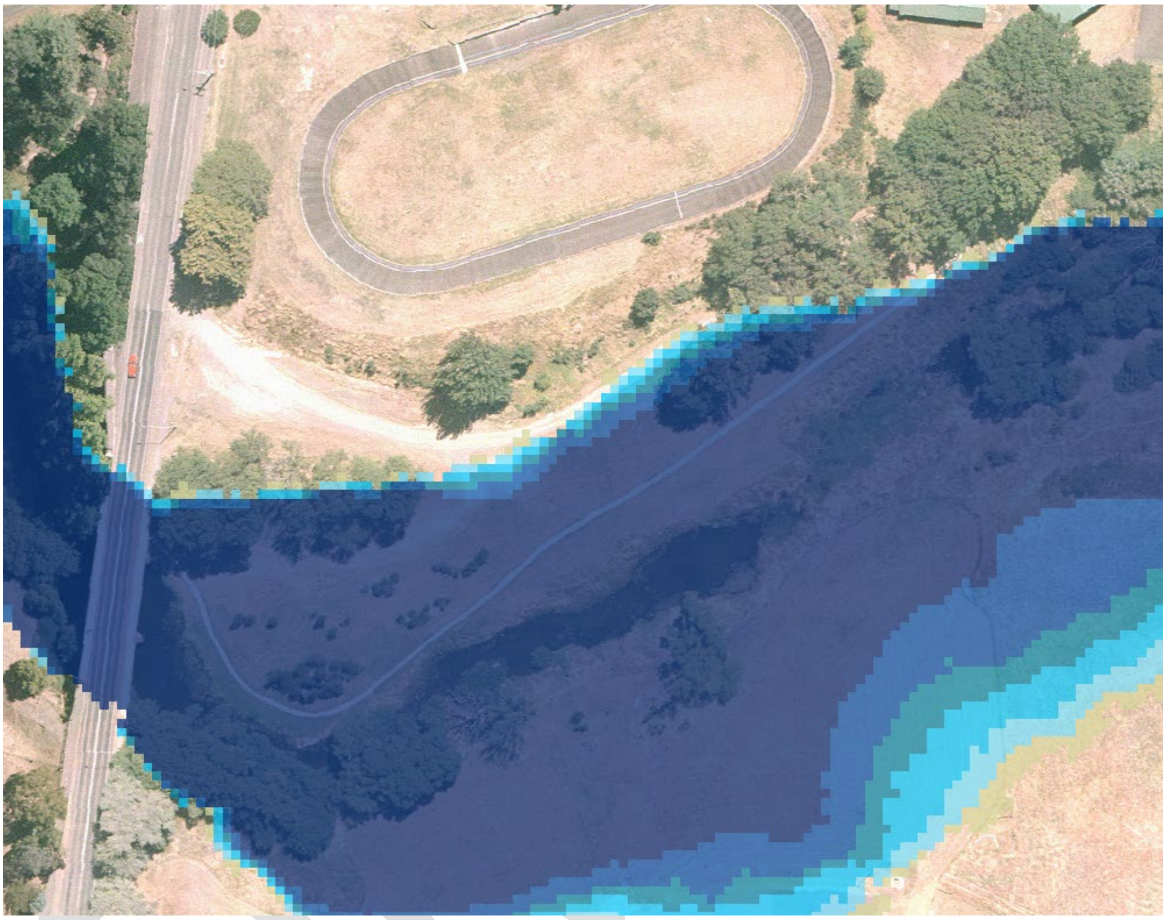


Figure 4-8 Modelled flood extent of the January 2011 event upstream of Mollison Street Bridge

3. The flood level was observed at a height just below the base of the northern bridge abutment of Mollison Street Bridge, which is reproduced by the model (Figure 4-8). A comparison of the modelled flood level to the LiDAR level adjacent to the bridge abutment indicates that the flood height is approximately 200mm below the abutment.

4. The original boardwalk along the northern river bank in the Kyneton Botanic Gardens was completely inundated during the 2011 event however the floodwater did not reach the level of the road that runs through the Kyneton Botanic Gardens. As a result, the boardwalk has since been replaced by a gravel walking track located higher up on the river bank. The aerial photography shown in Figure 4-9 was taken prior to the 2011 flood event and therefore shows the original boardwalk that was flooded.

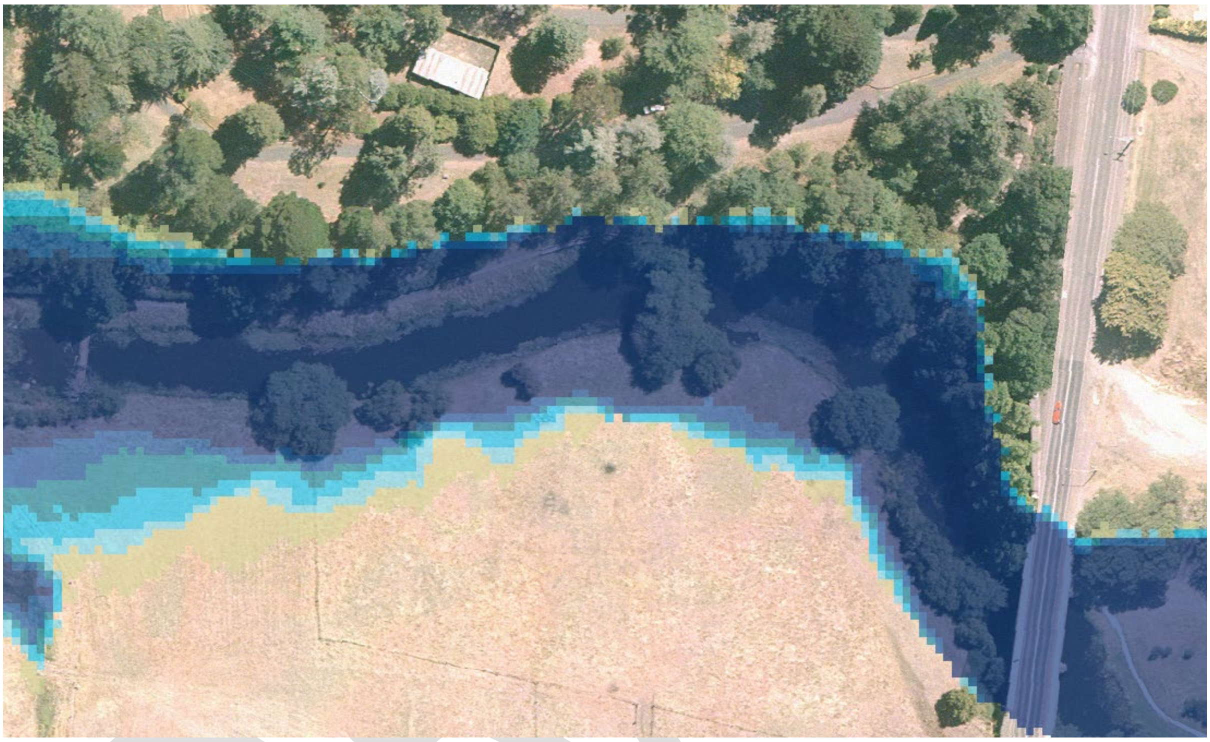


Figure 4-9 Modelled flood extent of the January 2011 event at the Kyneton Botanic Gardens

5. Although the low-lying area at the Greenway Lane weir was completely inundated, floodwater reportedly did not overtop Mill Street. Figure 4-10 below shows only minor flooding over Mill Street, generally less than 50mm in depth. An analysis of the flood levels and road surface levels indicated that they are approximately the same height. The slightly deeper areas shown along the road is where water has pooled in the adjacent table drains. Given the depth is only approximately 50mm and covers a small portion of Mill Street, the model is considered to achieve reasonable calibration in this area.

Figure 4-10 Modelled flood extent of the January 2011 event along Mill Street

6. Properties north of St Agnes Place experienced flooding through the rear of the properties, however there were no reported cases of above floor flooding of the dwellings. Figure 4-11 illustrates that the model reasonably replicates this observed behaviour with only one dwelling potentially having shallow floodwater up against it.

Figure 4-11 Modelled flood extent of the January 2011 event along rear of properties north of St Agnes Place

7. Most of the rear of the properties at the end of Jennings Street was inundated to significant depths as reflected in Figure 4-12. The model indicates depths of approximately 2m over these properties.

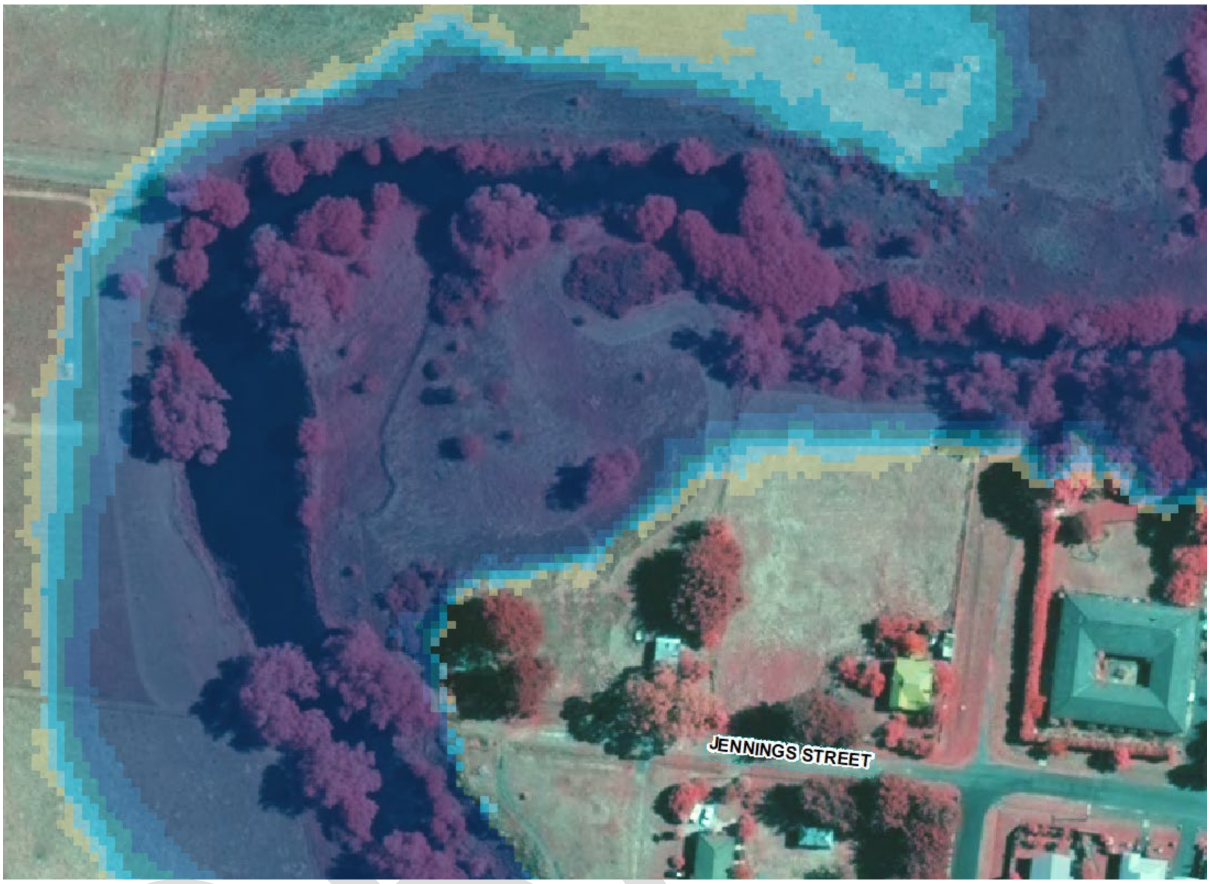


Figure 4-12 Modelled flood extent of the January 2011 event along rear of Jennings Street properties

8. The Quarry Reserve Park immediately upstream of the Piper Street Bridge, located on the eastern bank, was not flooded. Furthermore, the flood extent did not reach the walking path along the eastern bank. Figure 4-13 shows that the modelled flood extent in this area is in accordance with the observed behaviour.

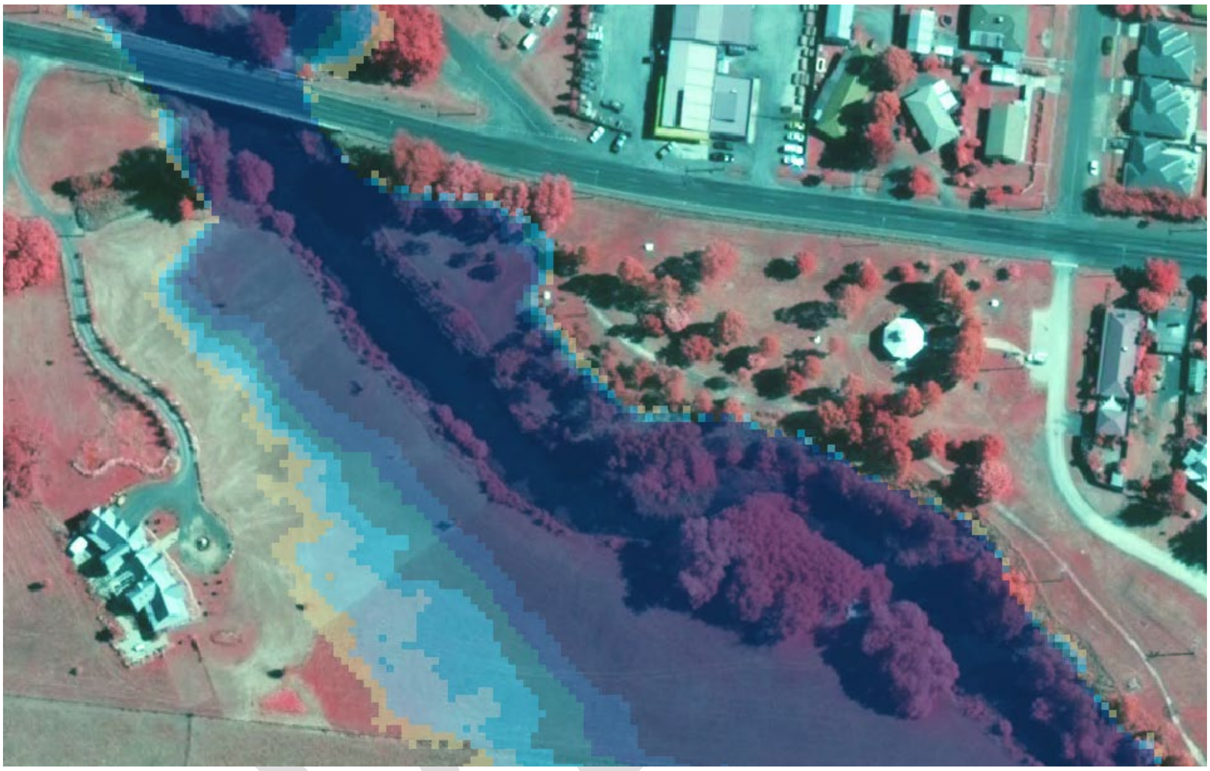


Figure 4-13 Modelled flood extent of the January 2011 event upstream of Piper Street Bridge

9. The horse racing stables along Campaspe Place experienced significant flooding and the horse pool was completely inundated. However, no dwellings were inundated in this area. The model replicates these observations as shown in Figure 4-14. It was also reported that the flood extent almost reached the weigh bridge site at 106-110 Beauchamp Street (located in the north-eastern corner of Figure 4-14). It is unclear how close the floodwater was to the weigh bridge itself and whether this was water backing up from the Campaspe River or stormwater flowing toward the river. Based on the modelled data the flood extent was within approximately 30m of the weigh bridge.

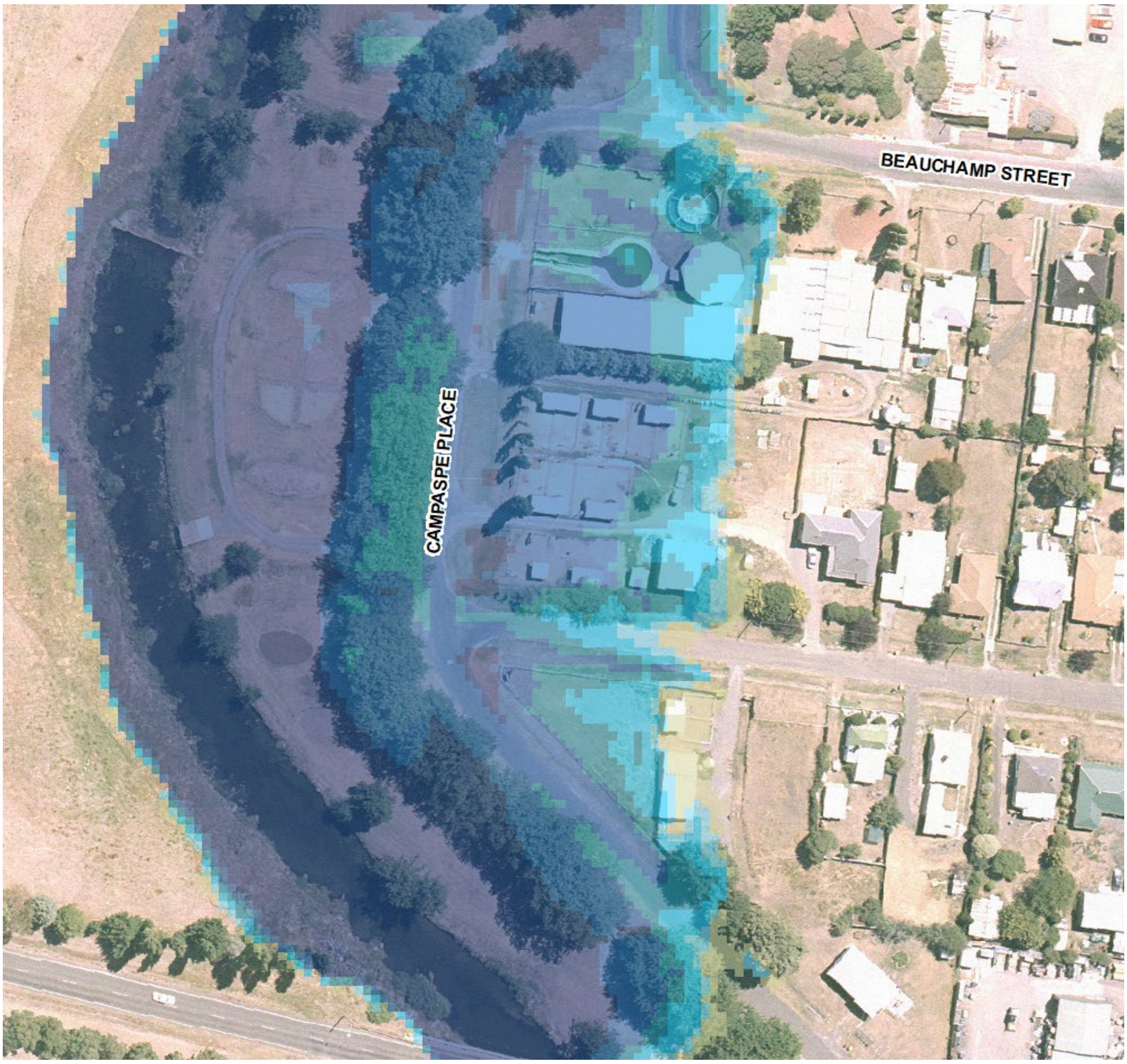


Figure 4-14 Modelled flood extent of the January 2011 event along Campaspe Place

10. A single flood mark on Post Office Creek was available to calibrate the January 2011 event as shown in Figure 4-15. The flood mark, located immediately upstream of Ebden Street, was surveyed with an accuracy of 30mm. A comparison to the modelled data shows that the results are approximately 800mm higher than the recorded 2011 flood mark.

Figure 4-15 Modelled flood extent of the January 2011 event on Post Office Creek. Location of surveyed flood mark is shown as a yellow dot

4.3.3 September 2010 Validation

The hydrographs input into the hydraulic model for the September 2010 event are shown in Figure 4-16. The mapping outputs were compared to the historical observations to validate the hydraulic model parameters as described below. The location of the available validation data is shown in Figure 4-4.

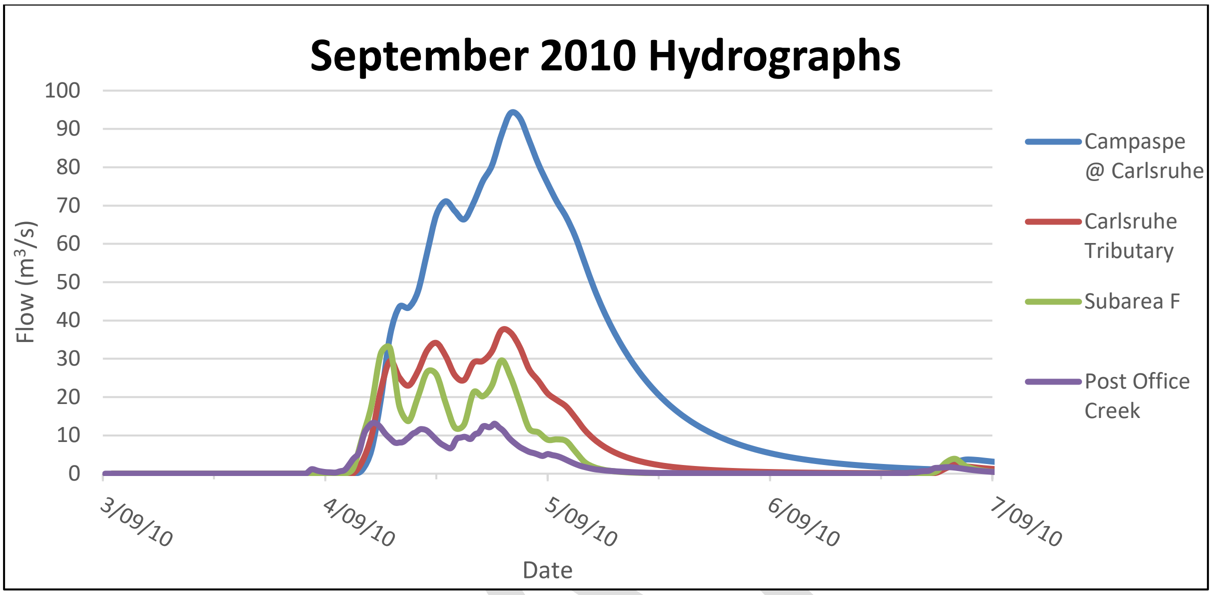


Figure 4-16 Inflow hydrographs for the September 2010 flood event

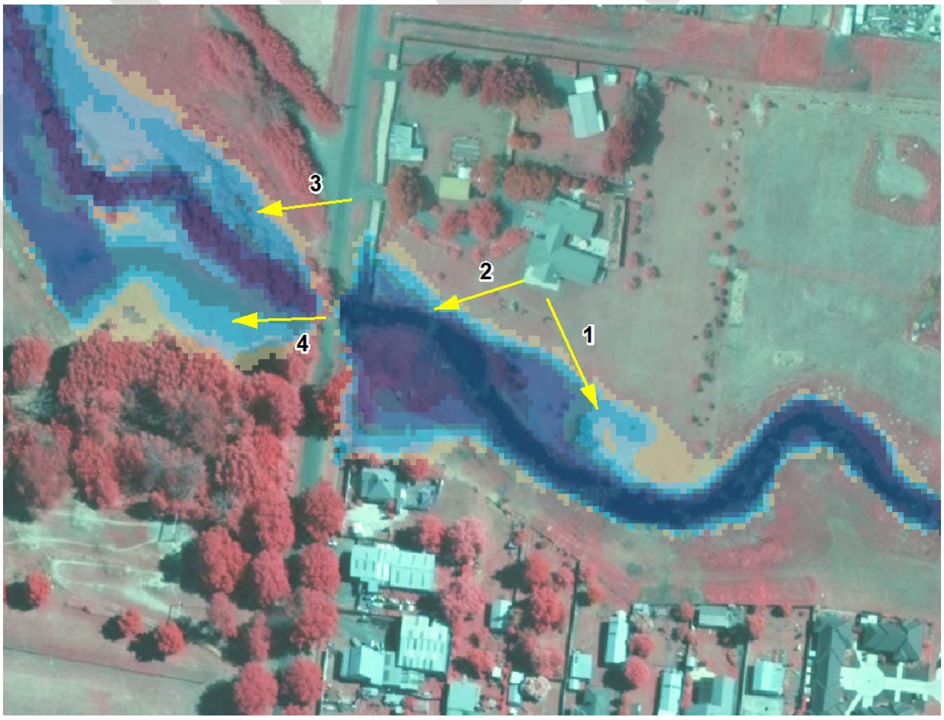


Figure 4-17 Modelled flood extent of the September 2010 event at Wedge Street. Approximate direction of the following photos is indicated by yellow arrows.

1. Further upstream of the Wedge Street crossing of Post Office Creek floodwater generally did not overtop the waterway banks during the September 2010 event as shown in Figure 4-18. Figure 4-17 shows that the model reproduced this behaviour, with water contained within the defined creek channel.

Figure 4-18 September 2010 event looking south-east on Post Office Creek upstream of Wedge Street (refer to Photo 1 in Figure 4-17)

2. Figure 4-19 was taken immediately upstream of the Wedge Street bridge looking downstream along Post Office Creek. It can be seen that the water level appears to reach the bridge soffit but does not overtop the bridge which is replicated by the hydraulic model. A comparison to Figure 4-17 shows that the modelled flood extends further south than what is shown in Figure 4-19. This difference could be because the photo was not taken at the flood peak or possibly due to an upstream blockage restricting the flow rate downstream.

Figure 4-19 September 2010 event looking downstream along Post Office Creek towards Wedge Street bridge (refer to Photo 2 in Figure 4-17)

3. The extent of flooding over the northern bank of Post Office Creek immediately downstream of Wedge Street can be seen in Figure 4-20. The modelled flood extent correlates reasonably well with the floodwater also extending just beyond the tree line (Figure 4-17).

Figure 4-20 September 2010 event looking downstream of Wedge Street bridge along the northern bank of Post Office Creek (refer to Photo 3 in Figure 4-17)

4. The hydraulic model reproduces the extensive flooding that occurred over the southern bank of Post Office Creek downstream of Wedge Street as shown through a comparison of Figure 4-17 and Figure 4-21. In particular, the modelled flood just extends to the furthest of the two power poles as shown in Figure 4-21.

Figure 4-21 September 2010 event looking downstream of Wedge Street bridge along the southern bank of Post Office Creek (refer to Photo 4 in Figure 4-17)

5. The photo in Figure 4-22 was taken from Burton Avenue looking north toward the Campaspe River. It should be noted that this photo would most likely not have captured the peak flood extent which is estimated to have occurred during the night. Consequently, the photo shows some areas that are clearly above the floodwaters whereas the model displays the entire area as completely inundated. Nevertheless, it can be clearly seen in the photo that floodwater did extend up to the road and also that water extended up to the base of a row of four trees (shown on the far right of the photo) which has been reproduced by the model. Hence, given the uncertainty in the observation, this calibration is considered reasonable.

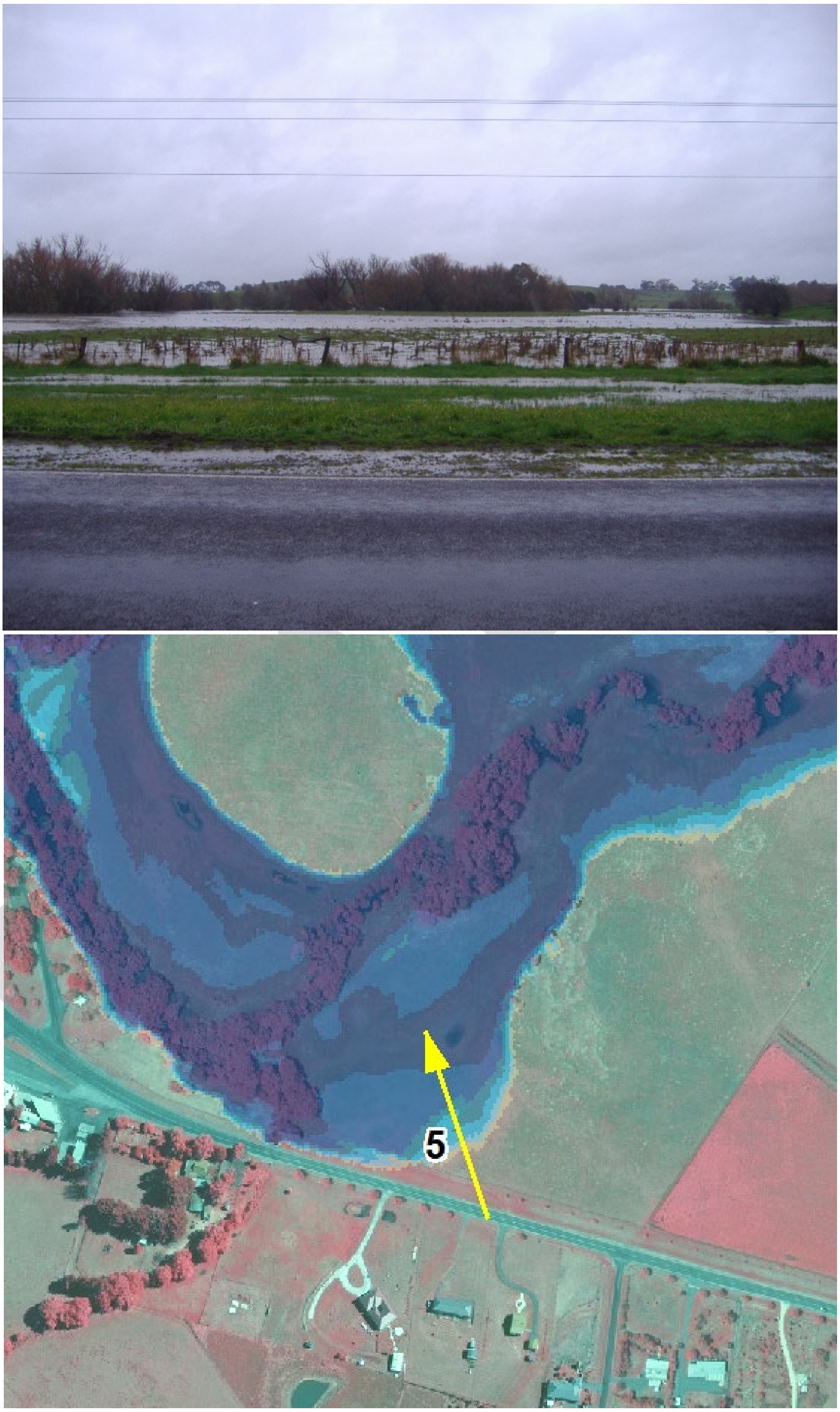


Figure 4-22 Campaspe River flood extent along Burton Avenue during the September 2010 event

4.3.4 September 2016 Validation

The hydrographs input into the hydraulic model for the September 2016 event are shown in Figure 4-23. The mapping outputs were compared to the historical observations to validate the hydraulic model parameters as described below. The location of the available validation data is shown in Figure 4-4.

It should be noted that the hydraulic model for the September 2016 event incorporates the recent willow removal works along sections of the Campaspe River by reducing the Manning's roughness value. Accordingly, the design hydraulic roughness grid (shown in Figure 4-3) was applicable for this event.

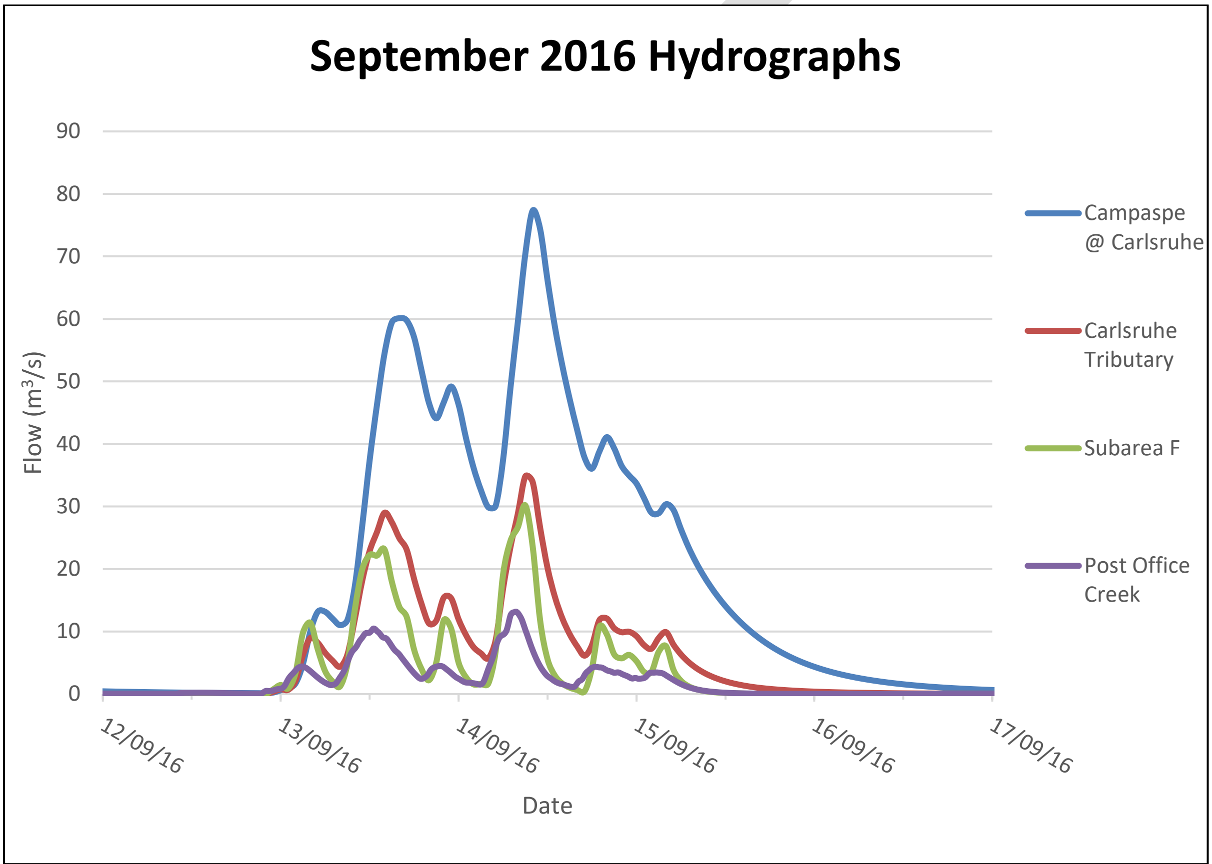


Figure 4-23 Inflow hydrographs for the September 2016 flood event

1. Figure 4-24 below shows a photo taken during the September 2016 flood event from Rennick Drive looking upstream along the Campaspe River. It should be noted that a subdivision has recently occurred on this site which is not shown on the older aerial photography. The subdivision itself has been developed outside the flood extent however the associated detention basin can be seen in the far right of the photo. Therefore, although minor alterations have since occurred in this area, the modelled flood extent still reasonably reflects the extent shown in the photo.

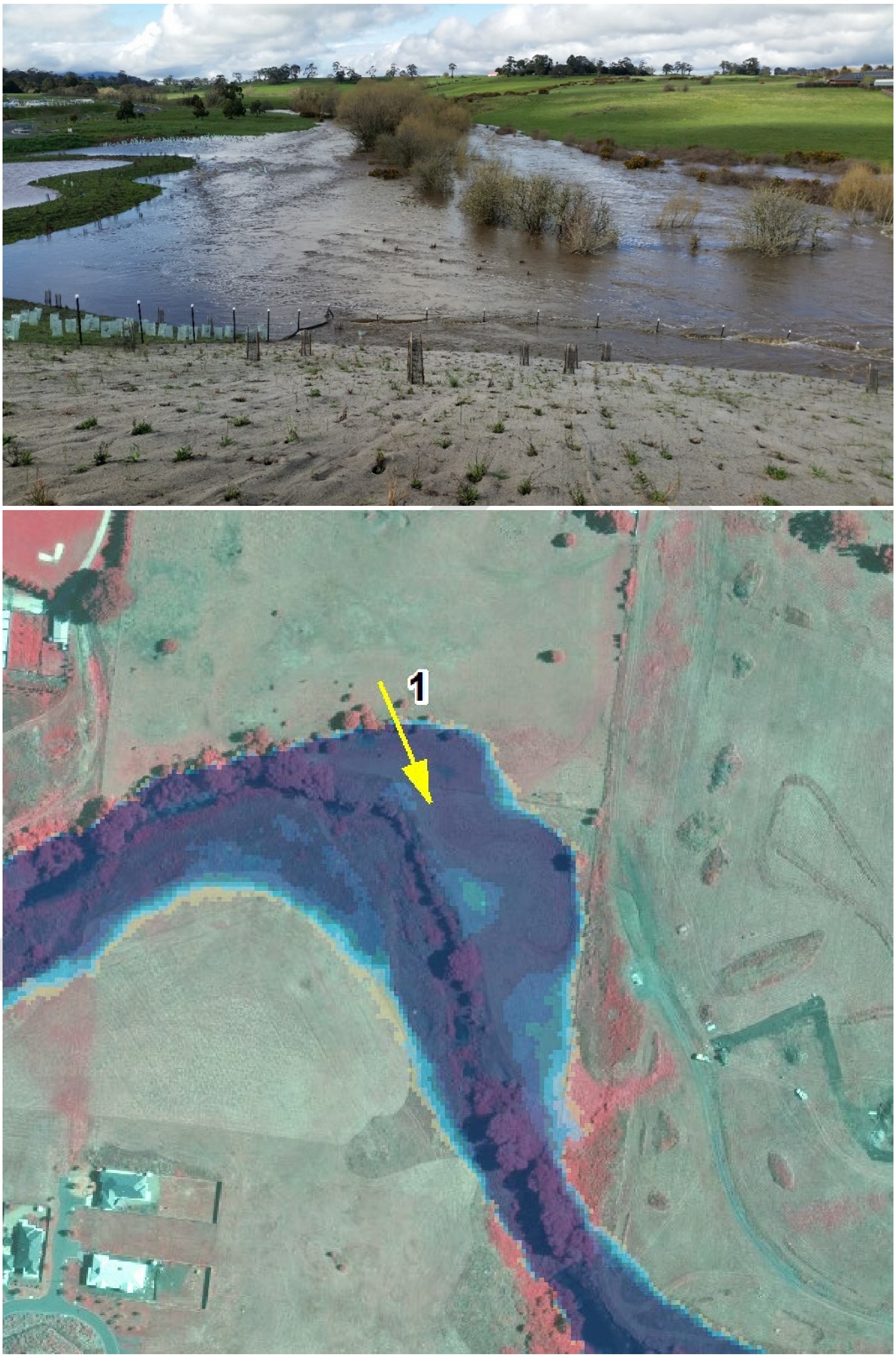


Figure 4-24 **Comparison of modelled flood extent to photo taken during the September 2016 event looking upstream on the Campaspe River from Rennick Drive**

2. The photo in Figure 4-25 was taken on the walking path adjacent to the Campaspe River, located near the north western corner of the Kyneton Botanic Gardens. The modelled data shows water abutting and overtopping this path in this area. This is in accordance with the photo which, although not taken at the height of the flood, clearly shows low areas of the path became completely inundated.

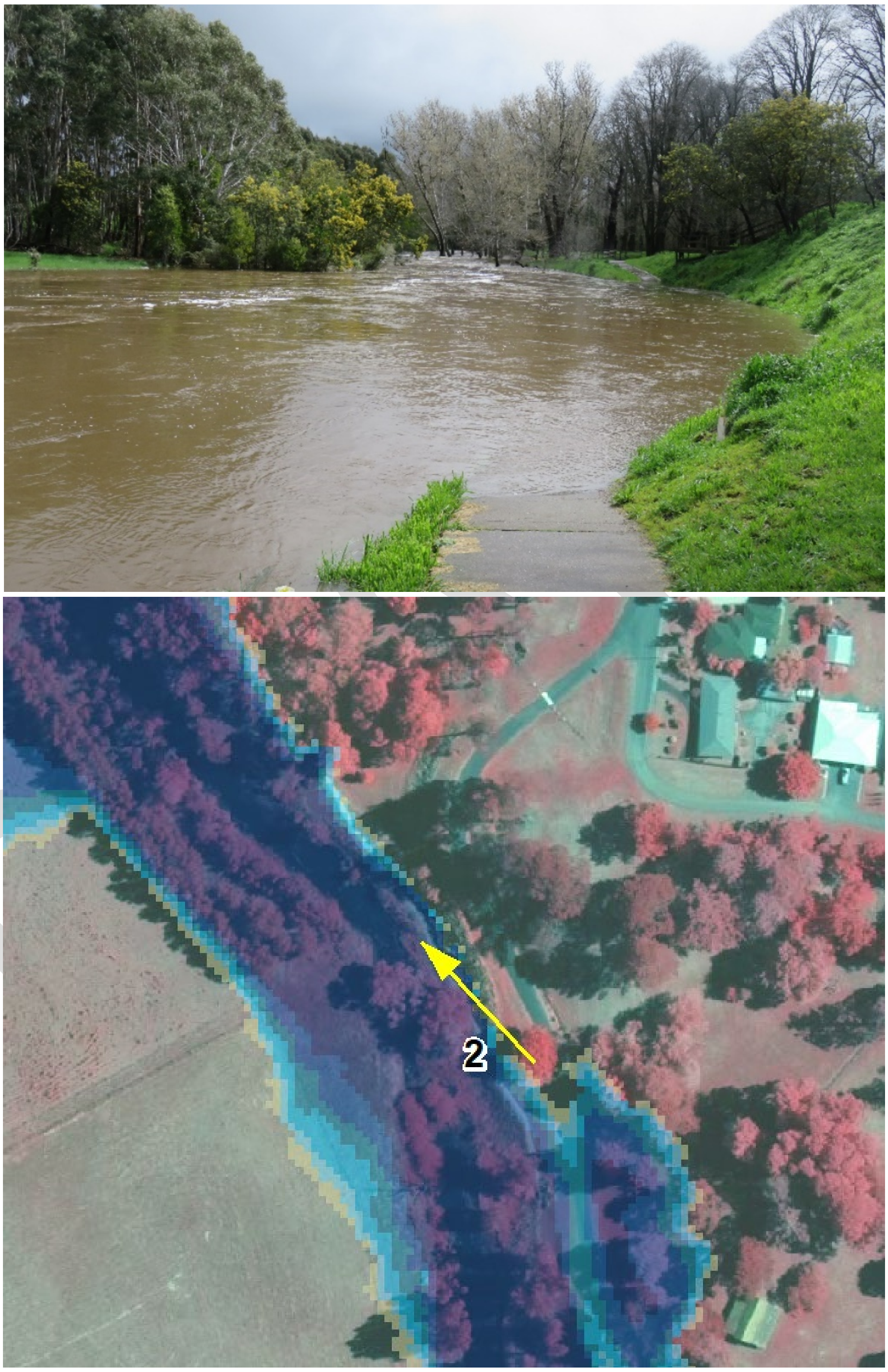


Figure 4-25 **Comparison of modelled flood extent to photo taken during the September 2016 event looking downstream on the Campaspe River from the Kyneton Botanic Gardens**

3. Figure 4-26 compares the modelled data at the Greenway Lane Weir to a photo taken during the September 2016 event. This photo was taken approximately 5 hours prior to the flood peak and hence the peak flood level and extent would have been considerably greater than what is shown in the photo. However, it can be seen that even prior to the peak, floodwater had begun to inundate the adjacent low lysing land which is reflected by the modelling. Additionally, observations at this site also indicated that this weir, which is over 1.5 metres high, became completely submerged during this flood resulting in a constant water surface slope across the weir. This flood behaviour was also replicated by the hydraulic model.

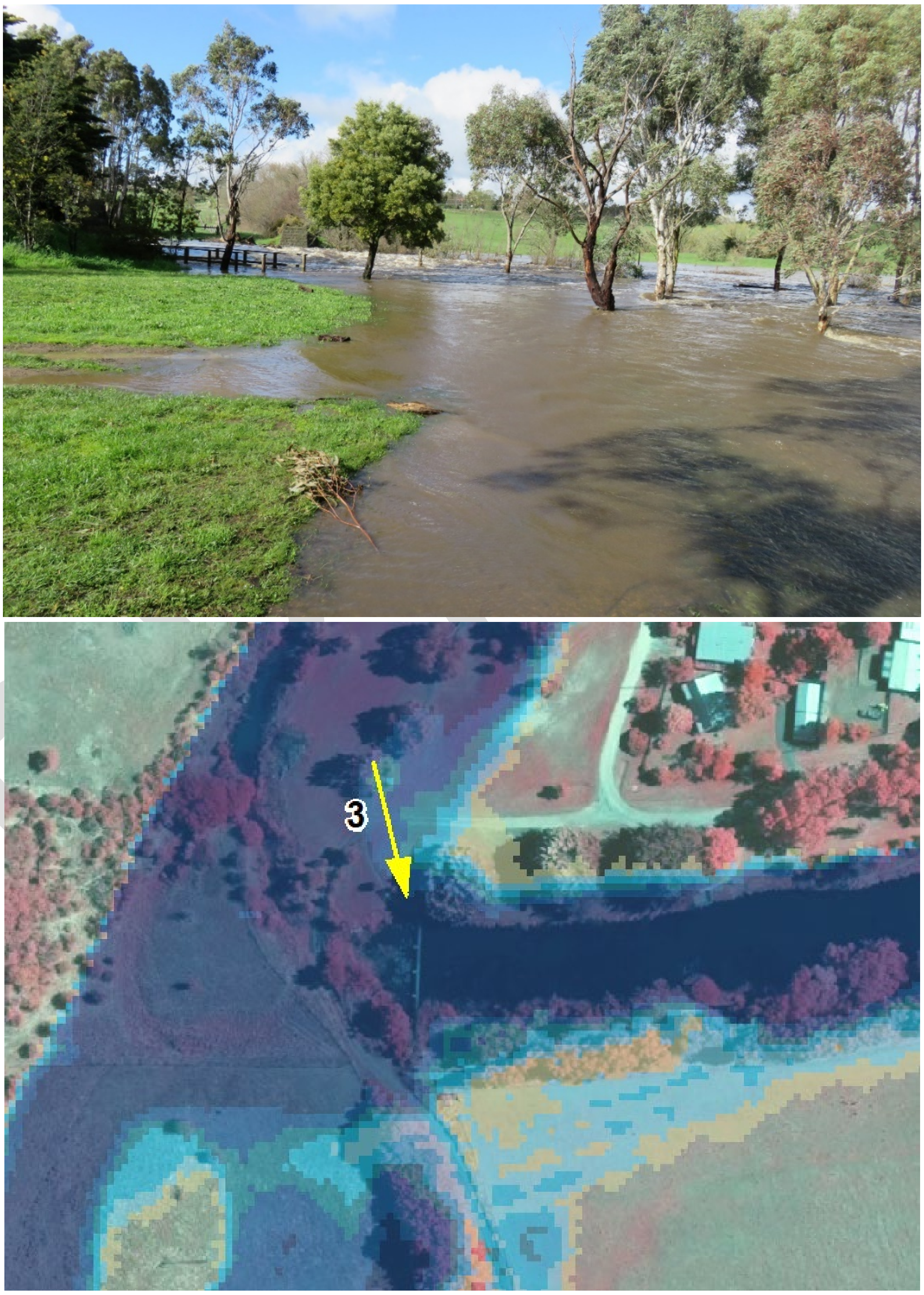


Figure 4-26 Comparison of modelled flood extent to photo taken during the September 2016 event of the Campaspe River at the Greenway Lane Weir

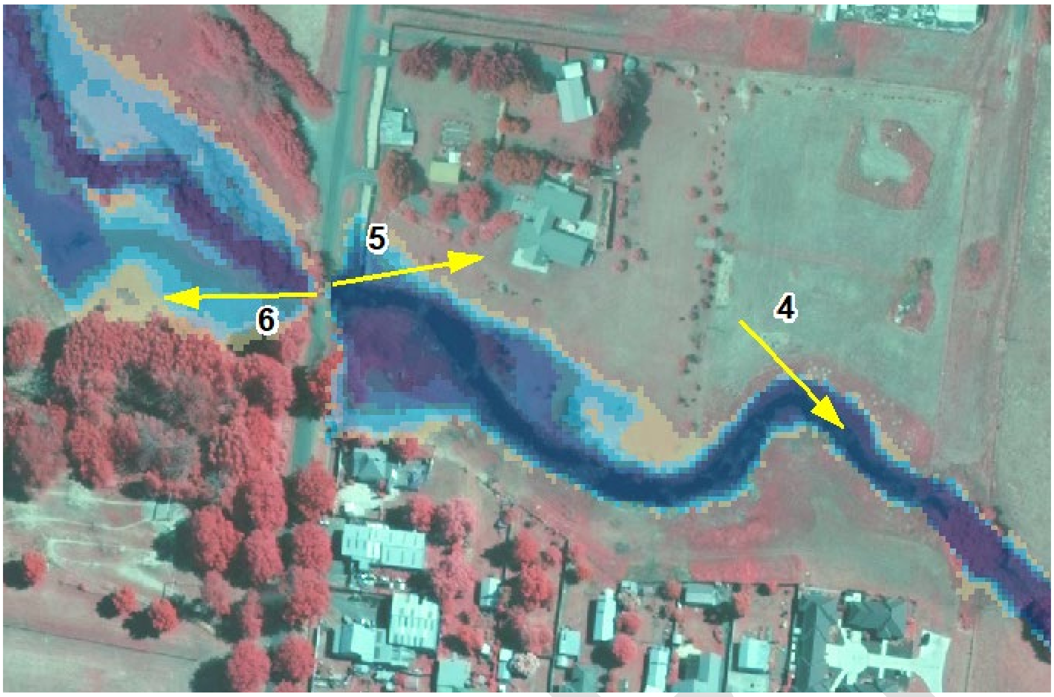


Figure 4-27 **Modelled flood extent of the September 2016 event at Wedge Street. Approximate direction of the following photos is indicated by yellow arrows.**

4. Floodwater was generally contained within the defined Post Office Creek channel upstream of Wedge Street. Figure 4-28 was taken looking upstream along Post Office Creek toward the end of Powlett Street. Figure 4-27 shows the modelled data in relation to the direction of this photo, indicating that the model reflects this flood behaviour.

Figure 4-28 **September 2016 event looking south-east on Post Office Creek upstream of Wedge Street (refer to Photo 4 in Figure 4-27)**

5. Figure 4-29 was taken on Wedge Street looking upstream along Post Office Creek toward the northern bank. The modelled flood extent shown in Figure 4-27 is in accordance with this observation, with floodwater inundating the adjacent garden area but not extending up to the existing dwelling.

Figure 4-29 September 2016 event looking toward northern bank of Post Office Creek immediately upstream of Wedge Street (refer to Photo 5 in Figure 4-27)

6. Figure 4-30 shows the extent of flooding over the southern bank of Post Office Creek immediately downstream of Wedge Street. The photo was taken the day before the peak however the flow rate is estimated to be close to the peak flow for this event. A comparison to Figure 4-27 shows that the observed flood extent is only slightly less than the extent modelled for the estimated peak flow as expected.

Figure 4-30 September 2016 event looking downstream of Wedge Street bridge along the southern bank of Post Office Creek (refer to Photo 6 in Figure 4-27)

4.3.5 Summary

The model results for the calibration (January 2011) and validation (September 2010 and September 2016) flood events replicate the observed flood behaviour along the Campaspe River and Post Office Creek reasonably accurately based on photographs and anecdotal observations. However, the January 2011 model results do not correlate to the surveyed flood mark on Post Office Creek. Reasons for this poor calibration may include:

- The flood mark is located near the Post Office Creek inflow boundary and model results within this proximity are inherently uncertain. Ideally the model inflow boundary would be located a sufficient distance from areas of interest to avoid boundary condition influences. However, due to a lack of available LiDAR data, the hydraulic model cannot be extended upstream any further.
- The available LiDAR is significantly obscured in some areas by the heavy vegetation within Post Office Creek. Although slight modifications have been made to ensure the model represents the topography it is possible that the LiDAR does not accurately represent the channel form in some areas.

- There is uncertainty regarding potential blockages of structures during the event. The waterway is heavily vegetated and crossed by several hydraulic structures that are susceptible to blockage. Blockages have the potential to significantly impact on upstream and downstream flood levels and flows. It is possible that one of the structures upstream of the flood mark was significantly blocked during the January 2011 flood event, resulting in reduced flows downstream and lower flood levels at the site.
- Only a single flood mark is available for calibration and hence it in cannot be validated. Validation of flood marks is important as there may be some uncertainty around the accuracy of the flood mark, particularly if it is based on debris marks. For example, a debris mark may be higher than the actual flood level due to wave action, or underestimate the flood level due to larger debris only being deposited as floodwaters subside. In this case, there are no details available regarding what this flood mark was based on.
- Due to the relatively small catchment size and significant proportion of impervious areas associated with urban land uses, runoff flows for Post Office Creek are correlated closely with rainfall temporal patterns. As the nearest pluviograph stations are approximately 12 kilometres away there exists some uncertainty relating to the applicability of these temporal patterns to the catchment. For example, the temporal pattern from pluviograph station 406266 was initially applied for the January 2011 calibration as it was the closest station. However, due to a short-duration high-intensity rainfall burst, this temporal pattern produced a significant peak flow, exceeding the 0.5% AEP design event, which was not experienced based on the available evidence. Therefore, the temporal pattern was derived instead from pluviograph station 406250 which was located at a similar distance from the catchment. This produced more realistic flows, one third of the size of the previous peak flow. Hence, the RORB model for Post Office Creek appears to be particularly sensitive to rainfall temporal patterns and there is uncertainty as to whether the nearby pluviograph stations provide representative patterns.

Methods to improve the calibration on Post Office Creek are discussed further in Section 0. Overall, the hydraulic model reproduces the observed flood behaviour during the three historical events reasonably well. Therefore, the hydraulic model is considered to be appropriate for use in generating design flood events.

4.4 Sensitivity Analysis

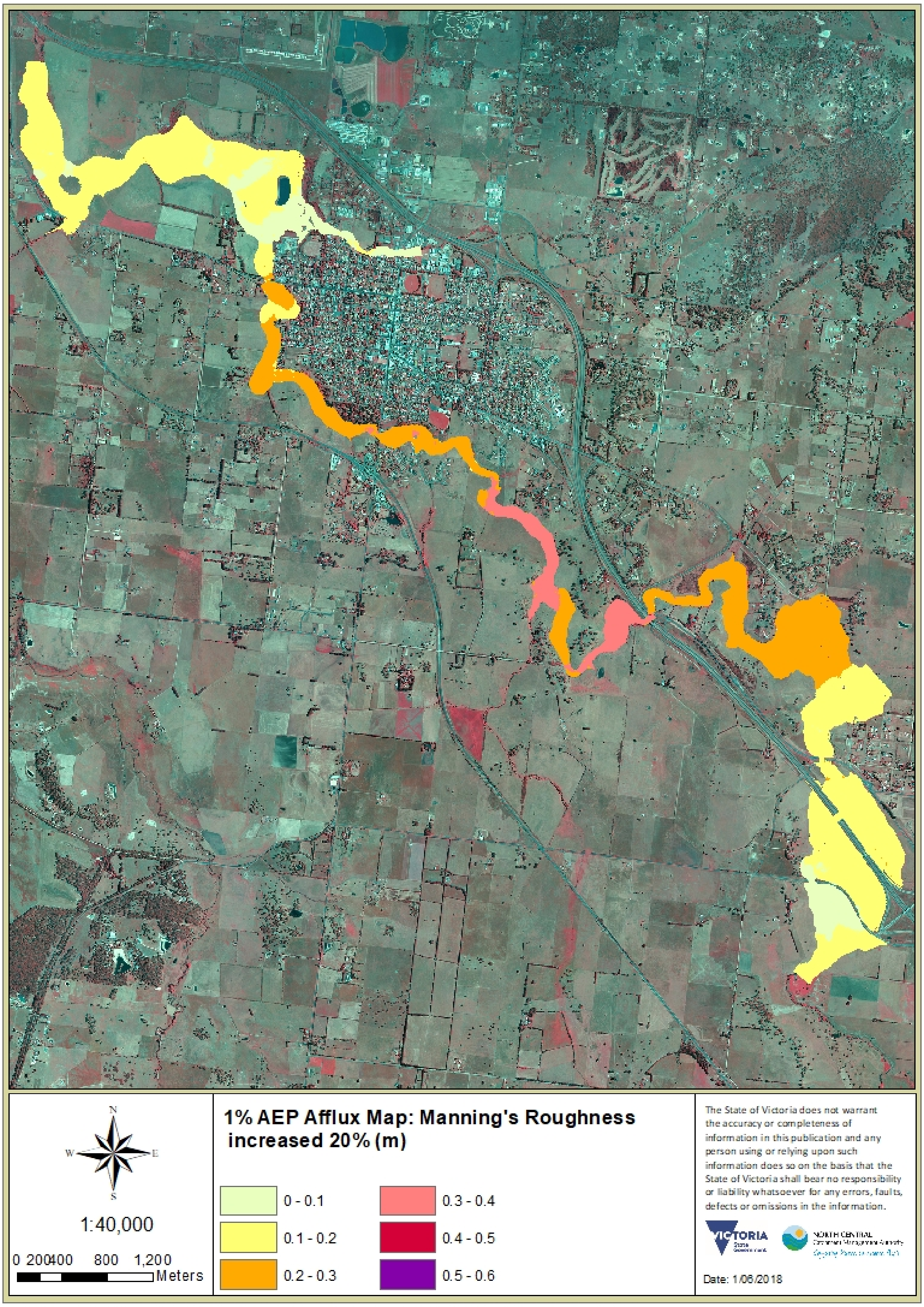
The hydraulic model sensitivity was tested by varying the Manning's roughness values, the downstream outflow boundary condition, the model inflows and the hydrograph volumes to determine the influence of these parameters on the model results. The sensitivity analysis was undertaken based on the 1% AEP design results. These various scenarios are detailed in the following sections.

4.4.1 Roughness Sensitivity

The model sensitivity to the Manning's roughness values was analysed by varying these values by 20% and comparing the results to the base case scenario. The Manning's roughness values adopted for the base case scenario are detailed in Section 4.2.2.6. With the roughness increased by 20% the flood levels were increased by an average of 0.18m. The maximum localised increase in flood height was 0.58m. Due to the steep slopes of the catchment the flood extent for both the Campaspe River and Post Office Creek was only slightly increased. Figure 4-31 below shows the afflux caused by increasing the Manning's roughness by 20%.

Similarly, with the Manning's values reduced by 20%, the flood levels were decreased by an average of 0.19m. Again, the difference in extent was very minor as shown in Figure 4-32. The maximum decrease in flood height was 1.0m, the location of which is shown in the red insert in Figure 4-32. The significant difference at this site is due to the topography of the floodplain. In the sensitivity scenario, the floodwater only briefly overtops a bank to fill this depression but does not maintain this height long enough to equalise the flood level adjacent to the depression.

Overall, a comparison of Figure 4-31 and Figure 4-32 indicate that there is minimal change in flood level based on the roughness selected for Post Office Creek and the wider sections of the Campaspe floodplain. However, the confined reaches of the Campaspe River through Kyneton do appear to be sensitive to the Manning's roughness value applied.



T:\GISData\FLOODPLAIN\MACEDON RANGES\KYNETON\KYNETON FLOOD STUDY 2014\CMA DATA\Kyneton_Study_(20180530_MGA55).mxd

Figure 4-31 **1% AEP afflux map comparing base case scenario to sensitivity scenario with Manning's roughness increased by 20% (Sensitivity Scenario – Base Case Scenario)**

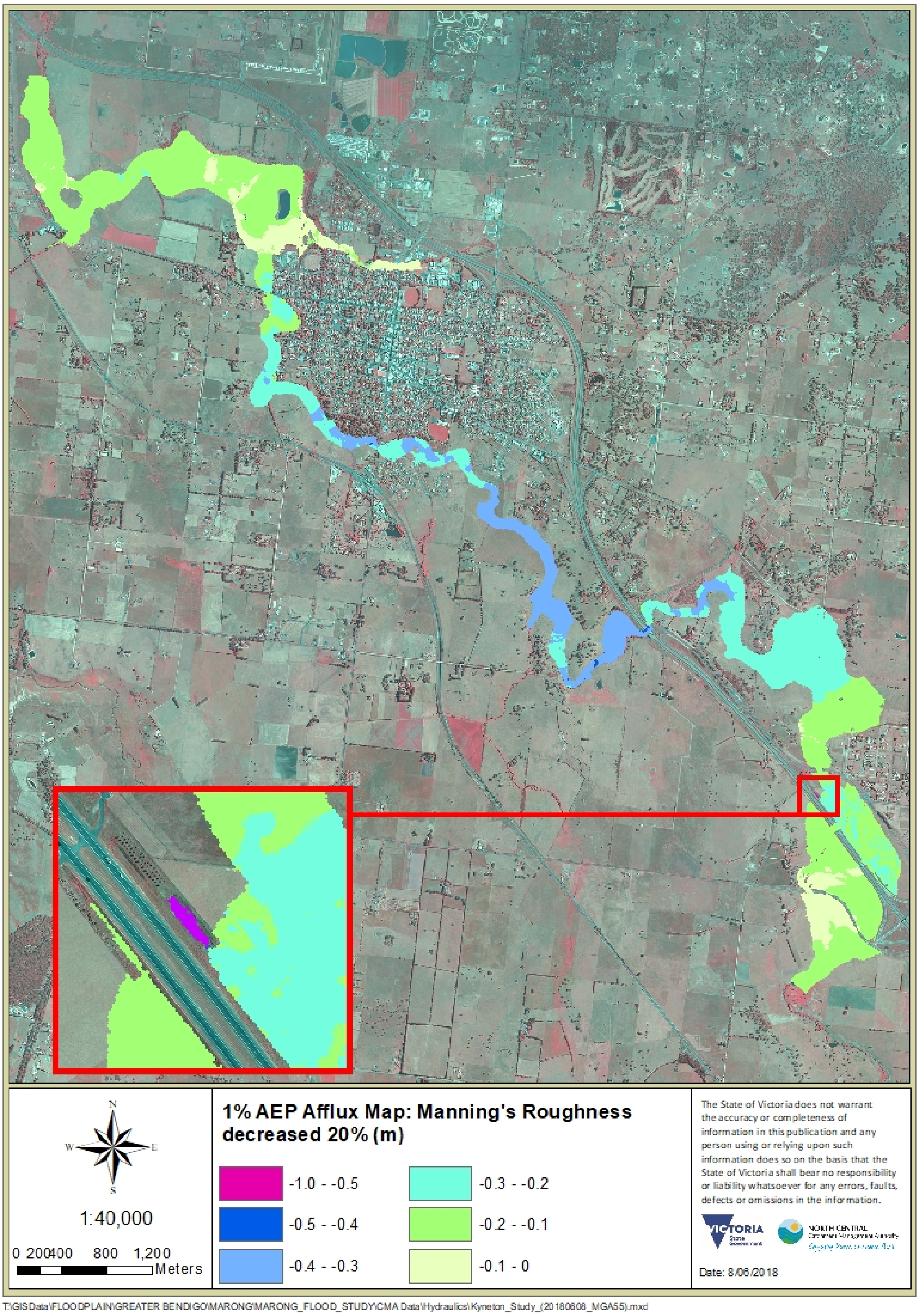


Figure 4-32 **1% AEP afflux map comparing base case scenario to sensitivity scenario with Manning's roughness decreased by 20% (Sensitivity Scenario – Base Case Scenario). The red insert shows the area of maximum difference.**

4.4.2 Outflow Boundary Condition Sensitivity

The sensitivity of the hydraulic model to the outflow boundary condition was also analysed. The base case scenario applied a water surface slope of 0.01 at the outflow boundary to determine the flow rate of water leaving the model. This was compared to two sensitivity scenarios, the first of which reduced the water surface slope to 0.002, and the second where it was increased to 0.02.

As expected, the flood levels near the outflow model boundary are increased when the boundary condition slope is reduced to 0.002 as shown in Figure 4-33. The afflux immediately upstream of the model boundary is significant, with flood levels approximately 1.0m higher than the base case scenario. However, the impacts are quickly dissipated further upstream of the boundary to less than a 50mm increase at a distance of 500m from the boundary. Due to the steep river banks this afflux only results in a relatively small increase in flood extent, generally less than 20m. Moreover, the additional area impacted is farm land with no development located within this vicinity.

Figure 4-34 shows the results due to increasing the outflow boundary slope to 0.02. As shown the flood levels are slightly reduced as compared to the base case scenario. The maximum decrease in flood level is approximately 0.25m immediately at the outflow boundary. At a distance of 70m upstream of the outflow boundary the difference in flood level is less than 50mm. Due to the relatively small decrease in flood level there is no significant change in flood extent for this scenario.

Overall, the extent of influence due to the outflow boundary is generally minor and restricted to farm land. Therefore, it is considered that the hydraulic model is not particularly sensitive to the outflow boundary conditions and the areas of interest for the model are not impacted.

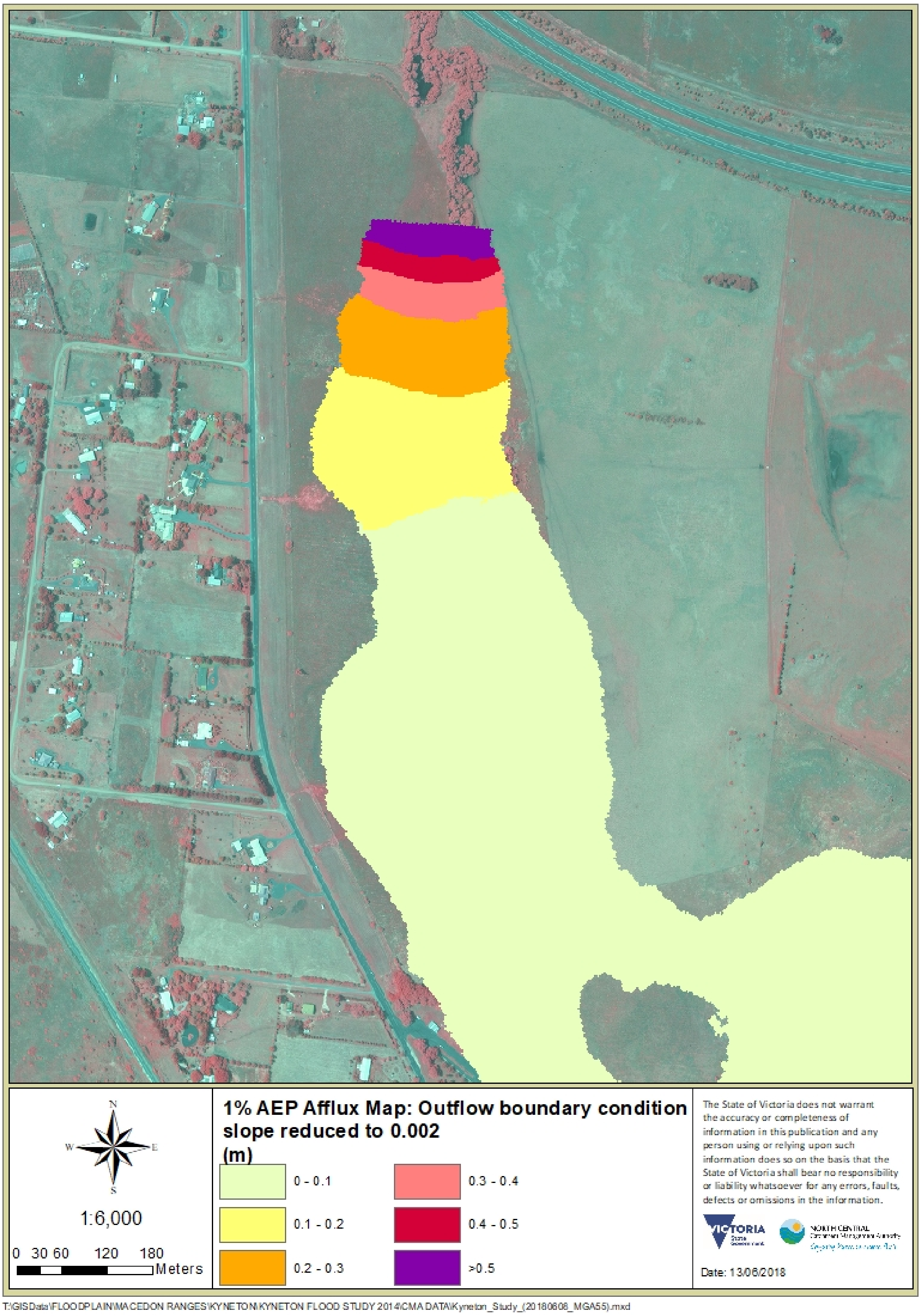
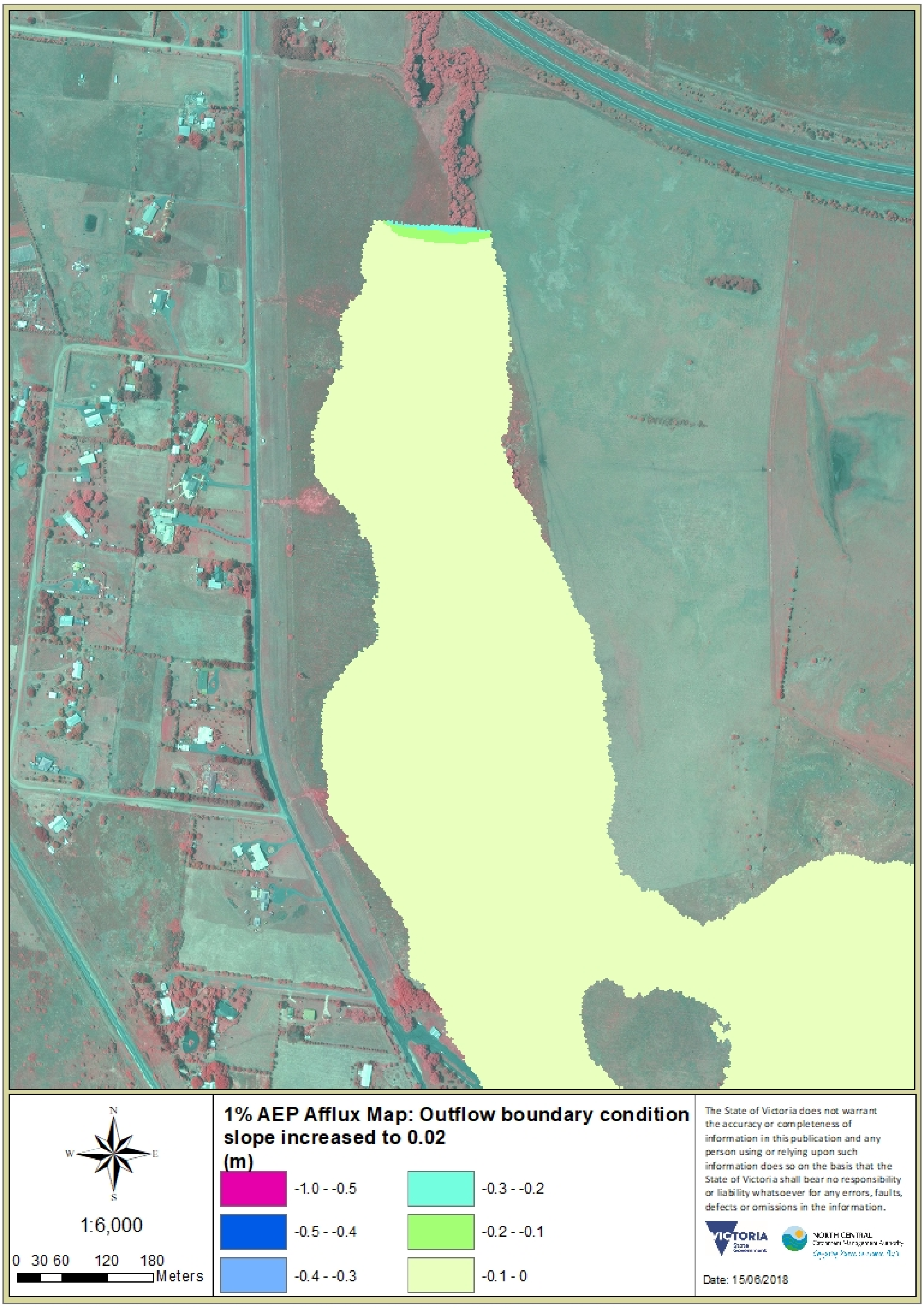


Figure 4-33 **1% AEP afflux map comparing base case scenario to sensitivity scenario with the outflow boundary slope reduced to 0.002 (Sensitivity Scenario – Base Case Scenario)**



T:\GISData\FLOODPLAIN\MACEDON RANGES\KYNETON\KYNETON FLOOD STUDY 2014\CMA DATA\Kyneton_Study_(20180608_MGA55).mxd

Figure 4-34 **1% AEP afflux map comparing base case scenario to sensitivity scenario with the outflow boundary slope increased to 0.02 (Sensitivity Scenario – Base Case Scenario)**

4.4.3 Model Inflow Sensitivity

The sensitivity of the hydraulic model to the inflows was tested by varying all inflow hydrographs by 10%. Figure 4-35 shows the afflux due to the inflow hydrographs being increased by 10%. The flood levels are only increased an average of 0.1m compared to the base case scenario, with localised increases on over 0.2m. Moreover, there is no material increase in flood extent.

The afflux results for a 10% reduction of the inflow hydrographs is shown in Figure 4-36. There is an average decrease of 0.1m in flood level and an overall minor decrease in flood extent. The areas impacted by variations in the model inflow appear to be reasonably consistent as shown by comparing Figure 4-35 and Figure 4-36. The Campaspe River reach through Kyneton appears to be the most sensitive to changes in flow, with levels varying by approximately 0.2m along this section due to a 10% increase or decrease in the inflow hydrographs.

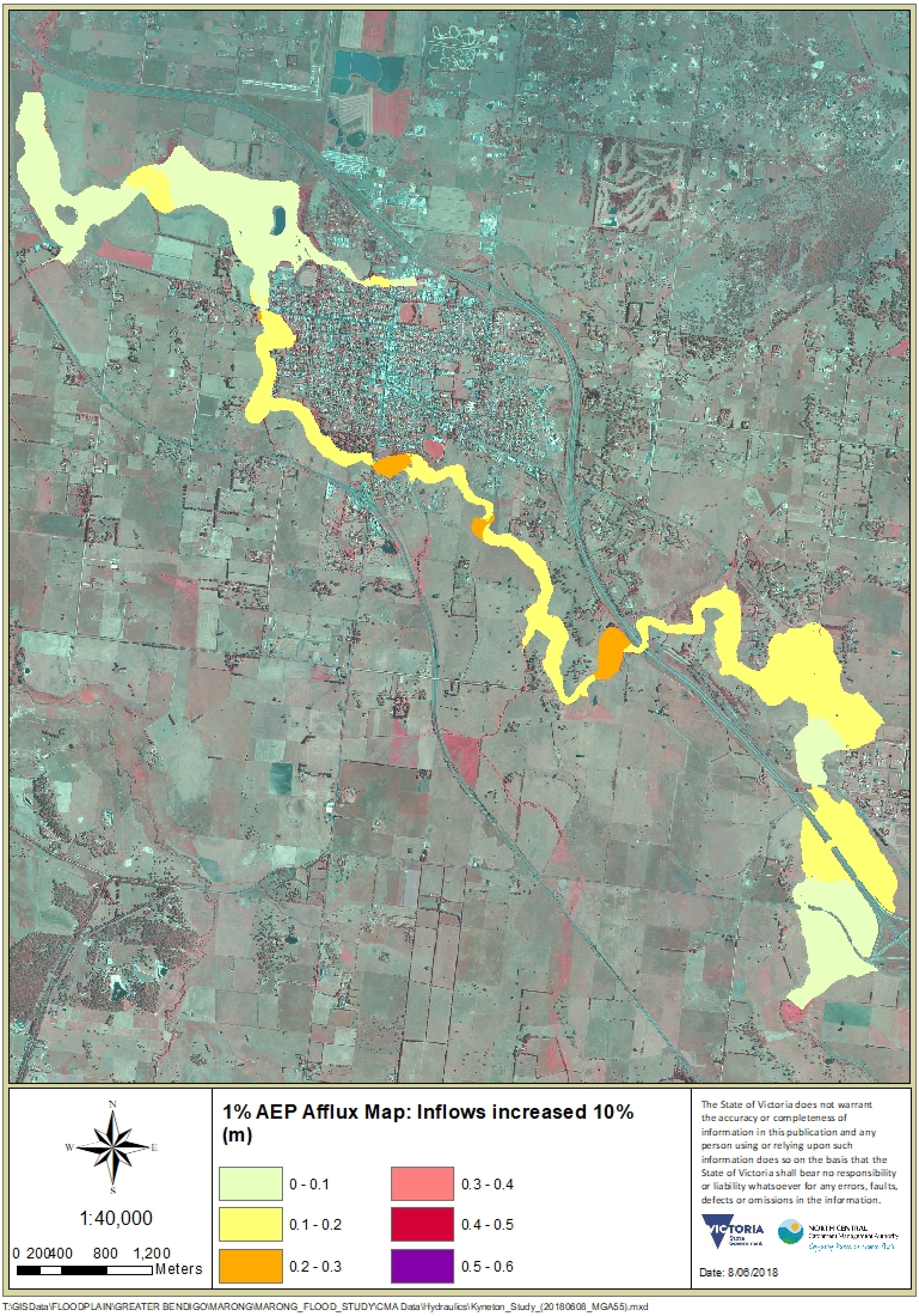
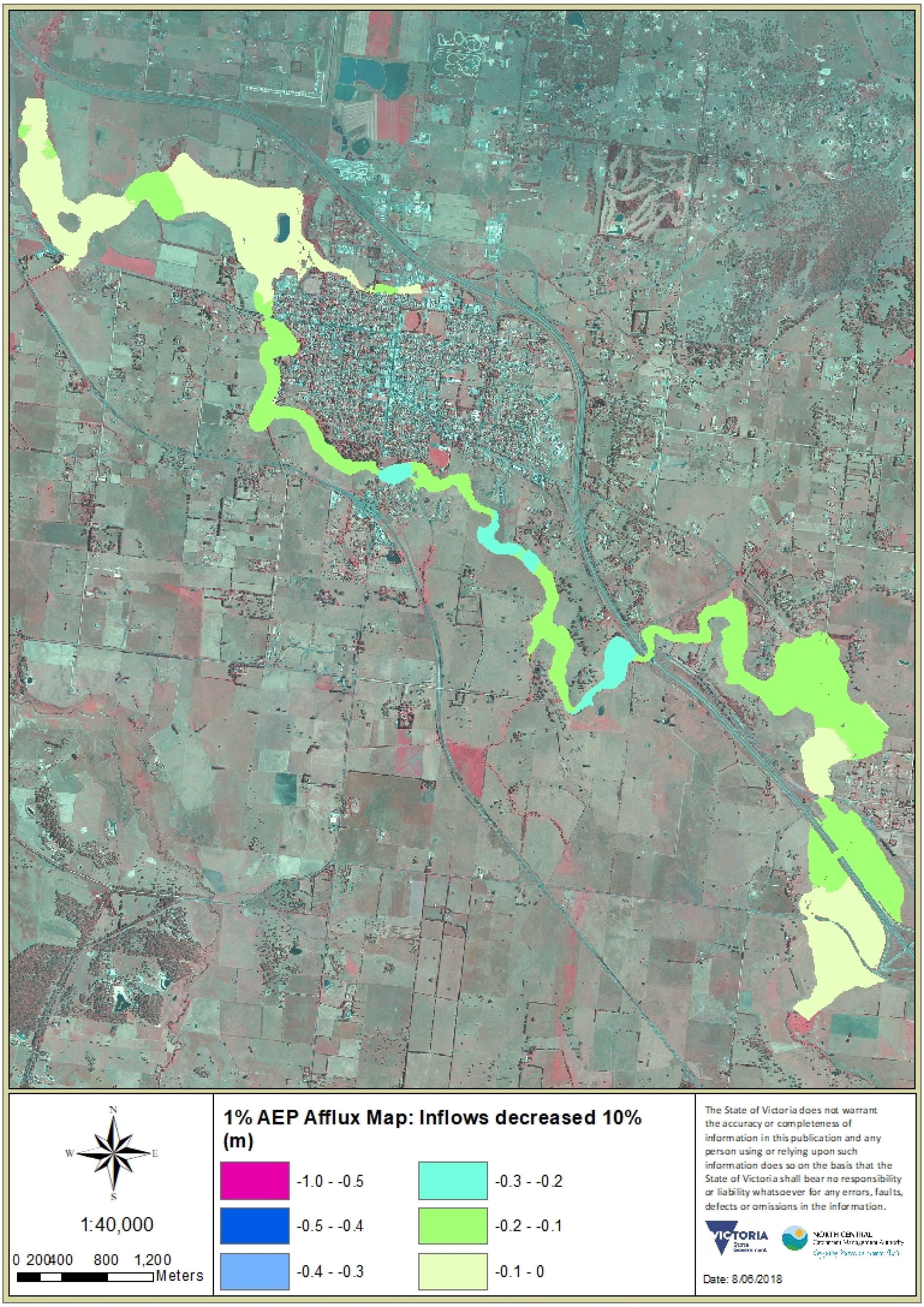


Figure 4-35 1% AEP afflux map comparing base case scenario to sensitivity scenario with inflows increased by 10% (Sensitivity Scenario – Base Case Scenario)



T:\GISData\FLOODPLAIN\GREATER BENDIGO\MARONG\MARONG_FLOOD_STUDY\CMA Data\Hydraulics\Kyneton_Study_(20180608_MGA55).mxd

Figure 4-36 1% AEP afflux map comparing base case scenario to sensitivity scenario with inflows decreased by 10% (Sensitivity Scenario – Base Case Scenario)

4.4.4 Hydrograph Volume Sensitivity

It is important to note that the preceding hydrologic analysis detailed in Section 3 is dependent on the implicit assumption that the peak flood levels on the floodplain occur coincidentally with the peak catchment flow rate. However, this is not necessarily the case. For some floodplains, peak flood conditions are controlled by the total hydrograph volume with peak water levels occurring significantly after the peak flow has passed. Hence, it is essential that sensitivity testing be undertaken to justify this assumption.

The sensitivity of the hydraulic model to hydrograph volume was tested by increasing the volume of the 1% AEP inflow hydrographs by 25%. The duration of the peak flow rate for each hydrograph was extended to achieve the 25% increase in volume. A comparison of the hydrographs at the Campaspe @ Carlsruhe model inflow location is shown in Figure 4-37 below to illustrate this method. It should be noted that this is a conservative approach to the volume analysis as the peak flow rate would typically be lower for a hydrograph with greater volume.

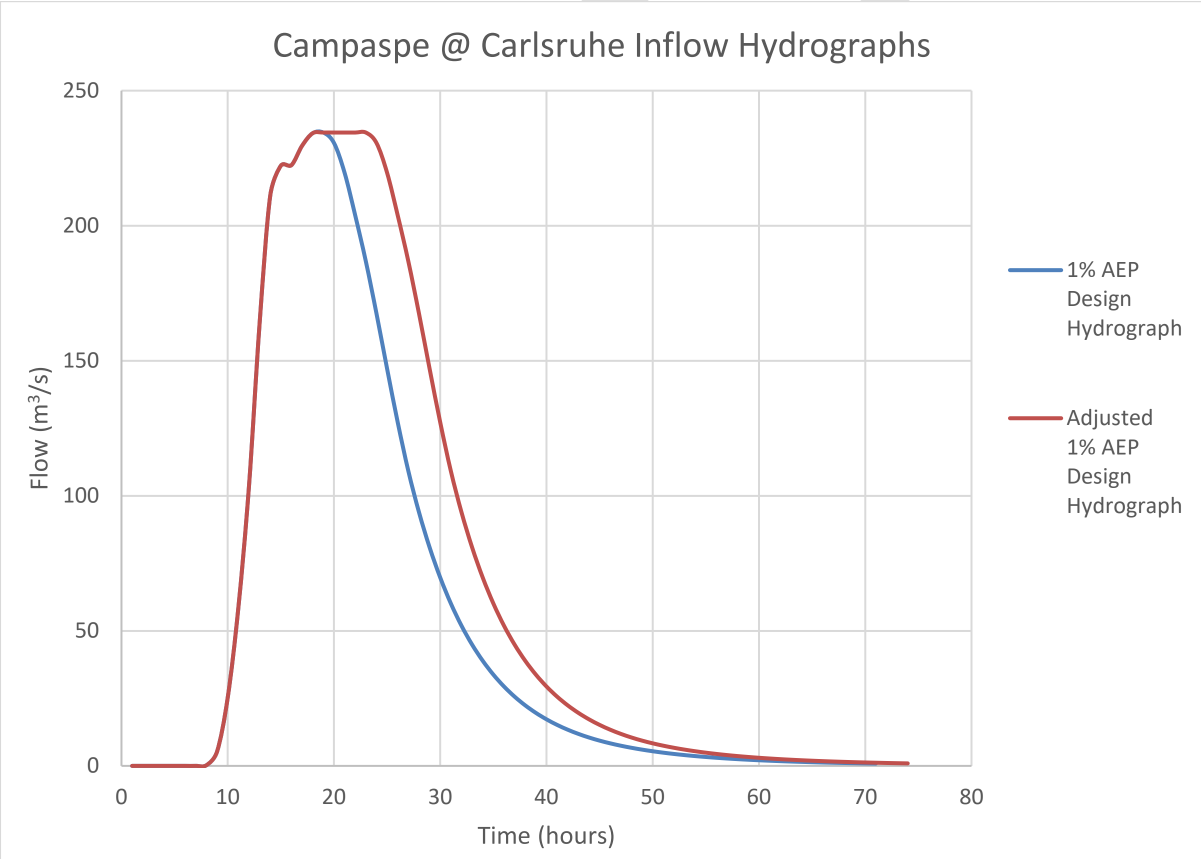
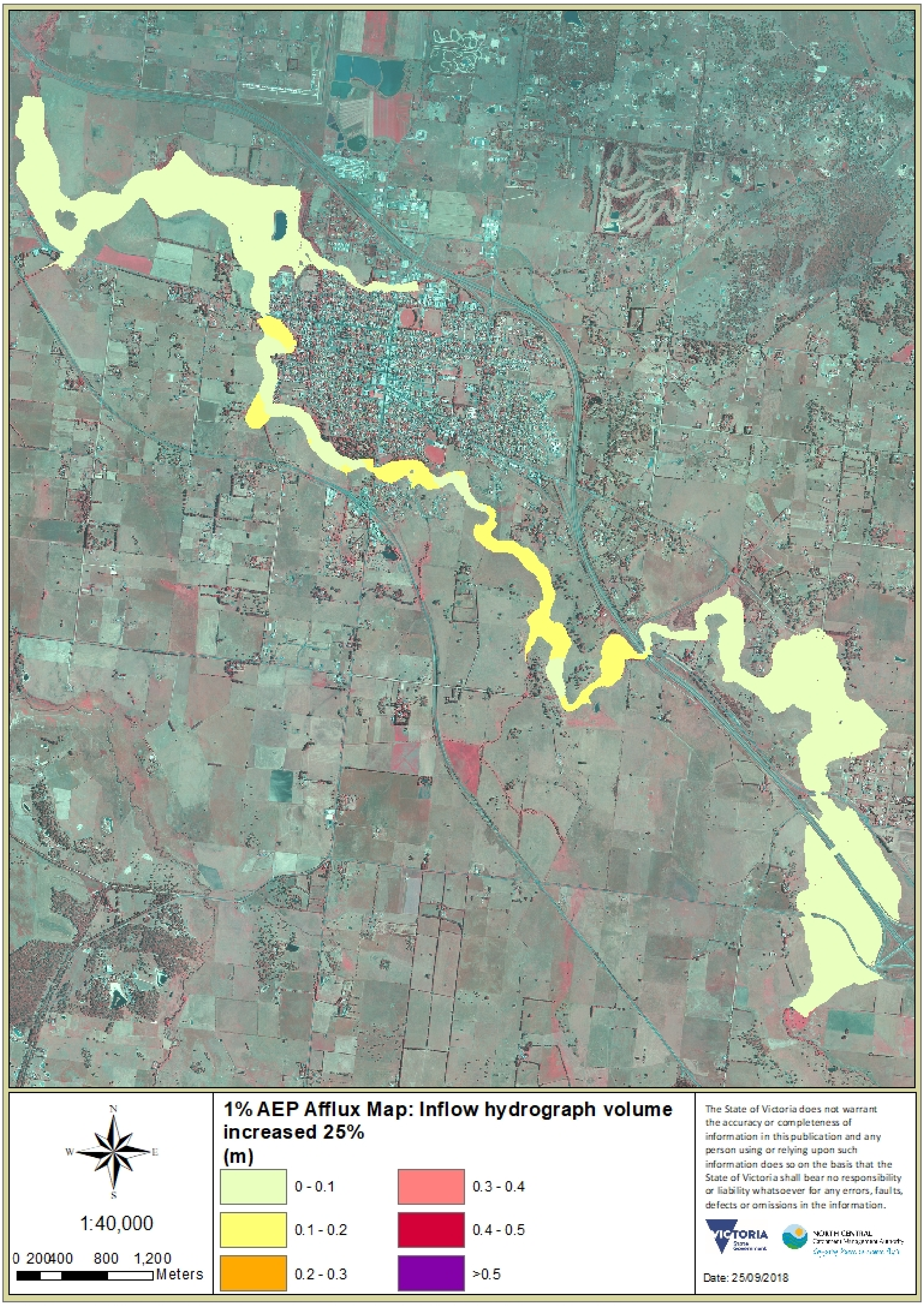


Figure 4-37 Campaspe @ Carlsruhe 1% AEP design inflow hydrograph volume increased by 25%

Figure 4-38 shows the difference in flood height due to the inflow hydrograph volumes being increased by 25%. There is an average increase of 50mm in flood height across the entire model domain, with the maximum afflux limited to less than 140mm, resulting in a negligible increase in flood extent.

Although there is a slight increase in flood levels the afflux is relatively minor particularly considering that this is a conservative estimation. In comparison, Section 4.4.3 discusses the impacts of increasing flows by 10%, which effectively increases the hydrograph volume by approximately only 10%. In that analysis, the average increase in flood levels was 100mm, twice the afflux caused by increasing the volume by 25%. Consequently, this indicates that peak flow rate has a greater influence than total hydrograph volume on flood behaviour. This is also consistent with the topography of this waterway reach, characterised by steep slopes with confined floodplains, which is typically associated with flood behaviour that is controlled by peak flow rate. Hence, this analysis demonstrates that the increase in flood level due to the increased hydrograph volume is negligible, thus validating the assumption that peak flood conditions for this floodplain are dependent on peak flow rate as opposed to hydrograph volume.



T:\GISData\FLOODPLAIN\MACEDON RANGES\KYNETON\KYNETON FLOOD STUDY 2014\CMA DATA\Kyneton_Study_(20180921_MGA55).mxd

Figure 4-38 **1% AEP afflux map comparing base case scenario to sensitivity scenario with inflow hydrographs increased by 25% (Sensitivity Scenario – Base Case Scenario)**

4.5 Design Flood Modelling

4.5.1 Coincidence of Campaspe River and Post Office Creek flows

There is a significant difference in catchment size between Post Office Creek and the Campaspe River ($12km^2$ and $233km^2$ respectively). Hence, for a given storm event, the peak flow from Post Office Creek will pass through Kyneton hours before the Campaspe River peaks within the Township. Consequently, flood interactions between the two waterways are limited. Furthermore, due to the steep gradient of both waterways and the fact that the location of the confluence is located approximately one kilometre north of the township, any localised impacts from backwater would be limited. Therefore, the design hydrographs for both the Campaspe River and Post Office Creek have simply been modelled together.

4.5.2 Blockage of Structures

Blockage of bridges and culverts were assessed in accordance with ARR, Book 6, Chapter 6. Blockage assessments were undertaken for three structures on the Campaspe River and three on Post Office Creek, as described in the following sections. These structures were selected based on the potential of the blockages to impact on urban areas. Details of these structures are provided in Section 2.1.2.

The blockage scenarios were modelled simultaneously on the Campaspe River and Post Office Creek since any impacts from the blocked structures are localised on both waterways and therefore the flood behaviour effects are independent. Regarding the blockage combinations that were simulated, it was considered that if multiple structures on the same waterway were blocked the restricted flows from the upstream blockages would lessen the impacts of downstream blockages. Hence, a single bridge on each waterway was modelled as blocked while the other structures remained clear for each scenario as this appears to result in the most adverse flood behaviour.

The ‘all clear’ base scenario was then augmented with the various blockage scenarios and an envelope of the maximum results was created for the 1% AEP flood event. This ensures that the impacts of individual blocked structures are properly simulated in the enveloped solution in addition to the ‘all clear’ flood impacts. However, it must therefore be noted when considering the results that in any single historic event, the recorded flood surface will likely only reach the envelope levels at some locations due to the variability in actual blockages (ARR, Book 6, Section 6.4.4.10).

4.5.2.1 Blockage determination for Campaspe River structures

A blockage assessment of the following three Campaspe River bridges was undertaken:

- S7 – Cobb and Co Road North Bridge
- S10 – Mollison Street Bridge
- S13 – Piper Street Bridge

These structures were specifically selected based on their proximity and potential impact on urban areas. Figure 2-5 and Figure 2-6 display the locations of these bridges. It should be noted that the Cobb and Co Road North Bridge (S7 in Figure 2-5) was selected to be block rather than the Calder Highway

North Bridge (S8) as it is located immediately upstream of the later bridge and has a shorter span. Therefore, any blockage is more likely to occur at this bridge and also hence reduce the potential for blockage at the downstream bridge.

The blockage assessment for the structures is detailed below:

1) Debris Types and Dimensions – estimated L_{10} = 3m, where L_{10} is defined as the average length of the longest 10% of the debris reaching the structure. An example of debris located upstream of Piper Street bridge is shown in Figure 4-39.
2) Debris Availability = Medium
3) Debris Mobility = High
4) Debris Transportability = High
5) 1% AEP Debris Potential = High (HHM from above assessment)
6) AEP Adjusted Debris Potential = High (for 5% - 0.5% AEP)
7) Most Likely Inlet Blockage, $B_{DES\%}$ = 10% (Clear width of inlet (bridge spans) is greater than 13m, hence $W>3*L_{10}$)

Hence, a separate simulation was modelled with each bridge having the determined blockage factor of 10% applied.

Figure 4-39 Debris during the September 2010 flood event (Kyneton Historical Society, 2010)

4.5.2.2 Blockage determination for Post Office Creek structures

Similarly, the blockage assessment of the four 1.8m diameter culvert structures on Mollison Street and Ebden Street (structures S15 and S16 respectively in Figure 2-6) is described below:

1) Debris Types and Dimensions – estimated L_{10} = 2m, where L_{10} is defined as the average length of the longest 10% of the debris reaching the structure
2) Debris Availability = High
3) Debris Mobility = Medium
4) Debris Transportability = Medium
5) 1% AEP Debris Potential = Medium (HMM from above assessment)
6) AEP Adjusted Debris Potential = Medium (for 5% - 0.5% AEP)
7) Most Likely Inlet Blockage, $B_{DES\%}$ = 50% (Clear width of inlet (i.e. culvert diameter) is 1.8m, hence $W<L_{10}$)

Figure 4-40 below shows an example of a blockage that has occurred at the Ebden Street culverts, indicating that the estimated culvert blockage of 50% is reasonable.

Figure 4-40 Blockage at the Ebden Street culverts

Additionally, for the single span bridge on Wedge Street (S17), the blockage assessment was as follows:

1) Debris Types and Dimensions – estimated L_{10} = 2m where L_{10} is defined as the average length of the longest 10% of the debris reaching the structure
2) Debris Availability = High
3) Debris Mobility = Medium

4) Debris Transportability = Medium
5) 1% AEP Debris Potential = Medium (HMM from above assessment)
6) AEP Adjusted Debris Potential = Medium (for 5% - 0.5% AEP)
7) Most Likely Inlet Blockage, $B_{DES\%}$ = 10% (Clear width of inlet (i.e. bridge span) is 5.1m, hence $L_{10} \leq W \leq 3*L_{10}$)

It should be noted that willow removal works have been undertaken along a segment of the waterway immediately upstream of the Wedge Street bridge. Anecdotal evidence provided by a local landowner suggests that minimal debris is transported along this section of waterway and that no significant blockage at this bridge has previously occurred. This is most likely due to the restriction caused by the upstream culvert structures at Ebden Street and Mollison Street which limit the debris that arrives at the Wedge Street bridge. Therefore, applying a 10% blockage factor is considered appropriate.

Hence, scenarios of each culvert structure on Post Office Creek with a 50% blockage were modelled separately, in addition to a scenario with the Wedge Street bridge blocked by a factor of 10%.

4.5.2.3 Impact of blockages

A comparison of the blockage scenarios to the base case scenario for the 1% AEP design event shows that the impacts of typical blockages on the Campaspe River bridges are minor with localised increases of less than 100mm upstream of the structures. However, the impacts of structure blockages on Post Office Creek are more significant due to the likelihood of a large blockage occurring. In particular, the afflux at Ebden Street due to a 50% blockage of the culverts increases upstream flood levels by up to 450mm.

4.5.3 Model Quality Assurance

To ensure the modelling was fit for purpose, the TUFLOW model results were assessed. Checks were made to ensure that input data such as topography, surface roughness, and hydraulic structures were appropriately represented by the hydraulic model. Model inflow and outflow boundaries were located a sufficient distance from areas of interest to ensure that the boundary conditions did not influence model results. The absence of any negative depth warnings or significant volume fluctuations for the modelled events also indicated the stability of the hydraulic model. Furthermore, the peak cumulative mass error for the various model scenarios were less than 0.25% and therefore within acceptable limits.

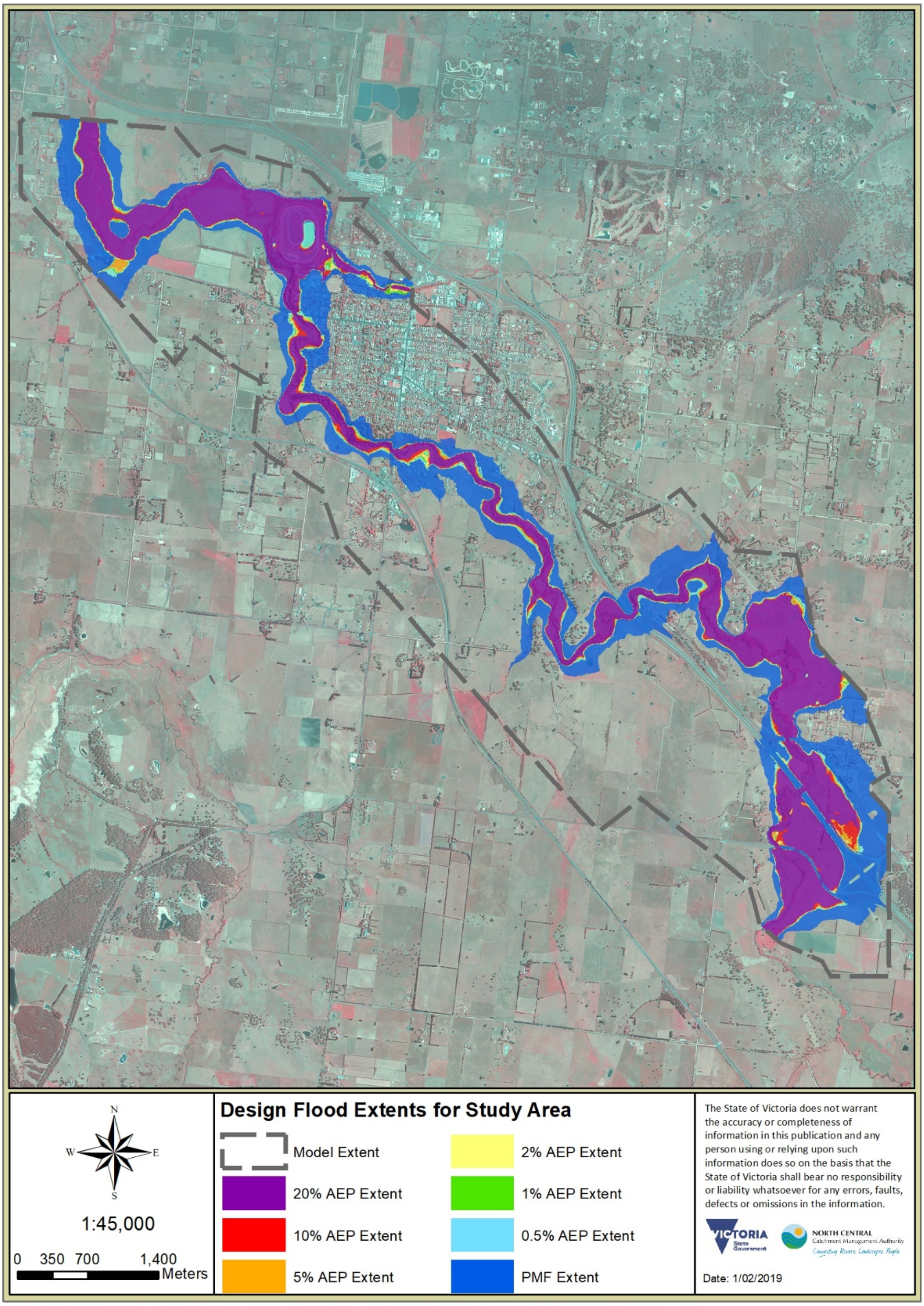
A review of the individual scenario outputs was undertaken to identify any discontinuities in flow behaviour as well as any other erroneous results that might indicate underlying issues with model inputs such as steep topography or unrealistic roughness values. The water surface elevations for each design event were also compared to events both rarer and more frequent events to ensure that the results were consistent. For instance, the 10% AEP flood levels were compared to the 20% AEP level to ensure that they were indeed higher at every point in the model. This analysis revealed that the 1% AEP flood levels were actually higher than the 0.5% AEP flood level on a section of Post Office Creek between Ebden Street and Mollison Street. This was due to the fact that blockages were only considered for the 1% AEP flood event. Therefore, since this was a localised area, the original 0.5%

AEP outputs were combined with the 1% AEP outputs and the highest critical values were selected for each grid to generate updated data for the 0.5% AEP event. This process ensured consistency between the datasets.

4.5.4 Design Results

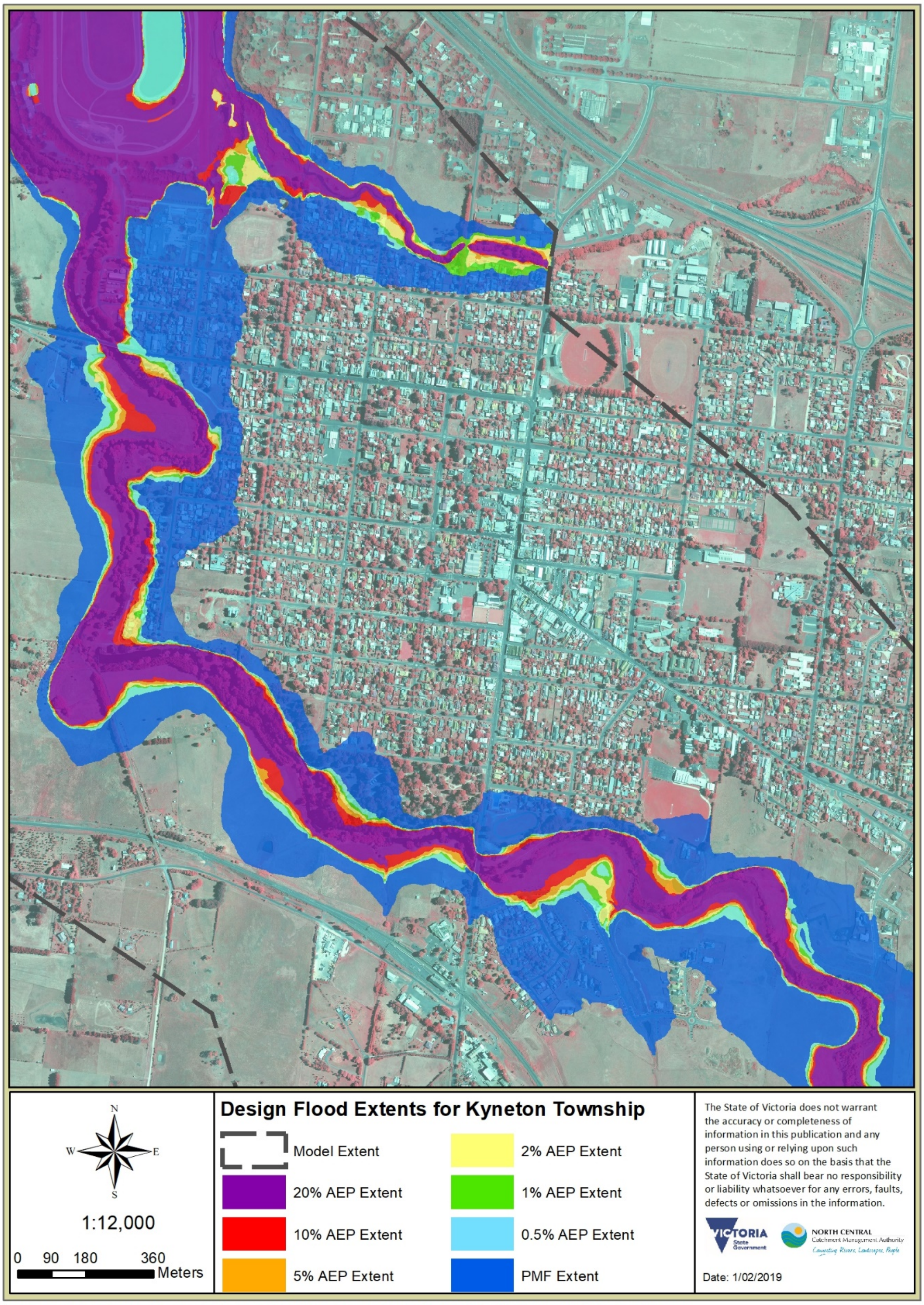
The hydraulic model was used to generate water surface elevations (flood levels), depths, velocities and hazard (depth multiplied by velocity) rasters for the 20%, 10%, 5%, 2%, 1% and 0.5% AEP flood events as well as the PMF. These outputs were then post-processed to generate flood extents, flood contours velocity vectors and longitudinal profiles for all design events. The extents produced from the raster data were smoothed using the Polynomial Approximation with Exponential Kernel (PAEK) algorithm and applying a tolerance of 20 metres. This provided a more realistic extent of flooding while still sufficiently preserving the definition of the raster data. Additionally, any small islands occurring within the flood extent with an area less than $400m^2$ were removed for clarity.

Figure 4-41 and Figure 4-42 shows all design flood extents overlayed on a single map for comparison. It can be seen that due to the confined floodplain along this reach of the Campaspe River there is not a substantial difference between the 20% AEP flood extent and the 0.5% AEP flood extent, although the average difference in flood level is 1 metre. The flood depth maps for each design event are shown in the Appendix (Section 7.1). A comparison of the longitudinal profiles for each design event is shown in Section 7.2.



T:\GISData\FLOODPLAIN\MACEDON RANGES\KYNETON\KYNETON FLOOD STUDY 2014\CMA DATA\Kyneton_Study_(20181102_MGA55)_Depth Maps.mxd

Figure 4-41 Design flood extents for study area



T:\GISData\FLOODPLAIN\MACEDON RANGES\KYNETON\KYNETON FLOOD STUDY 2014\CMA DATA\Kyneton_Study_(20181102_MGA55)_Depth Maps.mxd

Figure 4-42 Design flood extents for Kyneton Township

4.6 Design Flood Behaviour

Based on the design flood mapping for events ranging from the 20% AEP to the 0.5% AEP event (Figure 4-41 and Figure 4-42), it can be seen that flooding is generally confined through Kyneton Township by the steep banks of the Campaspe River. However, floodwaters do break out over the rural land surrounding Carlsruhe and also inundate the Racecourse north of Kyneton Township. Due to the defined nature of the floodplain, the increase in flood levels for rarer events typically do not result in a significant increase in flood extent and impacts. Flood depth maps for each event is shown in Section 7.1. The following comments summarise the key flood impacts for each design event.

20% AEP Flood Event

- Campaspe River
    - Some properties along Ebden Street and Pultney Street in Carlsruhe flooded. No buildings appear to be inundated.
    - Significant flooding on rural properties north of Carlsruhe.
    - Intersection of Trio Road and Murphys Road inundated to a depth of over 0.5 metres.
    - The rear of properties on Degraves Court flooded. Some outbuildings may be impacted.
    - Shallow inundation of St Agnes Place.
    - The rear of some properties along Mill Street flooded.
    - Some properties along Jennings Street inundated.
    - Properties along Campaspe Place and Lennox Street flooded.
    - Kyneton Racecourse entirely inundated.
    - Farm land downstream of Kyneton Township inundated. No buildings appear to be impacted

- Post Office Creek
    - Floodwater generally contained within creek.
    - Possible impacts to Hall Court properties fronting Post Office Creek.
    - Shallow flooding over Wedge Street bridge.
    - Properties immediately upstream of Wedge Street bridge may be impacted.

10% AEP Flood Event

The 10% AEP flood levels are on average 0.25 metres higher than the 20% AEP flood levels.

- Campaspe River
    - Impacts similar to 20% AEP flood event.
    - Campaspe Drive inundated to a depth of approximately 0.5 metres.
    - Franklin Place inundated.
    - St Agnes Place becomes inundated to a depth over 0.8 metres.
    - The rear of properties along Mill Street and St Agnes Place flooded. Dwellings may be impacted.

- Post Office Creek
  - Impacts similar to 20% AEP flood event.

5% AEP Flood Event

The 5% AEP flood levels are on average 0.2 metres higher than the 10% AEP flood levels.

- Campaspe River
  - Impacts similar to 10% AEP flood event.
  - Campaspe Drive inundated to a depth of over 1 metre.
  - Some properties near the Campaspe Drive and Windridge Way intersection flooded. Dwellings may be impacted.
  - St Agnes Place becomes inundated to a depth over 1 metre.
  - Property at the end of Argyle Lane (immediately upstream of Piper Street bridge) significantly impacted.
  - 171 and 185 Burton Avenue partially inundated. Dwellings potentially impacted.
  - Shallow inundation of Burton Avenue.

- Post Office Creek
  - Wedge Street bridge overtopped by approximately 0.2 metres.

2% AEP Flood Event

The 2% AEP flood levels are on average 0.15 metres higher than the 5% AEP flood levels.

- Campaspe River
  - Impacts similar to 5% AEP flood event.
  - Property at the end of Argyle Lane (immediately upstream of Piper Street bridge) significantly impacted. Dwelling potentially impacted.
  - Burton Avenue overtopped to a depth of approximately 0.3 metres.

- Post Office Creek
  - Shallow inundation on Johnson Court.
  - Mollison Street bridge overtopped by approximately 0.4 metres.
  - Flooding over Ward Street up to 0.5 metres.
  - Some properties along Ward Street inundated. Dwellings potentially impacted.
  - Properties at the end of Powlett Street may be impacted. Some dwellings potentially impacted.
  - Wedge Street bridge overtopped by approximately 0.4 metres.

1% AEP Flood Event

The 1% AEP flood levels are on average 0.1 metres higher than the 2% AEP flood levels.

- Campaspe River
  - Impacts similar to 2% AEP flood event.
  - Victoria Road inundated.
  - Greater flooding on Mill Street and St Agnes Place properties.

- Post Office Creek
  - Property immediately upstream of Mollison Street bridge significantly impacted. Dwelling potentially impacted.
  - Mollison Street bridge overtopped by approximately 0.6 metres.
  - Flooding on Ward Street over 1 metre.
  - Most properties along Ward Street inundated. Dwellings potentially impacted.
  - Ebden Street bridge overtopped by approximately 0.5 metres.
  - Property immediately downstream of Ebden Street bridge flooded. Dwelling potentially impacted.
  - Wedge Street bridge overtopped by approximately 0.5 metres.

0.5% AEP Flood Event

The 0.5% AEP flood levels are on average 0.35 metres higher than the 2% AEP flood levels.

- Campaspe River
  - Impacts similar to 1% AEP flood event.
  - Cobb and Co Road may overtop by 200mm between Nicholson Street and Three Chain Road.
  - Shallow flooding on Piper Street adjacent to Piper Street bridge.
  - Burton Avenue overtopped to a depth of over 0.5 metres.

- Post Office Creek
  - Mollison Street bridge overtopped by approximately 0.7 metres.
  - Ebden Street bridge overtopped by approximately 0.5 metres.
  - Property immediately downstream of Ebden Street bridge flooded. Dwelling potentially impacted.
  - Wedge Street bridge overtopped by approximately 0.5 metres.

5 Conclusion

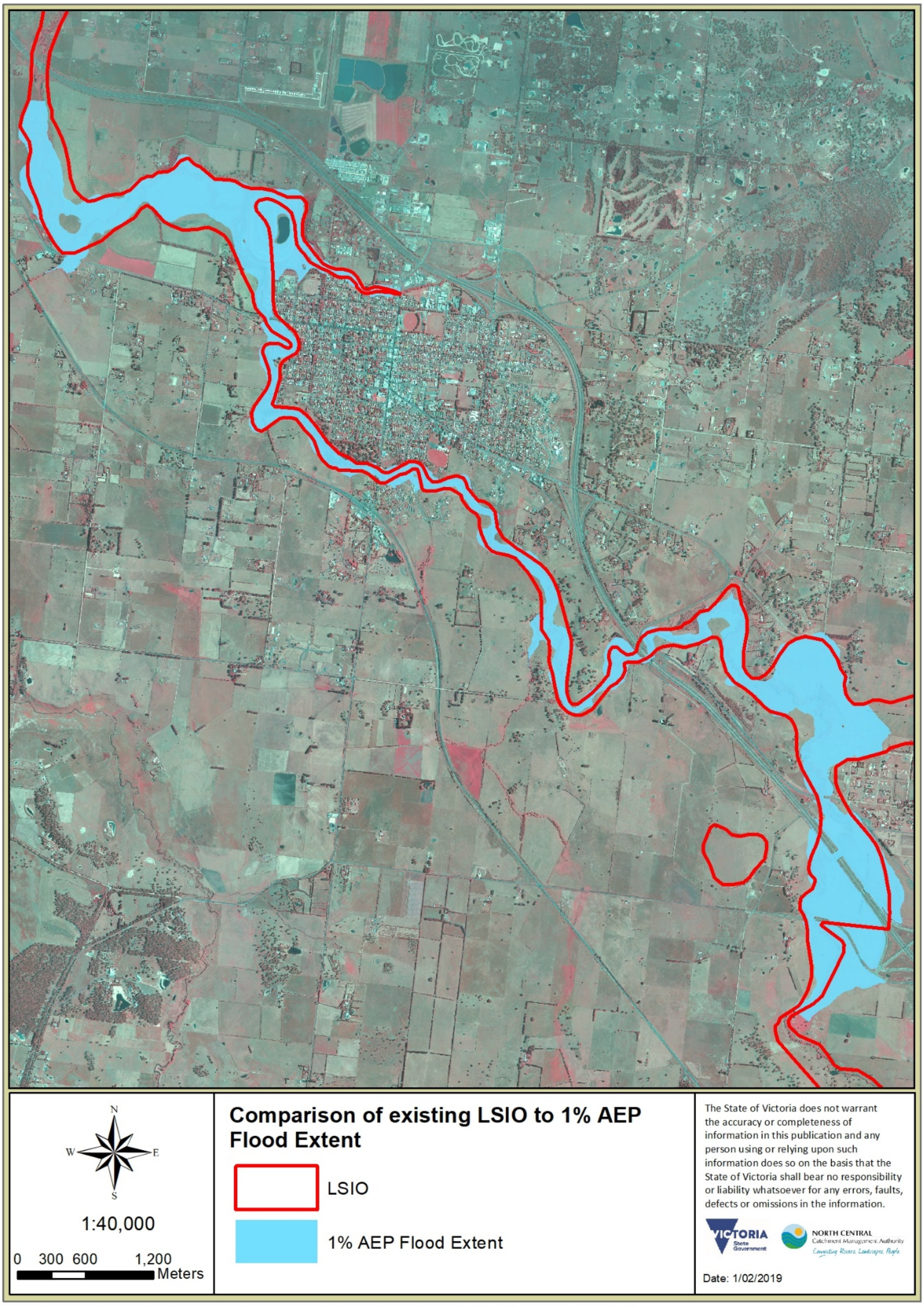
This report has documented the methodology and results of the Kyneton Flood Study. Through the development and calibration of hydrologic and hydraulic models, the flood behaviour has been determined for various design flood events ranging from the 20% AEP to the PMF. The model outputs generated for these design events include flood extents, levels, depths and velocities. These results will be used to update the available flood information for the township of Kyneton.

It should be noted that, although the model provides a reasonable indication of flooding for Post Office Creek, there were limits to the calibration of the hydrologic and hydraulic models due to a lack of data and historical information. Recommendations for a future study to improve the flood data along Post Office Creek include:

- Obtain additional LiDAR and survey of the creek area to:
    - Extend the hydraulic model. Preferably, the model should extend upstream of Baynton Road so that several upstream hydraulic structures, including the Calder Freeway culverts, can be considered in the hydraulic model. By incorporating the impact of these structure in restricting downstream flows a better calibration to historical events is likely to be achieved.
    - Improve quality of data. The existing LiDAR is obscured by vegetation and does not appear to accurately reflect the topography of the waterway in several places.

- With extended and improved terrain data, a refined hydraulic model for Post Office Creek could be undertaken. In addition to incorporating the additional upstream structures, a finer grid resolution for the 2D model would allow the creek channel to be more accurately defined.

It is recommended that the current Land Subject to Inundation Overlay (LSIO) be amended to reflect the 1% AEP design results determined by this study. This is in accordance with Policy 13a of the Victorian Floodplain Management Strategy (2016) which states that the 1% AEP flood will remain the design flood extent for the land use planning and building systems in Victoria. Although the existing LSIO covers the majority of the determined flood extent due to the confined nature of the floodplain, Figure 5-1 below shows that the extent should be refined by increasing the overlay in some areas and decreasing it in other areas.



T:\GISData\FLOODPLAIN\MACEDON RANGES\KYNETON\KYNETON FLOOD STUDY 2014\CMA DATA\Kyneton_Study_(20190201_MGA55).mxd

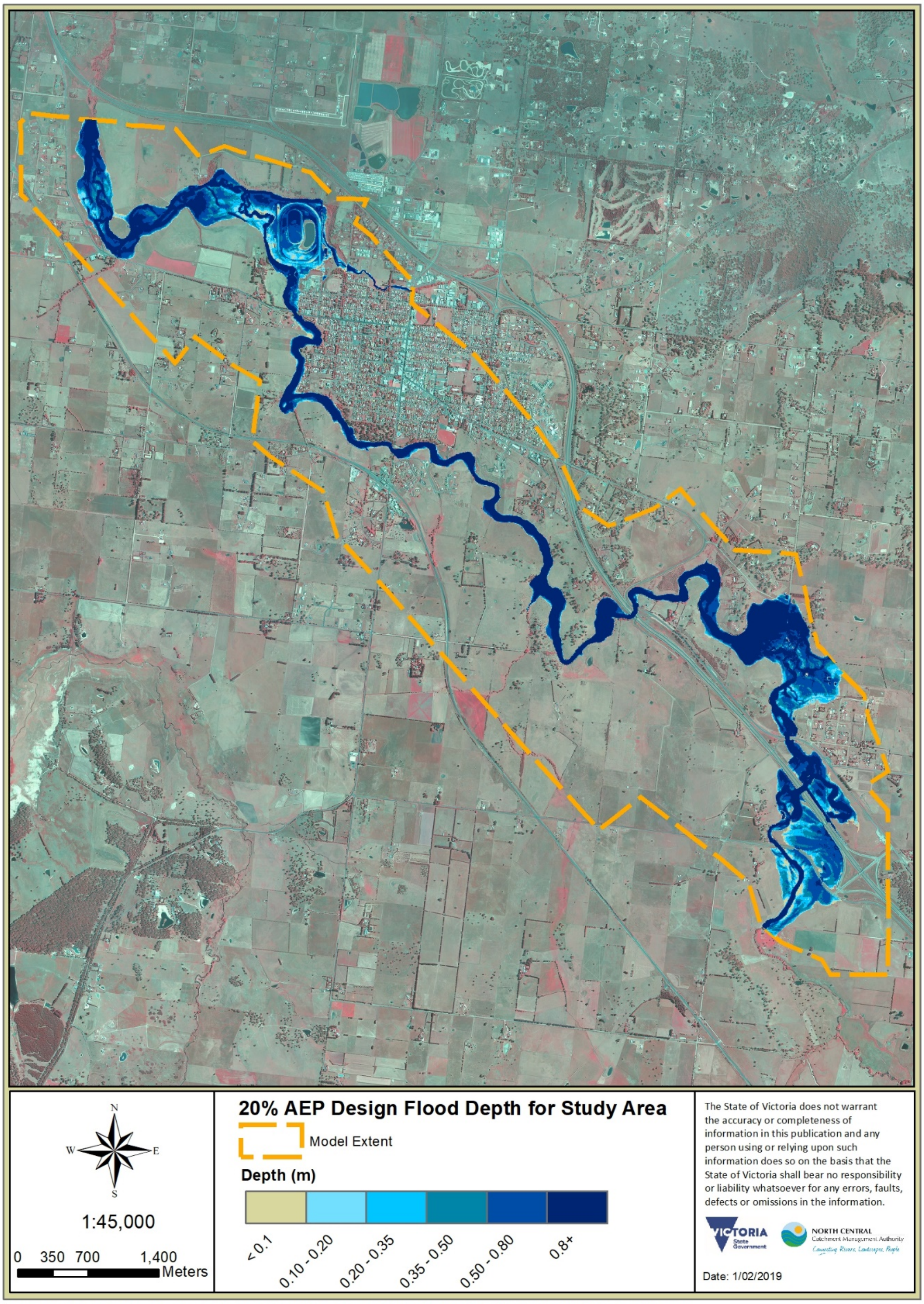
Figure 5-1 Comparison of existing LSIO to the 1% AEP design flood extent

6 Reference

- Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M & Testoni I (2016), *Australian Rainfall and Runoff: A Guide to Flood Estimation*, Commonwealth of Australia
- BMT WBM (2018), *TUFLOW User Manual Build 2017-09-AC,* https://www.tuflow.com/Download/TUFLOW/Releases/2017-09/TUFLOW%20Manual.2017-09.docx
- CMPS&F Environmental (1995), *Calder Highway Carlsruhe to Kyneton – Hydrologic and Hydraulic Investigations: Final Report*
- Concrete Pipe Association of Australasia (2012), *CPAA Design Value – Hydraulics of Precast Concrete Conduits*, http://www.cpaa.asn.au/images/publications/engineering_guidelines/cpaa_design_manual_hydraulics_jan13.pdf
- Department of Environment, Land, Water and Planning (2016), Victorian Floodplain Management Strategy, Victorian Government, Melbourne
- Earth Tech (2005), *River Walk Flood Study*
- Herschy, R (2003), *World Catalogue of Maximum Observed Floods,* International Association of Hydrological Sciences, UK, http://hydrologie.org/redbooks/a284/RB284.pdf
- Institution of Engineers Australia (1987), *Australian Rainfall and Runoff, Vol. 1&2*. (Ed: Pilgrim, D.H.)
- Joseph Bradley (1978), *Hydraulics of Bridge Waterways*, Federal Highway Administration, USA, http://www.ciccp.es/ImgWeb/Castilla%20y%20Leon/Documentaci%C3%B3n%20T%C3%A9cnica/Hydraulics%20of%20Bridge%20Waterways%20(1978).pdf
- Macedon Ranges Shire Council, Melbourne & North Central Catchment Management Authority (2013), *Flood Management Plan for Macedon Ranges Shire, Melbourne Water and North Central CMA*
- Rural Water Commission of Victoria (1990), *Victorian Surface Water Information to 1987 – Volume 4,* http://data.water.vic.gov.au/WebDocs/SW/VSWI-Volume4.pdf
- Scorah, M., P. Hill, S. Lang and R. Nathan (2016), *Addressing embedded bursts in design storms for flood hydrology*, 37th Hydrology & Water Resources Symposium Tasmania, Engineers Australia.
- Water Technology (2013), *Rochester Flood Management Plan*

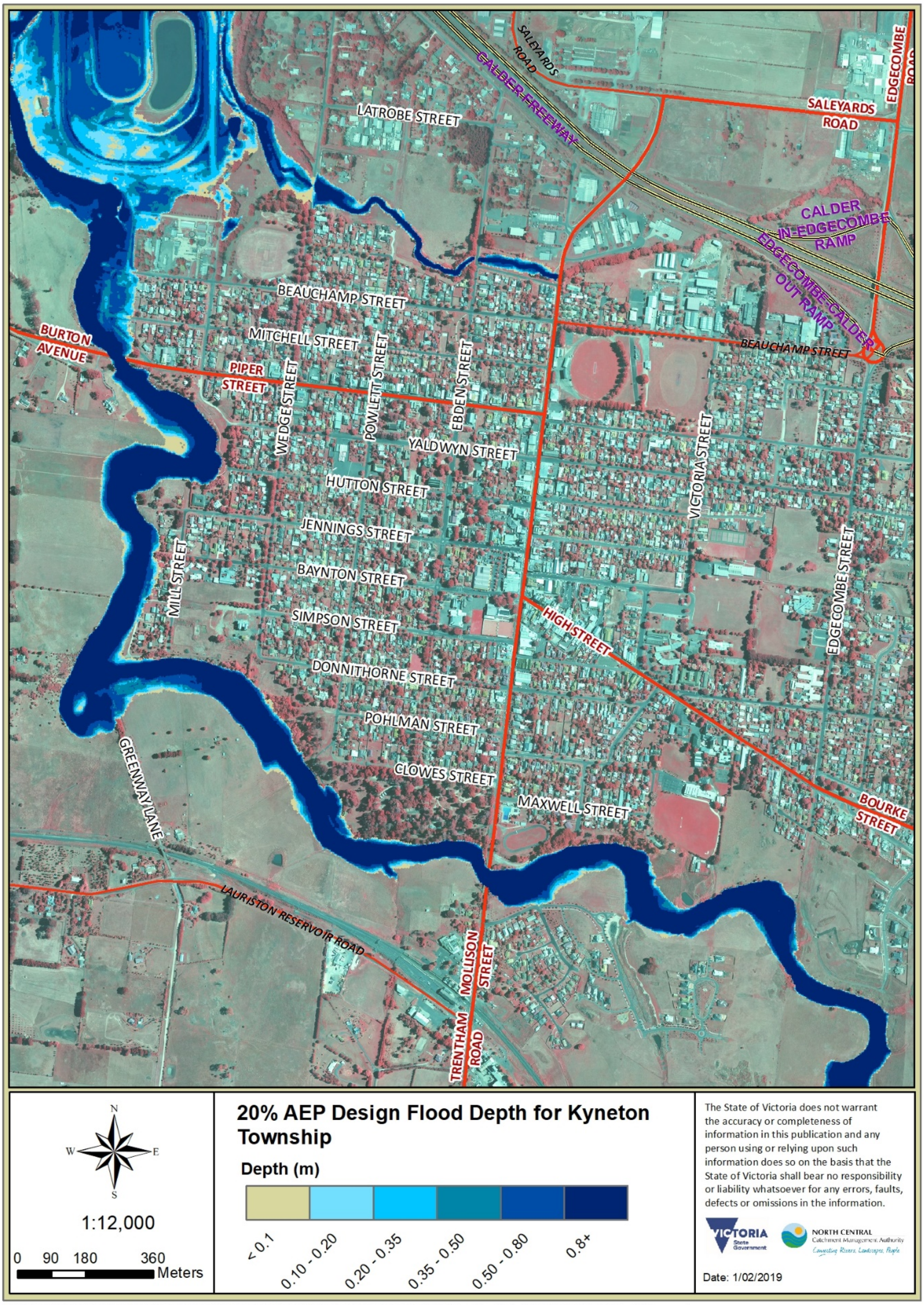
7 Appendix

7.1 Design Flood Depth Maps (20% AEP to PMF)



T:\GISData\FLOODPLAIN\MACEDON RANGES\KYNETON\KYNETON FLOOD STUDY 2014\CMA DATA\Kyneton_Study_(20181102_MGA55)_Depth Maps.mxd

Figure 7-1 20% AEP design flood depth for study extent



T:\GISData\FLOODPLAIN\MACEDON RANGES\KYNETON\KYNETON FLOOD STUDY 2014\CMA DATA\Kyneton_Study_(20181102_MGA55)_Depth Maps.mxd

Figure 7-2 20% AEP design flood depth for Kyneton Township

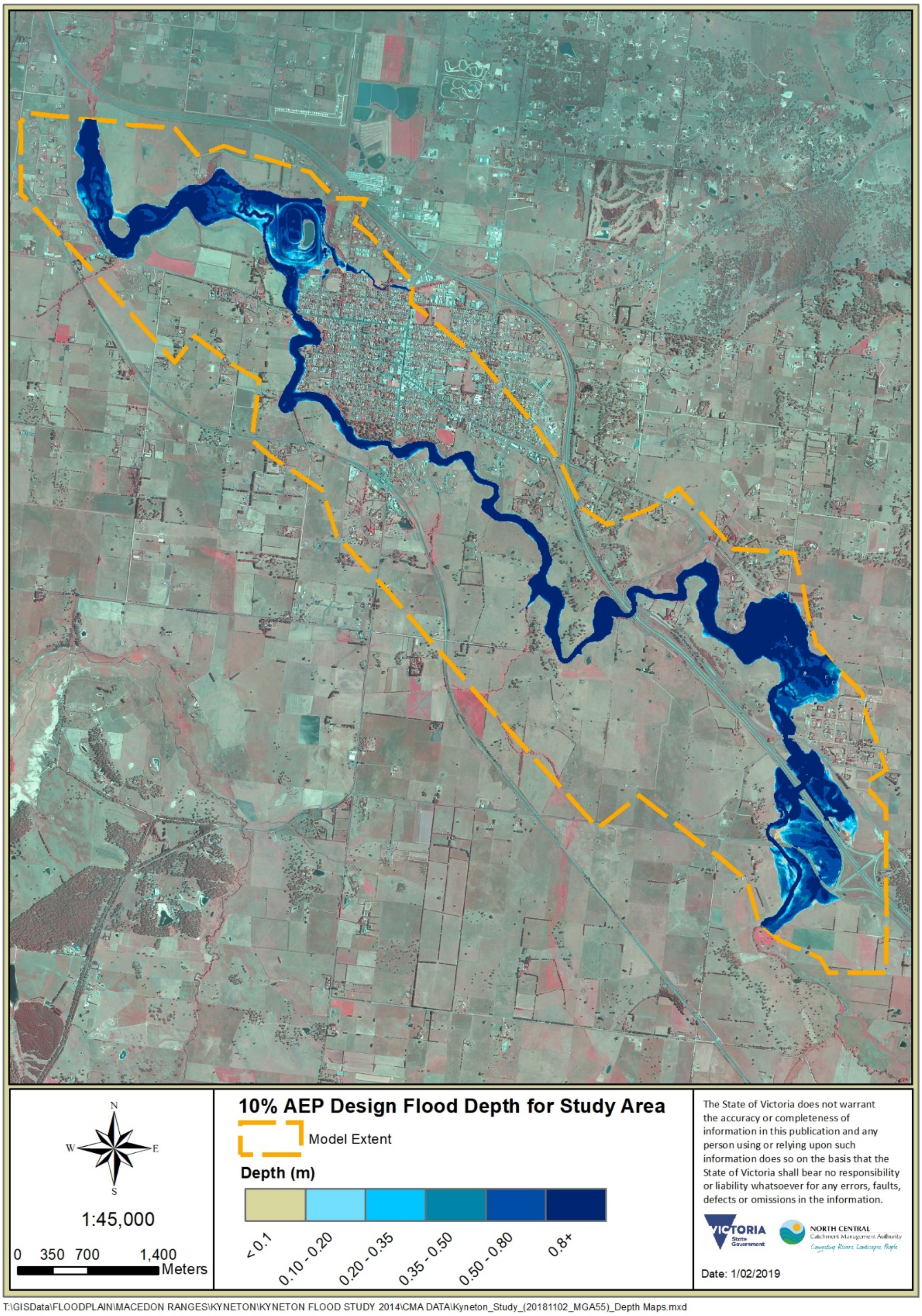


Figure 7-3 10% AEP design flood depth for study extent

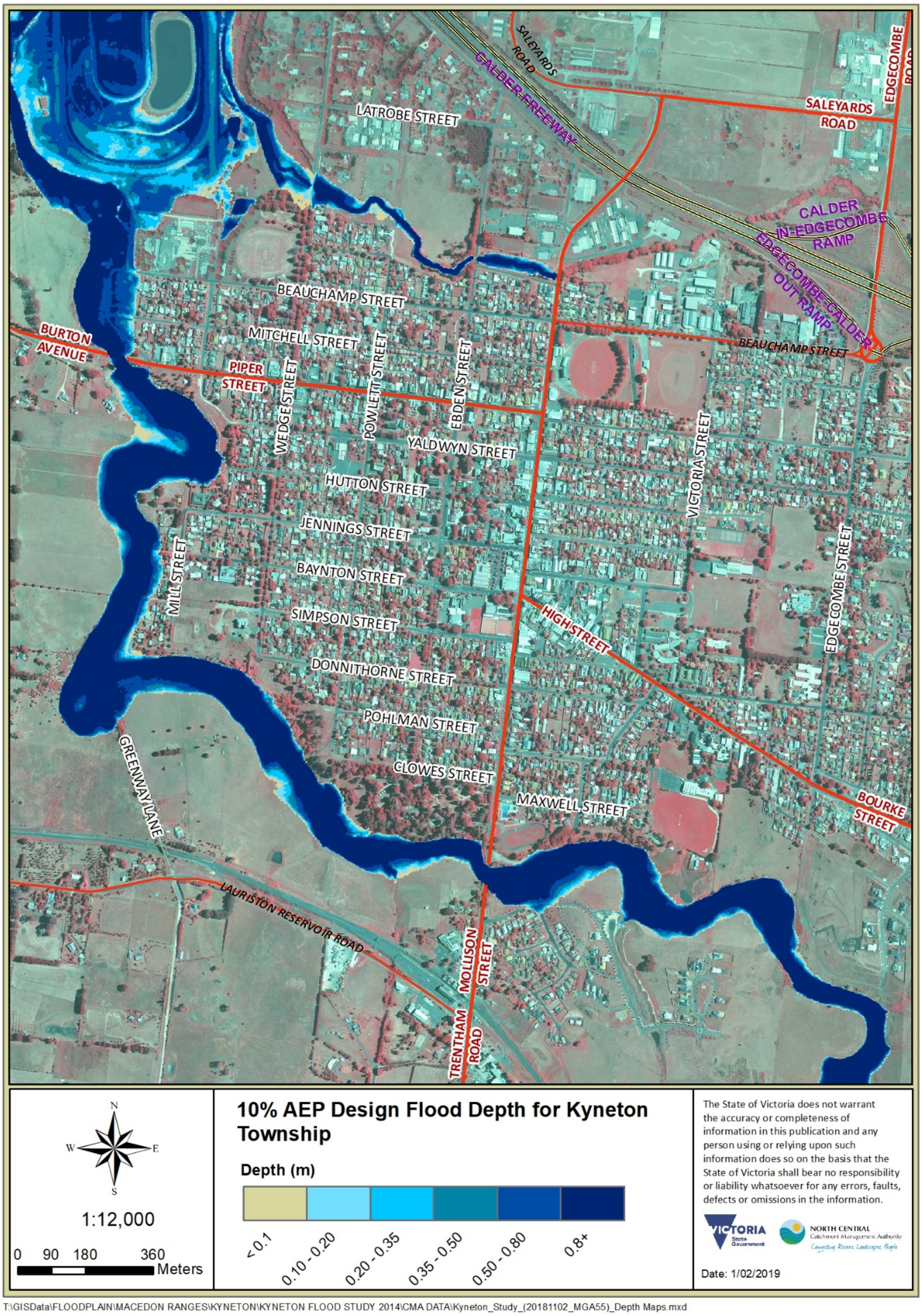


Figure 7-4 10% AEP design flood depth for Kyneton Township

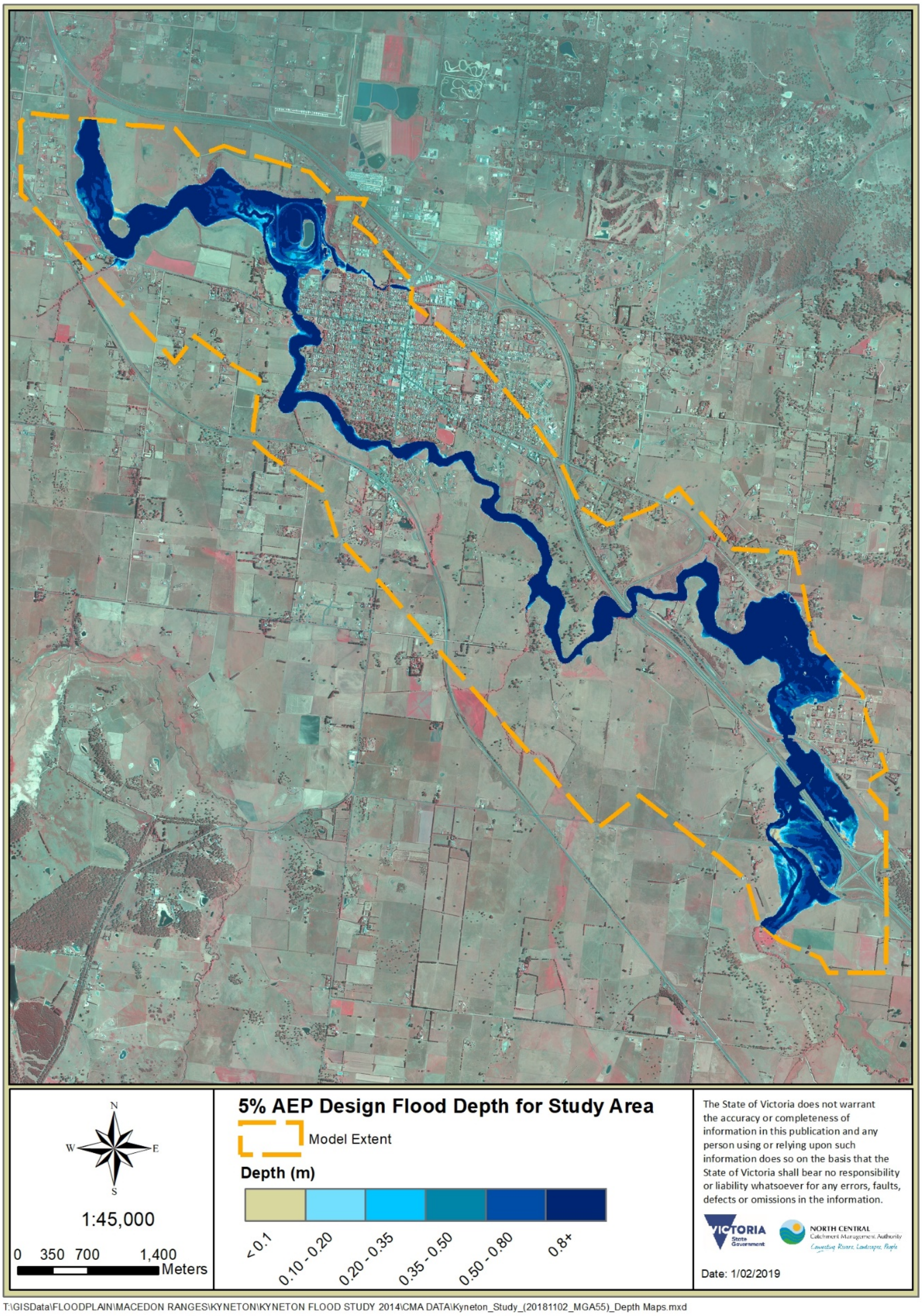


Figure 7-5 5% AEP design flood depth for study extent

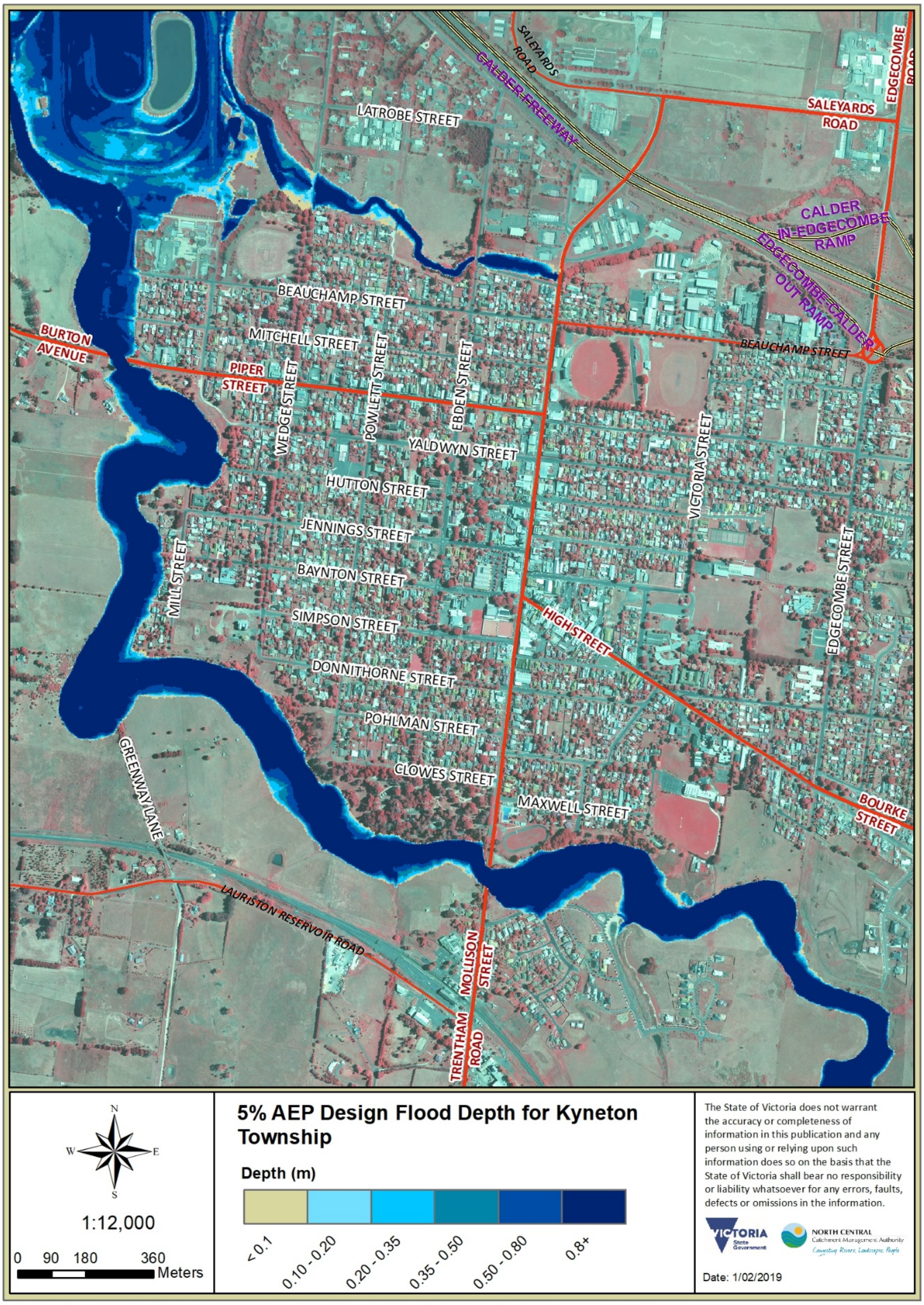


Figure 7-6 5% AEP design flood depth for Kyneton Township

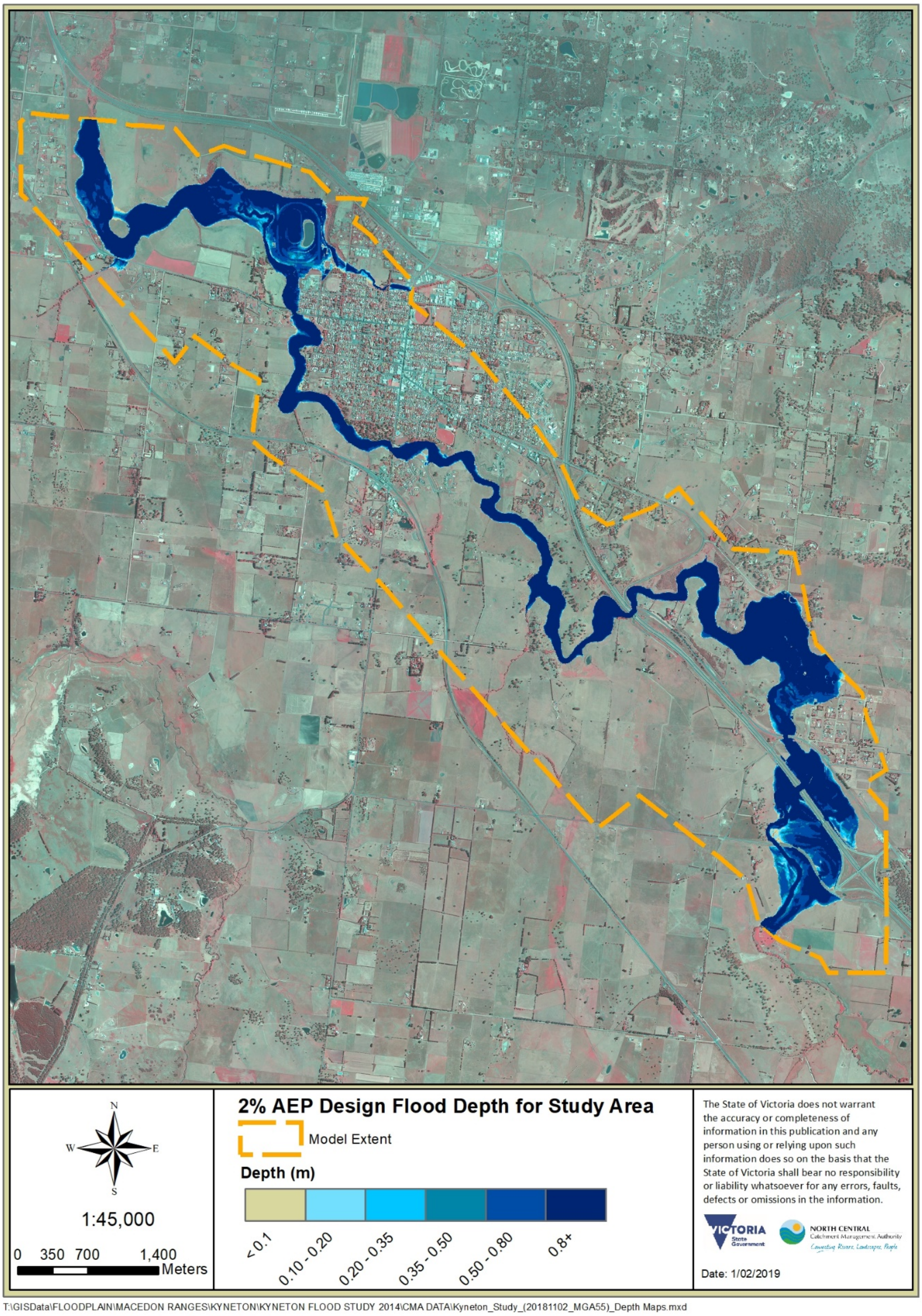


Figure 7-7 2% AEP design flood depth for study extent

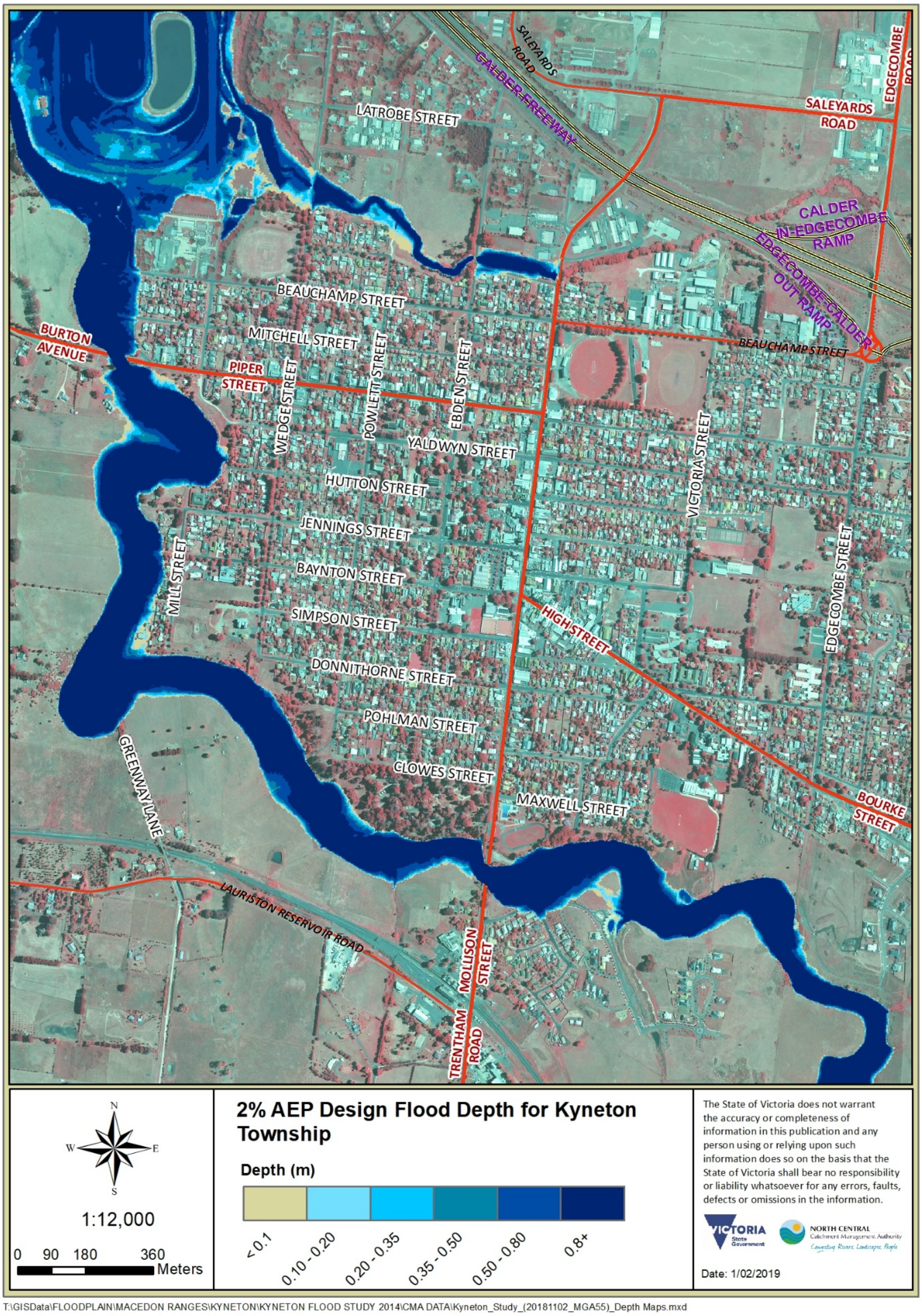


Figure 7-8 2% AEP design flood depth for Kyneton Township

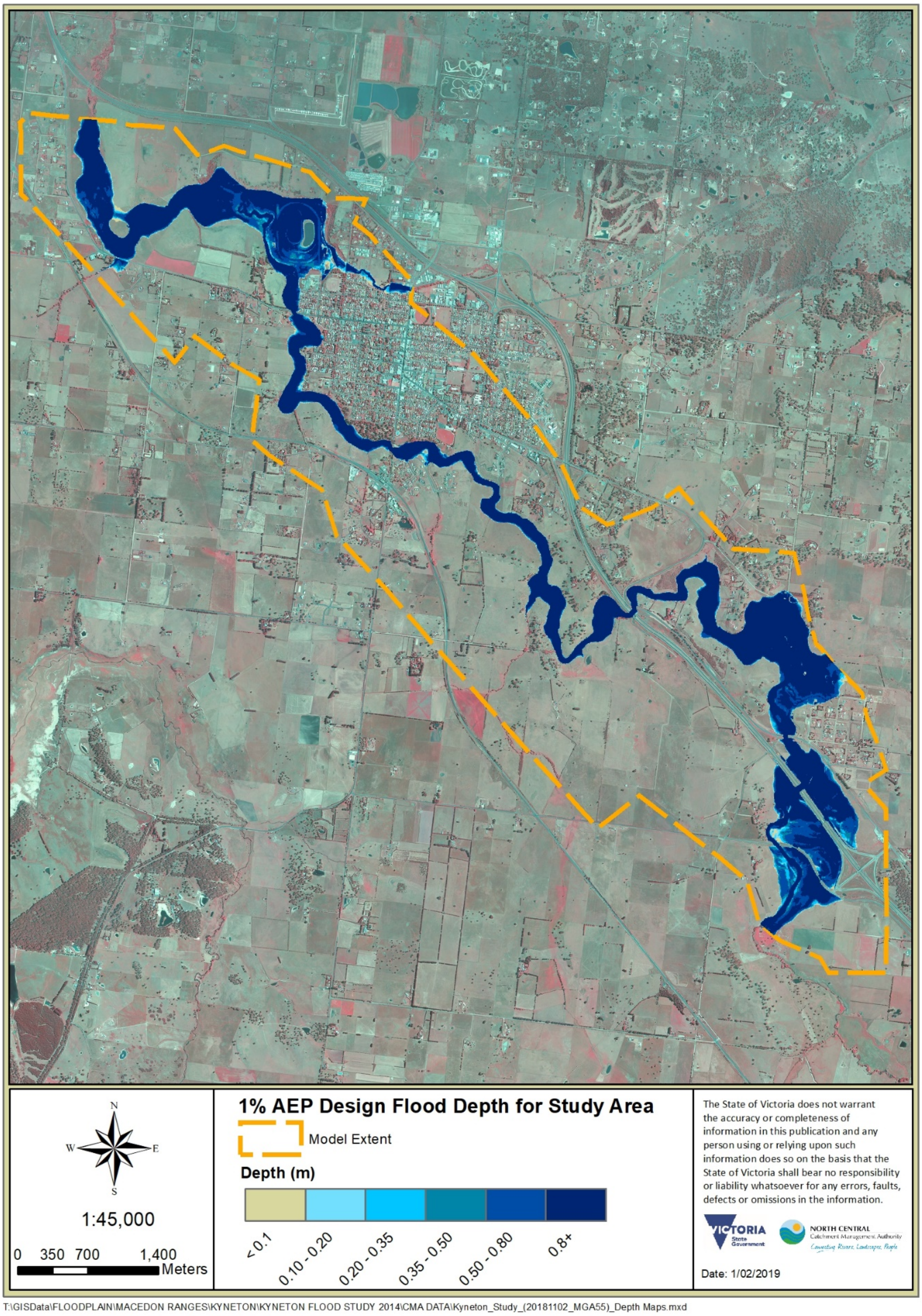


Figure 7-9 1% AEP design flood depth for study extent

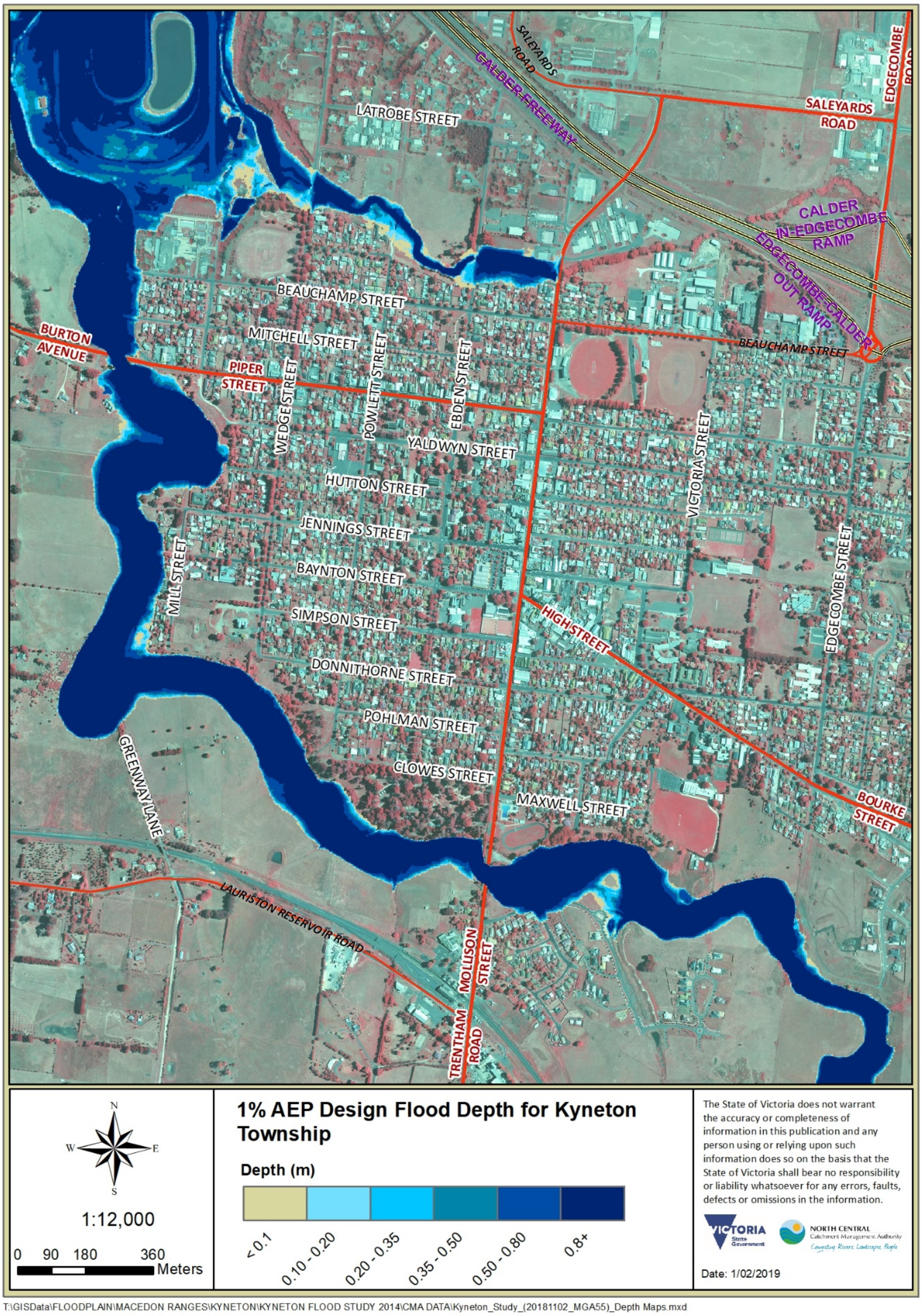


Figure 7-10 1% AEP design flood depth for Kyneton Township

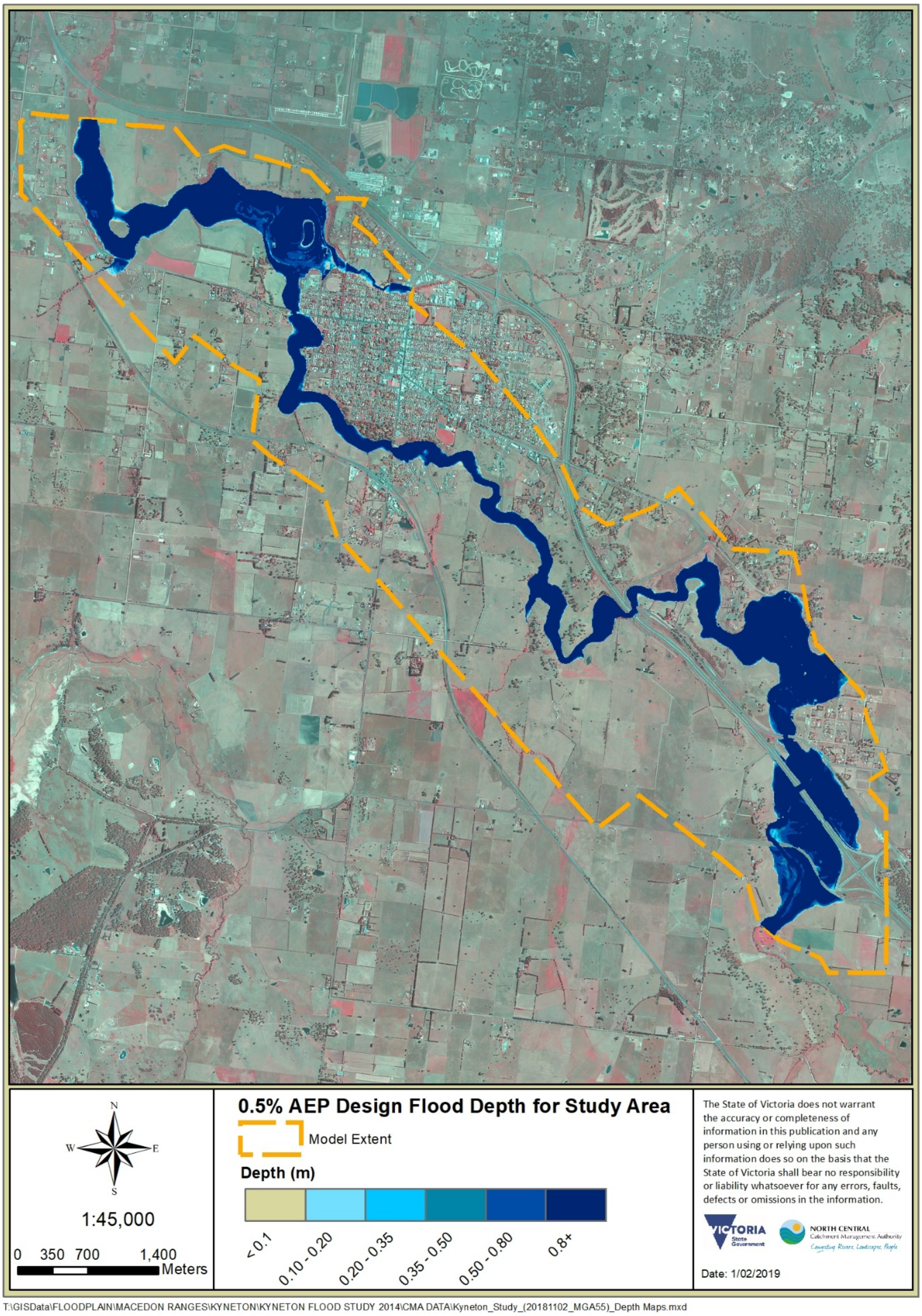
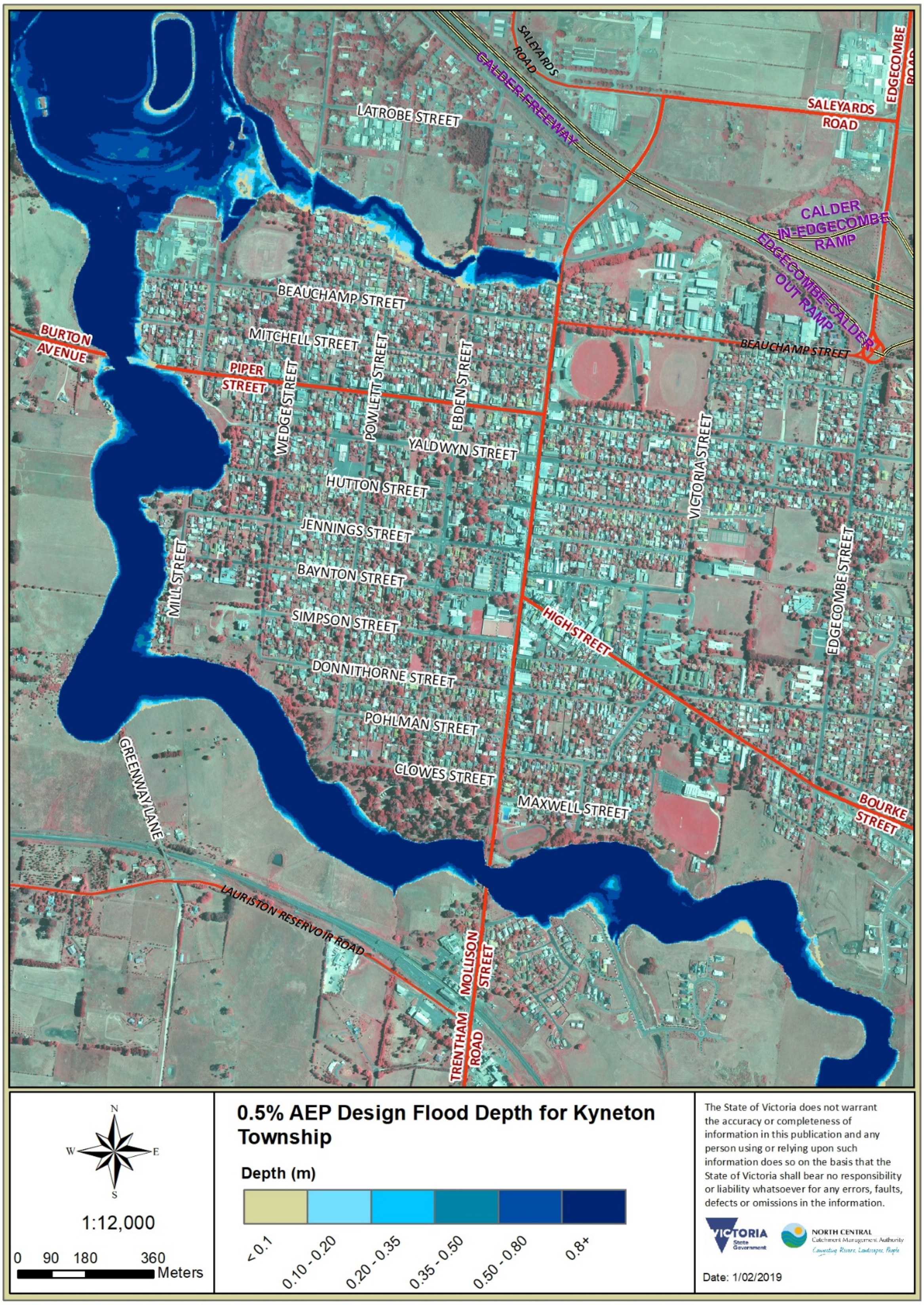
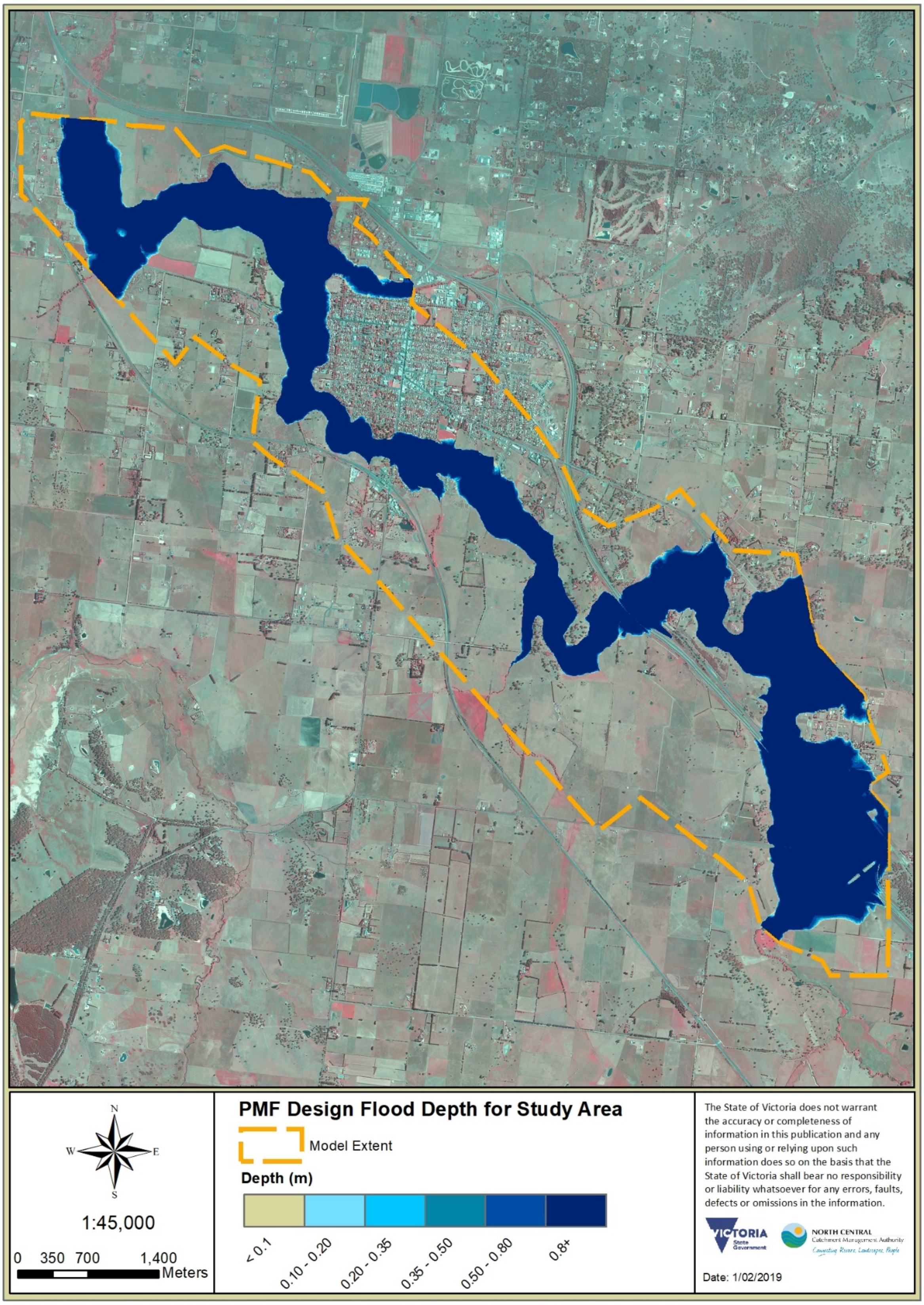


Figure 7-11 **0.5% AEP design flood depth for study extent**



T:\GISData\FLOODPLAIN\MACEDON RANGES\KYNETON\KYNETON FLOOD STUDY 2014\CMA DATA\Kyneton_Study_(20181102_MGA55)_Depth Maps.mxd

Figure 7-12 0.5% AEP design flood depth for Kyneton Township



T:\GISData\FLOODPLAIN\MACEDON RANGES\KYNETON\KYNETON FLOOD STUDY 2014\CMA DATA\Kyneton_Study_(20181102_MGA55)_Depth Maps.mxd

Figure 7-13 PMF design flood depth for study extent

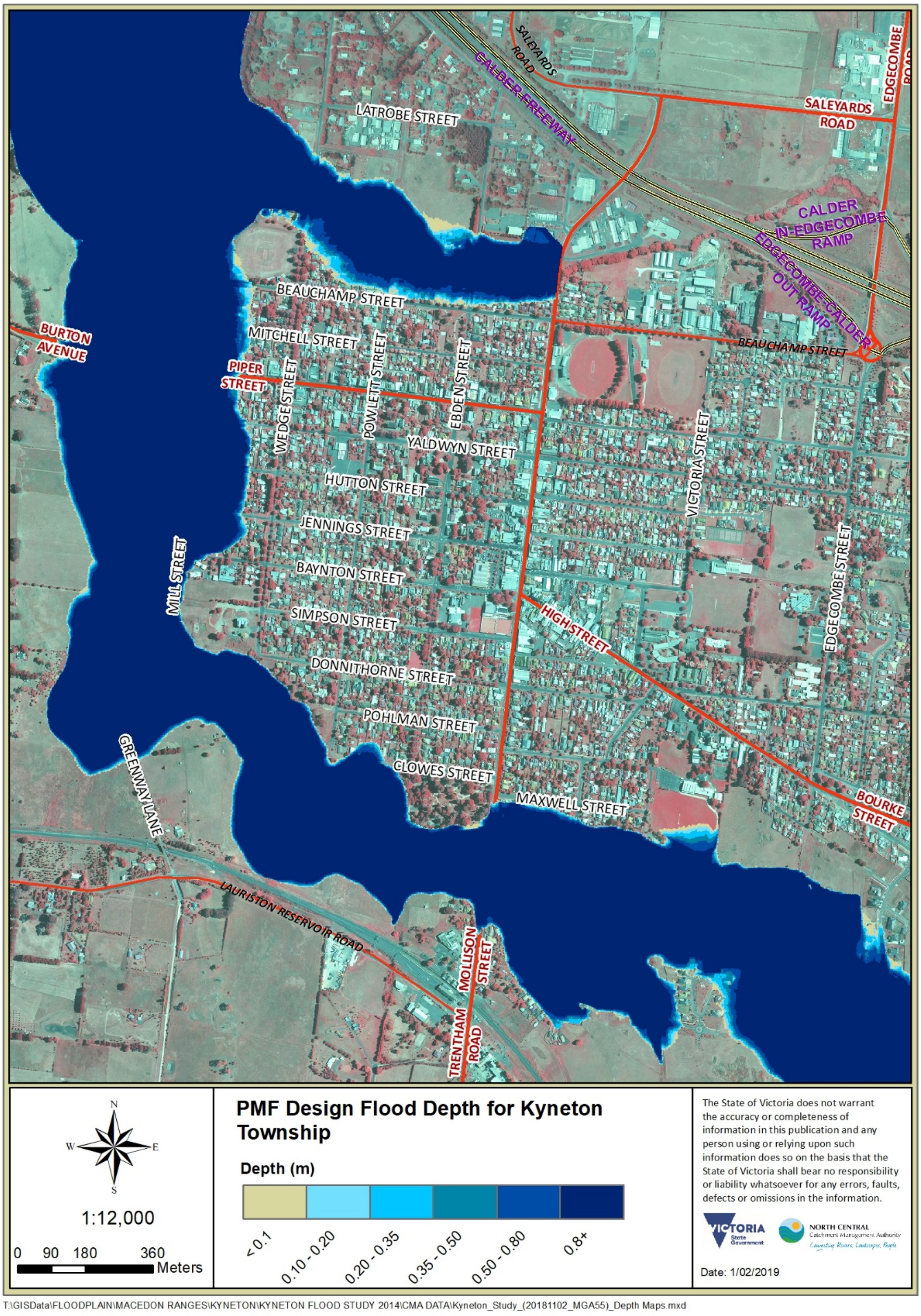
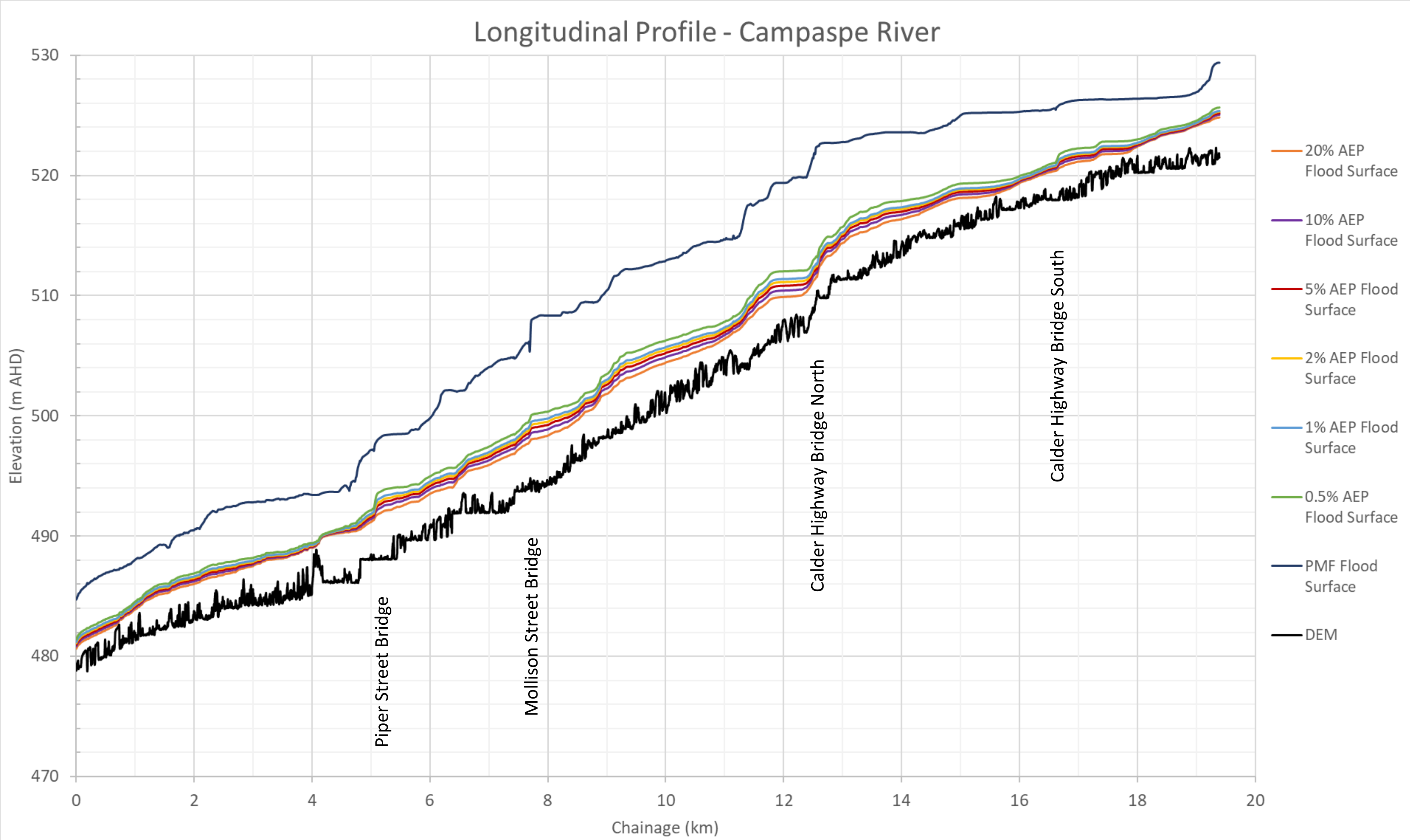
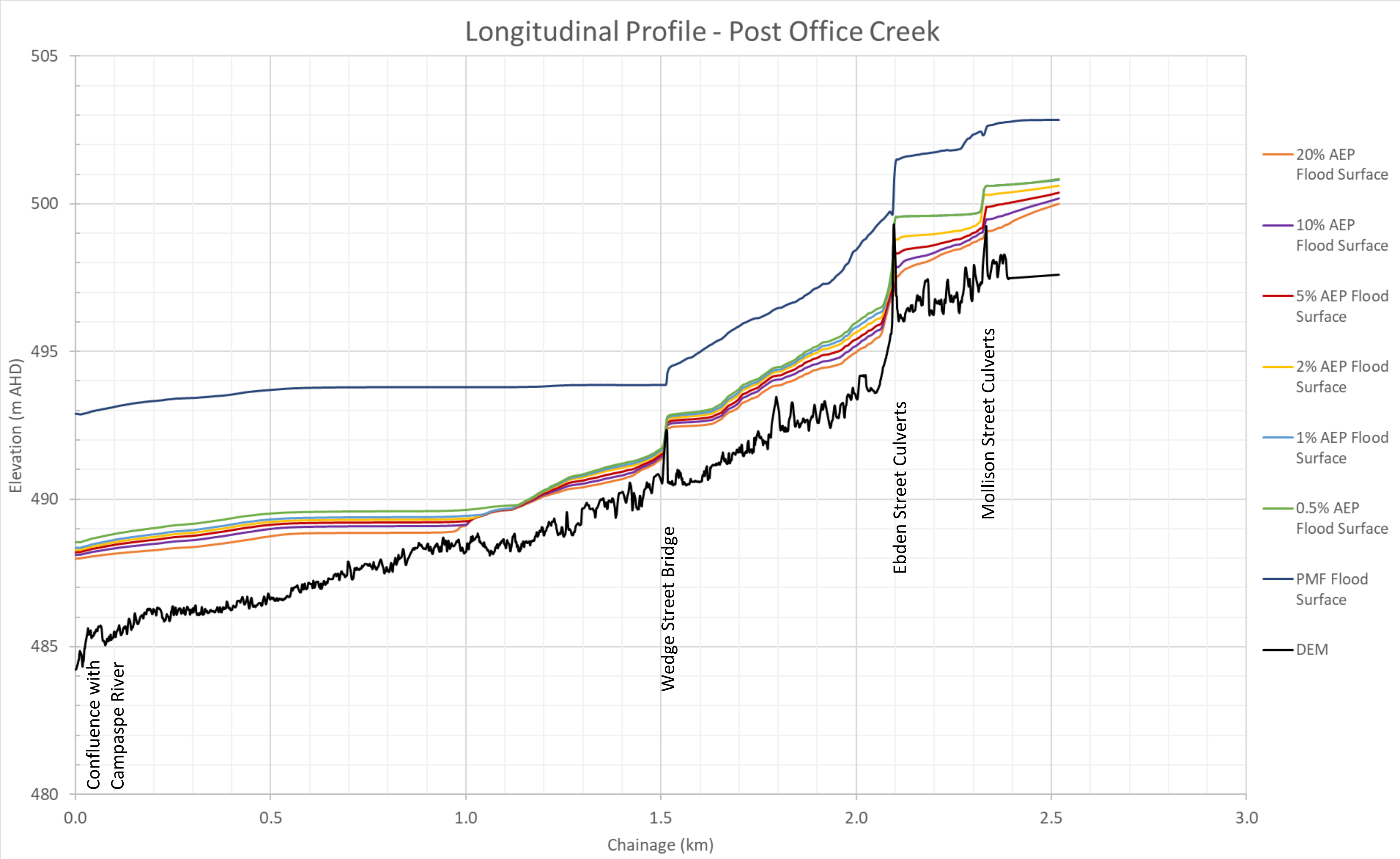


Figure 7-14 PMF design flood depth for Kyneton Township

7.2 Longitudinal Profiles (20% to 0.5% AEP Flood Events)


Longitudinal Profile - Campaspe River
Elevation (m AHD)
530
520
510
500
490
480
470
Chainage (km)
0
2
4
6
8
10
12
14
16
18
20
Piper Street Bridge
Mollison Street Bridge
Calder Highway Bridge North
Calder Highway Bridge South
20% AEP Flood Surface
10% AEP Flood Surface
5% AEP Flood Surface
2% AEP Flood Surface
1% AEP Flood Surface
0.5% AEP Flood Surface
PMF Flood Surface
DEM


Longitudinal Profile - Post Office Creek
505
500
495
490
485
480
Elevation (m AHD)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Chainage (km)
Confluence with Campaspe River
Wedge Street Bridge
Ebden Street Culverts
Mollison Street Culverts
20% AEP Flood Surface
10% AEP Flood Surface
5% AEP Flood Surface
2% AEP Flood Surface
1% AEP Flood Surface
0.5% AEP Flood Surface
PMF Flood Surface
DEM

7.3 Data Hub

7.3.1 Redesdale Data Hub Information

Australian Rainfall & Runoff Data Hub - Results

Input Data

Longitude	144.522
Latitude	-37.23
Selected Regions (clear)	
ARF Parameters	show
Storm Losses	show
Temporal Patterns	show
Areal Temporal Patterns	show
Interim Climate Change Factors	show
Baseflow Factors	show

Region Information

Data Category	Region
River Region	Campaspe River
ARF Parameters	Southern Temperate

Data

ARF Parameters

Long Duration ARF

$$Areal\ reduction\ factor = Min\left\{1, \left[1 - a\left(Area^{b} - c\log_{10}Duration\right) Duration^{-d} + eArea^{f} Duration^{g} \left(0.3 + \log_{10}AEP\right) + h10^{iArea\frac{Duration}{1440}} \left(0.3 + \log_{10}AEP\right)\right]\right\}$$

Zone	Southern Temperate
a	1.58E-01
b	2.76E-01
c	3.72E-01
d	3.15E-01
e	1.41E-04
f	4.10E-01
g	1.50E-01
h	1.00E-02
i	-2.70E-03

Short Duration ARF

$$ARF = Min\left[1, 1 - 0.287\left(Area^{0.265} - 0.439\log_{10}(Duration)\right) . Duration^{-0.36} + 2.26 \text{ x } 10^{-3} \text{ x } Area^{0.226} . Duration^{0.125}\left(0.3 + \log_{10}(AEP)\right) + 0.0141 \text{ x } Area^{0.213} \text{ x } 10^{-0.021\frac{(Duration-180)^2}{1440}}\left(0.3 + \log_{10}(AEP)\right)\right]$$

Layer Info

Time Accessed	08 June 2017 03:20PM

Version	2016_v1

Storm Losses

Storm Initial Losses (mm)	28.0
Storm Continuing Losses (mm/h)	4.0

Layer Info

Time Accessed	08 June 2017 03:20PM
Version	2016_v1

Temporal Patterns | Download (.zip)

CODE	MB
LABEL	Murray Basin

Layer Info

Time Accessed	08 June 2017 03:20PM
Version	2016_v1

Areal Temporal Patterns | Download (.zip)

CODE	MB
LABEL	Murray Basin

Layer Info

Time Accessed	08 June 2017 03:20PM
Version	2016_v1

Median Preburst Depths and Ratios

Values are of the format depth (ratio) with depth in mm

min (h)\AEP (%)	50	20	10	5	2	1
60 (1.0)	3.2 (0.198)	3.0 (0.127)	2.8 (0.098)	2.6 (0.077)	1.8 (0.043)	1.2 (0.024)
90 (1.5)	2.4 (0.125)	2.3 (0.085)	2.2 (0.068)	2.1 (0.055)	1.4 (0.03)	0.9 (0.017)
120 (2.0)	3.6 (0.175)	3.4 (0.118)	3.3 (0.093)	3.2 (0.075)	1.9 (0.038)	1.0 (0.017)
180 (3.0)	3.3 (0.136)	3.4 (0.101)	3.4 (0.085)	3.5 (0.072)	5.6 (0.095)	7.1 (0.105)
360 (6.0)	1.6 (0.052)	2.0 (0.046)	2.2 (0.043)	2.4 (0.041)	4.4 (0.06)	5.9 (0.07)
720 (12.0)	0.4 (0.009)	1.3 (0.024)	1.9 (0.029)	2.6 (0.033)	4.4 (0.048)	5.8 (0.055)
1080 (18.0)	0.0 (0.001)	0.6 (0.009)	0.9 (0.012)	1.2 (0.014)	2.2 (0.021)	3.0 (0.025)
1440 (24.0)	0.0 (0.0)	0.2 (0.002)	0.3 (0.003)	0.4 (0.004)	0.6 (0.005)	0.7 (0.006)
2160 (36.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
2880 (48.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
4320 (72.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)

Layer Info

Time Accessed	08 June 2017 03:20PM

10% Preburst Depths

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
90 (1.5)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
120 (2.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
180 (3.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
360 (6.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
720 (12.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
1080 (18.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
1440 (24.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
2160 (36.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
2880 (48.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
4320 (72.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)

Layer Info

Time Accessed	08 June 2017 03:20PM
Version	2016_v1

25% Preburst Depths

min (h)\AEP (%)	50	20	10	5	2	1
60 (1.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
90 (1.5)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
120 (2.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.001)	0.0 (0.0)	0.0 (0.0)
180 (3.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.001)	0.1 (0.001)
360 (6.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
720 (12.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
1080 (18.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
1440 (24.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
2160 (36.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
2880 (48.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
4320 (72.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)

Layer Info

Time Accessed	08 June 2017 03:20PM
Version	2016_v1

75% Preburst Depths

min (h) \AEP(%)	50	20	10	5	2	1
60 (1.0)	12.8 (0.78)	14.9 (0.635)	16.3 (0.567)	17.6 (0.514)	14.9 (0.353)	12.9 (0.264)
90 (1.5)	10.9 (0.582)	13.2 (0.495)	14.6 (0.451)	16.0 (0.415)	14.8 (0.311)	13.9 (0.252)
120 (2.0)	12.9 (0.619)	15.4 (0.528)	17.0 (0.481)	18.6 (0.443)	17.1 (0.331)	16.0 (0.267)
180 (3.0)	13.6 (0.567)	14.5 (0.436)	15.1 (0.374)	15.6 (0.328)	20.7 (0.353)	24.5 (0.362)
360 (6.0)	8.7 (0.278)	13.1 (0.307)	16.1 (0.313)	18.9 (0.313)	21.7 (0.296)	23.8 (0.283)
720 (12.0)	6.4 (0.154)	10.0 (0.18)	12.4 (0.188)	14.8 (0.191)	18.9 (0.204)	22.1 (0.21)
1080 (18.0)	5.0 (0.103)	7.7 (0.119)	9.5 (0.124)	11.3 (0.126)	15.1 (0.142)	18.0 (0.15)
1440 (24.0)	2.9 (0.054)	4.0 (0.056)	4.8 (0.057)	5.5 (0.056)	9.0 (0.077)	11.6 (0.088)
2160 (36.0)	0.2 (0.003)	1.3 (0.016)	2.0 (0.021)	2.7 (0.024)	5.7 (0.043)	7.9 (0.053)
2880 (48.0)	0.0 (0.0)	0.4 (0.005)	0.7 (0.006)	0.9 (0.008)	2.3 (0.016)	3.4 (0.021)
4320 (72.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.3 (0.002)	0.6 (0.003)

Layer Info

Time Accessed	08 June 2017 03:20PM
Version	2016_v1

90% Preburst Depths

min (h) \AEP(%)	50	20	10	5	2	1
60 (1.0)	22.9 (1.397)	27.0 (1.152)	29.8 (1.036)	32.4 (0.945)	30.3 (0.716)	28.7 (0.586)
90 (1.5)	21.9 (1.164)	25.7 (0.967)	28.2 (0.87)	30.7 (0.793)	29.0 (0.609)	27.8 (0.504)
120 (2.0)	24.8 (1.194)	29.2 (1.004)	32.2 (0.908)	35.0 (0.831)	41.1 (0.793)	45.7 (0.761)
180 (3.0)	24.6 (1.024)	28.3 (0.85)	30.8 (0.764)	33.1 (0.694)	43.4 (0.74)	51.1 (0.754)
360 (6.0)	22.3 (0.709)	28.0 (0.654)	31.7 (0.619)	35.3 (0.587)	43.0 (0.587)	48.7 (0.58)
720 (12.0)	27.1 (0.655)	29.8 (0.534)	31.5 (0.476)	33.2 (0.43)	38.6 (0.417)	42.7 (0.405)
1080 (18.0)	17.3 (0.358)	20.0 (0.308)	21.8 (0.283)	23.5 (0.263)	28.9 (0.272)	33.0 (0.275)
1440 (24.0)	16.2 (0.304)	18.8 (0.262)	20.5 (0.241)	22.2 (0.225)	26.9 (0.23)	30.4 (0.231)
2160 (36.0)	5.3 (0.087)	9.9 (0.121)	12.9 (0.133)	15.8 (0.141)	24.9 (0.188)	31.7 (0.214)
2880 (48.0)	4.0 (0.061)	6.6 (0.075)	8.4 (0.08)	10.0 (0.083)	18.4 (0.128)	24.7 (0.154)
4320 (72.0)	1.4 (0.019)	3.0 (0.031)	4.1 (0.035)	5.1 (0.038)	12.5 (0.079)	18.0 (0.101)

Layer Info

Time Accessed	08 June 2017 03:20PM
Version	2016_v1

Interim Climate Change Factors

Values are of the format temperature increase in degrees Celcius (% increase in rainfall)

	RCP 4.5	RCP6	RCP 8.5
2030	**0.85 (4.3%)**	0.845 (4.2%)	**0.974 (4.9%)**
2040	**1.086 (5.4%)**	1.05 (5.3%)	**1.341 (6.7%)**
2050	**1.303 (6.5%)**	1.283 (6.4%)	**1.734 (8.7%)**
2060	**1.478 (7.4%)**	1.539 (7.7%)	**2.212 (11.1%)**
2070	**1.629 (8.1%)**	1.775 (8.9%)	**2.753 (13.8%)**
2080	**1.741 (8.7%)**	2.036 (10.2%)	**3.26 (16.3%)**
2090	**1.793 (9.0%)**	2.316 (11.6%)	**3.748 (18.7%)**

Layer Info

Time Accessed	08 June 2017 03:20PM
Version	2016_v1
Note	ARR recommends the use of RCP4.5 and RCP 8.5 values

Baseflow Factors

DOWNSTREAM	11070.0
AREA_SQKM	741.635
CATCH_NO	11088.0
R3RUNOFF	0.096
R1RUNOFF	0.02

Layer Info

Time Accessed	08 June 2017 03:20PM
Version	2016_v1

Download TXT

Download PDF

7.3.2 Kyneton Data Hub Information

Australian Rainfall & Runoff Data Hub - Results

Input Data

Longitude	144.503
Latitude	-37.349
Selected Regions	
ARF Parameters	
Temporal Patterns	
Areal Temporal Patterns	
Interim Climate Change Factors	
Baseflow Factors	

Region Information

Data Category	**Region**
River Region	Campaspe River
ARF Parameters	Southern Temperate
Temporal Patterns	Murray Basin

Data

ARF Parameters

Long Duration ARF

$$Areal\ reduction\ factor = Min\left\{1, \left[1 - a\left(Area^{b} - c\log_{10}Duration\right)Duration^{-d} + eArea^{f}Duration^{g}\left(0.3 + \log_{10}AEP\right) + h10^{iArea\frac{Duration}{1440}}\left(0.3 + \log_{10}AEP\right)\right]\right\}$$

Zone	Southern Temperate
a	0.158
b	0.276
c	0.372
d	0.315
e	0.000141
f	0.41
g	0.15
h	0.01
i	-0.0027

Short Duration ARF

$$ARF = Min\left[1, 1 - 0.287\left(Area^{0.265} - 0.439\log_{10}(Duration)\right).Duration^{-0.36} + 2.26 \text{ x } 10^{-3} \text{ x } Area^{0.226}.Duration^{0.125}\left(0.3 + \log_{10}(AEP)\right) + 0.0141 \text{ x } Area^{0.213} \text{ x } 10^{-0.021\frac{(Duration-180)^2}{1440}}\left(0.3 + \log_{10}(AEP)\right)\right]$$

Layer Info

Time Accessed	29 June 2017 01:08PM
Version	2016_v1

Storm Losses

Note: Burst Loss = Storm Loss - Preburst

Note: These losses are only for rural use and are NOT FOR USE in urban areas

Storm Initial Losses (mm)	28.0
Storm Continuing Losses (mm/h)	4.0

Layer Info

Time Accessed	29 June 2017 01:08PM
Version	2016_v1

Temporal Patterns

code	MB
Label	Murray Basin

Layer Info

Time Accessed	29 June 2017 01:08PM
Version	2016_v2

Areal Temporal Patterns

code	MB
arealabel	Murray Basin

Layer Info

Time Accessed	29 June 2017 01:08PM
Version	2016_v2

Median Preburst Depths and Ratios

Values are of the format depth (ratio) with depth in mm

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	2.2 (0.135)	2.0 (0.088)	1.9 (0.068)	1.8 (0.054)	1.5 (0.037)	1.3 (0.027)
90 (1.5)	3.1 (0.164)	2.8 (0.107)	2.6 (0.083)	2.5 (0.065)	1.9 (0.041)	1.5 (0.028)
120 (2.0)	3.0 (0.147)	2.6 (0.09)	2.3 (0.066)	2.0 (0.048)	1.9 (0.039)	1.9 (0.033)
180 (3.0)	3.5 (0.145)	3.2 (0.098)	3.1 (0.078)	2.9 (0.063)	5.1 (0.089)	6.7 (0.102)
360 (6.0)	1.4 (0.044)	1.6 (0.038)	1.8 (0.034)	1.9 (0.032)	5.0 (0.068)	7.3 (0.087)
720 (12.0)	0.3 (0.006)	2.8 (0.048)	4.4 (0.065)	6.0 (0.075)	7.5 (0.078)	8.7 (0.079)
1080 (18.0)	0.3 (0.005)	1.4 (0.02)	2.1 (0.026)	2.8 (0.03)	4.6 (0.041)	6.0 (0.047)
1440 (24.0)	0.0 (0.0)	1.1 (0.015)	1.8 (0.02)	2.6 (0.024)	3.1 (0.024)	3.4 (0.024)
2160 (36.0)	0.0 (0.001)	0.1 (0.001)	0.1 (0.001)	0.2 (0.001)	0.4 (0.003)	0.6 (0.004)
2880 (48.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.1 (0.0)	0.0 (0.0)	0.0 (0.0)
4320 (72.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)

Layer Info

Time Accessed	29 June 2017 01:08PM
Version	2016_v2

Interim Climate Change Factors

Values are of the format temperature increase in degrees Celcius (% increase in rainfall)

	RCP 4.5	RCP6	RCP 8.5
2030	0.85 (4.3%)	0.845 (4.2%)	0.974 (4.9%)
2040	1.086 (5.4%)	1.05 (5.3%)	1.341 (6.7%)
2050	1.303 (6.5%)	1.283 (6.4%)	1.734 (8.7%)
2060	1.478 (7.4%)	1.539 (7.7%)	2.212 (11.1%)
2070	1.629 (8.1%)	1.775 (8.9%)	2.753 (13.8%)
2080	1.741 (8.7%)	2.036 (10.2%)	3.26 (16.3%)
2090	1.793 (9.0%)	2.316 (11.6%)	3.748 (18.7%)

Layer Info

Time Accessed	29 June 2017 01:08PM
Version	2016_v1
Note	ARR recommends the use of RCP4.5 and RCP 8.5 values

Baseflow Factors

downstream	11070
area_sqkm	741.6352
catch_no	11088
Volume Factor	0.09606
Peak Factor	0.019987

Layer Info

Time Accessed	29 June 2017 01:08PM
Version	2016_v1

7.3.3 Post Office Creek Data Hub Information

Australian Rainfall & Runoff Data Hub - Results

Input Data

Longitude	144.479
Latitude	-37.243
Selected Regions	
River Region	
ARF Parameters	
Temporal Patterns	
Areal Temporal Patterns	
Interim Climate Change Factors	
Baseflow Factors	

Region Information

Data Category	Region
River Region	Campaspe River
ARF Parameters	Southern Temperate
Temporal Patterns	Murray Basin

Data

River Region

division	Murray-Darling Basin
rivregnum	6
River Region	Campaspe River

Layer Info

Time Accessed	06 July 2017 09:44AM
Version	2016_v1

ARF Parameters

Long Duration ARF

$$Areal\ reduction\ factor = Min\left\{1, \left[1 - a\left(Area^{b} - c\log_{10}Duration\right)Duration^{-d} + eArea^{f}Duration^{g}\left(0.3 + \log_{10}AEP\right) + h10^{iArea\frac{Duration}{1440}}\left(0.3 + \log_{10}AEP\right)\right]\right\}$$

Zone	Southern Temperate
a	0.158
b	0.276
c	0.372
d	0.315
e	0.000141
f	0.41
g	0.15
h	0.01
i	-0.0027

Short Duration ARF

$$ARF = Min\left[1, 1 - 0.287\left(Area^{0.265} - 0.439\log_{10}(Duration)\right).Duration^{-0.36} + 2.26 \text{ x } 10^{-3} \text{ x } Area^{0.226}.Duration^{0.125}\left(0.3 + \log_{10}(AEP)\right) + 0.0141 \text{ x } Area^{0.213} \text{ x } 10^{-0.021\frac{(Duration-180)^2}{1440}}\left(0.3 + \log_{10}(AEP)\right)\right]$$

Layer Info

Time Accessed	06 July 2017 09:44AM
Version	2016_v1

Storm Losses

Note: Burst Loss = Storm Loss - Preburst

Note: These losses are only for rural use and are NOT FOR USE in urban areas

Storm Initial Losses (mm)	28.0
Storm Continuing Losses (mm/h)	4.0

Layer Info

Time Accessed	06 July 2017 09:44AM
Version	2016_v1

Temporal Patterns

code	MB
Label	Murray Basin

Layer Info

Time Accessed	06 July 2017 09:44AM
Version	2016_v2

Areal Temporal Patterns

code	MB
arealabel	Murray Basin

Layer Info

Time Accessed	06 July 2017 09:44AM
Version	2016_v2

BOM IFD Depths

Click here to obtain the IFD depths for catchment centroid from the BoM website

Layer Info

Time Accessed	06 July 2017 09:44AM
Version	2016_v2

Median Preburst Depths and Ratios

Values are of the format depth (ratio) with depth in mm

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	3.1 (0.185)	2.7 (0.113)	2.4 (0.084)	2.2 (0.064)	1.6 (0.038)	1.2 (0.024)
90 (1.5)	2.4 (0.125)	2.5 (0.092)	2.5 (0.077)	2.6 (0.066)	1.8 (0.038)	1.3 (0.023)
120 (2.0)	3.5 (0.165)	3.1 (0.104)	2.8 (0.078)	2.5 (0.06)	1.9 (0.036)	1.4 (0.023)
180 (3.0)	3.2 (0.133)	3.2 (0.096)	3.2 (0.079)	3.2 (0.066)	5.6 (0.094)	7.4 (0.108)
360 (6.0)	1.4 (0.043)	1.6 (0.038)	1.8 (0.035)	2.0 (0.033)	4.8 (0.065)	6.9 (0.082)
720 (12.0)	0.3 (0.007)	2.0 (0.035)	3.1 (0.046)	4.1 (0.053)	5.9 (0.063)	7.2 (0.067)
1080 (18.0)	0.1 (0.003)	0.9 (0.014)	1.5 (0.019)	2.0 (0.022)	3.1 (0.029)	4.0 (0.033)
1440 (24.0)	0.0 (0.0)	0.5 (0.007)	0.9 (0.01)	1.2 (0.012)	1.6 (0.013)	1.8 (0.014)
2160 (36.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.2 (0.001)	0.3 (0.002)
2880 (48.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
4320 (72.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)

Layer Info

Time Accessed	06 July 2017 09:44AM
Version	2016_v2

10% Preburst Depths

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
90 (1.5)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
120 (2.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
180 (3.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
360 (6.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
720 (12.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
1080 (18.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
1440 (24.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
2160 (36.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
2880 (48.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
4320 (72.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)

Layer Info

Time Accessed	06 July 2017 09:44AM
Version	2016_v2

25% Preburst Depths

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	0.0 (0.001)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
90 (1.5)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
120 (2.0)	0.0 (0.001)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
180 (3.0)	0.0 (0.002)	0.0 (0.001)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.001)
360 (6.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
720 (12.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
1080 (18.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
1440 (24.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
2160 (36.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
2880 (48.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
4320 (72.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)

Layer Info

Time Accessed	06 July 2017 09:44AM
Version	2016_v2

75% Preburst Depths

min (h)\AEP (%)	50	20	10	5	2	1
60 (1.0)	11.7 (0.708)	14.0 (0.594)	15.6 (0.539)	17.1 (0.495)	14.7 (0.344)	12.9 (0.261)
90 (1.5)	10.7 (0.567)	13.1 (0.491)	14.7 (0.451)	16.3 (0.417)	14.9 (0.31)	13.9 (0.249)
120 (2.0)	12.9 (0.614)	14.9 (0.508)	16.3 (0.456)	17.6 (0.415)	16.7 (0.32)	16.1 (0.266)
180 (3.0)	13.4 (0.55)	14.4 (0.43)	15.2 (0.373)	15.8 (0.329)	20.5 (0.346)	23.9 (0.35)
360 (6.0)	9.1 (0.285)	13.2 (0.305)	15.9 (0.307)	18.5 (0.304)	21.9 (0.296)	24.4 (0.287)
720 (12.0)	6.2 (0.147)	10.5 (0.186)	13.4 (0.2)	16.2 (0.206)	21.4 (0.228)	25.4 (0.238)
1080 (18.0)	4.3 (0.087)	8.1 (0.122)	10.6 (0.136)	13.0 (0.143)	16.8 (0.155)	19.6 (0.161)
1440 (24.0)	2.8 (0.052)	4.8 (0.066)	6.1 (0.07)	7.3 (0.073)	10.8 (0.091)	13.4 (0.1)
2160 (36.0)	1.8 (0.029)	2.8 (0.034)	3.5 (0.035)	4.2 (0.036)	7.3 (0.054)	9.7 (0.064)
2880 (48.0)	0.1 (0.001)	0.6 (0.006)	0.9 (0.008)	1.2 (0.009)	2.8 (0.019)	4.0 (0.025)
4320 (72.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.1 (0.0)	0.6 (0.004)	1.0 (0.005)

Layer Info

Time Accessed	06 July 2017 09:44AM
Version	2016_v2

90% Preburst Depths

min (h)\AEP (%)	50	20	10	5	2	1
60 (1.0)	22.4 (1.359)	27.3 (1.154)	30.5 (1.052)	33.6 (0.97)	29.3 (0.686)	26.1 (0.528)
90 (1.5)	23.0 (1.215)	26.7 (0.994)	29.1 (0.888)	31.4 (0.804)	28.9 (0.6)	27.0 (0.484)
120 (2.0)	24.4 (1.164)	28.0 (0.954)	30.4 (0.851)	32.7 (0.77)	41.8 (0.8)	48.7 (0.803)
180 (3.0)	23.5 (0.968)	27.8 (0.827)	30.6 (0.754)	33.4 (0.693)	42.9 (0.726)	50.0 (0.732)
360 (6.0)	23.6 (0.74)	29.4 (0.68)	33.3 (0.643)	37.0 (0.608)	45.5 (0.615)	51.9 (0.611)
720 (12.0)	21.9 (0.522)	29.5 (0.523)	34.6 (0.514)	39.4 (0.503)	44.5 (0.474)	48.4 (0.454)
1080 (18.0)	16.3 (0.331)	20.3 (0.307)	22.9 (0.293)	25.4 (0.28)	31.3 (0.29)	35.7 (0.293)
1440 (24.0)	14.7 (0.271)	19.3 (0.264)	22.3 (0.258)	25.2 (0.252)	28.9 (0.243)	31.7 (0.237)
2160 (36.0)	11.7 (0.189)	14.9 (0.178)	17.0 (0.172)	19.0 (0.166)	26.1 (0.194)	31.5 (0.208)
2880 (48.0)	9.4 (0.14)	10.5 (0.116)	11.2 (0.104)	11.9 (0.096)	20.2 (0.138)	26.4 (0.161)
4320 (72.0)	1.9 (0.026)	5.8 (0.058)	8.3 (0.07)	10.7 (0.079)	17.0 (0.105)	21.7 (0.12)

Layer Info

Time Accessed	06 July 2017 09:44AM
Version	2016_v2

Interim Climate Change Factors

Values are of the format temperature increase in degrees Celcius (% increase in rainfall)

	RCP 4.5	RCP6	RCP 8.5
2030	0.85 (4.3%)	0.845 (4.2%)	0.974 (4.9%)
2040	1.086 (5.4%)	1.05 (5.3%)	1.341 (6.7%)
2050	1.303 (6.5%)	1.283 (6.4%)	1.734 (8.7%)
2060	1.478 (7.4%)	1.539 (7.7%)	2.212 (11.1%)
2070	1.629 (8.1%)	1.775 (8.9%)	2.753 (13.8%)
2080	1.741 (8.7%)	2.036 (10.2%)	3.26 (16.3%)
2090	1.793 (9.0%)	2.316 (11.6%)	3.748 (18.7%)

Layer Info

Time Accessed	06 July 2017 09:44AM
Version	2016_v1
Note	ARR recommends the use of RCP4.5 and RCP 8.5 values

Baseflow Factors

downstream	11070
area_sqkm	741.6352
catch_no	11088
Volume Factor	0.09606
Peak Factor	0.019987

Layer Info

Time Accessed	06 July 2017 09:44AM
Version	2016_v1