Shepparton Mooroopna Flood Mapping and Flood Intelligence

Data Review, Hydrology and Hydraulic Model Calibration

April 2017

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PROJECT DETAILS

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Client	Greater Shepparton City Council
Client Project Manager	Greg McKenzie
Water Technology Project Manager	Ben Tate
Report Authors	Richard Connell, Ben Tate, Lachlan Inglis
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15 Business Park Drive
Notting Hill VIC 3168

Telephone (03) 8526 0800
Fax (03) 9558 9365

ACN No. 093 377 283
ABN No. 60 093 377 283

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1. INTRODUCTION

Water Technology was commissioned by the Greater Shepparton City Council to undertake the Shepparton Mooroopna Flood Mapping and Intelligence Project. This study involves detailed hydrological and hydraulic modelling of the Goulburn River, Seven Creeks and the Broken River and flood mapping. The main outcome of the study is to update information for use in sharing flood intelligence for the Shepparton Mooroopna area.

As part of the initial scoping work, the data required for modelling and mapping was collated and reviewed. This report documents the data reviewed, identifies gaps and methods used to infill the missing data, and describes the hydrology and hydraulic model development and calibration. The model was calibrated to the large flood event of October 1993 and smaller September 2010 flood event in tandem, with validation to the May 1974 flood event.

2. STUDY AREA

Shepparton and Mooroopna are situated on the Goulburn River at the confluence with the Broken River and Seven Creeks. The study area in the tender documentation extended upstream of Shepparton to Toolamba and downstream of Shepparton to Loch Garry on the Goulburn River, upstream of Shepparton to Kialla East on the Broken River and upstream of Shepparton to Kialla West on Seven Creeks.

To model the Goulburn River, Broken River and Seven Creeks system, the hydraulic model was extended upstream to suitable boundary locations, Murchison on the Goulburn River, Gowangardie on the Broken River and upstream of Kialla West on the Seven Creeks. The final modelled area was thus significantly larger than the original tendered study area, Figure 2-1 clearly demonstrates this. The larger model area has been separated into three separate hydraulic models. Two coarser resolution models were developed to better understand the flood behaviour between Murchison and Kialla West on the Goulburn River, and Gowangardie to Kialla East on the Broken River. These two coarser models study the routing between the upstream gauges and Shepparton, and on the Broken River allow a better understanding of cross-catchment flows leaving the river between Gowangardie and Shepparton. A higher resolution model of the flood mapping area from Kialla East on the Broken River and Kialla West on the Goulburn River and Seven Creeks, down to Loch Garry on the Goulburn River was developed.

The hydrology of the system was considered across an even wider area, with many gauges outside the extended study area analysed.

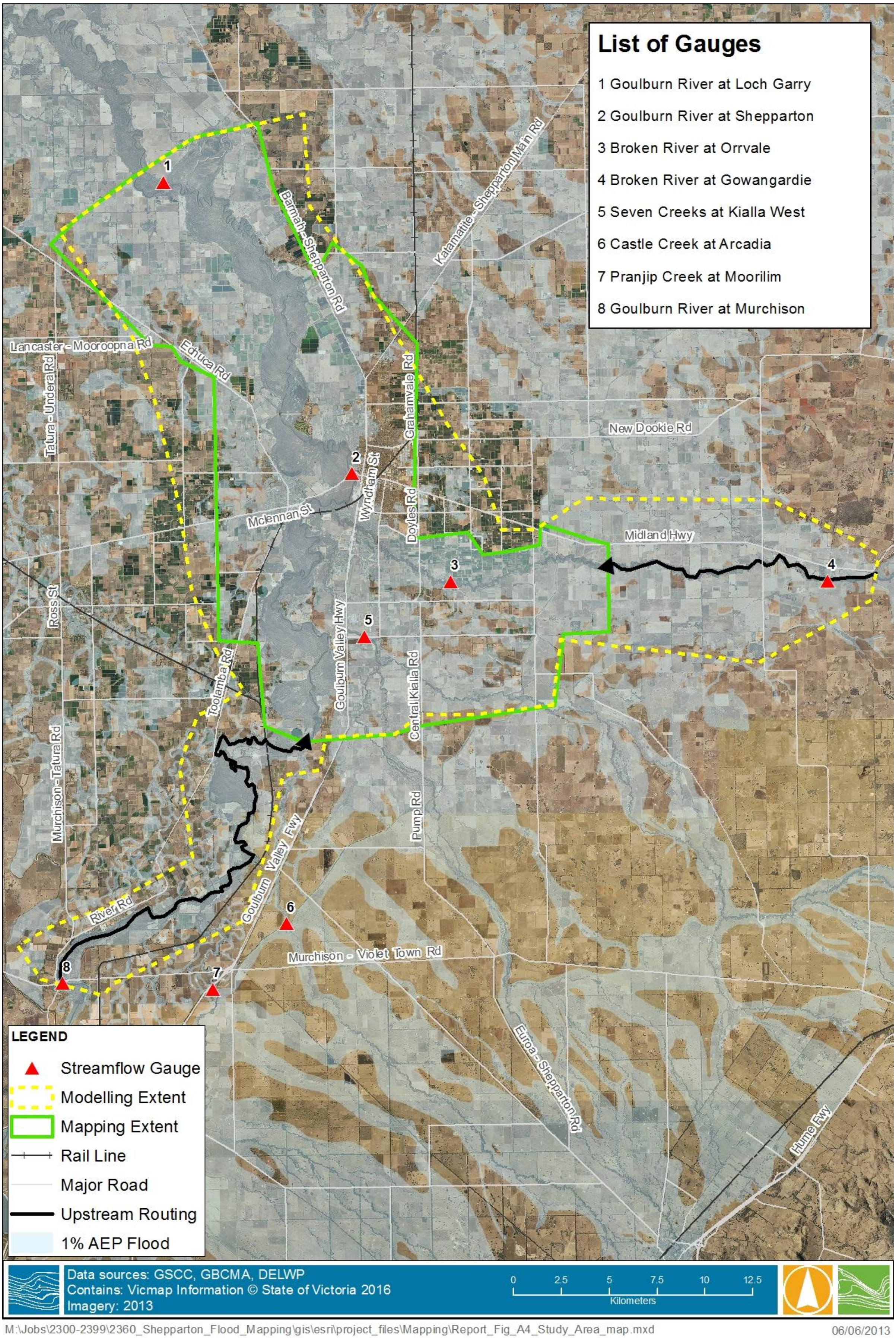


Figure 2-1 Study area extent, revised model area and gauge locations

3. DATA REVIEW AND ASSESSMENT

3.1 Overview

A large amount of information was available within the study area and broader catchment to assist in this study. A significant amount of hydrological data was collated and analysed along with many different topographical datasets. Shepparton and Mooroopna have a long history of flooding so many historical accounts of flooding and observed data was collated. Information from Goulburn Broken Catchment Management Authority (GBCMA), Greater Shepparton City Council (GSCC), the Department of Land Water and Planning (DELWP), the hydrographers (Ventia), Goulburn Murray Water (GMW) and VicRoads was collated. The Shepparton Mooroopna Floodplain Management Study by Sinclair Knight & Merz (SKM, 2002) was a major study that considered the issues of flooding in the study area, and as such was reviewed in detail.

3.2 Hydrological Data

3.2.1 Streamflow Data

Streamflow data is required for the hydrological analysis. The details of the streamflow gauging stations used in this analysis are listed in Table 3-1. These streamflow gauging details include the period of continuous streamflow record for each gauge. The continuous period of record is the period of systematic recording of streamflow via a daily read staff gauge or a continuous recorder. For some streamflow gauges, records are available during flood events only. Streamflow data records have been sourced from the DELWP water data portal.

Rating tables for the various stream gauges within the catchment were provided by Goulburn Broken CMA, DELWP and Ventia. During the calibration stage of the project it was found that recent changes to rating tables applied back over the entire record of data at some gauge sites has significantly changed the peak flow record for some historic flood events. This has a significantly large impact on the results of any flood frequency analysis and resultant design flows for this study. As such this is discussed in detail in Section 4.2.

Table 3-1 Streamflow gauge details

Station Name	Station No.	Area (km^2)	Period of record	Additional data since 2002 study
Broken River at Benalla	404203	1,461	1886 – 1961 (partial)* Oct 1977 to current	2002-now
Broken River at Casey's Weir (Goorambat)Headwater Gauge	404216	1,924	February 1888 to June 1916. July 1979 to current	2002 - now
Broken River at Casey's Weir (Goorambat) Tailwater Gauge	404200	1,924	July 1916 to June 1979	N/A
Broken River at Gowangardie	404224	2,396	January 1978 to July 1985 August 1991 to current	Not used in SKM study
Broken River at Orrvale	404222	2,508	June 1977 to current	2002 - now
Goulburn River at Goulburn Weir	405253	10,627	March 1967 to October 1985	N/A
Goulburn River at Murchison	405200	10,772	June 1881 to March 1967 November 1984 to current	2002 - now
Goulburn River at Kialla West	405270	12,038	June 1977 to August 1985	N/A
Goulburn River at Shepparton	405204	16,125	June 1921 to current	2002 - now

Station Name	Station No.	Area (km^2)	Period of record	Additional data since 2002 study
Goulburn River at Loch Garry	405276	16,490	Feb-1978 to current	2002 - now
Seven Creeks at Euroa Township	405237	332	May 1963 to current	2002 - now
Seven Creeks at Kialla West	405269	1,505	June 1977 to current	2002 - now
Pranjip Creek at Moorilim	405226	787	December 1957 to current	Not used in SKM study
Castle Creek at Arcadia	405246	164	June 1970 to current	Not used in SKM study

* note that the rating curve for Benalla pre-1970 is no longer valid due to construction of the lake

3.2.2 Peak Design Flow Estimates

The SKM (2002) study undertook a detailed flood frequency analysis for many gauges on the Goulburn and Broken Rivers. This analysis is provided below in Table 3-2. Note that for some sites the adopted design flows were from a combination of methods, unless otherwise indicated, estimates were from flood frequency analysis over the gauge period.

Table 3-2 SKM (2002) Adopted Design Peak Flow Estimates (ML/d)

	Goulburn River at Murchison	Broken River at Casey's Weir	Broken River at Benalla	Seven Creeks at Euroa	Goulburn River at Shepparton
Period for FFA	1956-1999, 1916 (1)	1889-1999	1955-1999, 1916, 1921	1956-1999, 1916, 1921	1921-1999, 1916
20%	51,900	23,300	30,900	11,800	73,400
10%	68,400	31,400	45,500	16,200	102,000
5%	87,000	40,500	61,600	20,200	137,000
2%	114,000	54,500	85,600(2)	25,800(3)	180,000
1%	134,000	66,900	106,000(2)	34,000(3)	219,000
0.5%	158,000	81,200	128,000	42,900(3)	261,000
0.2%	192,000	103,000	161,000	56,300(3)	336,000

(1) 1956-1999 chosen for FFA as it is period after construction of Big Eildon dam.

(2) Estimate adopted from calibrated rainfall-runoff modelling by Willing and Partners (1998) study instead of FFA.

(3) Estimate adopted from calibrated rainfall-runoff modelling by SKM (1997) study instead of FFA.

The SKM (2002) study used a number of regression equations to transpose the peak design flows from the above mentioned gauges to the boundaries of their study area. As volume is just as important as peak flow in large flat floodplains, the frequency analysis and transposition was repeated for five day volumes.

3.2.3 Design Flow Hydrographs

To determine a design hydrograph the SKM (2002) study scaled historic hydrographs to represent the design peak flow and 5 day volume. The 1974 hydrograph was adopted for the Goulburn River and the 1993 hydrographs for the Broken River and Seven Creeks.

The timing of the three major contributing catchments has a large impact on the resulting flood at Shepparton. The SKM (2002) study found that the peak flow of Seven Creeks at Kialla West generally occurs between 6-24 hours earlier than the Broken River at Orrvale, the study adopted the median 15 hour time offset for the peak flow for design purposes. The relative timing of the Goulburn and

Broken River flood peaks was also investigated, however a lack of data hindered this assessment. A lag time of 33 hours was assumed between Goulburn Weir and Kialla West and 30 hours between Murchison and Kialla West. It was estimated that the peak flow in the Goulburn at Kialla West occurred approximately 15 hours after the peak flow on the Broken River at Orrvale for the 1974 event, with a 60 hour lag in the 1993 event. This longer lag in 1993 was attributed to the impact of Eildon attenuating the flood in the upper catchment, with the lower catchment having a smaller contribution to the Goulburn flows. For design purposes the 15 hour time lag from the 1974 event was adopted. A number of design flood scenarios were developed using various combinations of Goulburn River, Broken River and Seven Creeks flows for a given design event at the Shepparton gauge. A similar approach in adopting appropriate timing for design events for the current investigation is discussed in more detail in Section 4.4.2 with the timing tested in the hydraulic model to assess the sensitivity on flood levels shown in Section 4.4.3.

3.3 Topographic and Physical Survey

Several sources of topographic/survey data were obtained to prepare the hydraulic model. Most of the data was provided by GBCMA and GSCC. These include:

- Light detection and ranging (LiDAR) data
- Pipe Drainage Networks
- Survey Cross sections
- Photogrammetry
- Feature survey of Shepparton Mooroopna Causeway
- Feature survey of strategic levees downstream of Shepparton

3.3.1 LiDAR Data

LiDAR data for the region was made available by Goulburn Broken CMA and DELWP. A summary of available digital elevation model (DEM) data sets is summarised below in Table 3-3.

Table 3-3 Available Digital Elevation Model Data Sets

DEM Data Set	Resolution	Year Flown	Vertical Accuracy
Fugro Spatial Systems (FSS)	1 m & 5 m DEM	2007	± 0.10 m
Index of Stream Condition (ISC)	1 m DEM	2011	± 0.15 m
Floodplain Set I	1 m DEM	2011	± 0.10 m
Think Spatial UAV	1m DEM	2013	± 0.15 m
VicMap Elevation	20 m DEM		
Geoscience Australia	1 Second DEM		

Figure 3-2 shows the extent of available DEMs used in the hydraulic modelling.

The 5 m/1 m Fugro Spatial Systems (FSS) data contained many gaps and ‘holes’ within the DEM. These were removed (using 12d terrain software) by creating a Triangulated Irregular Network (TIN) across the surrounding data points and exporting as a new DEM.

A comparison of the Floodplain (FP) and FSS datasets was undertaken in ARCGIS for a location where there was overlap. Both datasets had the same 1 m grid resolution. Very little elevation difference was observed where the two datasets overlapped, with the differences mostly present in channels and water bodies as well as variations in crop development. An example of the comparison in DEMs is shown in Figure 3-3. Areas of river channel, dense vegetation and crops showed elevated surface levels in the FP LiDAR compared to the FSS LiDAR, which indicated that the FSS LiDAR may be closer to the true ground level in these locations. Therefore, the FSS LiDAR was used in preference to the FP LiDAR where there was overlap.

The Index of Stream Condition (ISC) data follows the alignment of major waterways, but doesn't extend far onto the floodplain. This data set was found to be the most consistent with the feature survey of the causeway, whereas the FSS LiDAR data set was found to be lower than the feature survey. This is demonstrated by the analysis of the feature survey along the Shepparton Mooroopna Causeway shown in Figure 3-1 and summarised in Table 3-4.

The ISC LiDAR is on average 0.2 cm lower than the feature survey and the FSS LiDAR is on average 7.8 cm lower. For this reason the ISC LiDAR data set was chosen as the basis for the final model topography and the other data sets were adjusted to match. Several checks were carried out along the interface of the different datasets, and following this analysis it was decided to raise the FSS and FP LiDAR datasets by 10 cm to ensure a smooth transition between the different data sets. The final composition of the LiDAR used in the topography is shown in Figure 3-4.

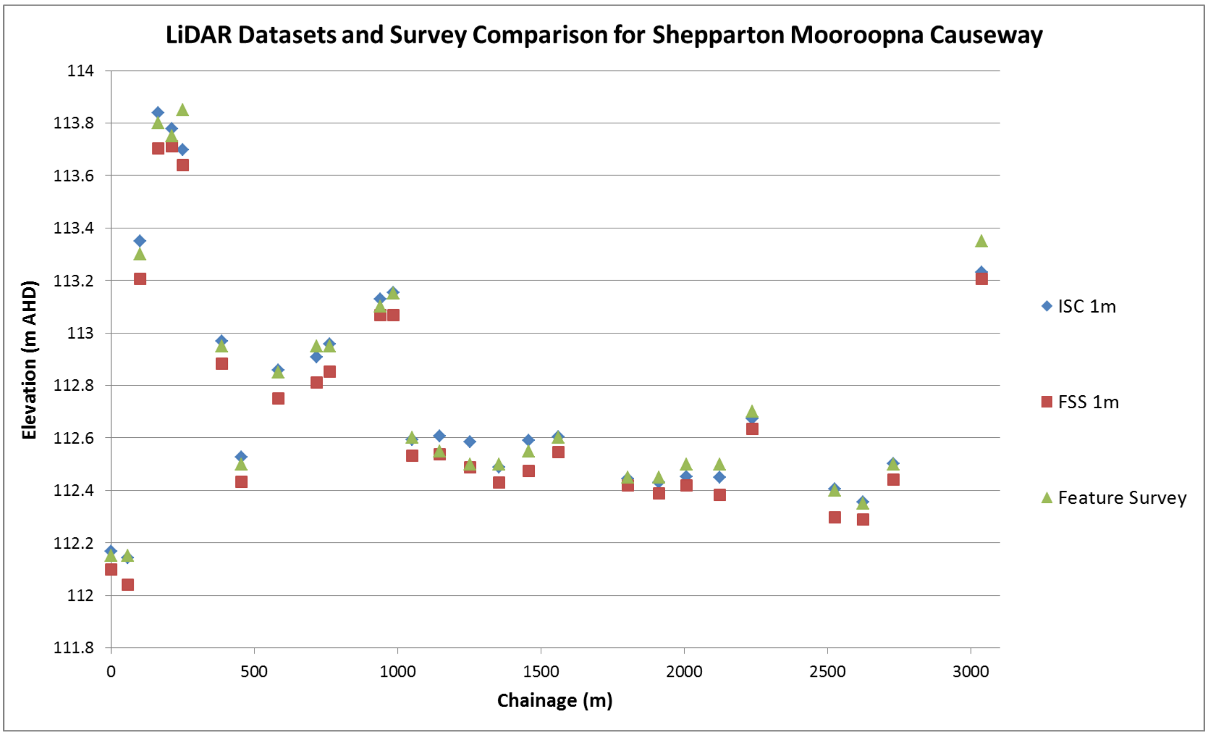


Figure 3-1 Survey and LiDAR elevation comparison along Shepparton-Mooroopna Causeway

Table 3-4 Survey and LiDAR elevation differences for Shepparton Mooroopna Causeway

Points Chainage	ISC 1 m Difference (cm)	FSS 1 m Difference (cm)	Points Chainage	ISC 1 m Difference (cm)	FSS 1 m Difference (cm)
0	1.7	-5.2	1146	5.8	-1.1
59	-0.7	-11.0	1253	8.6	-1.1
101	4.9	-9.3	1354	-1.3	-7.0
166	3.8	-9.7	1457	3.9	-7.5
212	2.9	-3.7	1560	0.5	-5.4
252	-15.1	-21.1	1805	-0.6	-3.1
387	1.9	-6.6	1913	-2.1	-6.2
456	2.7	-6.8	2009	-4.8	-8.2
584	0.8	-10.0	2125	-5.0	-11.7
718	-4.3	-14.0	2238	-2.6	-6.5
764	0.7	-9.6	2526	0.5	-10.2
940	2.8	-3.1	2623	0.4	-6.0
987	0.3	-8.2	2731	0.1	-5.9
1051	-0.8	-6.9	3039	-11.8	-14.5
			Mean	**-0.2**	**-7.8**

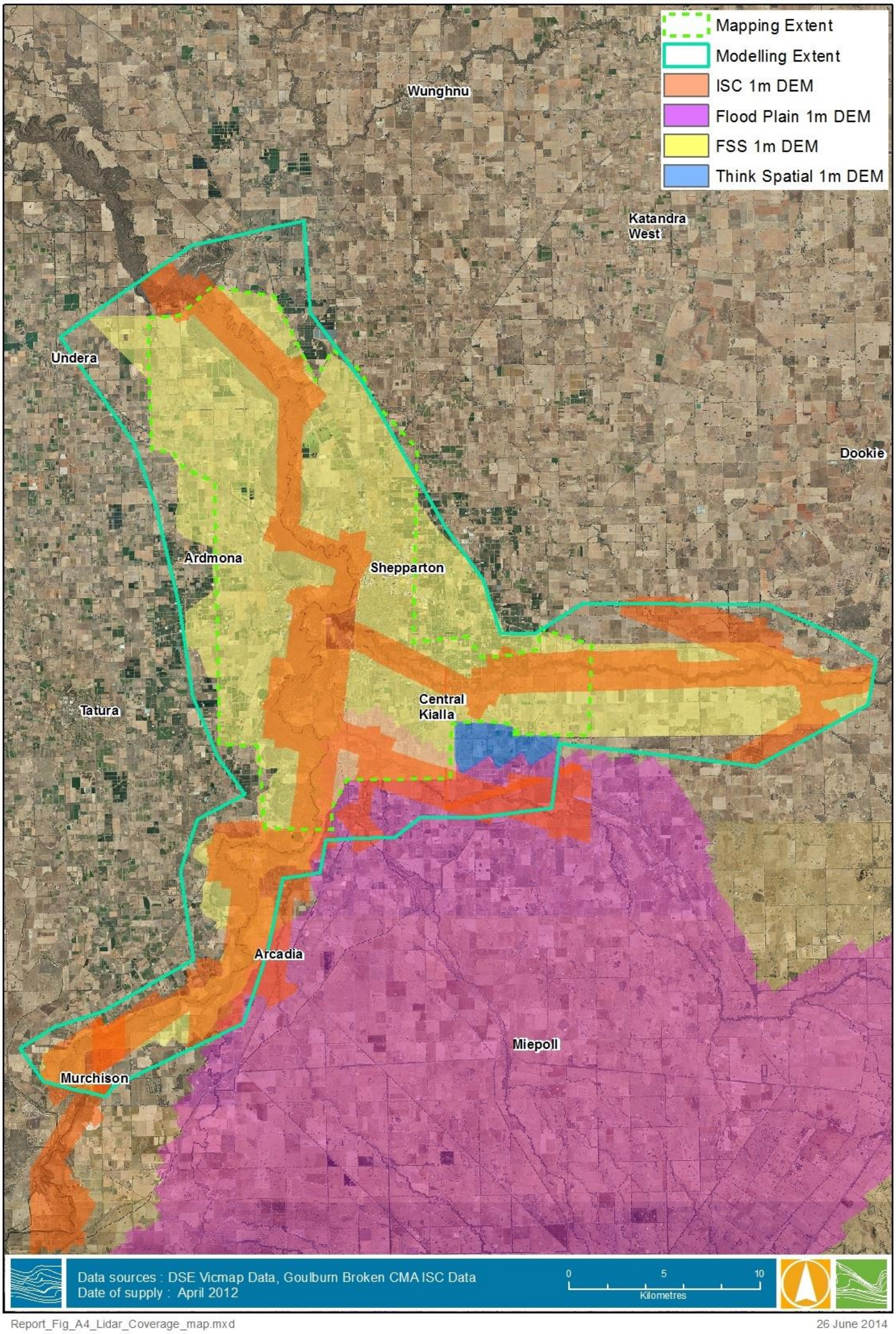


Figure 3-2 Extent of Available DEM's

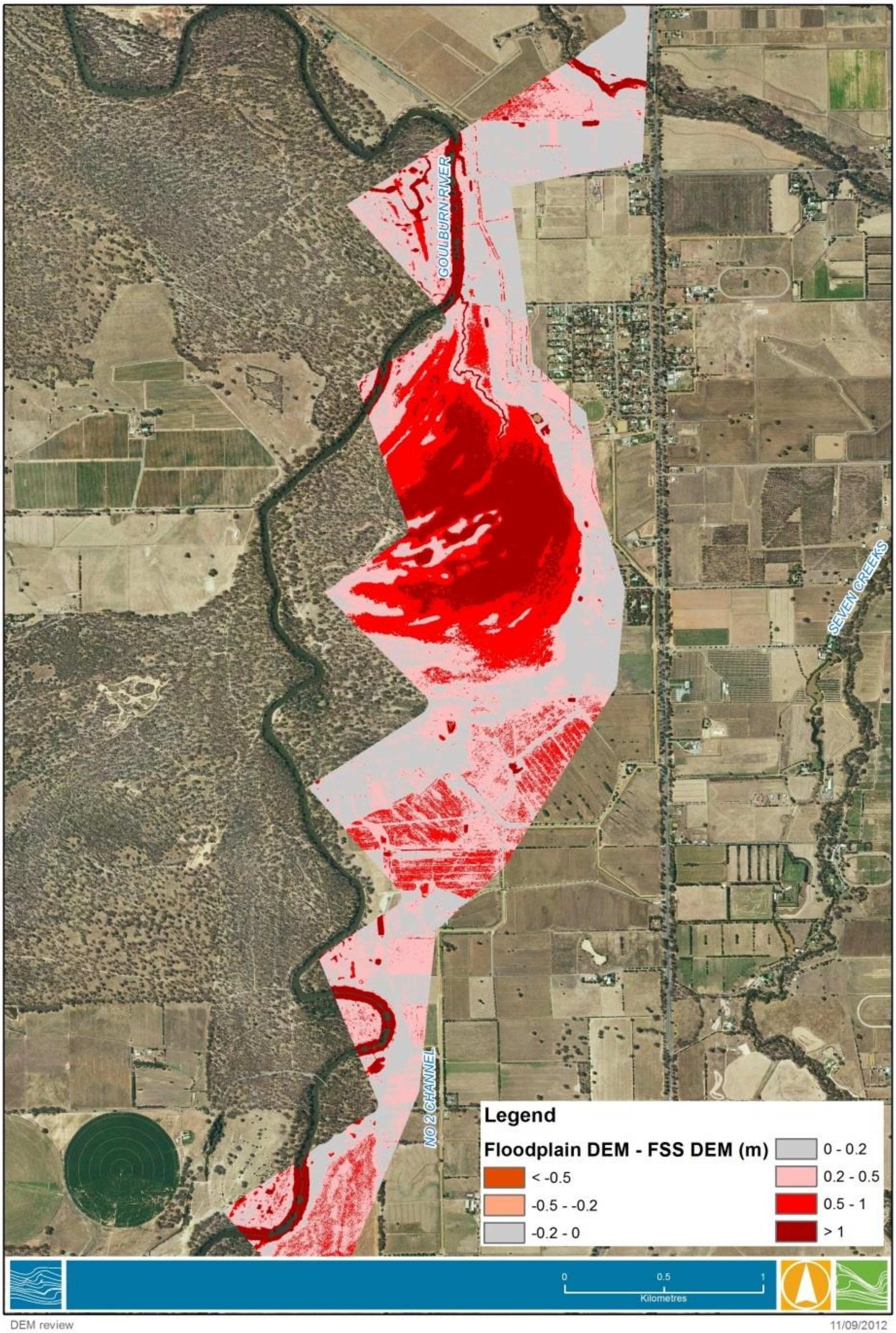


Figure 3-3 Vertical Comparison of Floodplain DEM and FSS DEM

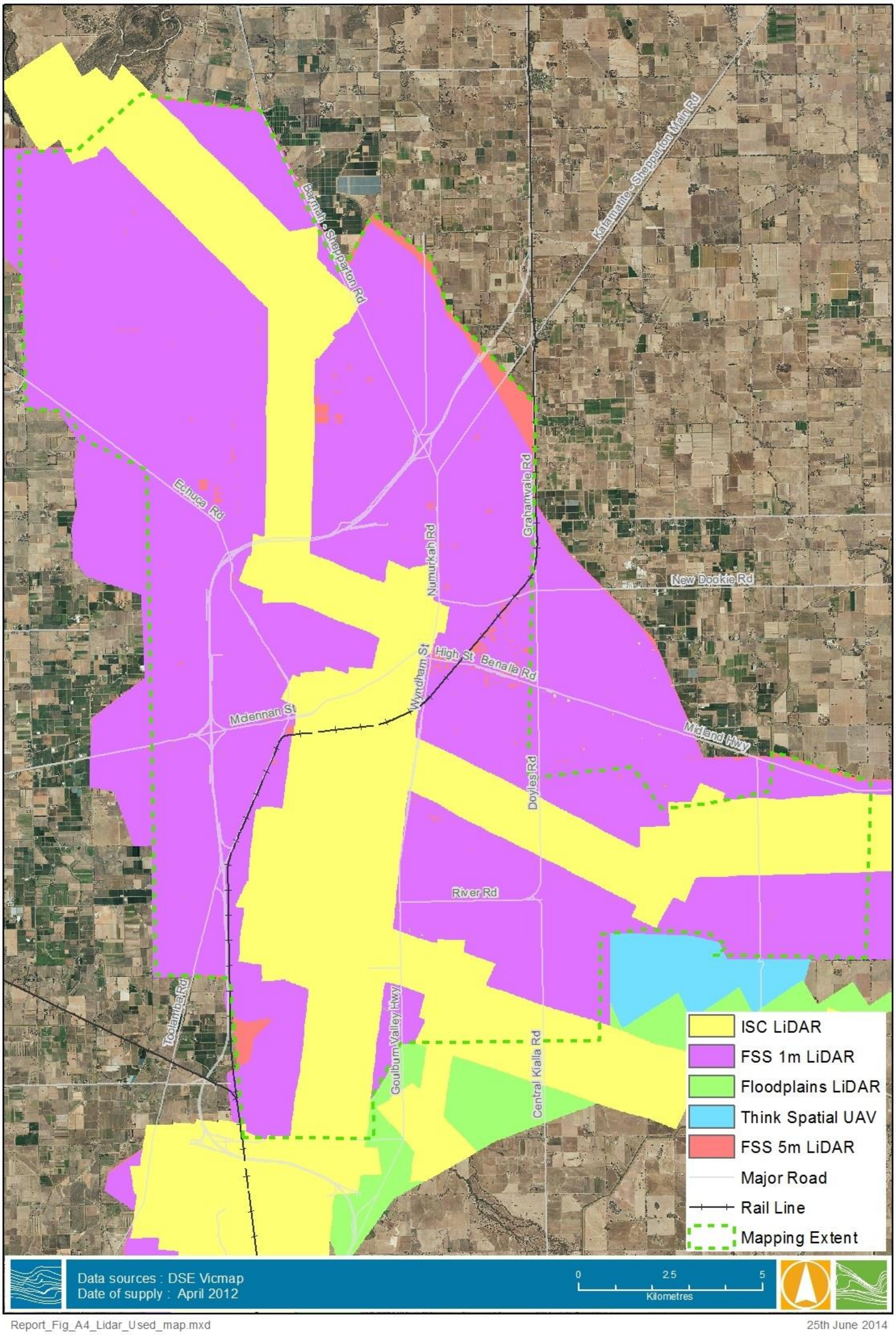


Figure 3-4 DEMs used in hydraulic model topography

3.3.2 Stormwater Drainage

Details of the underground drainage network are important for the establishment of the hydraulic model and identification of flood related drainage issues. It should be noted however that this study was not intended to consider the entire stormwater system. The study focussed on larger riverine flood events and included pipes greater than 600 mm in diameter, as identified within the GSCC GIS database. Other smaller pipes were included in areas where it was thought that they may be important in conveying floodwaters from the river to low lying areas that would be otherwise disconnected. The steering committee were consulted on the pipes and pits that were to be included in the hydraulic modelling and approved the selection.

Greater Shepparton City Council supplied their drainage network layout for Shepparton and Mooroopna. The network was received in an ESRI shapefile format of the pipes and pits.

The shapefiles indicated conduit/pit locations and conduit sizes for constructed pipes. The shapefile drainage network consisted of 8,091 conduits of which 8,078 had recorded diameters and 3,055 had recorded inverts. The drainage network is shown in Figure 3-6.

In addition to the underground pipe network, several major culverts, siphons and bridges were incorporated into the hydraulic model. Existing survey of the Shepparton-Mooroopna Causeway was available from earlier studies, with several hardcopy plans also made available from VicRoads, VicTrack and GMW.

3.3.3 Feature Survey

Two sets of feature survey were made available for this study, the survey of the Shepparton Mooroopna Causeway and the survey of the strategic levees downstream of Shepparton on the Goulburn River. The survey of the causeway was used to verify the LiDAR data as mentioned in Section 3.3.1. The survey of the levees was stamped onto the hydraulic model topography as thin break lines in the TUFLOW model topography. The location of the feature survey is shown in Figure 3-8.

Although the feature survey defined crest elevation for the causeway, the waterway openings were not defined. The Goulburn Broken CMA took photos and sketched up dimensions of all the waterway openings so that they could be well defined within the hydraulic model. The sketch provided for Daintons Bridge over the Goulburn River is shown in Figure 3-5. This is the main bridge crossing the Goulburn River, which was modelled as a 1D structure within the 1D Goulburn River Channel network. The remaining waterway openings were modelled as 2D structures, applying form losses, blockage from piers and the bridge deck and rails.

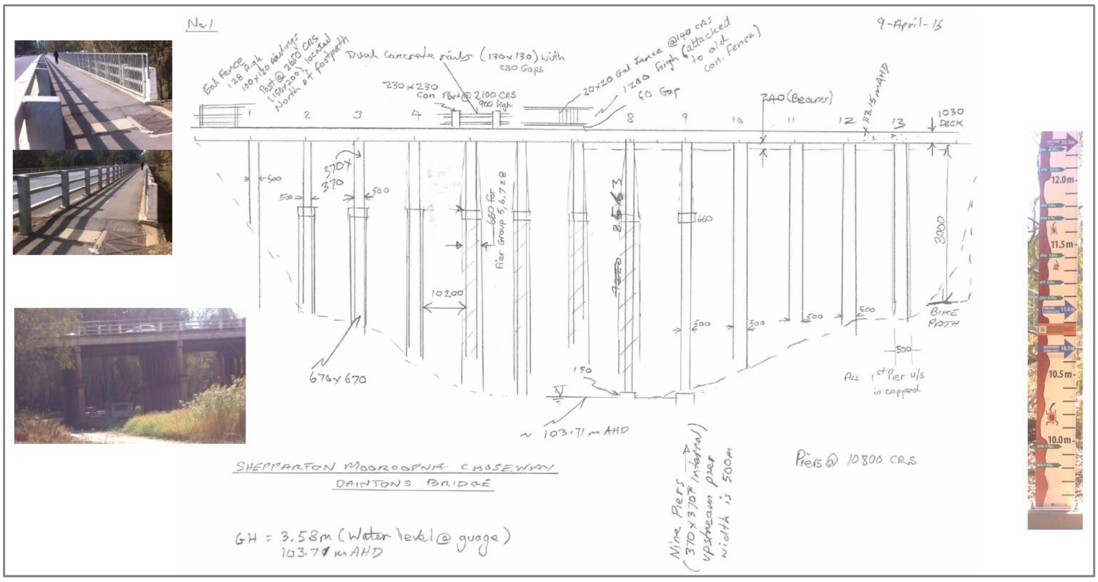


Figure 3-5 Daintons Bridge sketch provided by Goulburn Broken CMA

3.3.4 Irrigation Channels

The Goulburn Broken CMA provided ESRI shapefiles of the irrigation network. The data showed the location of channels and is shown in Figure 3-8. The irrigation channel banks form hydraulic barriers across the floodplain and were stamped onto the topography as thin break lines in the TUFLOW hydraulic model. The irrigation channel banks were digitised using the irrigation channel network shapefiles, refined using the 1 m LiDAR datasets.

Very limited information other than the alignment of these irrigation and drain features was available other than that extracted from LiDAR information. Some feature survey was available for small sections of channel bank and was included.

3.3.5 Aerial Photos and Observed Flood Extents

Aerial photos of the 1974 and 1993 flood events were received from Goulburn Broken CMA as well as digitised flood extents generated from the photos. Aerial photos for the 2010 flood event were sourced from NearMap and flood extents were digitised from this photography. The flood extents and photos were used to validate the hydraulic model for the calibration events. It should be noted that these images do not necessarily represent the peak of the flood event, with the 1993 image taken 2 days after the flood peak.

A recent aerial photo from 14th December 2013 was used for mapping purposes as a background image. This image was supplied by the Goulburn Broken CMA.

3.3.6 Observed Flood Levels and Floor Levels

The Goulburn Broken CMA provided flood levels from the Victorian Flood Database (VFD) which contain levels for a range of events including the 1974, 1993 and 2010 events. These levels were used to calibrate the hydraulic model. Figure 3-9 shows the available observed levels for the three calibration events.

3.3.7 Waterway survey

State Rivers and Water Supply Commission (SRWC) survey was used to define the channel invert within the waterway. This survey was undertaken for the 1982 Shepparton Mooroopna Flood Study undertaken by Sinclair Knight and Partners (SKP).

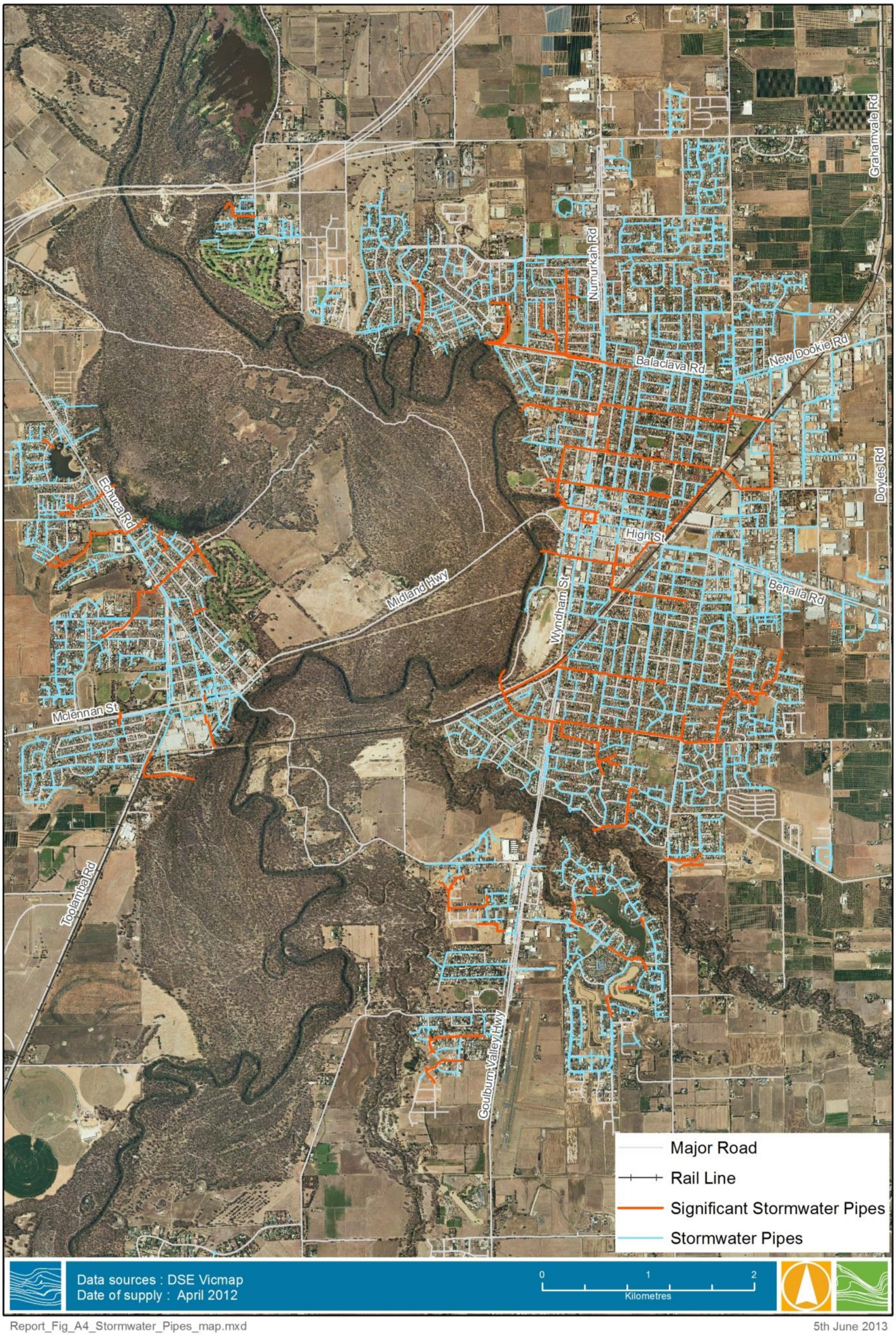


Figure 3-6 Shepparton-Mooroopna Stormwater Pipe System

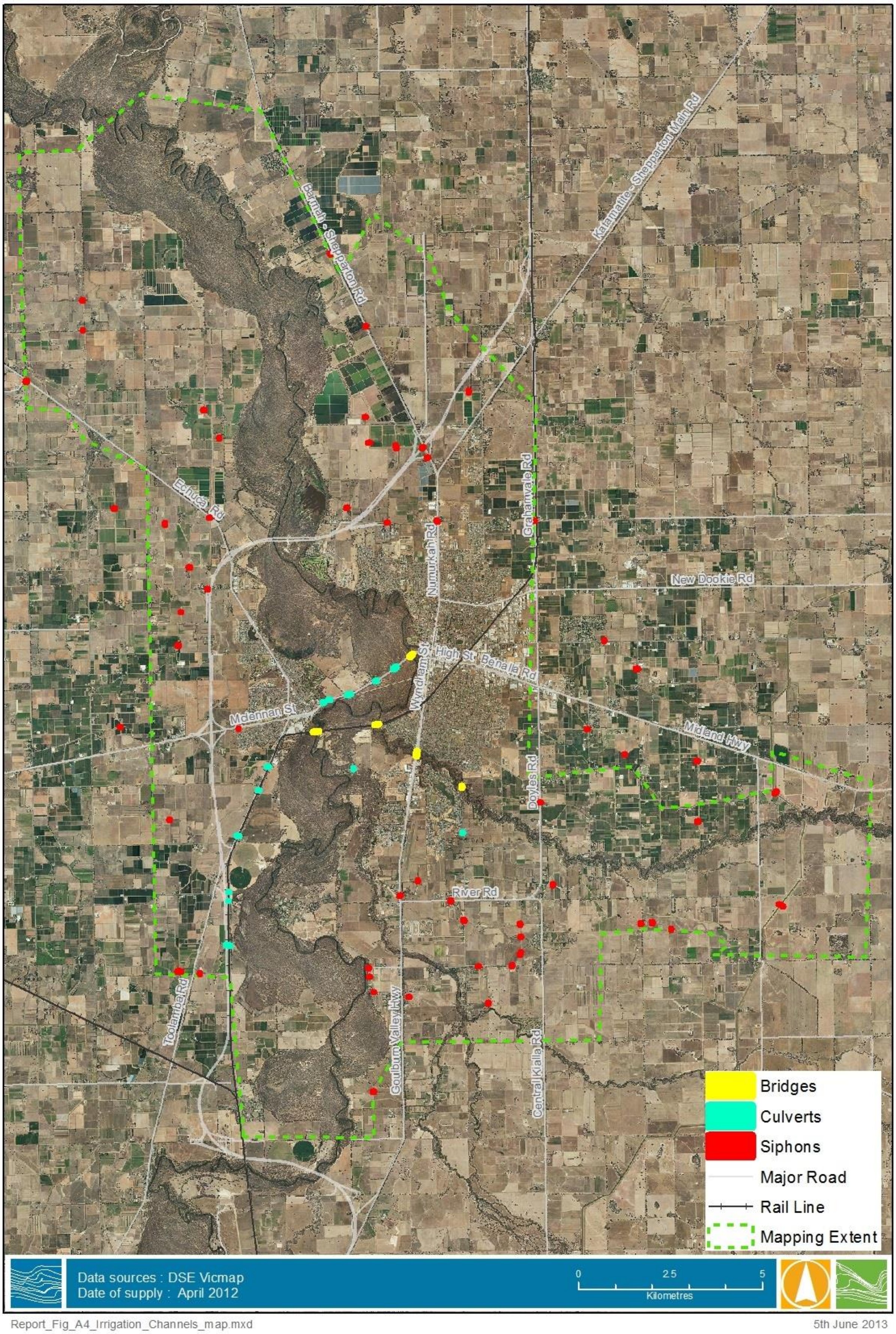


Figure 3-7 Shepparton-Mooroopna Culverts, Siphons and Bridges

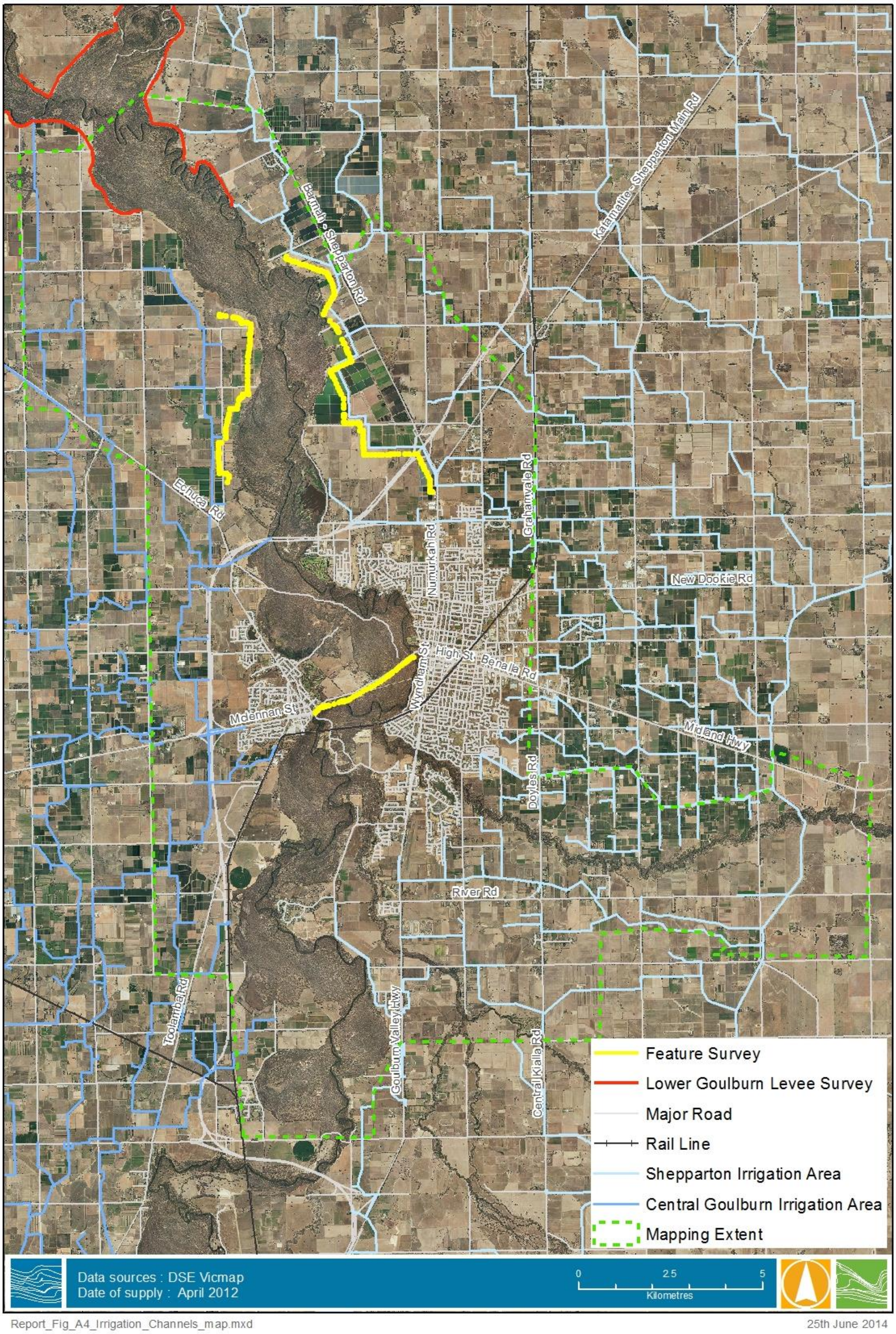


Figure 3-8 Shepparton-Mooroopna irrigation channels and feature survey

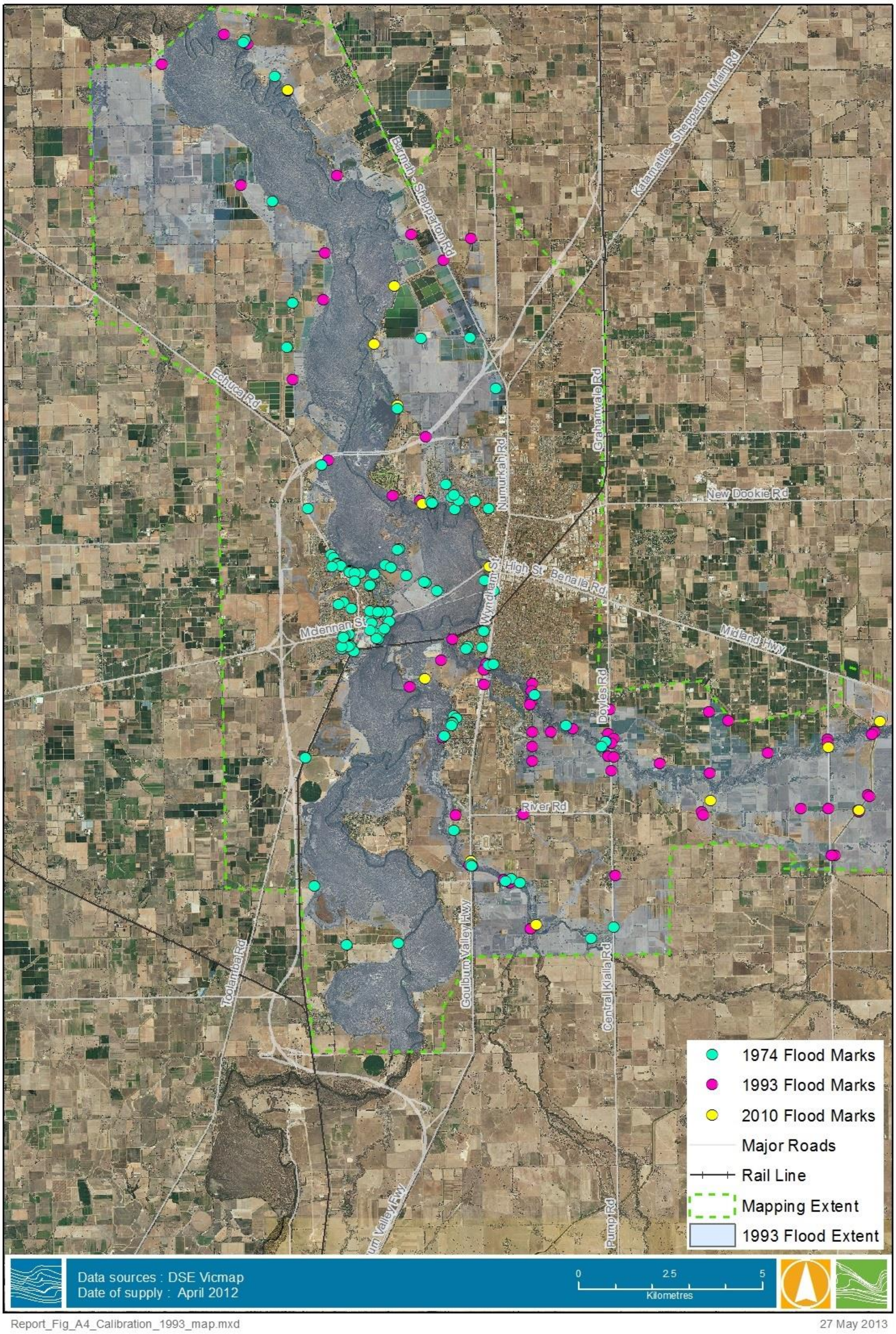


Figure 3-9 Shepparton-Mooroopna Observed Flood Levels

4. HYDROLOGY

4.1 Overview

Shepparton and Mooroopna are located on the floodplain of the Goulburn River, Broken River and Seven Creeks. The total catchment area of the Goulburn River at Shepparton is approximately 16,000 km^2 (2,525 km^2 in the Broken River catchment, 1,510 km^2 in the Seven Creeks/Honeysuckle Creek catchment, 800 km^2 in the Pranjip Creek catchment and 280 km^2 on the Castle Creek catchment). Given the size of the combined catchment of the Goulburn River upstream of Shepparton and the significance of Lake Eildon in the upper catchment, it was considered impractical to develop a single hydrological model of the area. Any model of the whole area would require numerous assumptions about design considerations and may not make the best use of available streamflow information. Furthermore, cross catchment flows are likely to occur in the Broken River and Seven Creeks catchments because of the extremely flat floodplain, making calibration of a hydrological model impractical. This was demonstrated in earlier hydrological studies of the Seven Creeks catchment. Given the uncertainties regarding an appropriate spatial and temporal distribution of design rainfall over such a large and varying catchment, not to mention drawdown considerations of Lake Eildon, it is considered more practical and a more efficient approach to update the methodology adopted in the SKM (2002) study, which used Flood Frequency Analysis of the long period of gauge records.

The hydrology approach adopted by SKM (2002) was robust but was improved and updated to take advantage of:

- Additional data from rainfall and streamflow events between 2002 and present day;
- New techniques and research undertaken as part of the revision of Australian Rainfall and Runoff; and,
- Inclusion of outputs from studies completed since 2002.

More specifically the hydrology approach outlined in the study was similar to SKM (2002) with the following updates and enhancements:

- The historical flow series used in flood frequency analysis was updated to include events up to 2012, including the large event in September 2010;
- Flood frequency analysis used updated procedures as outlined in the revised edition of Australian Rainfall and Runoff for fitting design distributions.
- Streamflow gauge ratings were reviewed, with the most appropriate streamflow data utilised;
- Additional routing was carried out within hydraulic models from established gauge locations to the township model boundary to aid in adopting time lags between upstream gauges and model inflow boundaries;
- Specific modelling of major breakouts from the Broken River to the Broken Creek catchment was completed for a range of events; and,
- Recent flood events and available hydrodynamic modelling of the Goulburn was utilised to inform timing of coincident flows for design purposes.

The following sections summarise the hydrological analysis that was undertaken as part of this project, building on the review of previous work undertaken in the SKM (2002) study.

Based on the availability of flood data (aerial imagery, survey and anecdotal evidence), the October 1993, September 2010 and May 1974 events have been used to calibrate the hydraulic model. There is an emphasis on these events in the following discussion around hydrology.

4.2 Rating Curve Review

4.2.1 Overview

Streamflow data was collated for all relevant gauges in the catchment from the Water Information Management System (http://data.water.vic.gov.au/monitoring.htm), and directly from DELWP. The data was compared and it was found that the two datasets had significant discrepancies in the instantaneous peak flows and average daily computed flow. Upon further analysis, a similar trend was observed across most gauges assessed. It was identified that this discrepancy was due to recent rating curve revisions, some of which had been applied back over the entire gauge period. To illustrate this, the 1916 peak flow was revised for the Goulburn River at Murchison gauge from 195,000 ML/d to 311,000 ML/d, close to a 60% increase in the peak flow. If this flow increase was adopted it would mean that the revised 1% AEP flow would be larger than the previous 0.2% AEP flow, dramatically changing design flood levels and influencing planning decisions. As the Goulburn River at Murchison gauge was used to produce the upstream model inflows on the Goulburn River, it was decided to undertake a thorough review of the rating curve using a detailed hydraulic model of Murchison.

4.2.2 Recent Changes to the Rating Curves

DELWP supplied rating curves along with instantaneous and daily mean streamflow records for all relevant gauges requested. Figure 4-1 below compares rating curves at different time periods (1974, 1993, 2010 and current) for all relevant gauges.

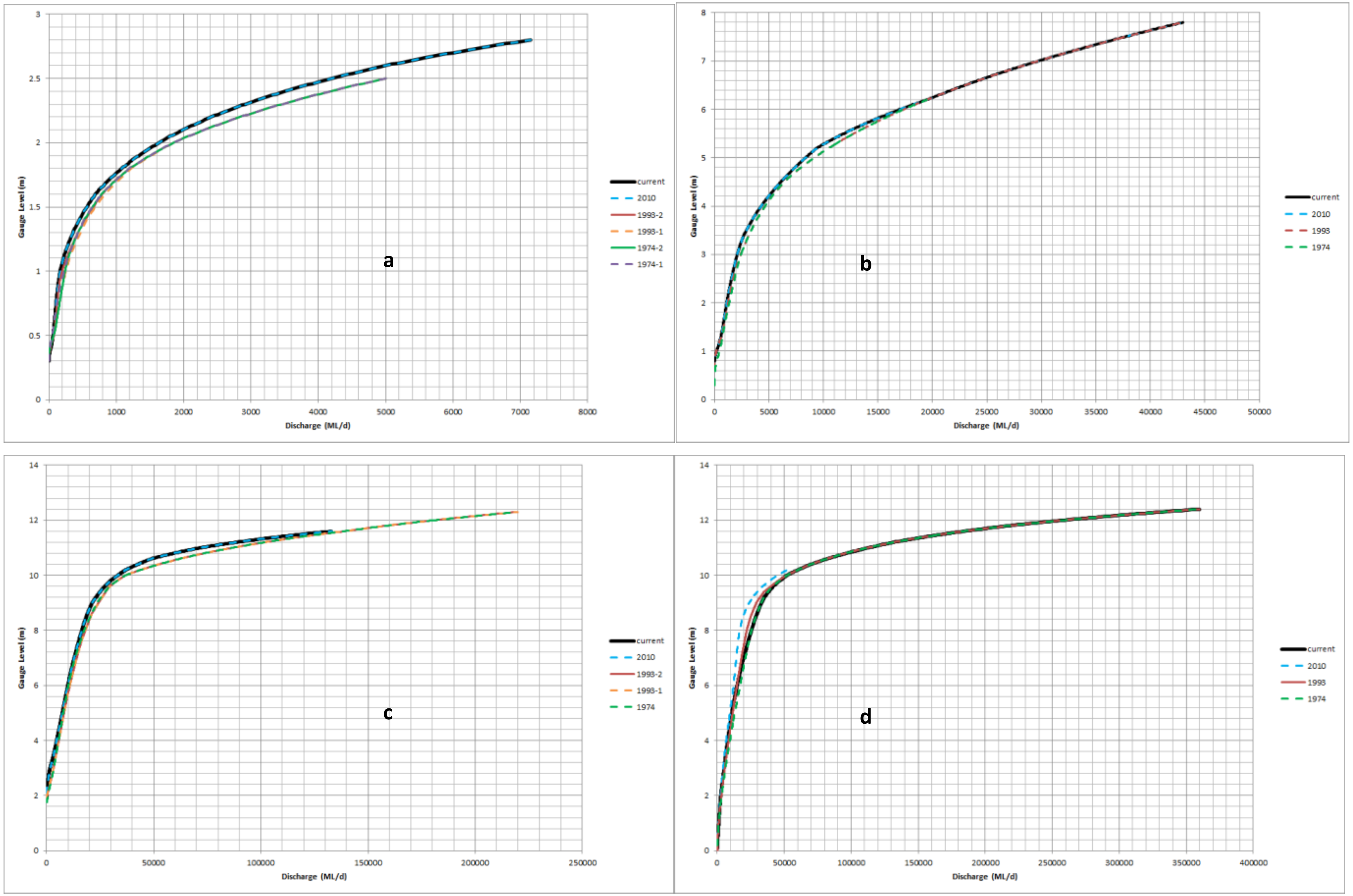
All the rating curves have experienced significant change over the past 40 years. Of interest was the Goulburn River at Murchison rating curve. Although the rating curve has not experienced much change in the high flow section of the rating curve since 1974, when comparing previously accepted estimates of the largest historic flood events to flows estimated using the recent rating curves, major discrepancies were identified.

4.2.3 Goulburn River at Murchison

The rating curve review and update of the Goulburn River at Murchison gauge is fully detailed in the Murchison Flood Mapping Study Report (Water Technology, 2014) and is summarised below.

The Murchison gauge has operated since 1887 and has a significant number of gauging's (610) that have made up the current rating curve (rating table number 73.00). The current rating curve is considered reliable up to a relative gauge height of 11.6 m or 184,000 ML/d. The highest gauging used to construct the rating curve was taken in 1974 at a relative gauge height of 10.915 m and approximately 100,000 ML/d, so even within the 'reliable' section of the curve there has been some extrapolation. The rating curve has been extrapolated out to a gauge height of 12.4 m or 360,000 ML/d.

The need to complete a full rating curve review became apparent when comparing the previously accepted flow estimates of the largest of the historic flood events with flows estimated using the extrapolated section of the current rating curve. The previously accepted flow estimate for the 1916 flood was 195,000 ML/d at Murchison. Using the current rating curve the 1916 flow is estimated at 311,000 ML/d. This increase in the flow of the 1916 event and other large events would have a significant impact on flood frequency analysis and design flood flows if adopted. This revised flow for 1916 did not correspond with other regional flow estimates on the Goulburn River, i.e. it was significantly larger than upstream and downstream gauge readings, warranting further investigation.


a
Gauge Level (m)
Discharge (ML/d)
current
2010
1993-2
1993-1
1974-2
1974-1
b
Gauge Level (m)
Discharge (ML/d)
current
2010
1993
1974
c
Gauge Level (m)
Discharge (ML/d)
current
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1974
d
Gauge Level (m)
Discharge (ML/d)
current
2010
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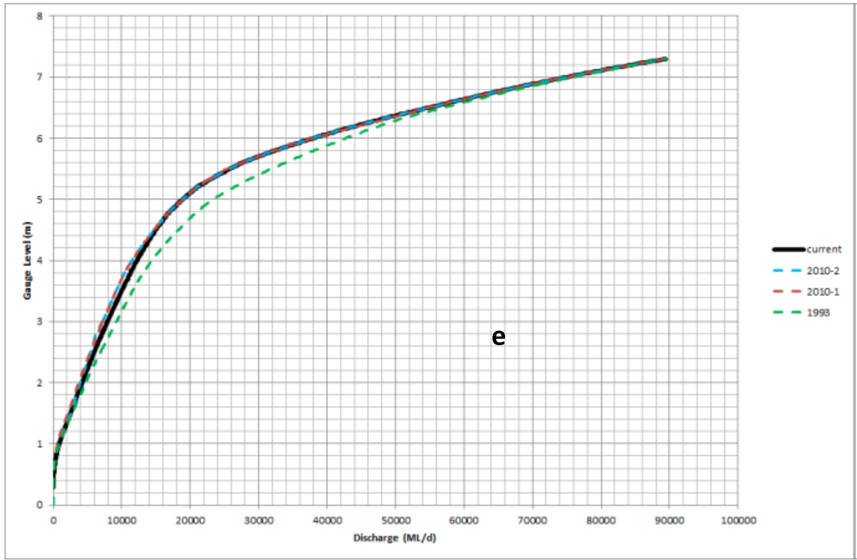

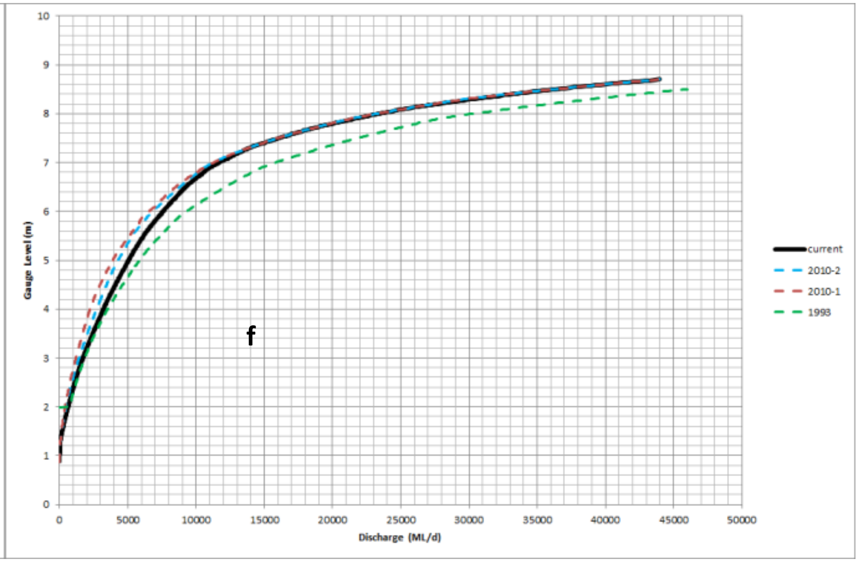

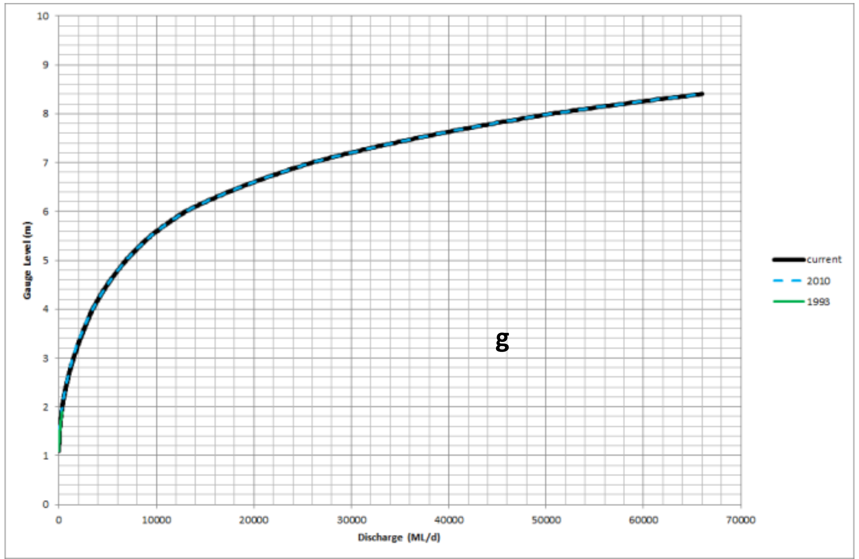

a. 405246 Castle Creek @ Arcadia
b. 405226 Pranjip Creek @ Moorilim
c. 405204 Goulburn River @ Shepparton
d. 405200 Goulburn River @ Murchison
e. 405224 Broken River @ Gowangardie
f. 405222 Broken River at Orrvale
g. 405269 Seven Creeks @ Kialla West

Figure 4-1 Recent Rating Curve Revisions

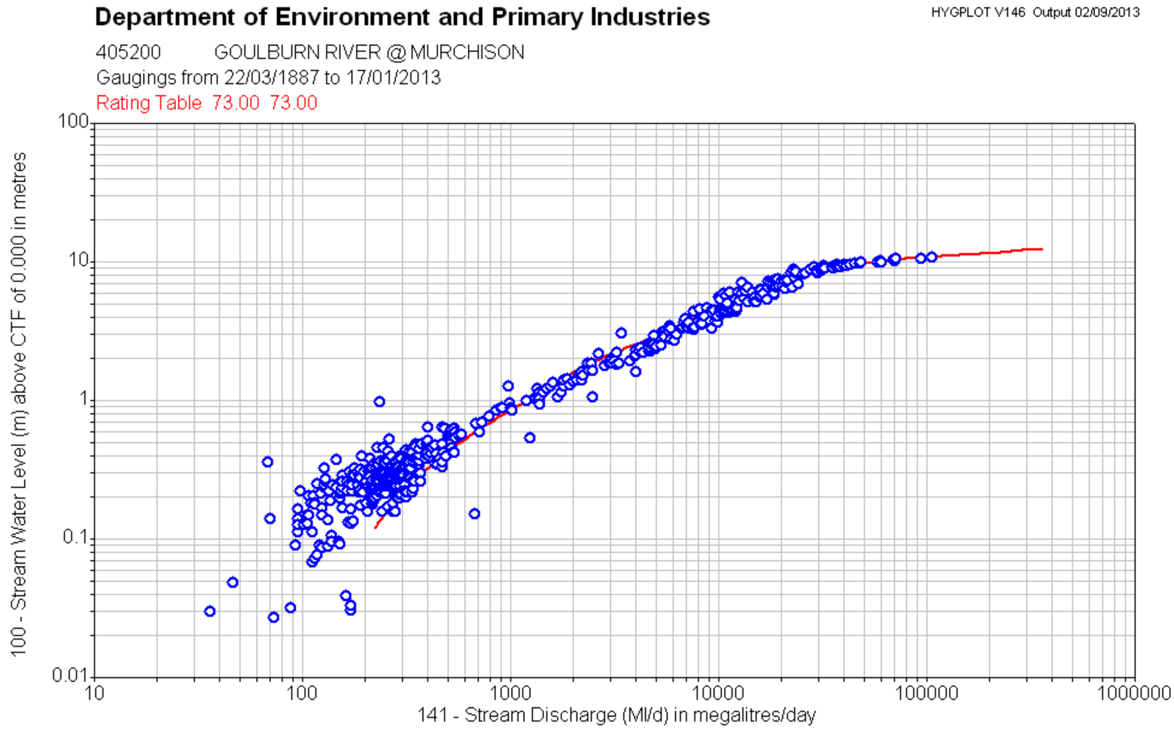

Figure 4-2 Murchison Current Rating Curve and Gaugings (Source: DELWP)

Murchison Hydraulic Modelling

A 1D-2D TUFLOW model with a grid resolution of 5 m was developed of the Murchison area, extending 4 km upstream and over 11 km downstream of the gauge site. The model was calibrated at the gauge site using the water levels and flows available for the 1974, 1993 and 2010 events. Another three steady state flows were run through the model to provide further verification points along the rating curve at the gauge site. The downstream boundary was set as a constant water level, 1.68 m below the level expected at the Murchison gauge site, based on analysis of the water surface profile captured by the ISC LiDAR. Although this is a simplistic assumption, it was tested through sensitivity and was shown to not unduly impact on model results due to its distance downstream.

The water levels predicted by the model at the gauge site for each flow are shown in Table 4-1 and Figure 4-3. The level for the 20,000 ML/d flow was 0.16 m lower than the current rating curve, however it was well within the envelope formed by the historic gaugings. It is understood that there exists a very large hysteresis loop in the rating curve at the site, with flows measured on the rising limb of a flood being very different to the flows measured on the falling limb of the flood. The levels were within 0.1 m of the current rating curve for flows from 50,000 to 100,000 ML/d. Given that gaugings only extend up to 100,000 ML/d this is a good calibration result, with the rating curve well-matched within this flow range. Above this flow, the modelled levels started to diverge from the rating curve significantly, and the modelled level for a flow of 184,000 ML/d was 0.62 m higher than the rating curve. This indicates that the extrapolation of the current rating curve above this flow is most likely overestimate flows for a given level.

This result is supported by a comparison of upstream and downstream gauges and previously adopted lower flow rates for the larger historic events.

Table 4-1 Murchison Calibration Results

Flow	Level (current rating curve)		Year	Tailwater level	Modelled Level		Deviation from Rating Curve	Deviation from historic levels
(ML/d)	(m)*	(m AHD)		(m AHD)	(m)	(m AHD)	(m)	(m)
20,000	7.04	115.72		114.04	6.88	115.56	-0.16	-
50,200	9.93	118.61**	2010	116.93	9.87	118.55	-0.06	-0.27 (2010)
63,500	10.27	118.95	1993	117.27	10.22	118.90	-0.05	-0.05 (1993)
100,000	10.84	119.52		117.84	10.93	119.61	+0.09	-
142,000	11.29	119.97	1974	118.29	11.64	120.32	+0.35	+0.35
184,000	11.60	120.28		118.60	12.22	120.90	+0.62	-

** Gauge zero 108.679 m AHD*

*** A level of 118.82 was measured in 2010 with a flow of 50,200 ML/d for this event; the current rating curve differs slightly.*

Two sensitivity analyses were performed by reducing the tailwater level by 1 m and increasing roughness by 25%. Figure 4-3 shows the results of the sensitivity analysis. This showed that the model is moderately sensitive to the adopted roughness values with water levels raised by between 0.18 and 0.35 m at the gauge location. It showed that the model is sensitive to the tailwater condition at low flows but less sensitive at high flows. Even with variation in the possible modelled rating curve, it is clear that the current rating curve over predicts flow at high stages.

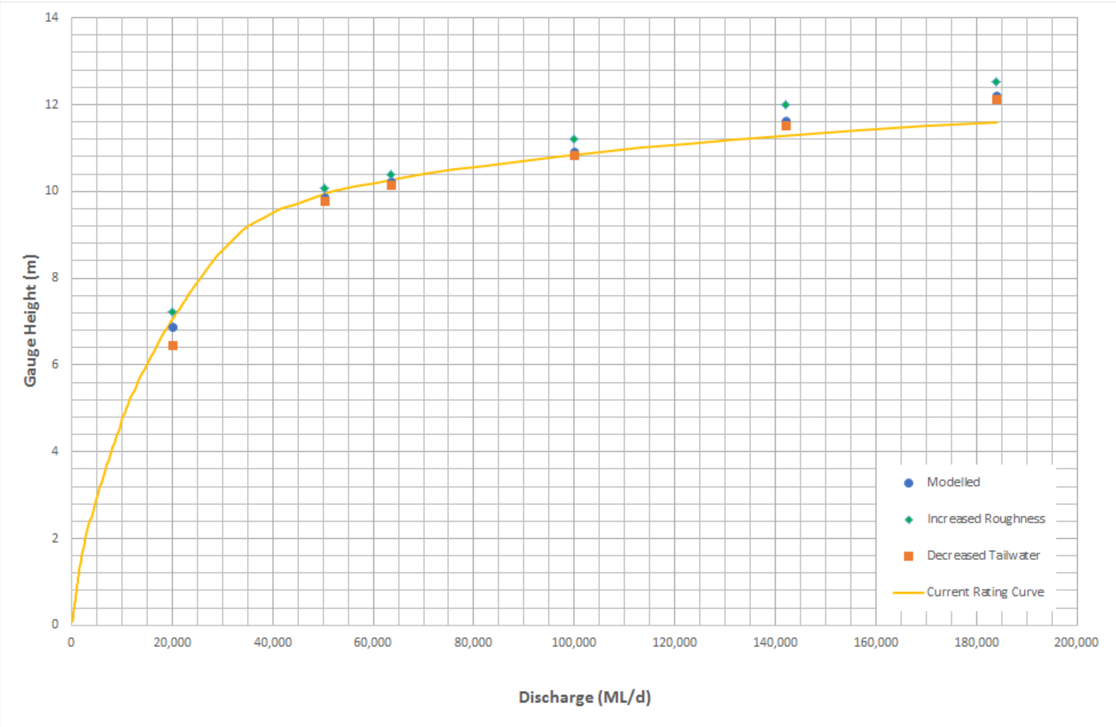

Figure 4-3 Murchison Calibration and Sensitivity Results

Given the good calibration to the high reliability section of the rating curve between 20,000 and 100,000 ML/d, and the relative insensitivity to tailwater conditions and roughness, the calibration was adequate for simulation of flood levels at the Murchison gauge. It was concluded that the current rating curve significantly overestimates the flow for a given stage at high flows. A revised rating curve was developed using the existing rating curve up to a relative gauge level of 10.5 m or 76,000 ML/d, with the rest of the curve interpolated between the modelled points from 10.5 m to 12.22 m.

The revised rating curve resulted in the 1916 flood level of 12.22 m having a peak flow of 178,000 ML/d which is much closer to the previously accepted estimate of 195,000 ML/d as compared to the current rating curves estimate of 311,000 ML/d. This flow estimate for 1916 is in line with estimates for other upstream and downstream gauges, so provides further confidence that the revised rating curve is an improvement on the current rating curve for the Goulburn River at Murchison.

At the time of writing this report Ventia was in the process of updating the rating curve in the extrapolated region using the modelled rating curve as suggested. This will be back dated to 2010 and will be used as part of the official rating curve at this site into the future. Ventia have created a new quality code to indicate the flow is based on a modelled extrapolated rating curve.

It should be noted that the recorded peak flow for 1975 at this gauge was found to be inconsistent with upstream and downstream gauges. The method adopted for estimating the actual 1975 peak flow is discussed below, in Section 4.3.4.

4.3 Design Flow Estimates

Flood frequency analysis was previously undertaken for the Shepparton Mooroopna Floodplain Management Study (SKM, 2002), which included flow data up till 1999. The flood frequency analysis was updated for this study utilising additional data from 2000 to 2012. There were also some issues identified with the flow gauging data which resulted in changes to the peak flow magnitudes included in the annual series.

4.3.1 Method

The following streamflow gauges were subject to a flood frequency analysis and revised estimates of design flood peaks were calculated:

- Broken River @ Benalla (404203)
- Broken River @ Orrvale (404222)
- Goulburn River @ Goulburn Weir (405253)
- Goulburn River @ Murchison (405200)
- Goulburn River @ Shepparton (405204)
- Seven Creeks @ Kialla West (405269)
- Pranjip Creek @ Moorilim (405226)
- Castle Creek @ Arcadia (405246)

Design peak flow estimates were derived directly from flood frequency analysis for most of the gauges. Estimates for the Broken River @ Benalla were adopted directly from SKM (2002), as there was no new flow data available. Estimates for Seven Creeks at Kialla West were derived from a regression with upstream gauges, due to a lack of long-term gauge data at Kialla West. This was possible because there was a good gauge record at Euroa and a strong relationship between peak flows at the two gauges. For gauges at the hydraulic model boundaries flood frequency analysis on volume was also carried out to enable design volumes to be estimated along with the design peak flows.

4.3.2 Data Review

A summary of the available gauge data for relevant gauges on the Goulburn River, Broken River and Seven Creeks is provided in Table 4-2 below. There were some discrepancies between the period of data available, and the recorded flow magnitudes, in the SKM (2002) report and the currently available dataset, this is described further below. A summary of the data used by SKM (2002) is provided in Table 4-3 below.

Table 4-2 Available gauge data

Gauge Number	Gauge Name	DELWP Water Monitoring Portal	RWC Blue Book (RWC, 1990)	Historic Peaks
404203	Broken River @ Benalla	1978-1981, 1983-1984, 1993, 1995-1996, 1998	1956, 1958, 1964 (Inst Flow) 1955-1964 (MDF)	
404200	Broken River @ Goorambat (Casey Weir T. Gauge)	1916-1973 (MDF) 1973-1979		
404216	Broken River @ Goorambat (Casey Weir H. Gauge)	1888-1916 (MDF) 1972-2013	1888-1916 (MDF) 1979-1986	
404222	Broken River @ Orrvale	1977-2013		1993
404224	Broken River @ Gownagardie	1991-2013		1993
405253	Goulburn River @ Goulburn Weir	1974-1980 1967-1980 (MDF)	1967-1984 (MDF)	1916, 1934, 1956
405200	Goulburn River @ Murchison	1881-2013		1916
405204	Goulburn River @ Shepparton	1974-2013 1921-2013 (MDF)	1921-1984	1916
405237	Seven Creeks @ Euroa	1973-2013 1963-1973 (MDF)		
405269	Seven Creeks @ Kialla West	2003-2013		1974#, 1993#
405226	Pranjip Creek @ Moorilim	1974-2013	1958-1986	
405246	Castle Creek @ Arcadia	1974-2013	1970-1986	

#Based on SKM Hydraulic Modelling (SKM, 2002)

Table 4-3 Data used by SKM (2002) for flood frequency analysis

Gauge Number	Gauge Name	Annual Series	Historic Peaks	1% AEP Flow (ML/d)
404203	Broken River @ Benalla	1955-1999	1916, 1921	103,000
404200	Broken River @ Casey's Weir	1889-1999		66,900
404222	Broken River @ Orrvale*	1955-1999	1916, 1921	43,500
405253	Goulburn River @ Goulburn Weir			N/A
405200	Goulburn River @ Murchison	1956-1999	1916	134,000
405204	Goulburn River @ Shepparton	1921-1999	1916	219,000
405269	Seven Creeks @ Kialla West**	1963-1995	1916	69,900

** Regression with Broken River @ Benalla using 1977-1993 data*

*** Regression with Seven Creeks @ Euroa using 1977-1996 data*

Broken River @ Benalla (404203)

Some discrepancies between DELWP and RWC data, and the flows reported in SKM (2002) were found.

- DELWP has only recorded 20 years of instantaneous flow data between 1978 and 1998, of which almost 10 years is classified as missing data.
- 10 years of mean daily flow data are recorded in the RWC Blue Book from 1955 to 1964 (RWC, 1990)No information was available on historic floods.
- The SKM analysis used a full annual series of peak flows from 1955-1998 plus historic peaks in 1916 and 1921
- The peak flow for 1993 provided by DELWP was confirmed to be the same as the flow reported in SKM (2002)
- Most of the flow data used by SKM (2002) could not be located

Goulburn River @ Murchison (405200)

Some discrepancies between DELWP and RWC data, and the flows reported in SKM (2002) were found.

- There are no records in the Blue Book from 1967-1984
- The Victorian Water Resources Data Warehouse station level and instantaneous flow data set is missing from January 1970 to June 1977 and January 1981 to November 1984
- The DELWP Instantaneous flow dataset is complete from 1881-2013
- The 1916 flow in the DELWP dataset was 311,000 ML/d, compared to 195,000 ML/d (average daily flow) in the Blue Book
- The 1974 flow in the DELWP dataset was 142,000 ML/d, compared with 111,000 in SKM (2002)
- The 1975 flow in the DELWP dataset was 411,000 ML/d, significantly larger than 1975 flows at upstream and downstream gauges, and larger than the 1916 largest event on record

A revised rating curve was developed for the high flow region of this gauge through 1D/2D hydraulic modelling for the Murchison Flood Mapping (Water Technology, 2014), see Section 4.2. With the revised rating curve applied to the DELWP dataset, the 1916 flow is estimated at 178,180 ML/d and the 1974 flow is estimated at 117,860 ML/d, which are more consistent with the data in SKM (2002). The flood frequency analysis for this gauge was undertaken with both the raw DELWP dataset and the revised rating curve dataset for comparison. Additional information from the Granite Creeks regional flood mapping study was used to provide an input for Pranjip and Castle Creek, tributaries of the Goulburn River between Murchison and the Seven Creeks outfall.

4.3.3 Broken River Flows

The Broken River inflow boundary to the detailed Shepparton-Mooroopna hydraulic model was located approximately 1.5 km upstream of the East Goulburn Main Channel. To determine design hydrology for this location, a coarse Broken River model was developed from Gowangardie to downstream of the East Goulburn Main Channel. This model determined the magnitude of flow splits leaving the Broken River, and was used to determine the ratio of flows between the downstream Broken River at Orrvale gauge and the inflow boundary to the detailed Shepparton-Mooroopna flood model. A flood frequency analysis on the Orrvale gauge was completed and flows at the inflow boundary upstream of the East Goulburn Main Channel were scaled up using the ratio determined from the coarse Broken River modelling. The inflow boundary was scaled up as it was demonstrated that breakouts occur away from the river and the East Goulburn Main Channel redirects some of the flow, reducing the flow passing the Orrvale gauge. This is discussed further in Section 5.4.1

Broken River @ Benalla (404203)

Given that there is no additional recent flow data available, and less historic data is now available compared to what was used in SKM (2002), the SKM peak flow estimates were adopted (Table 4-4). The approximate AEP for a selection of recorded floods are provided in Table 4-5.

Table 4-4 Design peak flows for Broken River @ Benalla (404203)

AEP	ARI (1 in X years)	Peak Flow (ML/d) Adopted from SKM (2002)
20%	5	30,900
10%	10	45,500
5%	20	61,600
2%	50	83,400
1%	100	103,000
0.5%	200	128,000
0.2%	500	161,000

Table 4-5 Highest recorded flows and corresponding AEP for Broken River @ Benalla (404203)

Year	Peak Flow (ML/d)	Approx. AEP
1993	112,000	1-0.5%
1981	41,400	20-10%
1956	37,700	20-10%
1996	33,400	20-10%

Broken River @ Casey Weir (404200/404216)

The Casey Weir gauge was reviewed and an initial flood frequency analysis was performed. It was found that the design flows were significantly different to that obtained in the previous SKM (2002) study, and were significantly different to the downstream Orrvale gauge. On inspection of the rating curve it was found that the maximum gauged level at 1.9 m or 17,000 ML/d was sufficiently low, that 44 years within the annual series exceeded the maximum gauging. The reliability of the rating curve was questionable, and further analysis was not completed as it added no value to the project.

Broken River @ Gowangardie (404224)

The Gowangardie gauge has a very short period of record, not enough to allow a flood frequency analysis to be performed with any degree of certainty. No further analysis was completed for this gauge.

Broken River @ Orrvale (404222)

Peak flows from 1978-2012 (35 years) were used for the annual series.

Log Pearson III and GEV distributions were fitted. 11 low flows less than 4,000 ML/d were omitted from the fitting of the distribution, as they appeared to follow a different distribution to the higher flows. The GEV distribution was judged to have the best fit (Figure 4-4). The resulting peak flow estimates are provided in Table 4-6. The resulting 1% AEP flow was broadly consistent with (but slightly higher than) the SKM (2002) estimate, which was derived from a regression relationship with Broken River at Benalla.

Under this distribution the 1993 flood has an AEP of between 2% and 1%, and the 1981, 2010 and 1996 floods have an AEP between 10% and 5% (Table 4-7).

Table 4-6 Design peak flows for Broken River @ Orrvale (404222)

AEP	ARI (1 in X years)	GEV Peak Flow (ML/d) 11 low flows censored
20%	5	17,900
10%	10	24,800
5%	20	31,600
2%	50	40,800
1%	100	48,000
0.5%	200	55,400
0.2%	500	65,600

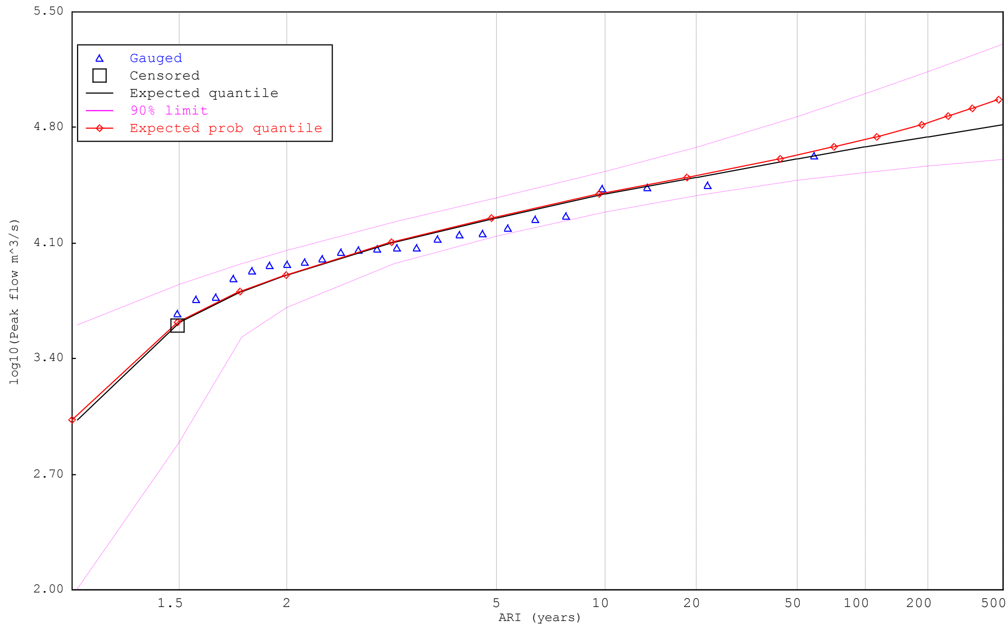

Figure 4-4 GEV distribution fitted to annual series for Broken River @ Orrvale (404222)

Table 4-7 Highest recorded flows and corresponding AEP for Broken River @ Orrvale (404222)

Year	Peak Flow (ML/d)	Approx. AEP
1993	42,900	2-1%
1981	28,300	10-5%
2010	27,300	10-5%
1996	27,100	10-5%

4.3.4 Goulburn River Flows

The Goulburn River at Murchison gauge was the focus for defining the historic and design flows for the Goulburn River. The detailed Shepparton-Mooroopna flood model boundary on the Goulburn River was located in Toolamba between the railway bridge crossing and Bridge Road at the location of the Castle Creek confluence. Historic hydrographs from Murchison were routed through the coarse Goulburn River model to the detailed Shepparton-Mooroopna flood model inflow boundary to provide an estimate of model routing time. These routing times were also applied to the design hydrographs from Murchison to the model inflow boundary. The tributary inflows from Castle Creek and Pranjip Creek were also assessed, but after an analysis of both gauges it was found the gauge rating curve for both sites had a high degree of uncertainty associated with flood flows. Given their contributions are relatively small, a simplified approach of adding a small flow contribution from the two gauges to the design event was adopted. This is discussed further in Section 5.4.1

Goulburn River @ Goulburn Weir (405253)

Instantaneous flow data was available for 1968-1969 and 1975-1979. Mean daily flow was available from 1967-1984. A regression analysis was undertaken on the coincident instantaneous flows and mean daily flows, and the relationship INSTANTANEOUS=1.0862*AVERAGE DAILY was derived ($r^2 = 0.99$). This was applied to the mean daily flow data to fill in the missing years in the instantaneous flow annual maximum series. The resulting annual series had 18 years of data from 1967-1984. This limited flow record may result in high uncertainty of peak flow estimates, particularly for large events.

The 1974 flood was the highest recorded flow at Goulburn Weir. The 1975 peak at Goulburn Weir was a much lower event (72,000 ML/d), compared to the recorded flow at Murchison in the DELWP Water Information Monitoring System database (411,000 ML/d). This indicates a possible error in one or both measurements as the flow between Goulburn Weir and Murchison is not likely to be very different. The peak at Goulburn Weir is coded as “Unedited data”. This provides further weight to the earlier discussion regarding the overestimation of flows using the current rating curve at Murchison. The revised 1975 peak flow at Murchison correcting for the revised rating curve was 223,000 ML/d, which is still significantly higher than the Goulburn Weir recorded flow, it is likely that the Goulburn Weir flow may be underestimated for this event.

The 1974 flow was exceeded at Murchison three times in the period 1881-1966, and never in the period 1985-2012. The three floods in 1916, 1934 and 1956 were included as peaks over the threshold of 121,000 ML/d (the 1974 flow at Goulburn Weir), as there is good evidence of a strong correlation between flows at Murchison and Goulburn Weir.

Log Pearson III and GEV distributions were fitted. The GEV distribution was judged to have the best fit (Figure 4-5). The resulting peak flow estimates are provided in Table 4-8. Approximate AEPs for recorded floods are provided in Table 4-9.

Table 4-8 Design peak flows for Goulburn River @ Goulburn Weir (405253)

AEP	ARI (1 in X years)	GEV Peak Flow (ML/d) 3 peaks over threshold and 111 peaks under threshold of 121,000 ML/d
20%	5	59,500
10%	10	80,000
5%	20	101,600
2%	50	132,600
1%	100	158,400
0.5%	200	186,500
0.2%	500	227,700

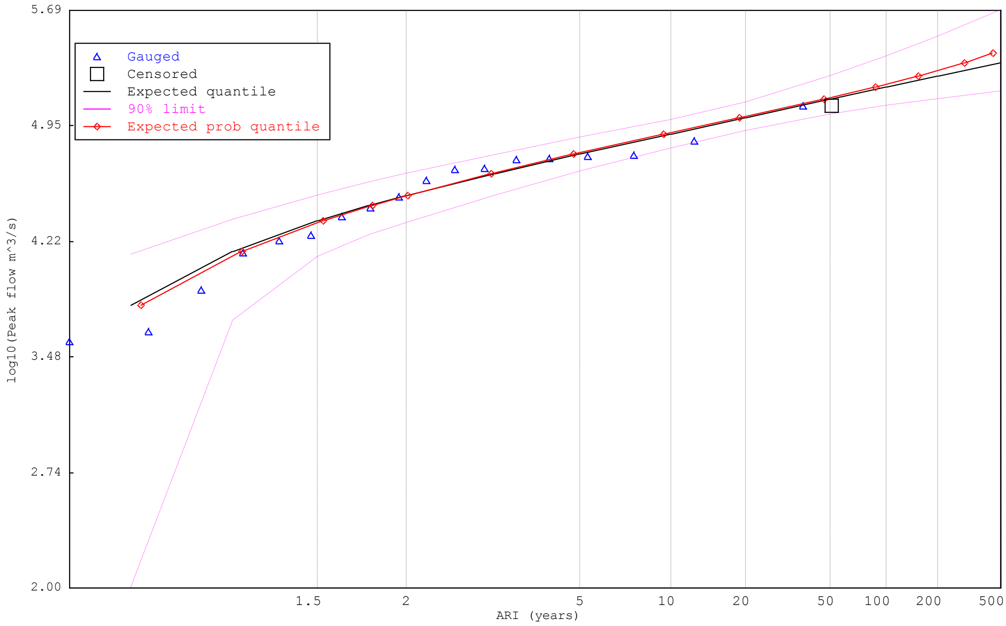

Figure 4-5 GEV distribution fitted to annual series for Goulburn River @ Goulburn Weir (405253)

Table 4-9 Highest recorded flows and corresponding AEP for Goulburn River @ Goulburn Weir (405253)

Year	Peak Flow (ML/d)	Approx. AEP
1974	120,600	5-2%
1981	59,000	20%

Goulburn River @ Murchison (405200)

SKM (2002) adopted data from 1956-1999 plus 1916 peak. There is evidence in the record that moderate flood flows were smaller after 1956 (after construction of Big Eildon dam) than before. The 1916 event was included for the following reason (SKM 2002, p. 22):

> *The rainfall spatial pattern for the 1916 event (SKP 1982) indicates significant rainfall fell downstream of Eildon. The 1916 event occurred in September, a time of year where the storage*

level in Lake Eildon is usually high. Given the size, the spatial rainfall pattern and time of year the event occurred, it is considered reasonable to assume the presence of Big Eildon, if constructed, may have had little impact on the peak flow at Murchison for the 1916 event. As a result, the peak flow for the 1916 event is included in the frequency analysis without modification.

As discussed previously in Section 4.2.3, a review of the Murchison rating curve was conducted and was found that the rating curve required revision for high levels in the extrapolated area of the curve. Flood frequency analysis was performed on the Murchison gauge data using both the raw data from the DELWP Water Information Monitoring System and with the gauge record adjusted using the recommended revised rating curve. The flood frequency analysis for both sets of analysis are provided below, but it is recommended that the revised rating curve flows be adopted for this study.

Existing Rating Curve Peak Flow Analysis

The annual series was constructed using the same period of record used by SKM (2002), post-Big Eildon Dam from 1956, plus 1916, and extended through to 2012. The analysis was also run on the entire record from 1881 to 2012 for comparison.

The 1984 maximum was missing from the gauge record and the 1975 peak flow was inconsistent with upstream and downstream gauges. The 1984 peak was excluded from the annual series. The 1975 peak was estimated from the upstream gauge at Goulburn Weir (405253). Monthly maximum flows at Murchison have a strong correlation with Goulburn Weir, following the relationship MURCHISON=0.8585*GOULBURN WEIR ($R^2 = 0.98$). A 1975 peak flow of 62,200 ML/d was adopted using this relationship.

The GEV distribution was adopted following initial trials of GEV and Log Pearson III. Low flows below 6,000 ML/d were considered “non-flood” years and excluded from the distribution fitting. There were 8 of these non-flood years over the 1956-2012 period and a further two over the pre-1956 period.

The adopted distribution is shown in Figure 4-6 and Figure 4-7, with the resulting peak flow estimates provided in Table 4-10.

Approximate AEPs for recorded floods are given in Table 4-11. Upon review of the results it was noted that the full record period produced peak flow estimates that were higher for events between 20% and 5% AEP, and lower for the larger events as compared to the post-Big Eildon record FFA. This may be explained by the lower range of peak annual flows being slightly higher pre 1956 due to the reduced storage of Eildon and the lack of any large floods above 100,000 ML/d in the 35 year period between 1881 to 1915. The two time periods have been combined, adopting the post-dam period for events up to the 1% AEP and for the rare 0.5% and 0.2% AEP events the full period of record was adopted. The rationale behind this decision is that in the rare events the impact of the dam would be minimal and the full record can be used in the annual series.

Table 4-10 Design peak flows for Goulburn River @ Murchison (405200), existing rating curve

AEP	ARI (1 in X years)	GEV Peak Flow (ML/d) Post-Big Eildon Record 1956-2012 plus 1916 8 low flows censored, 74 flows below 1916 threshold censored	GEV Peak Flow (ML/d) Entire Record 1881-2012 10 low flows censored	Adopted Peak Flow (ML/d)
20%	5	49,900	59,800	49,900
10%	10	74,700	82,800	74,700
5%	20	105,500	108,500	105,500
2%	50	158,400	147,800	158,400
1%	100	210,800	182,700	210,800
0.5%	200	277,100	222,900	222,900
0.2%	500	392,800	285,400	285,400

Table 4-11 Highest recorded flows and corresponding AEP for Goulburn River @ Murchison (405200), existing rating curve

Year	Peak Flow (ML/d)	Approx. AEP
1916	311,000	<0.2%
1956	154,000	5-2%
1974	142,000	5-2%
1993	80,200	10-5%

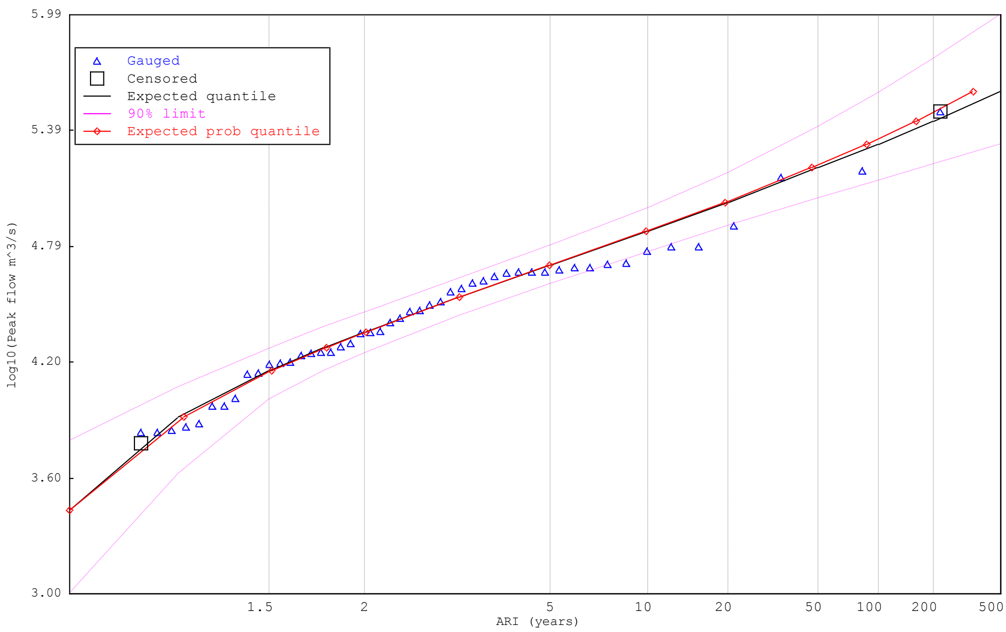

Figure 4-6 GEV distribution fitted to annual series for Goulburn River @ Murchison (405200), DELWP rating curve, post-Big Eildon record plus 1916

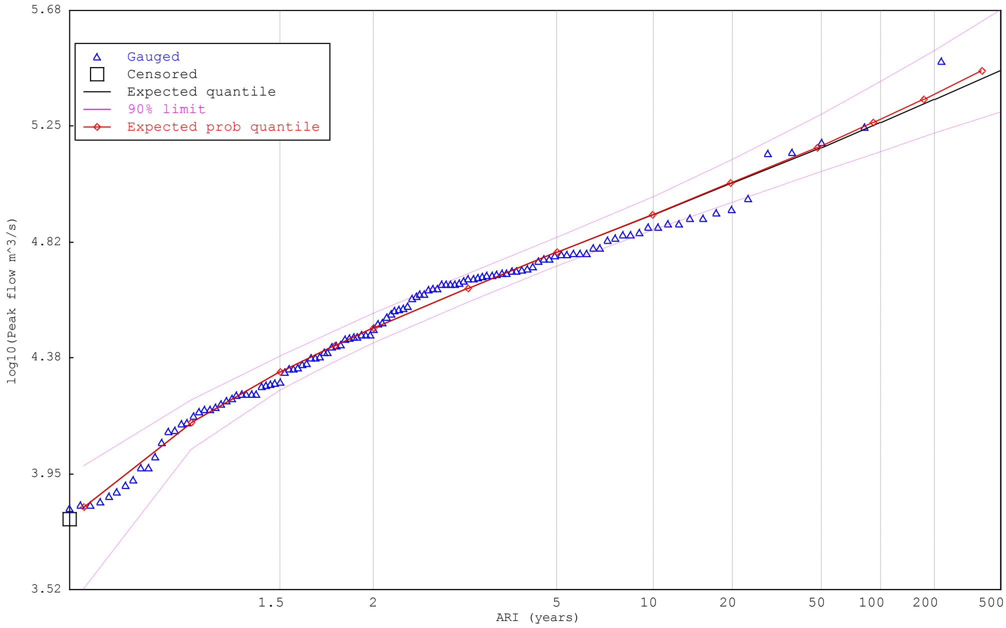

Figure 4-7 GEV distribution fitted to annual series for Goulburn River @ Murchison (405200), DELWP rating curve, entire record

Revised Rating Curve Peak Flow Analysis

The annual maximum flow series was constructed utilising the revised rating curve discussed in Section 4.2.3 for levels in the extrapolated region of the rating curve. The time periods, peaks over threshold and low flow censoring treatment was the same as the FFA for the existing rating curve analysis described above.

The adopted distribution is shown in Figure 4-8 and Figure 4-9, with the resulting peak flow estimates provided in Table 4-12. Approximate AEPs for recorded floods are provided in Table 4-13.

Similarly to the existing rating curve FFA, the full record period produced peak flow estimates that were lower than or equal to the post-Big Eildon record FFA for 2% AEP flows and above. The two time periods were again combined, adopting the post-dam period for events up to the 1% AEP and the full period for the rare 0.5% and 0.2% AEP events.

As seen in Table 4-10 and Table 4-12, using the revised rating curve to construct the annual series significantly reduces the design flows at Murchison for events greater in magnitude than a 10% AEP. It has been demonstrated that the revised rating curve is a better representation of the stage-flow relationship for larger events than the current rating curve, fitting with the regional hydrology upstream and downstream, and the understanding of historic flood flows in the Goulburn River. At the time of finalising this report it is understood that Ventia will be updating the rating curve for this location using the modelled rating curve in the extrapolated region of the curve. Table 4-12 was adopted for design flows at Murchison for the purposes of this study.

Table 4-12 Design peak flows for Goulburn River @ Murchison (405200), revised rating curve data

AEP	ARI (1 in X years)	GEV Peak Flow (ML/d) Post-Big Eildon Record 1956-2012 plus 1916 8 low flows censored, 74 flows below 1916 threshold censored	GEV Peak Flow (ML/d) Entire Record 1881-2012 10 low flows censored	Adopted Peak Flow (ML/d)
20%	5	49,100	59,700	49,100
10%	10	69,000	78,600	69,000
5%	20	90,900	97,700	90,900
2%	50	123,900	123,900	123,900
1%	100	152,600	144,700	152,600
0.5%	200	185,200	166,500	166,500
0.2%	500	235,200	196,900	196,900

Table 4-13 Highest recorded flows and corresponding AEP for Goulburn River @ Murchison (405200), revised rating curve data

Year	Peak Flow (ML/d)	Approx. AEP
1916	178,200	0.5-0.2%
1956	123,200	2%
1974	117,900	5-2%
1993	80,000	10-5%

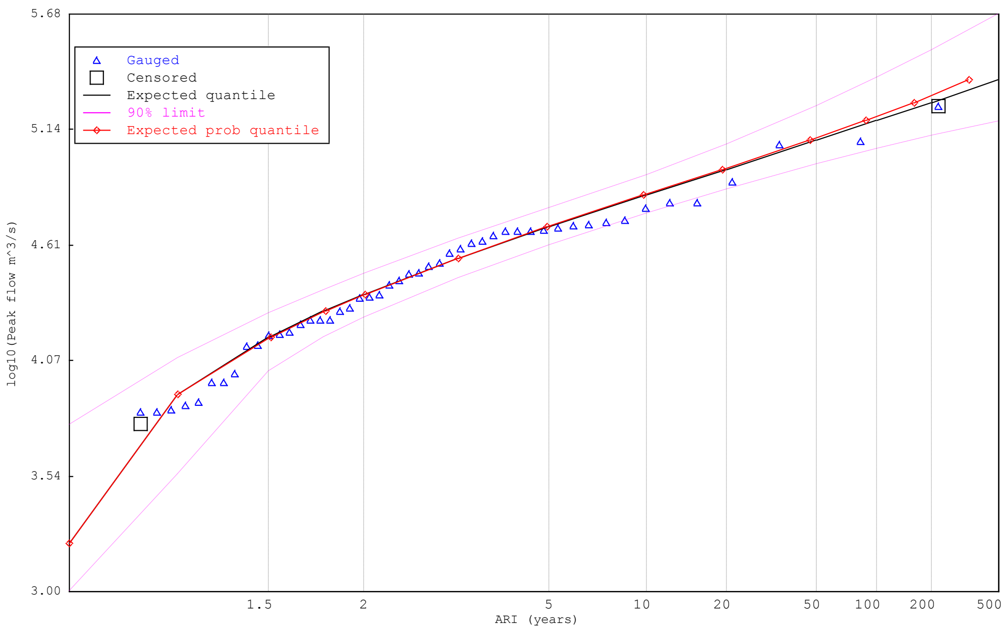

Figure 4-8 GEV distribution fitted to annual series for Goulburn River @ Murchison (405200), revised rating curve data, post-Big Eildon record plus 1916

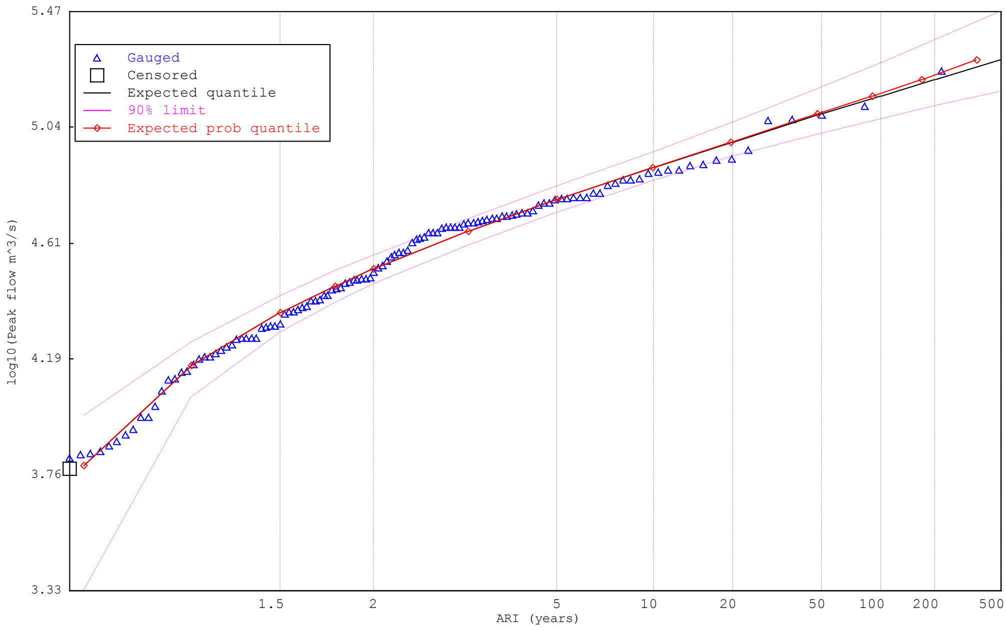

Figure 4-9 **GEV distribution fitted to annual series for Goulburn River @ Murchison (405200), revised rating curve data, entire record**

Volume Analysis

A flood frequency analysis on 5 day volume was undertaken using the revised rating curve. The Log Pearson III distribution was found to provide a much better fit than the GEV distribution, the Log Pearson III distribution was adopted. Similar to the flood frequency on peak flow the 1975 and 1984 events were excluded from the analysis due to missing data. Unlike the flood frequency analysis on peak flow, the analysis was performed on the entire period of record. Inconsistencies in the volumes arrived if the entire record analysis was combined with the post Big Eildon analysis. The analysis on the entire record had much tighter confidence intervals and is thought to provide reasonable results (Figure 4-10). The resulting five day volume estimates are provided in Table 4-14.

Table 4-14 **Goulburn River @ Murchison 5 day volume**

AEP	ARI (1 in X years)	LPIII 5 day volume (ML)
20%	5	244,500
10%	10	312,100
5%	20	375,000
2%	50	452,600
1%	100	507,500
0.5%	200	559,600
0.2%	500	624,400

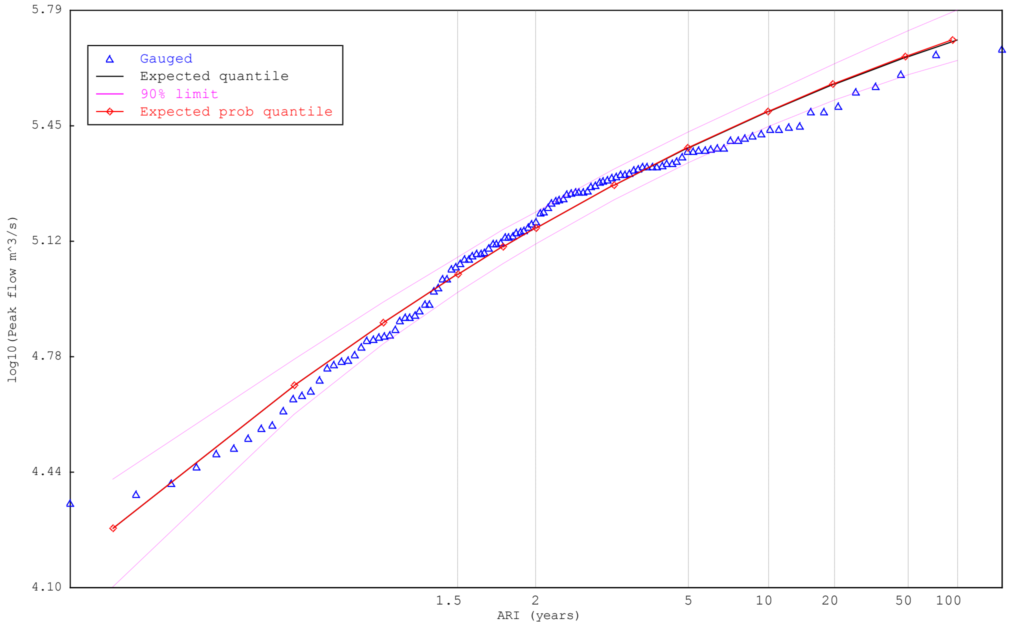

Figure 4-10 LPIII distribution fitted to annual series of 5 day volumes for Goulburn River @ Murchison 405200 (Source: DELWP)

Goulburn River @ Shepparton (405204)

Instantaneous flow data was available for 1941-1968 and 1974-2012. Mean daily flow was available from 1921-2012. A regression analysis was undertaken on the coincident instantaneous flows and mean daily flows, and the relationship INSTANTANEOUS=1.071*AVERAGE DAILY was derived (r^2 = 0.99). This was applied to the mean daily flow data to fill in the missing years in the instantaneous flow annual maximum series. The resulting annual series had 92 years of data from 1921-2012.

The 1974 peak flow was revised down in the latest DELWP data to 191,000 ML/d (from 214,000 ML/d in the SKM study in 2002) and the 1993 peak flow to 150,000 ML/d in the latest DELWP data (from 160,500 ML/d in the SKM study in 2002). It is presumed this revision in flow is due to changes in the rating curves applied back to the historic record.

An estimate of the 1916 peak of 233,300 ML/d from SKM (2002) was used as a historic peak. This is lower than the previous estimate of the 1916 flood of 267,000 ML/d in the Shepparton Flood Study by Sinclair Knight and Partners (SKP 1982). Again, it is presumed that this reduction in the historic peak flow is due to a change in the upper end of the rating curve applied back to the historic flow series.

Log Pearson III and GEV distributions were fitted. 15 low flows less than 10,000 ML/d were omitted from the fitting of the distribution, as they appeared to follow a different distribution to the higher flows. The GEV distribution was judged to have the best fit (Figure 4-11). The resulting peak flow estimates are provided in Table 4-15. Approximate AEPs for recorded floods are provided in Table 4-16.

Table 4-15 Design peak flows for Goulburn River @ Shepparton (405204)

AEP	ARI (1 in X years)	GEV Peak Flow (ML/d) 15 low flows censored, 39 flows below 1916 threshold censored
20%	5	70,000
10%	10	97,800
5%	20	128,200
2%	50	173,800
1%	100	213,200
0.5%	200	257,800
0.2%	500	325,700

Table 4-16 Highest recorded flows and corresponding AEP for Goulburn River @ Shepparton (405204)

Year	Peak Flow (ML/d)	Approx. AEP
1916	233,300	1-0.5%
1974	191,000	2-1%
1939	161,000	5-2%
1993	150,000	5-2%
1956	121,000	10-5%

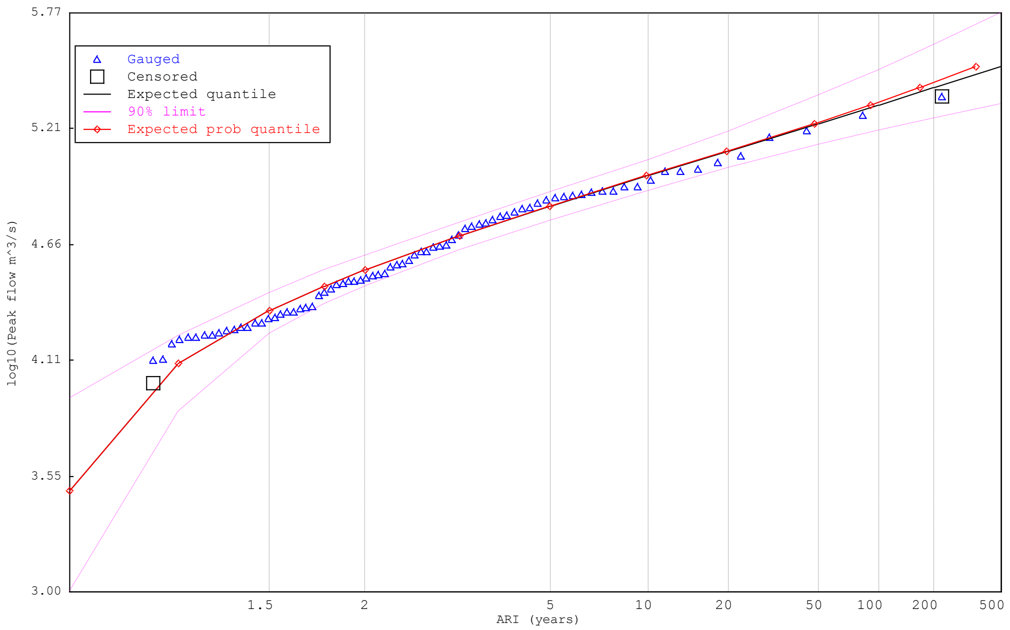

Figure 4-11 GEV distribution fitted to annual series for Goulburn River @ Shepparton (405204)

Pranjip Creek @ Moorilim (405226)

Peak Flow Analysis

Instantaneous flow data was available for 1974-2013. Monthly maximum instantaneous flow was available in the RWC Blue Book for 1965-1986 and monthly maximum mean daily flow was available for 1958-1986 (RWC, 1990). A regression analysis was undertaken on the coincident maximum annual instantaneous flows and mean daily flows, and the relationship INSTANTANEOUS = 1.125*AVERAGE DAILY was derived. This was applied to the mean daily flow data to fill in the missing years in the instantaneous flow annual maximum series. The resulting annual series had 56 years of data from 1958-2013.

Log Pearson III and GEV distributions were fitted. Nine low flow outliers were detected using the multiple Grubbs Beck test and were censored. The LPIII distribution was judged to have the best fit (Figure 4-12). The resulting peak flow estimates are provided in Table 4-17. The AEP of the highest recorded flood events is provided in Table 4-18.

Table 4-17 Design peak flows for Pranjip Creek @ Moorilim (405226)

AEP	ARI	LPIII Peak Flow (ML/d)
20%	5	6,200
10%	10	6,400
5%	20	12,800
2%	50	17,200
1%	100	20,400
0.5%	200	23,500
0.2%	500	27,400

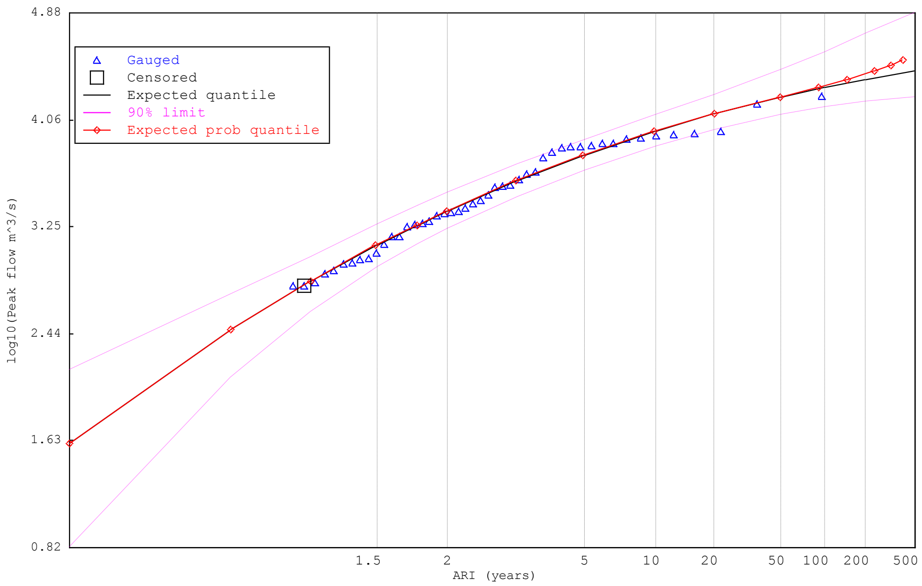

Figure 4-12 Log Pearson III distribution fitted to annual series for Pranjip Creek @ Moorilim (405226)

Table 4-18 Highest recorded flows and corresponding AEP for Pranjip Creek @ Moorilim (405226)

Year	Peak Flow (ML/d)	Approx. AEP
1974	17,444	2%
1993	15,209	5-2%
1973	9,410	10-5%

The Pranjip Creek @ Moorilim gauge rating curve is coded as extrapolated for flows above 7,400 ML/d, this equates to an event slightly larger than a 10% AEP event. The gauge is located at a siphon on the East Goulburn Main Channel, where the channel runs under Pranjip Creek. Immediately downstream the creek is crossed by the Goulburn Valley Highway. It is likely that at high flows, these structures have an impact on the hydraulics of the floodplain, and that without high flow gauging there is likely to be significant uncertainty in the recorded flows and thus the flood frequency analysis on peak flow. The Granite Creeks Regional Flood Mapping Study (Water Technology, ongoing at time of writing this report), has shown that the East Goulburn Main Channel does impact on flood flows, at Pranjip Creek.

Volume Analysis

A flood frequency analysis of three day volumes was undertaken for the period from 1958-2013, using mean daily flows for 1958-1973 and instantaneous flows for 1974-2013.

Log Pearson III and GEV distributions were fitted. 22 low outliers were detected using the multiple Grubbs Beck test, and were censored. The GEV distribution was judged to have the best fit (Figure 4-13). The resulting three day volume estimates are provided in Table 4-19.

Table 4-19 Design 3 day volumes for Pranjip Creek @ Moorilim (405226)

AEP	ARI (1 in X years)	GEV 3 Day Volume (ML)
20%	5	14,700
10%	10	20,100
5%	20	25,200
2%	50	31,600
1%	100	36,200
0.5%	200	40,600
0.2%	500	46,200

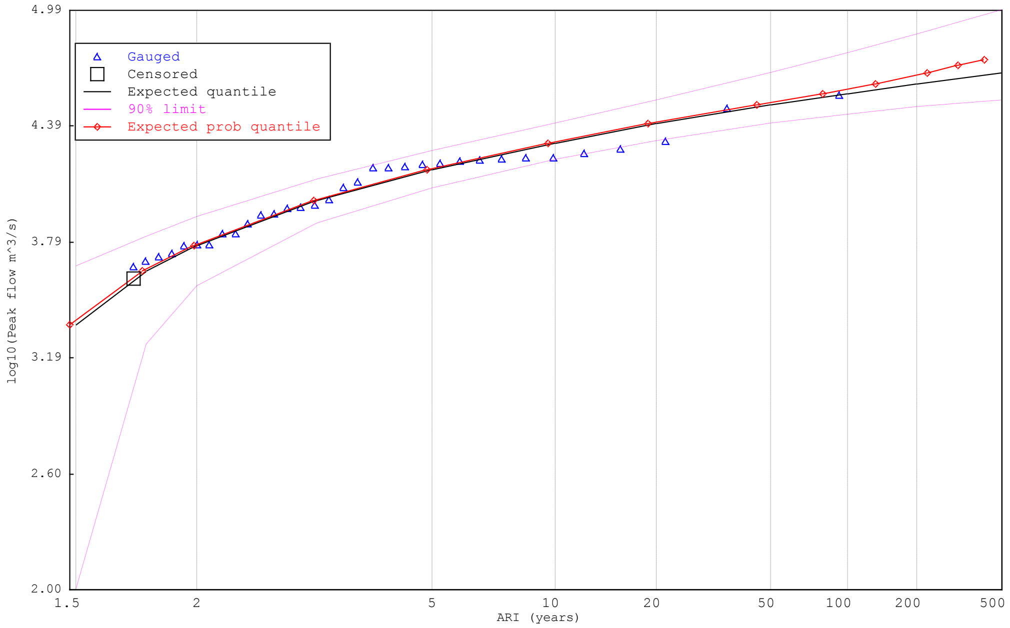

Figure 4-13 **GEV distribution fitted to annual series of 3 day volumes for Pranjip Creek @ Moorilim (405226)**

Castle Creek @ Arcadia (405246)

Peak Flow Analysis

Instantaneous flow data was available for 1974-2013. Monthly maximum instantaneous flow was available in the RWC Blue Book for 1970-1986 (RWC, 1990). The resulting annual series had 42 years of data from 1970-2013, with two years of data missing in 1989 and 1990. These two years were excluded completely from the series (i.e. the series length was shortened by two years).

Log Pearson III and GEV distributions were fitted. 21 low outliers were detected using the multiple Grubbs Beck test. This was not thought to be reasonable as it left a very small sample size in the annual series. The number of flows censored was reduced to 12 after inspection of the annual series. The GEV distribution was judged to have the best fit (Figure 4-14). The resulting peak flow estimates are provided in Table 4-20.

Table 4-20 **Design peak flows for Castle Creek @ Arcadia (405246)**

AEP	ARI (1 in X years)	GEV Peak Flow (ML/d)
20%	5	2,400
10%	10	3,200
5%	20	4,000
2%	50	5,000
1%	100	5,700
0.5%	200	6,400
0.2%	500	7,300

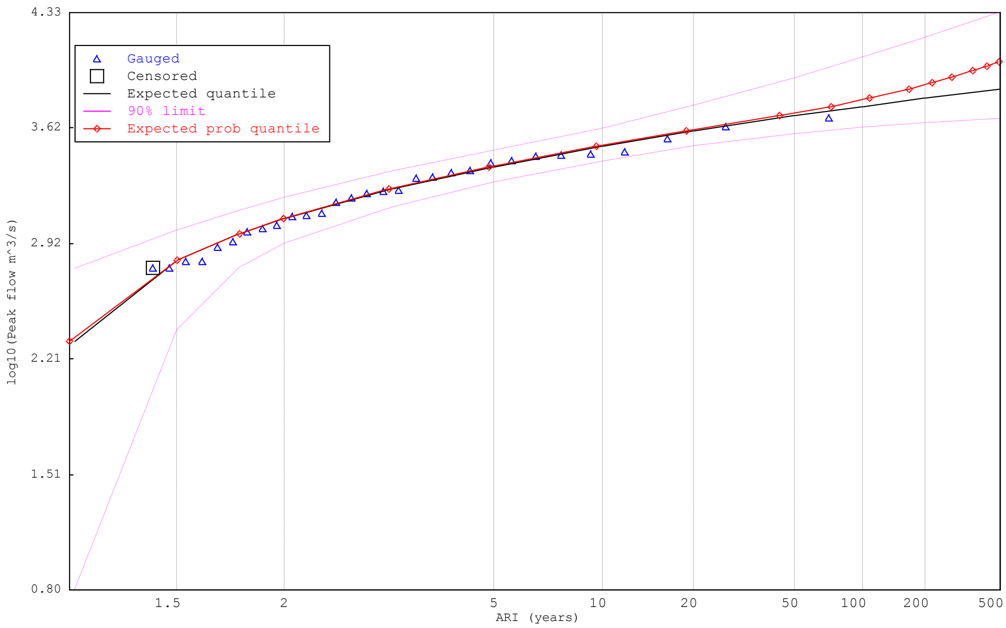

Figure 4-14 GEV distribution fitted to annual series for Castle Creek @ Arcadia (405246)

Table 4-21 Highest recorded flows and corresponding AEP for Castle Creek @ Arcadia (405246)

Year	Peak Flow (ML/d)	Approx. AEP
1993	4,835	2%
1974	4,264	5%
2012	3,606	10-5%
2010*	3,034	10%

* note that the peak flow recorded in 2010 occurred in December. This report uses the September 2010 flood event as a calibration event.

The Castle Creek @ Arcadia gauge rating curve is coded as extrapolated for flows above 2,410 ML/d, this equates to a 20% AEP event. The gauge is located at a siphon on the East Goulburn Main Channel, where the channel runs under Castle Creek. It is likely that at high flows, the channel has an impact on the hydraulics of the floodplain, and that without high flow gauging there is likely to be significant uncertainty in the recorded flows and thus the flood frequency analysis on peak flow. The Granite Creeks Regional Flood Mapping Study (Water Technology, ongoing at time of writing this report), has shown that the East Goulburn Main Channel does impact on flood flows, at Castle Creek and Pranjip Creek.

Volume Analysis

A flood frequency analysis of three day volumes was undertaken for the period from 1970-2013, using mean daily flows for 1970-1973 and instantaneous flows for 1974-2013. The resulting annual series had two years of missing data in 1989 and 1990; these years were excluded completely from the series.

Log Pearson III and GEV distributions were fitted. 19 low outliers were detected using the multiple Grubbs Beck test. This was not thought to be reasonable due to the small sample size remaining, and the number of flows censored was reduced to 11, after inspection of the annual series. The Log

Pearson III distribution was judged to have the best fit (Figure 4-15). The resulting three-day volume estimates are provided in Table 4-22.

Table 4-22 Design 3 day volumes for Castle Creek @ Arcadia (405246)

AEP	ARI (1 in X years)	LPIII 3 Day Volume (ML)
20%	5	4,100
10%	10	5,700
5%	20	7,200
2%	50	8,900
1%	100	10,000
0.5%	200	11,000
0.2%	500	12,100

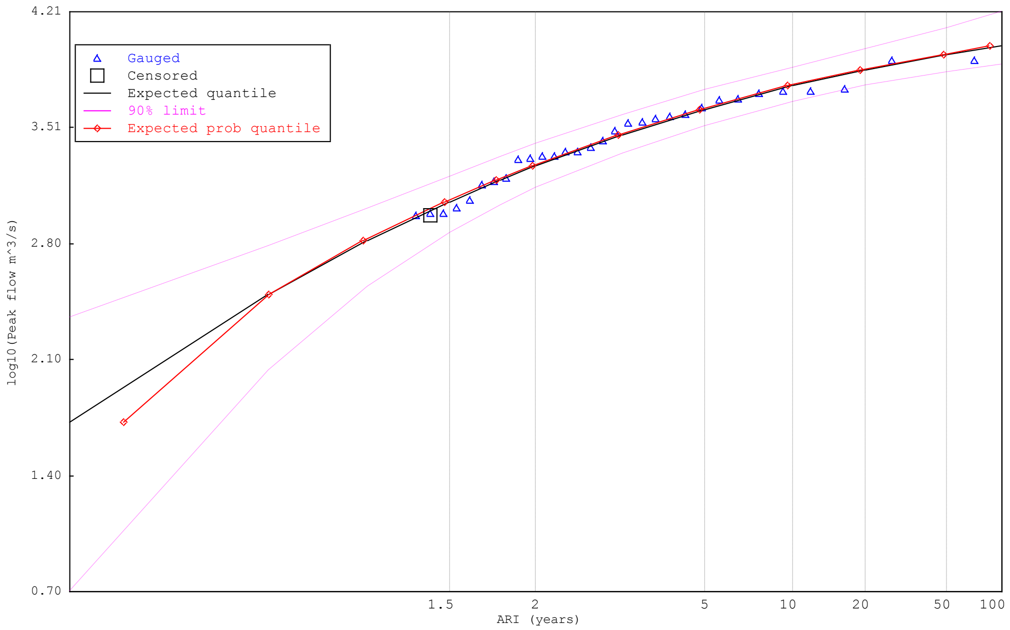

Figure 4-15 LPIII distribution fitted to annual series of 3 day volumes for Castle Creek @ Arcadia (405246)

4.3.5 Seven Creeks Flows

The Seven Creeks system has a large catchment area with Seven Creeks and Honeysuckle Creek combining 1.8 km upstream of the Seven Creeks @ Kialla West gauge. The inflow boundaries to the detailed Shepparton-Mooroopna hydraulic model are further upstream on both these tributaries. The Honeysuckle Creek inflow boundary is located upstream of the Shepparton-Euroa Road, and the Seven Creeks inflow boundaries are split on the two anabranches of the creek upstream of Union Road.

To develop historic and design flows for Seven Creeks and Honeysuckle Creek, the Seven Creeks at Kialla West gauge was used as a combined flow, which was then split evenly between the two tributaries. The even split was based on the catchment areas which were roughly the same. This split was later verified through hydraulic model calibration.

In order to develop the combined flow a regression analysis was used with upstream gauges in both catchments to extend the estimated streamflow record for the Seven Creeks at Kialla West streamflow gauge. Without this regression analysis the period of record was too short to complete a flood frequency analysis. This catchment has significant cross-catchment flows making hydrological catchment modelling difficult, necessitating the flood frequency approach.

Seven Creeks @ Kialla West (405269)

The quality of the DELWP data is questionable up to 2003, as peak flows appear to be missing. There is only 10 years of data available between 2003-2012, this does not constitute sufficient record for undertaking a flood frequency analysis. Regressions with two upstream gauges (Seven Creeks @ Euroa and Stony Creek @ Tamleugh) were developed to infill the years between 1977-2003 to extend the Seven Creeks @ Kialla West annual series. The gauge Stony Creek @ Tamleugh is named incorrectly, it is on Honeysuckle Creek downstream of the confluence with Stony Creek. This was raised with the Regional Water Monitoring Partnership during the Granite Creeks Regional Flood Mapping Study, and it is recommended that the name be changed to avoid confusion in the future.

The regression was undertaken between monthly maximum flows at each gauge from 1977-2013. As the data from the three gauges had significant periods of missing data there was a very limited period where all three gauges overlapped. This meant that a multiple regression relationship could not be established, instead a linear relationship was established between Seven Creeks @ Kialla West and each of the two upstream gauges. The maximum of the two regression equations was then used to infill the annual series for the Seven Creeks @ Kialla West gauge. The following relationships were produced: KIALLA WEST=2.20*EUROA (r^2 = 0.83) and KIALLA WEST = 2.613 * TAMLEUGH (r^2 = 0.88).

A flood frequency analysis on the extended gauge record was then undertaken and fitted using Log Pearson III and GEV distributions. The Log Pearson III distribution was judged to have the best fit and is shown in Figure 4-16. The resulting peak flow estimates are provided in Table 4-23. Approximate AEPs for the three flood events that are to be calibrated are provided in Table 4-24.

The resultant design flows in this analysis are slightly higher than those estimated in the SKM (2002) study. They are still relatively similar, and are thought to provide reasonable design estimates. The flow values calculated from the flood frequency analysis are to be placed several kilometres upstream of the Seven Creeks @ Kialla West (405269) streamflow gauge on the Seven Creeks and Honeysuckle System. It is likely that some attenuation may occur between the inflow locations and the streamflow gauge.

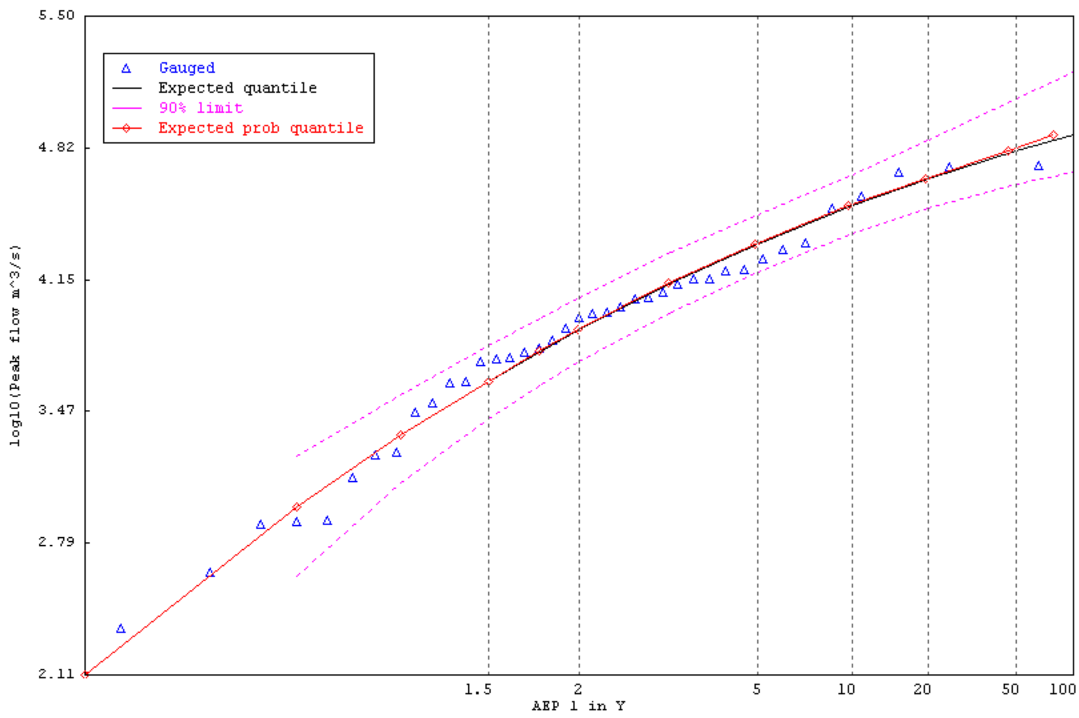

Figure 4-16 **Log Pearson III distribution fitted to annual series for Seven Creeks @ Kialla West (405269)**

Table 4-23 **Design peak flows for Seven Creeks @ Kialla West (405269)**

AEP	ARI (1 in X years)	SKM (2002) Method Peak Flow (ML/d)	Updated Method Peak Flow (ML/d)
20%	5	17,000	21,400
10%	10	27,100	33,400
5%	20	38,700	46,300
2%	50	56,600	64,100
1%	100	72,300	77,700
0.5%	200	89,600	91,200
0.2%	500	115,000	108,703

Table 4-24 **Highest recorded flows and corresponding AEP for Seven Creeks @ Kialla West (405269)**

Year	Peak Flow (ML/d)	Approx. AEP
1993	62,000*	2%
1974	50,000**	5-2%
2010	20,500	20%

*Estimated based on Shepparton-Mooroopna Flood Study (SKM, 2002)

**Estimated based on Regression Equation with Seven Creeks at Euroa streamflow gauge

4.3.6 Comparison with SKM (2002)

Comparisons between the current 1% AEP peak flow estimates at streamflow gauges throughout the catchment with the SKM (2002) estimates are provided in Table 4-25. Most of the estimates were broadly consistent. The estimates for Goulburn River @ Murchison diverge, but using the updated rating curve at Murchison, the results are more consistent with the SKM (2002) estimate.

Table 4-25 Comparison of updated design peak flows with SKM (2002) estimates

Gauge	1% AEP Flow (Updated FFA)	1% AEP Flow (SKM 2002)
Broken River @ Casey's Weir	^	66,900
Broken River @ Orrvale*	48,000	43,500
Goulburn River @ Goulburn Weir	158,400	-
Goulburn River @ Murchison*	152,600	134,000
Goulburn River @ Shepparton	213,200	219,000
Seven Creeks @ Kialla West	77,700	69,900

* *Updated FFA estimate using revised rating curve from hydraulic modelling*
^ *1% Flow at Casey's Weir not reliable due to poor rating curve*

4.4 Design Flow Hydrographs

To determine a design hydrograph the SKM (2002) study scaled historic hydrographs to represent the design peak flow and 5 day volume. The 1974 hydrograph was adopted for the Goulburn River and the 1993 hydrographs for the Broken River and Seven Creeks. The design hydrographs were scaled and lagged from the upstream gauges to the model boundary.

The major difference between the hydraulic model inflow hydrographs of this study and that of the SKM (2002) study is that in this study coarse hydraulic models were developed to route flows from the upstream gauges to the model boundaries of the detailed Shepparton-Mooroopna flood model. This allowed a more accurate lag time to be applied to the historic and design hydrographs developed at gauges and transferred to the model boundaries. It also allowed for an improved understanding of breakout flows and the impact of structures such as the East Goulburn Main Channel. Another difference was when considering the volume on the Goulburn River a 5 day volume was considered but on the Broken River and on the smaller tributaries 3 day volume was considered as the large historic hydrographs all showed a duration closer to 3 days than 5 days.

4.4.1 Previous Approach

The timing of the three major contributing catchments has a large impact on the resulting flood at Shepparton. The SKM (2002) study found that the peak flow of Seven Creeks at Kialla West generally occurs between 6-24 hours earlier than the Broken River at Orrvale, the study adopted the median 15 hour time offset for the peak flow for design purposes. The relative timing of the Goulburn and Broken Rivers was also investigated, however a lack of data did hinder this assessment. A lag time of 33 hours was assumed between Goulburn Weir and Kialla West, 30 hours between Murchison and Kialla West. It was estimated that the peak flow in the Goulburn at Kialla West occurred approximately 15 hours after the peak flow on the Broken River at Orrvale for the 1974 event, with a 60 hour lag in the 1993 event. This longer lag in 1993 was attributed to the impact of Eildon attenuating the flood in the upper catchment, with the lower catchment having a smaller contribution to the Goulburn flows. For design purposes the 15 hour time lag from the 1974 event was adopted. A number of design flood scenarios were developed using various combinations of Goulburn River, Broken River and Seven Creeks flows for a given AEP event at the Shepparton gauge.

4.4.2 Current Approach

Coarse hydraulic models were developed for the Goulburn River from Murchison to downstream of Toolamba, and on the Broken River from upstream of Gowangardie streamflow gauge to downstream of the East Goulburn Main Channel. The detailed Shepparton-Mooroopna model had hydraulic model boundaries located at Toolamba on the Goulburn River at the confluence with Castle Creek, upstream of the East Goulburn Main Channel on the Broken River, a Broken River breakout flow boundary 2.5km south of the Broken River, upstream of Shepparton-Euroa Road on Honeysuckle Creek, and upstream of Union Road on the two Seven Creeks anabranches.

Using results from the coarse hydraulic modelling for a range of flows, the hydrographs developed for Seven Creeks at Kialla West, Broken River at Orrvale and Goulburn River at Murchison were scaled and lagged to allow for the floodplain characteristics between the gauge locations and the inflow boundary locations. For design purposes, typical relative hydrograph timing was applied to represent a likely design event scenario. It must be noted that every flood is different, and the subtleties in tributary timing may result in differences in the resulting flood levels at Shepparton. This has been tested through this study during sensitivity analysis and is discussed further in Section 4.4.3

It is accepted that various combinations of hydrograph peak flows, volume, shape and timing with tributaries will lead to significant differences in flood level throughout the study area. The hydrology documented above has estimated various design peak flows and volumes for all modelled tributaries. The combination of these inputs will be discussed further below in Section 6, with the timing of hydrographs summarised below.

Similar to the earlier SKM (2002) study, for design events the 1974 hydrograph shape was scaled for the Goulburn River, and the 1993 hydrographs scaled for Broken River and Seven Creeks.

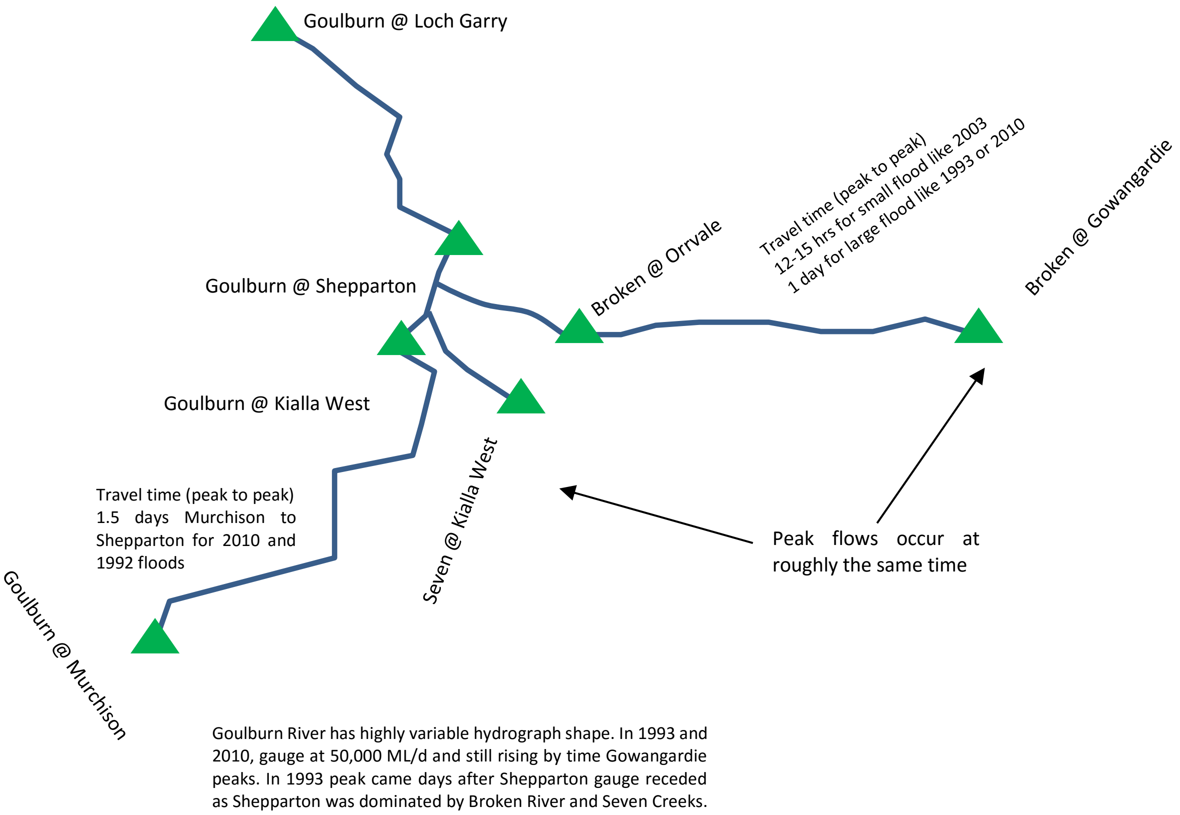

Figure 4-17 Analysis of Historic Floods for Tributary Timing

The final design modelling adopted tributary timing as follows.

- Seven Creeks model inflow peaks first
- Broken River model inflow peaks 26 hours after Seven Creeks model inflow
- Goulburn River model inflow peaks 10 hours after Broken River model inflow
- Broken River breakout model inflow peaks 24 hours after the Broken River model inflow

The above timings are based on an analysis of historic events routed through the coarse upper floodplain models from Murchison on the Goulburn River and Gowangardie on the Broken River. The timings of the peaks are different to that of the previous SKM study because of the new information from the coarse floodplain models and the change in inflow boundary locations. Consensus was reached on this approach with Greater Shepparton City Council and Goulburn Broken CMA.

A matrix of design hydrograph combinations are currently being modelled in the hydraulic model. The aim is to provide increments at thirteen different water level heights at the Goulburn River at Shepparton gauge. The approximate water level heights are 9.5 (minor flood level), 10.1, 10.5, 10.7 (moderate flood level), 10.9, 11.0 (major flood level), 11.1, 11.3, 11.5, 11.7, 11.9, 12.1, 12.2, 12.3 and 12.5 m. These incremental water level heights at the Goulburn River at Shepparton gauge will be produced by one of the three tributary flow combinations, assuming the Goulburn River is dominant, the Broken River/Seven Creeks is dominant, and a neutral or no dominance scenario. This will produce 15 suites of flood maps per flow dominance scenario including maximum depth, velocity, water surface and flood hazard mapping. This will produce 180 mapping outputs in total. An additional Probable Maximum Flood (PMF) suite of flood maps will also be produced.

The below matrix will be compiled somewhat iteratively based on the model results as they come through. Flows will be adjusted to result in the target Goulburn River at Shepparton gauge level.

Table 4-26 Flood Mapping Tributary Flow Combination Matrix (indicative only, will be revised)

Event	Goulburn River @ Shepparton Gauge Level (m)	Goulburn Dominant			Broken/Seven Dominant			Neutral		
		Goulburn River Flow	Broken River Flow	Seven Creeks Flow	Goulburn River Flow	Broken River Flow	Seven Creeks Flow	Goulburn River Flow	Broken River Flow	Seven Creeks Flow
Minor Flood	9.5	1EY	2EY	2EY	1EY	50%	50%	1EY	1EY	1EY
	10.1	50%	1EY	1EY	1EY	5%	2%	1EY	50%	1EY
	10.5	50-20%	50%	1EY	50%	50-20%	50%	50-20%	1EY	50-20%
Moderate Flood	10.7	50-20%	20%	1EY-50%	50%	20%	50-20%	50-20%	50-20%	20%
	10.9	20-10%	20%	1EY-50%	50%	10%	20-10%	20%	20%	20%
Major Flood (2010)	11.0	20-10%	20%	50%	50%	5%	10%	20-10%	20-10%	20%
	11.1	10%	10%	50%	50-20%	5%	10%	10%	10%	20%
	11.3	10-5%	10%	50-20%	20%	5-2%	5%	10%	10%	10%
	11.5	5%	5%	20%	20%	2%	5%	10-5%	10-5%	10-5%
(1993)	11.7	5-2%	5%	20-10%	10%	2%	5%	5%	10-5%	10-5%
	11.9	2%	5%	10%	5-2%	2%	5-2%	5%	5-2%	5%
(1974)	12.1	2-1%	5-2%	10-5%	5%	1%	2%	2%	1%	5%
	12.2	1%	5- 2%	10-5%	2%	1%	1%	2-1%	2-1%	2-1%
	12.3	0.5%	1%	1%	1%	0.5%	0.5%	1%	1%	1%
	12.5	0.2%	1%	1%	1%	0.2%	0.2%	0.5%	0.5%	0.5%
	PMF	1,330,000 (ML/D)	388,000 (ML/D)	622,000 (ML/D)	Note that Broken/Seven dominant may not be able to achieve some of the higher gauge levels at Shepparton, which are driven by Goulburn floods. This table will be revised during the hydraulic stage.					

4.4.3 Timing Sensitivity Analysis

Two preliminary design events were modelled using the timing methodology mentioned above. A further sensitivity analysis of the timing of the three peak inflows entering the model was undertaken to assess the water level differences experienced downstream of the confluences of the Goulburn River with the Seven Creeks and Broken River. While it is unlikely that a flood occurring in Shepparton would have the peak flow from the three river systems combining at the same time, it is important to assess the impact that a combination of this nature can have.

A comparison of the Goulburn River dominant and Broken River/ Seven Creeks dominant flood events with the adopted design hydrograph timing against the adopted design hydrographs phased so the peaks at the inflow locations align is shown in Figure 4-18. The impact at the Shepparton gauge for the aligned peak events is shown in Table 4-27. Note that the tributary inflows could be phased so that the peaks at Shepparton aligned more closely and the impacts on level at the Shepparton gauge may be more pronounced than the scenario presented. This will be investigated further during the modelling stage.

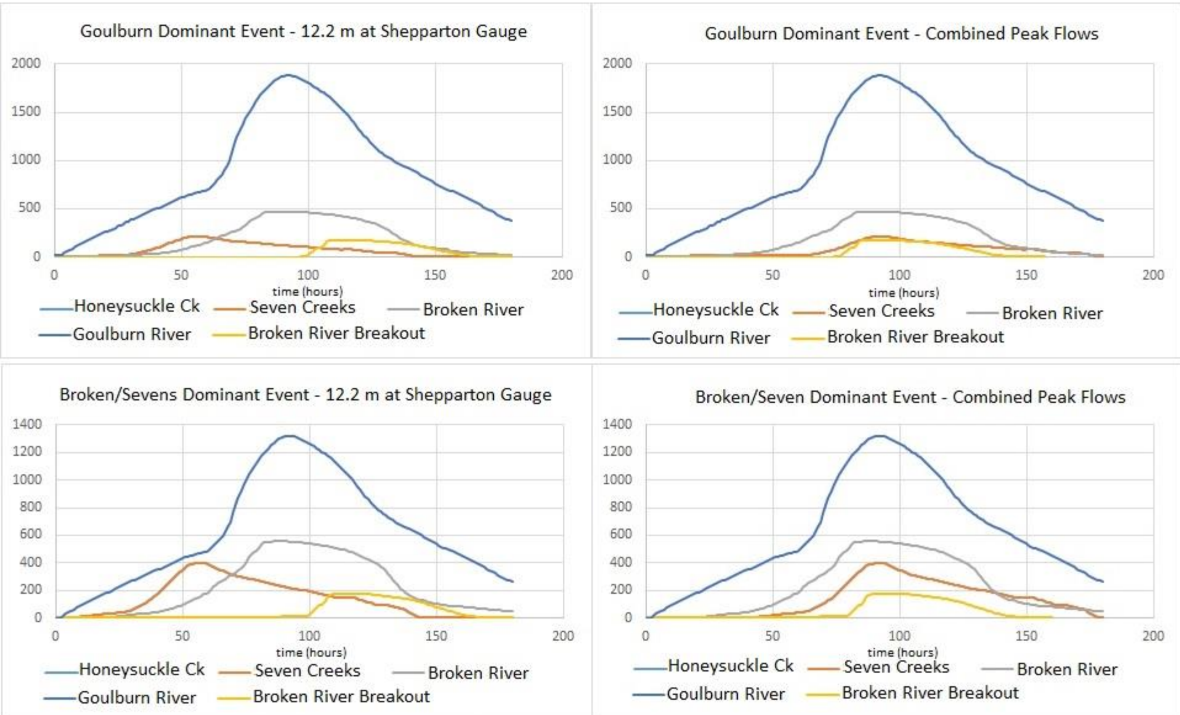

Figure 4-18 Inflow hydrographs for sensitivity analysis of tributary timing

Table 4-27 Impacts of tributary timing at the Shepparton Gauge

Flow Event	Flood Level at Shepparton Gauge (m AHD)	Flow at Shepparton Gauge (ML/d)
Goulburn Dominant	112.28	222,100
Goulburn Dominant Combined Peak	112.38	241,800
Broken/Seven Dominant	112.21	205,718
Broken/Seven Dominant Combined Peak	112.36	237,600

As expected, more closely aligning the peaks of the inflows resulted in an increase in flood levels at Shepparton. It showed not only an increase downstream of the confluences but also back up the three individual systems. The Broken/Seven dominant event showed a larger increase in flood levels (generally 100-200 mm) compared to the Goulburn dominant event (50-100 mm increase). The peak flows through the causeway increase significantly with the peaks aligned. Given the various catchment sizes of the contributing tributaries it is unlikely that they will align perfectly, and the design assumption is based on observations from historic events.

4.4.4 Summary of Adopted Design Inflows

A summary of the adopted design inflows is provided in Table 4-28. It should be noted that these values are slightly higher than the design estimates provided for the three waterways in Section 4.3. This is due to losses in peak flow rates across the catchment area and is explained in more detail in Section 5.3.5. The Broken River flows quoted include the two inflows from the Broken River while the Seven Creeks inflow includes both the Seven Creeks and Honeysuckle Creek systems.

Table 4-28 Adopted Design Event Inflow Summary

AEP%	Goulburn River (ML/d)	Broken River (ML/d)	Seven Creeks (ML/d)
20	49,100	17,900	22,500
10	69,000	29,600	35,100
5	90,900	42,700	48,600
2	123,900	53,800	67,300
1	162,600	63,430	82,100
0.50	176,500	72,680	95,760

5. HYDRAULIC MODELLING

5.1 Approach

A detailed combined 1D-2D hydraulic model of the township and surrounding floodplain was developed for the determination of flood levels and extents over a range of flood events primarily to inform flood intelligence for the study area. The calibrated hydraulic model simulates flood flow behaviour of the Goulburn River, Broken River and Seven Creeks as well as the overbank flow throughout the floodplain. The hydraulic modelling approach consisted of the following components:

- One dimensional (1D) hydraulic model of key hydraulic structures, pipes and river channels;
- Two dimensional (2D) hydraulic model of remaining waterways and the broader floodplain; and
- Links between the 1D and 2D hydraulic models to integrate the 1D hydraulic components with the broader floodplain flow.

The hydraulic modelling software TUFLOW developed by BMT-WBM was used for this study. TUFLOW is a state-of-the-art tool for floodplain modelling that combines the dynamic coupling of the 1D ESTRY river model and 2D TUFLOW model systems. Through coupling of these two systems it is possible to accurately represent river, pipe and floodplain processes.

The initial hydraulic analysis was carried out for the May 1974, October 1993 and September 2010 flood events, with the model calibrated to reproduce the observed flood heights and extents.

5.2 Information Used

The key information used to develop and run the hydraulic model is discussed below.

5.2.1 LiDAR data

LiDAR data for the region was made available from three different data sets, referred to as floodplain (FP), Fugro Spatial Systems (FSS) and Index of Stream Condition (ISC). A comparison of these datasets was undertaken as described in section 3.3.1. The 1m ISC DEM was approximately 100 mm above the FSS and FP DEMs. The available LiDAR grids are shown in Section Figure 3-2.

After careful analysis it was decided to use the ISC DEM as the base data set as it correlated the best with the feature survey, and compliment the ISC with the FSS and FP DEMS respectively. The FSS data was raised 100 mm to ensure there was no banding where the two datasets met.

5.2.2 Field Survey

Key survey data collated for the study included:

- Culvert crossings and bridge structure survey;
- Floor level survey of affected properties;
- Feature survey of the Shepparton Mooroopna causeway;
- Photos and sketches of the Shepparton Mooroopna causeway waterways;
- Feature survey of strategic levees downstream of Shepparton;
- Survey of key local drainage assets; and
- Flood marks for the May 1974, October 1993 and September 2010 events.
- SR&WSC Waterway cross sections used as part of SKP 1982

5.2.3 Hydrological Data

As part of the current study a detailed hydrologic analysis of the study area was undertaken and is detailed in Section 4.

The hydrology data was used as the input boundaries to the hydraulic model for the Goulburn River at Murchison, the Broken River at Gowangardie, Castle Creek at Arcadia, Pranjip Creek at Moorilim, and Seven Creeks at Kialla West. The hydrology of these boundaries was derived for a range of design events and the available gauge data was used directly for the May 1974, October 1993 and September 2010 calibration events. The rating curve for the Goulburn River at Loch Garry was used as the downstream model boundary, and the Goulburn River at Shepparton and the Broken River at Orrvale were used for calibration. Details of these gauges and the relevant available calibration data is shown in Table 5-1 and the locations of the gauges are shown in Figure 5-1.

Table 5-1 Available gauge data and peak flow data for calibration events

Site Number	Site Name	Catchment Area (Km^2)	Peak Flow 2010 (ML/d)	Peak Flow 1993 (ML/d)	Peak Flow 1974 (ML/d)
405246	CASTLE CREEK @ ARCADIA	164	2,870*	4,840	4,260
405226	PRANJIP CREEK @ MOORILIM	787	7,310	15,200	17,400
405269	SEVEN CREEKS @ KIALLA WEST	1,505	20,500	N/A	N/A
404224	BROKEN RIVER @ GOWANGARDIE	2,396	51,100	59,600	N/A
404222	BROKEN RIVER @ ORRVALE	2,508	27,300	42,900	N/A
405200	GOULBURN RIVER @ MURCHISON	10,772	50,200	63,500	117,900
405204	GOULBURN RIVER @ SHEPPARTON	16,125	78,600	150,000	191,000
405276	GOULBURN RIVER @ LOCH GARRY	16,490	57,100	97,400	N/A

* note that a higher flow was recorded in December 2010, however this investigation utilises the September 2010 event for calibration

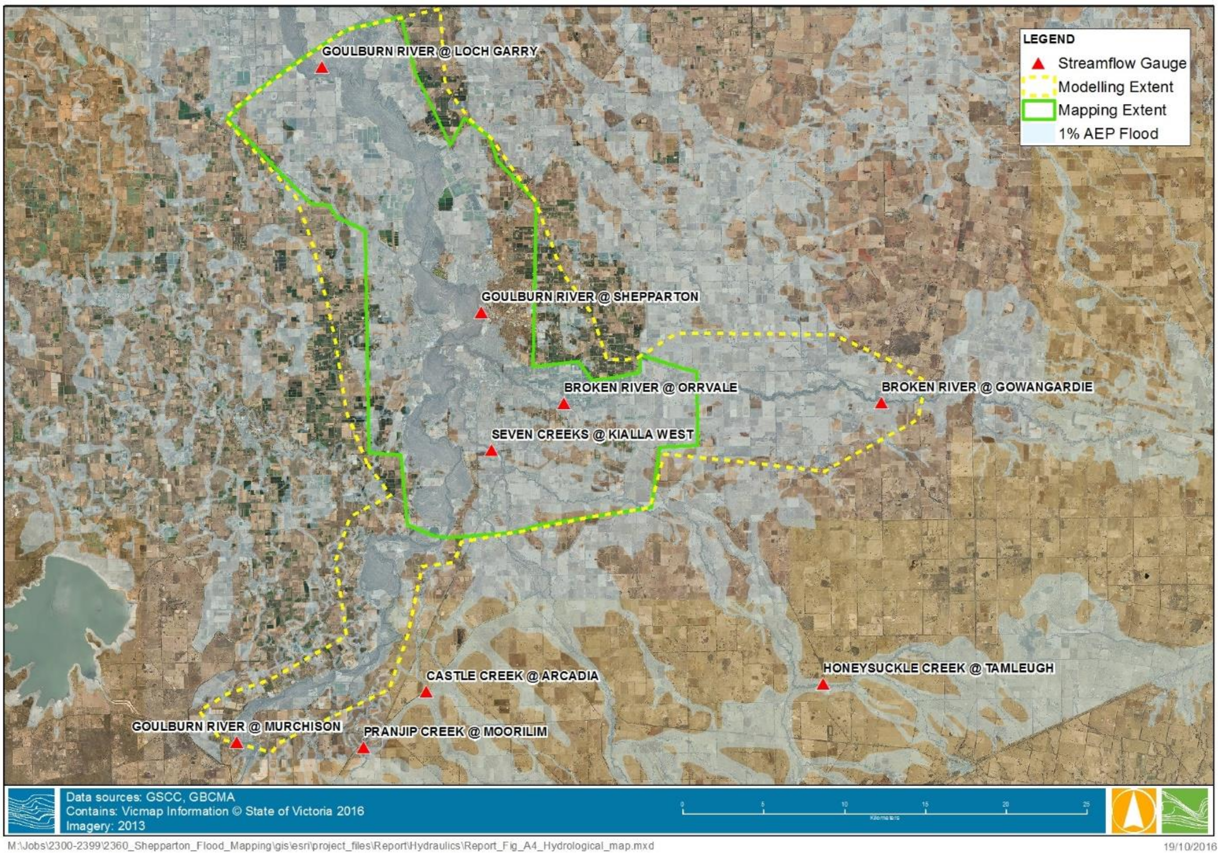

Figure 5-1 Location of available stream gauges and mapping extent

5.3 Hydraulic Model Development

5.3.1 Overview

Due to the complex nature of the floodplain within the study area, several hydraulic modelling options were tested. Through this process, several grid sizes and model extents were trialled. A single model extending from Murchison to Loch Garry on the Goulburn River and Gowangardie to Shepparton on the Broken River was initially trialled, but due to long run times this was split into three separate models. The approach then adopted two coarse resolution models on the upstream extents of the rivers, routing flows to the detailed Shepparton model area. The detailed Shepparton model initially represented the Goulburn River in 2D, but was converted to a 1D-2D model to improve run times. The 2D grid resolution was tested also to optimise the balance between run time and resolution. This section describes the final hydraulic model development.

5.3.2 Topography

The model covers a large area surrounding Shepparton, extending approximately 30 km up the Goulburn River to Murchison, approximately 26 km up the Broken River to Gowangardie Weir, and approximately 18 km downstream of Shepparton to Loch Garry. Shepparton is located at the confluence of the Goulburn and Broken Rivers, with heights ranging across the area from 116 m AHD near Murchison to around 105 m AHD within Loch Garry. Across the floodplain there are several small ephemeral watercourses, structures, irrigation channels, levees, railways and roadways which all influence flood behaviour, as well as the pipe drainage network within Shepparton itself.

To best represent the region, while allowing for reasonable run times, the model topography was split into three separate hydraulic models. To extend the model to the Murchison and Gowangardie gauges upstream of Shepparton, two smaller models were constructed using a 20 m grid resolution to route the flows from the gauges to the flood mapping extent along the Goulburn and Broken Rivers respectively. In both models the river channel was represented by a 1D channel and allowed to surcharge onto the 2D floodplain.

The larger Shepparton model covers the flood mapping extent with both the Goulburn and Broken Rivers and Seven Creeks again represented in 1D. The grid resolution for this model was split into two sections so that the Shepparton Township and surrounds could be modelled at the higher 10 m resolution and the routing downstream to Loch Garry is modelled at the lower 20 m grid resolution. The change in grid resolution occurs approximately 250 m north of Wanganui Road. The schematisation of the hydraulic model is shown in Figure 5-2 below.

Cross sectional survey was used to 'stamp' in the geometry of the main waterway channels on the LiDAR, so that the conveyance was accurately represented.

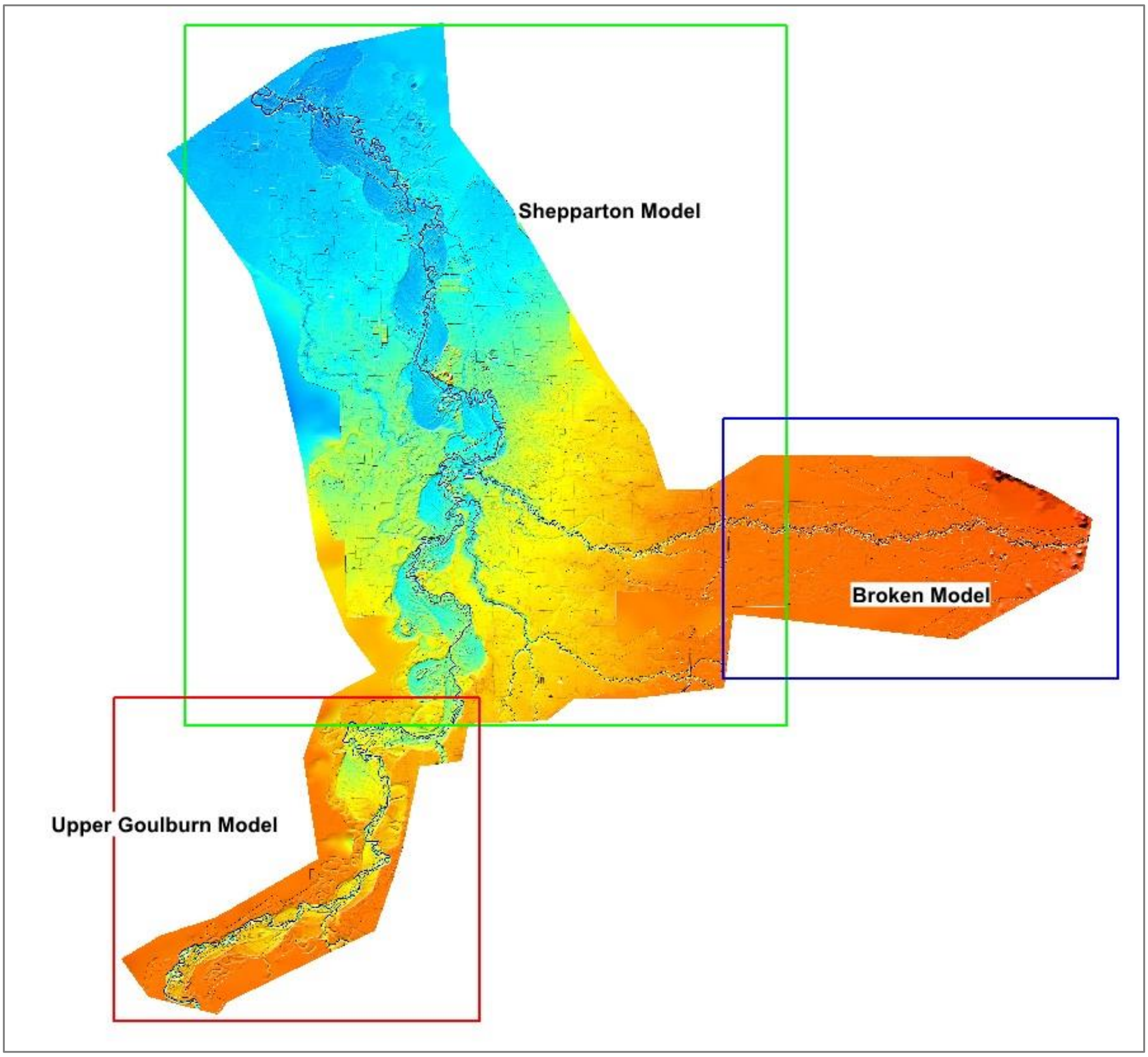

Figure 5-2 Shepparton Hydraulic Model Schematisation

5.3.3 Key Structures

Information about the key hydraulic structures within the floodplain including dimensions and inverts were required for input into the hydraulic model. The main structures within the study area were:

- East Goulburn Main Channel and the associated syphons;
- The railway bridge over the Goulburn and Broken Rivers;
- The causeway between Shepparton and Mooroopna across the Goulburn River floodplain;
- The levees adjacent to the Goulburn River;
- The Shepparton and Mooroopna drainage pipe network;
- The channels associated with the irrigation network;
- The Goulburn Valley Highway bridge over the Broken River; and
- Numerous drainage structures at various locations in the floodplain, such as culverts associated with the railway, roads and the irrigation channels.

Cross section details, dimensions and/or obverts of several hydraulic structures were not known and required estimation. LiDAR data was used to estimate invert levels and cross sections, and various imagery was used to estimate structure dimensions. It is expected that this method of estimating the structure inverts and dimensions will be accurate to +/-150 mm and as such will not have a significant impact on the model accuracy.

The main opening in the causeway (Daintons Bridge) is modelled as a (BW) Bridge Weir Structure in the 1D domain. The remaining openings in the midland highway have been modelled using an increased roughness.

A number of other bridges within the model extent were modelled as openings as identified in the LiDAR.

5.3.4 Hydraulic Roughness

The variation in hydraulic roughness within the study area was schematised as two separate roughness layers, one representing all the roads and the other representing the other various hydraulic roughness values (e.g. floodplain, channels, vegetation etc.). Areas with different roughness types were identified using aerial photographs and VicMap data layers. The values adopted for the two-dimensional hydraulic model are summarised in Table 5-2 and shown in Figure 5-3 below. These values were based on standard industry roughness values and were modified during the calibration process. The values adopted are reasonable estimates of hydraulic roughness given the floodplain condition.

Table 5-2 Hydraulic Roughness Parameters

Land Type	Roughness (Manning's "n")
Roads	0.015
Crops	0.05
Medium Density Vegetation	0.07
High Density Vegetation	0.10
Stagnant Water Bodies	0.03
Residential	0.06
Industrial	0.06
Cleared Land/Open Space	0.04
Goulburn River Channel	0.065
Seven Creeks Channel	0.06
Broken River	0.10
Pipes/Culverts	0.012

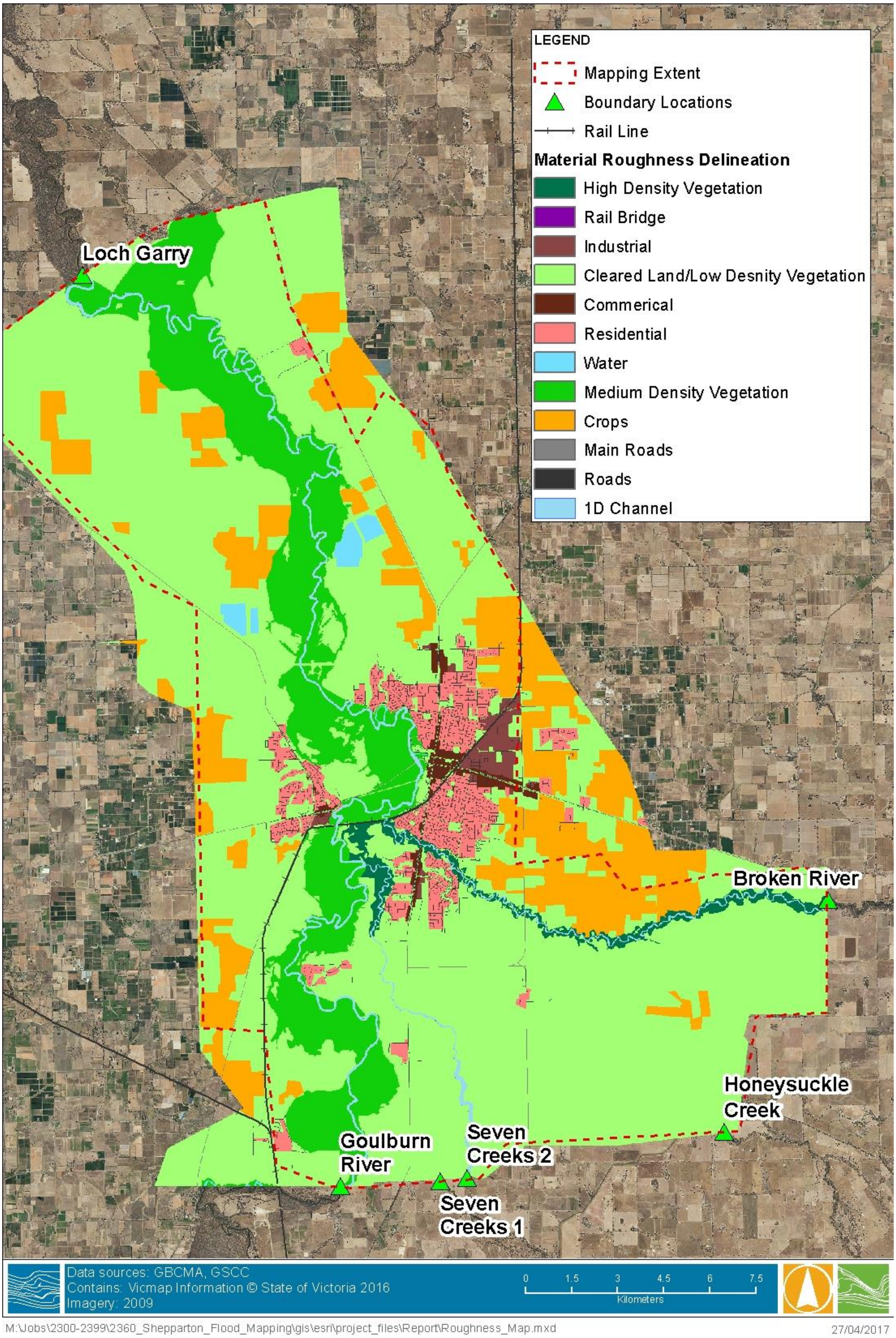

Figure 5-3 2D Material Roughness Map

5.3.5 Boundary Conditions

Broken Model

The Broken Model was developed with a single inflow boundary for the Broken River at Gowangardie Weir, located 26 km upstream of the confluence with the Goulburn River. This location was specifically chosen so that recorded flows from the gauge at Gowangardie could be input directly into the model for historical events, and flood frequency analysis at the gauge can be used for the design events.

The downstream boundaries were placed just downstream of the East Goulburn Main Channel, defined using the automatically generated Q-H relationships. Hydrographs for the Shepparton model were extracted for the Broken River and a breakout flow to the south upstream of the East Goulburn Main Channel. This structure acts as a major hydraulic control on the floodplain and greatly influences flow paths during overbank flow events. This barrier makes an ideal location to separate the hydraulic models.

Flow from the Broken River can also overtop the Goulburn Valley Highway just to the north of the river and enter the Pine Lodge Creek system. Another Q-H boundary has been placed on Pine Lodge Creek to the north to take this flow out of the model, and a flow extraction line (TUFLOW PO line) has been placed there to quantify this breakout.

Upper Goulburn Model

Like the Broken Model, the Upper Goulburn Model has an inflow point on the Goulburn River at Murchison. There are also two tributaries that enter downstream of Murchison, being Pranjip Creek at Moorilim and Castle Creek at Arcadia. All three of these flows are taken directly from the gauged data for the historical events and from the flood frequency analysis for the design events.

The downstream Q-H boundary of the Upper Goulburn Model has been placed just upstream of the flood mapping extent, downstream of Bridge Road near Toolamba. The flow extraction line for the Shepparton model has been placed just upstream of Bridge Road.

Shepparton Model

The Shepparton Model has numerous inflow boundaries not only from the Upper Goulburn and Broken models, but for Seven Creeks as well. The Shepparton model overlaps the Upper Goulburn and Broken Models and uses the flows extracted from those models as the upstream boundaries for the Goulburn River, the Broken River and the breakout south of the Broken River upstream of the East Goulburn Main Channel.

As the gauge for Seven Creeks at Kialla West is within the flood mapping extent, the inflow boundary was split into three boundaries upstream of the confluence of Seven Creeks and Honeysuckle Creek The catchment areas for Honeysuckle Creek and Seven Creeks at this point are approximately the same, so the inflows have been split evenly between Honeysuckle Creek and Seven Creeks, with the Seven Creeks inflow split evenly again between the two branches. The inflows at these boundaries had to be scaled up slightly to ensure that the flow at the gauge was accurately reproduced (to account for floodplain storage between the boundary inflows and the streamflow gauge). A similar approach was adopted for the Broken River inflows, with the main Broken River inflow placed upstream of the East Goulburn Main Channel and a secondary Broken River inflow placed south of the Broken River. With both inflows upstream of the Broken River at Orrvale streamflow gauge, flows were scaled up to simulate the design flow estimates from the flood frequency analysis at the streamflow gauge.

The downstream extent of the model incorporates Loch Garry and the gauge on the Goulburn River approximately 18 km downstream of Shepparton. The rating curve from the gauge has been used for the Q-H relationship on the Goulburn River downstream boundary. There is also an automatically generated Q-H boundary on the floodplain adjacent to the Goulburn River outside of the levee to the south west, and another Q-H boundary on the floodplain north of the Loch Garry levee. An automatically generated Q-H boundary has also been used for the structure within Loch Garry to estimate the operation of the weir during flood events. All model boundary locations are shown in Figure 5-4.

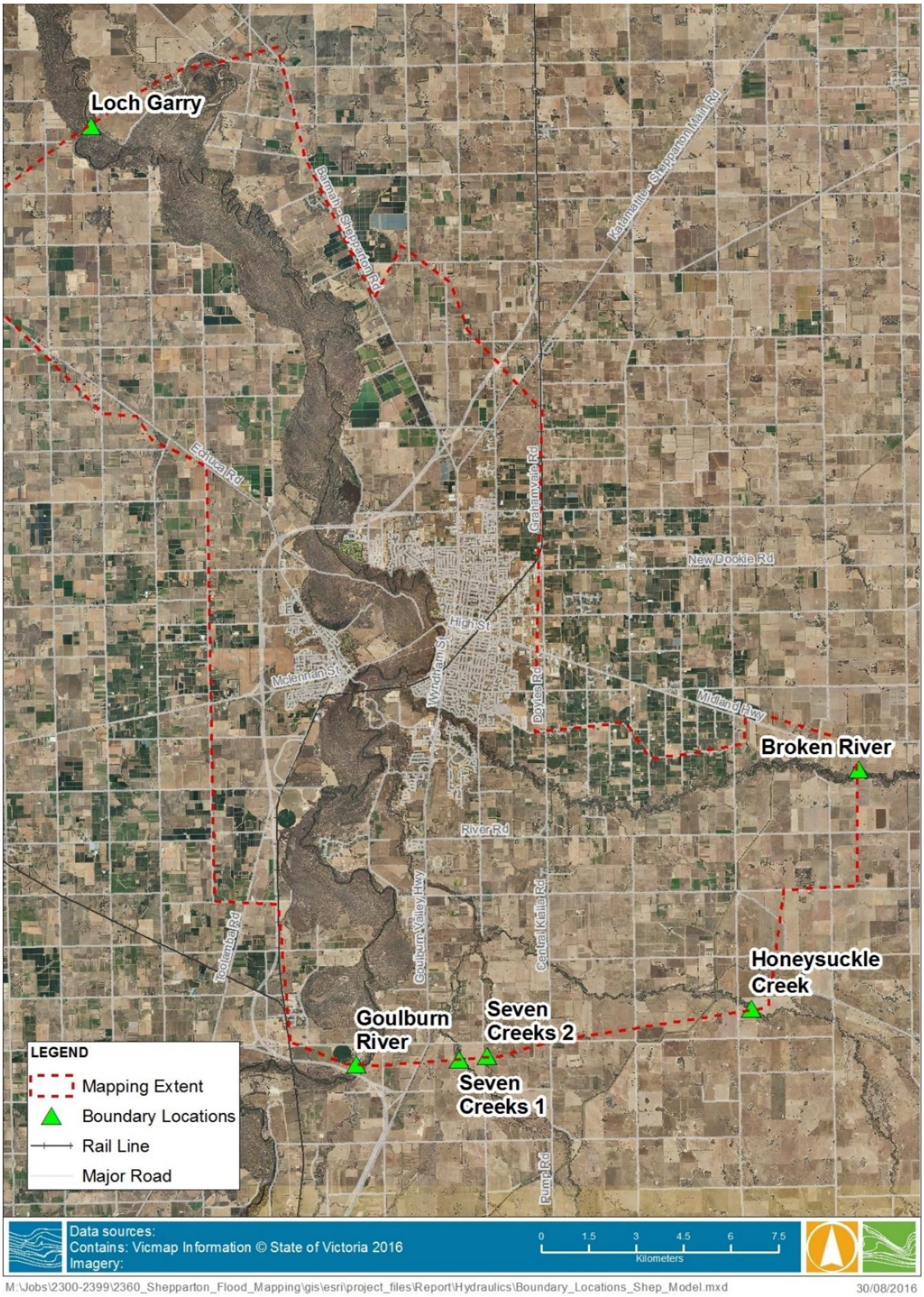

Figure 5-4 Shepparton Mooroopna Hydraulic Model Boundary Locations

5.3.6 1D Pipe Network

There are areas of Shepparton and Mooroopna that are inundated due to the backflow of pipes and council has requested that these be included in the hydraulic model. The entire stormwater network was provided by Council and after discussions with Council and the Goulburn Broken CMA it was decided to include those pipes greater than 600 mm in diameter. Figure 5-5 shows the selected pipes from the stormwater water pipe system for Shepparton and Mooroopna have been included in the TUFLOW hydraulic model.

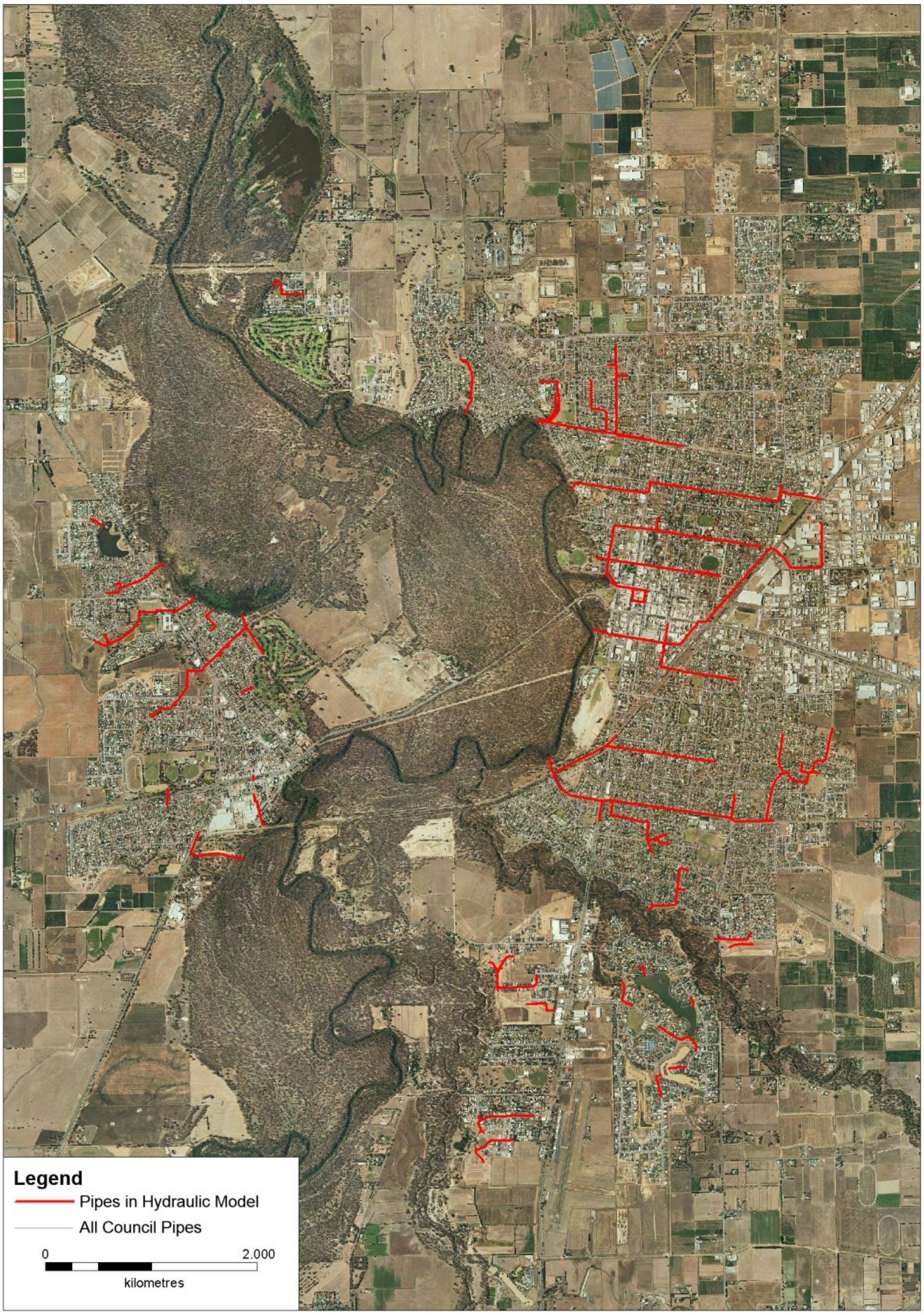

Figure 5-5 Selected Shepparton and Mooroopna Stormwater Pipes

5.4 Hydraulic Model Calibration

5.4.1 Overview

The following Section discusses the fine-tuning of the hydraulic model parameters through calibration against observed flood data. The model was calibrated to the large flood event of October 1993 and smaller September 2010 flood event in tandem, with validation to the May 1974 flood event. Surveyed flood marks (provided by the Goulburn Broken CMA), gauged river heights and aerial photographs of the floods were the basis for comparison to the modelled results.

Several sensitivity runs were undertaken with minor changes to the model parameters to get a better match to gauged river levels, surveyed flood levels and flood extents, namely:

- Adjusting the Broken River channel cross section near the East Goulburn Main Channel to allow more flow to pass through the gap in the high channel banks. The East Goulburn Main Channel creates a major hydraulic barrier so time was spent ensuring the afflux across the structure was modelled correctly.
- Adding the culverts under the railway line near Toolamba Road.
- Increased the Goulburn River and Seven Creeks roughness from 0.045 to 0.06 and the Broken River roughness from 0.06 to 0.10 (reasonable given the dense vegetation and woody debris along the channel). This helped raise flood levels to better match the observed flood levels for the 1993 and 2010 calibration events.
- Incorporating crest levels from the Goulburn River levee survey downstream of Shepparton from the Goulburn River Levee Audit project completed on behalf of the Goulburn Broken CMA. This provided a more accurate representation of the levee, which was otherwise not adequately defined in 2D at the model resolution.
- Layered flow constrictions and form losses were applied along the Shepparton-Mooroopna Causeway waterway opening after receiving the detailed gauging record from the 1974 flood event and structure details from the Goulburn Broken CMA. This additional information helped refine the flow through the causeway structures.

The final roughness parameters determined from the calibration process are shown in Table 5-2.

It should be noted that while flood mark survey is available for the calibration events there is inherent inaccuracies in the collection of those levels. The levels are primarily based on flood debris marks which may be significantly higher or lower than the true peak due to several reasons such as debris piling up on the upstream side of an obstruction or debris collecting on the recession of a flood, and obstructions causing a bow wave effect (with higher levels on the upstream face and lower on the downstream face).

A certain degree of engineering judgement is required in the collection of this data and inaccuracies in the data at some locations are likely.

5.4.2 September 2010 Calibration

15 flood marks within the flood mapping extent from the September 2010 flood event were collected by the Goulburn Broken CMA. These flood marks were complimented with aerial photography of the flood extent and river gauge data to check the modelled flood extent.

The 15 survey flood marks located within the study area were compared to the modelled flood levels:

- 12 points were within +/- 200 mm;
- 2 points had modelled water levels with a difference greater than 200 mm;
- On average the model levels were 49 mm higher than the observed flood marks.

The overall trend showed that the modelled flood levels were slightly higher than the surveyed flood levels. All modelled flood levels were well within the error threshold for the hydraulic model calibration for the September 2010 flood event.

Figure 5-6 below shows a plot of the water level for the gauge on the Goulburn River at Shepparton comparing the model results to the gauged data. The graphs show that the rising limb of the modelled hydrograph arrives slightly earlier than the gauged data; nevertheless, the peak elevation is well represented in the model. A calibration plot for the September 2010 flood event is shown in Figure 5-9. The aerial imagery obtained after the flood peak from Nearmap (Figure 5-10) shows the flood extent matches well around the Kialla West area along the Broken River.

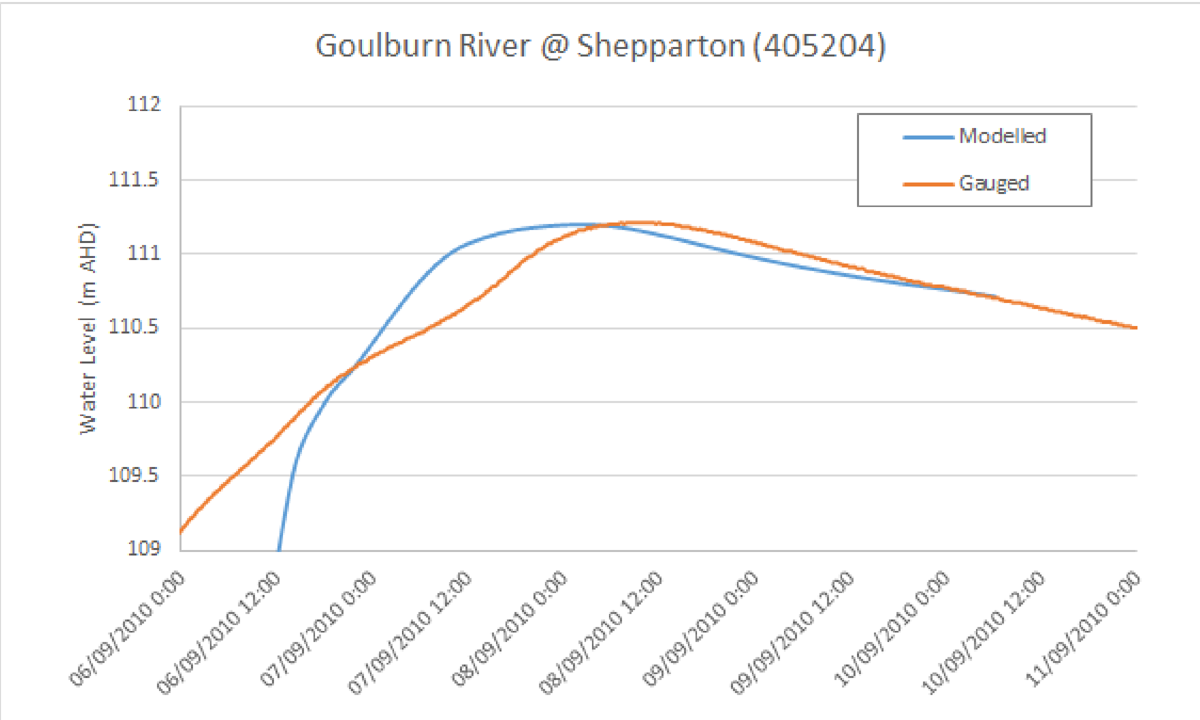

Figure 5-6 Comparison between modelled and gauged water levels for the Goulburn River at Shepparton during the September 2010 event

Figure 5-7 below shows a plot of the water level for the gauge on the Broken River at Orrvale comparing the model results to the gauged data. The graphs show that the rising limb of the modelled hydrograph compares well with the gauged data, the peak elevation is well represented in the model, and only the falling limb not comparing well, receding quicker than the gauged data. Figure 5-8 shows the comparison of the modelled and gauged water levels at the Seven Creeks at Kialla streamflow gauge. This shows the modelled peak flood level being slightly lower (110 mm) compared to the gauged flood level. The rising limb is not shown in this plot as the final calibration run utilised a hot start initial condition at 6:00am on the 6th September 2010.

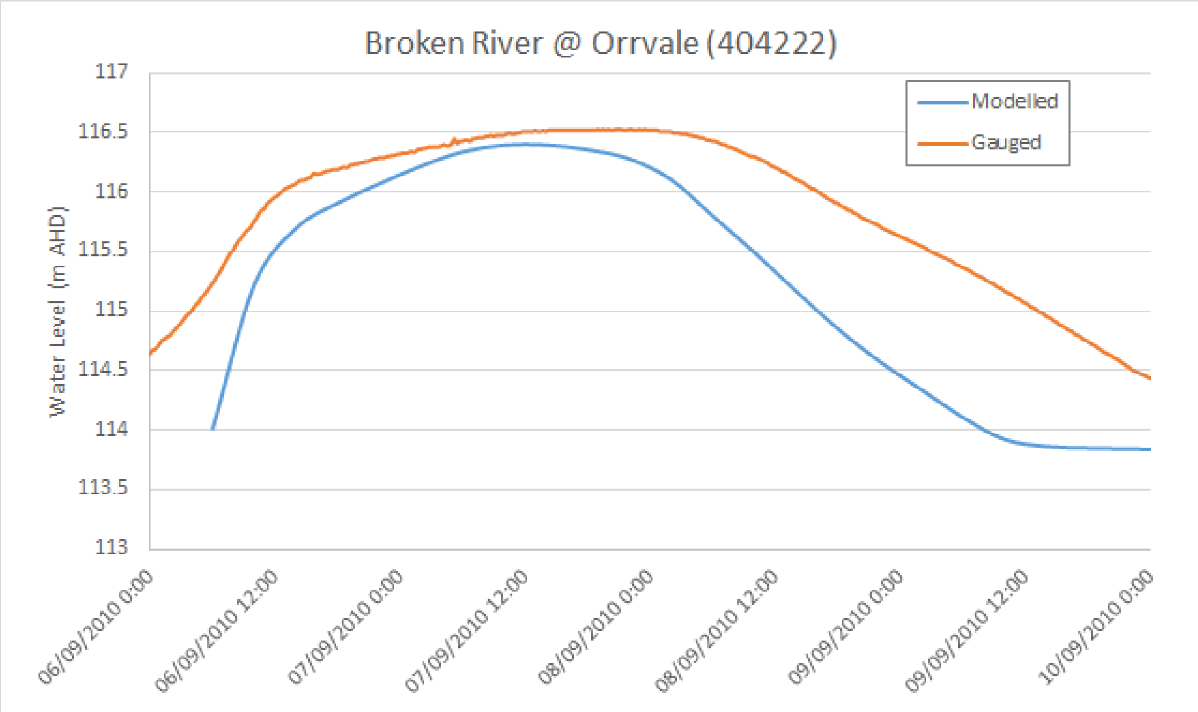

Figure 5-7 Comparison between modelled and gauged water levels for the Broken River at Orrvale during the September 2010 event

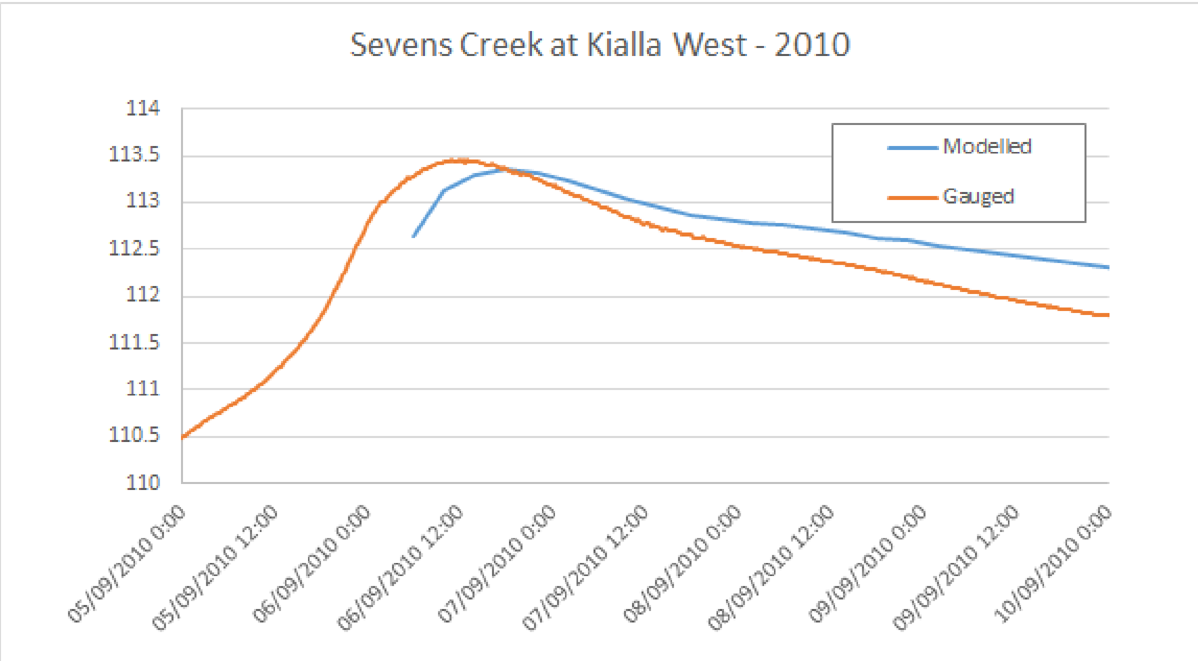

Figure 5-8 Comparison between modelled and gauged water levels for the Seven Creeks at Kialla West during the September 2010 event

The modelled flood extent matched very well with observations, gauged river heights and aerial photographs, and was deemed an acceptable calibration result. Figure 5-11 shows the water surface profiles along the three main waterways. These are plotted with the chainage distance of the waterway along the x – axis and the running distances (provided by the GBCMA) have also been included at key features along the waterways.

Flood Behaviour

Heavy rainfall occurred in in the north east of Victoria on Saturday 4th and Sunday 5th September 2010, particularly in the alpine areas including the upper Goulburn and Broken catchments. On Monday 6th September, the Seven Creeks at Kialla West peaked just above the major flood level of 6.6 m, and shortly after the Broken River at Orrvale peaked overnight at 8.19 m, again above the major flood level of 7.9 m. On Wednesday 8th September, the Goulburn River at Shepparton also peaked just above the major flood level of 11 m. Minor tributaries into the Goulburn, Castle Creek and Pranjip Creek, also flooded.

The September 2010 event was mostly contained within the lower floodplain area on the Goulburn River, however low lying areas near the Broken River were inundated. The SES advised that 13 houses and 31 structures were damaged by the floods. Approximately 30 local roads were closed due to flooding, however all major roads surrounding Shepparton remained open for the duration of the event. Figure 5-9 below shows the modelled peak flood extent which was consistent with the observed flood extent. This shows that low lying areas between Archer Road and the East Goulburn Main Channel along the Broken River were inundated during the event, whilst areas outside of the Goulburn River lower floodplain were not affected.

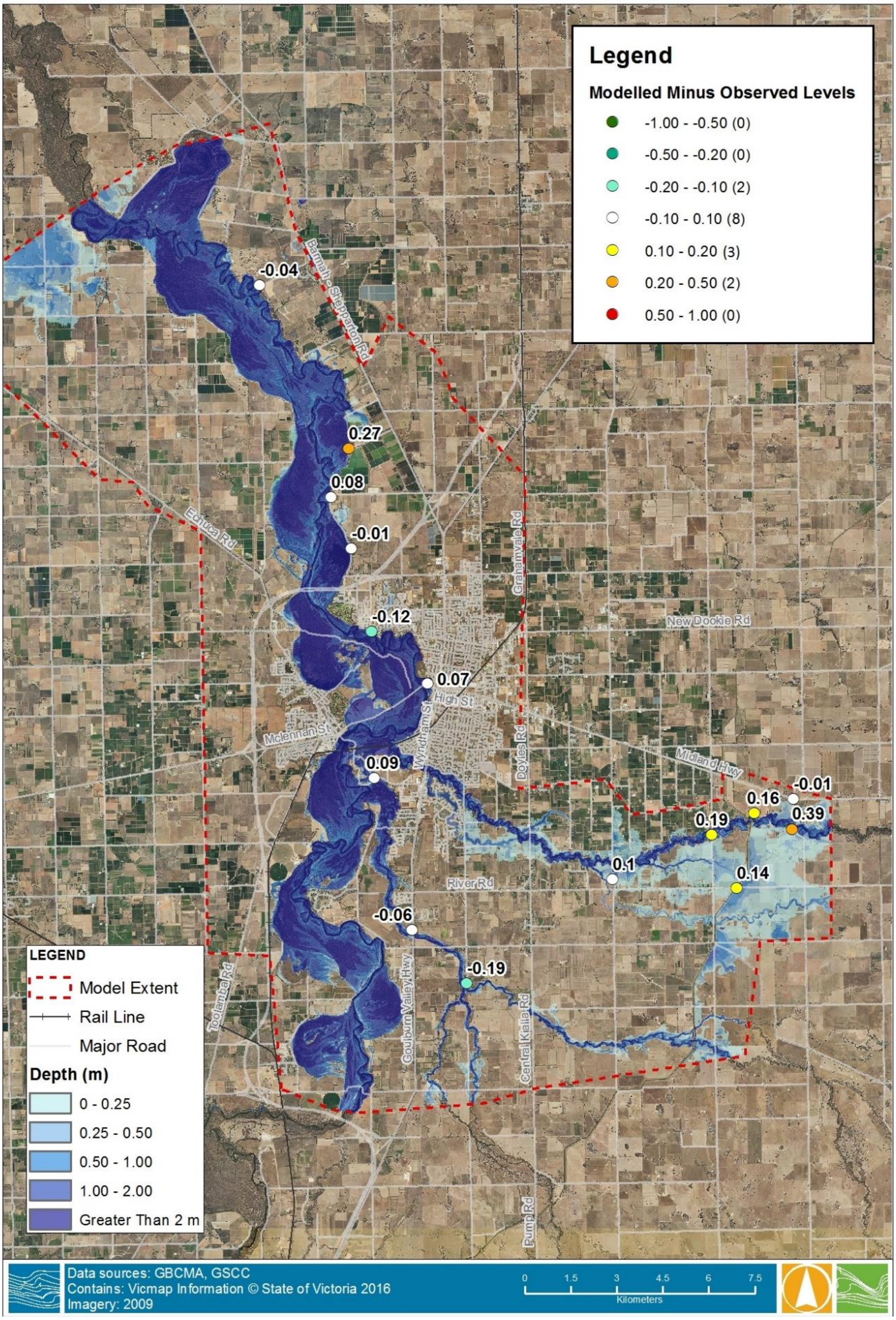

Figure 5-9 Hydraulic model calibration plot – September 2010

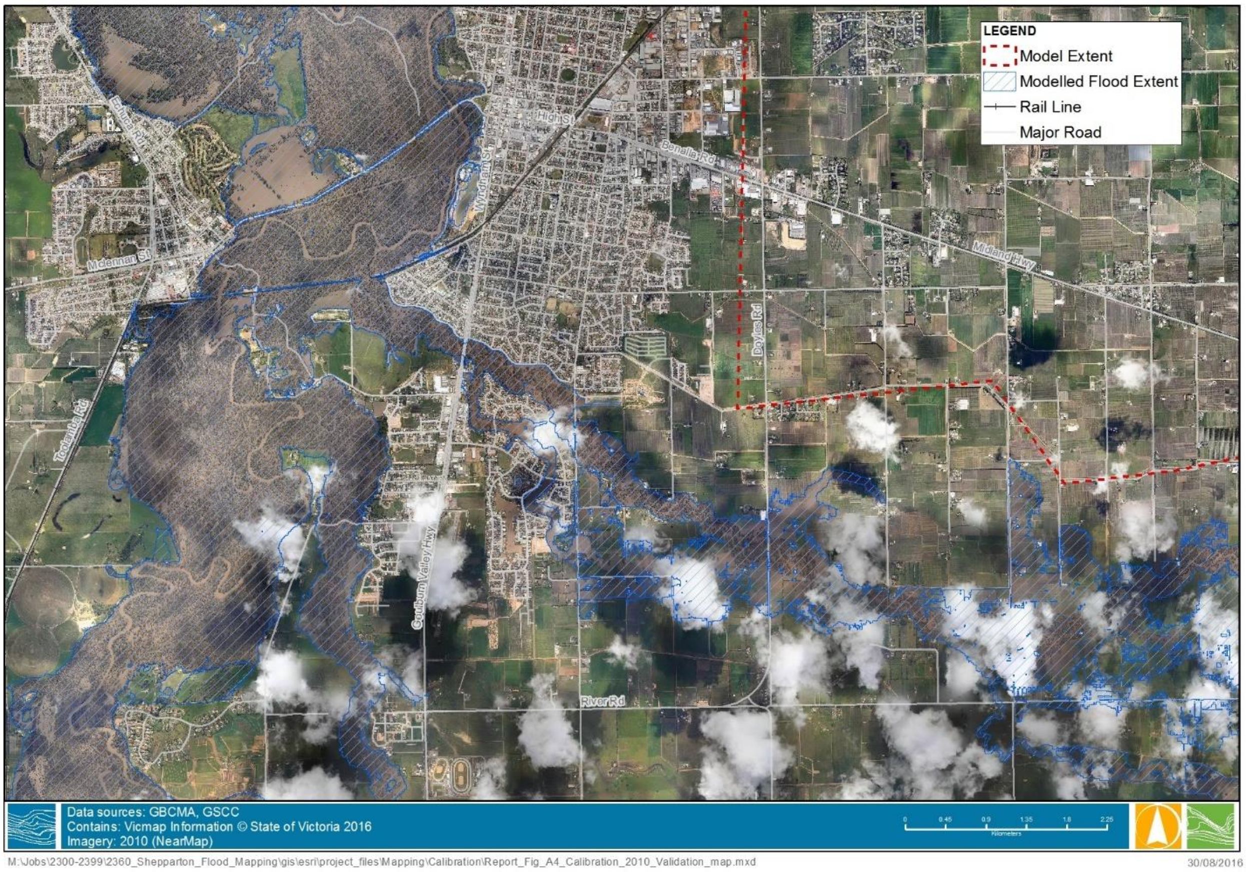

Figure 5-10 September 2010 Modelled Flood Extent Aerial Imagery Validation (Source: NearMap)

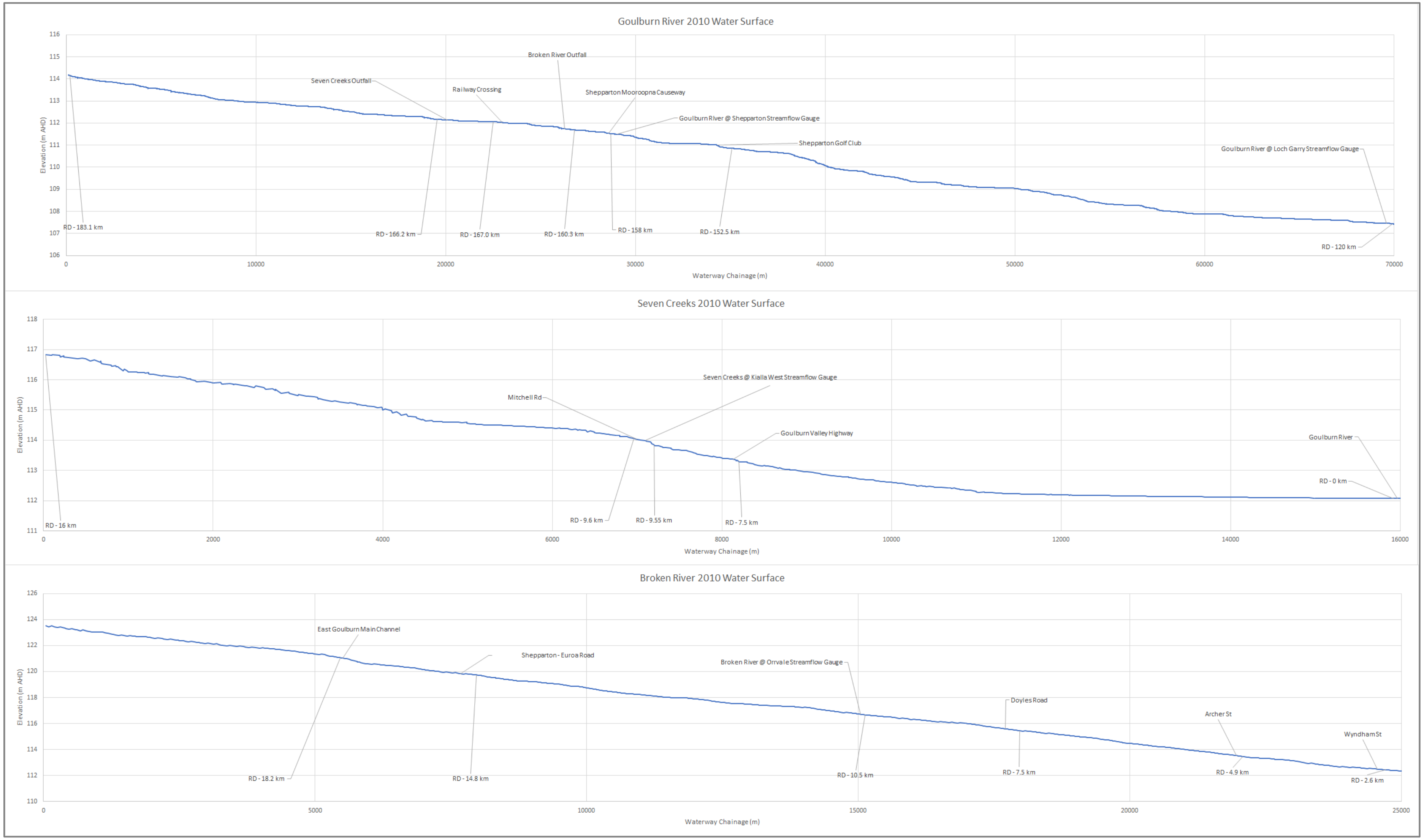

Figure 5-11 September 2010 – Water Surface Profiles

5.4.3 October 1993 Calibration

Many survey flood marks were collected for the October 1993 flood event. In total, there were 66 survey points to which the model results were calibrated, giving confidence in the reliability of the reproduced flood behaviour. Calibration plots of the October 1993 flood event are shown in Figure 5-15. Of the 66 survey flood marks located within the study area:

- 32 (48%) points were within +/- 100 mm;
- 19 (29%) points were within +/- 100 - 200 mm;
- 8 (12%) points were within +/- 200 - 300 mm;
- 4 (6%) points were below 300mm and were mainly near the Broken River just upstream of the confluence with the Goulburn River;
- 3 (5%) points were above 300mm; and
- On average the 66 observed flood levels that sit within the modelled flood extent showed no overall difference above or below the surveyed flood marks, with a standard deviation of 219 mm.

The overall trend showed that the modelled flood levels had no bias higher or lower than the surveyed flood levels and were predominately well within the satisfactory error interval expected for flood modelling scenarios.

Figure 5-12 below shows a plot of the water level for the gauge on the Goulburn River at Shepparton comparing the model results to the gauged data. The graphs show that the rising and falling limbs of the modelled hydrograph are well represented within the model, and the peak elevation is approximately 110 mm higher than the gauged data.

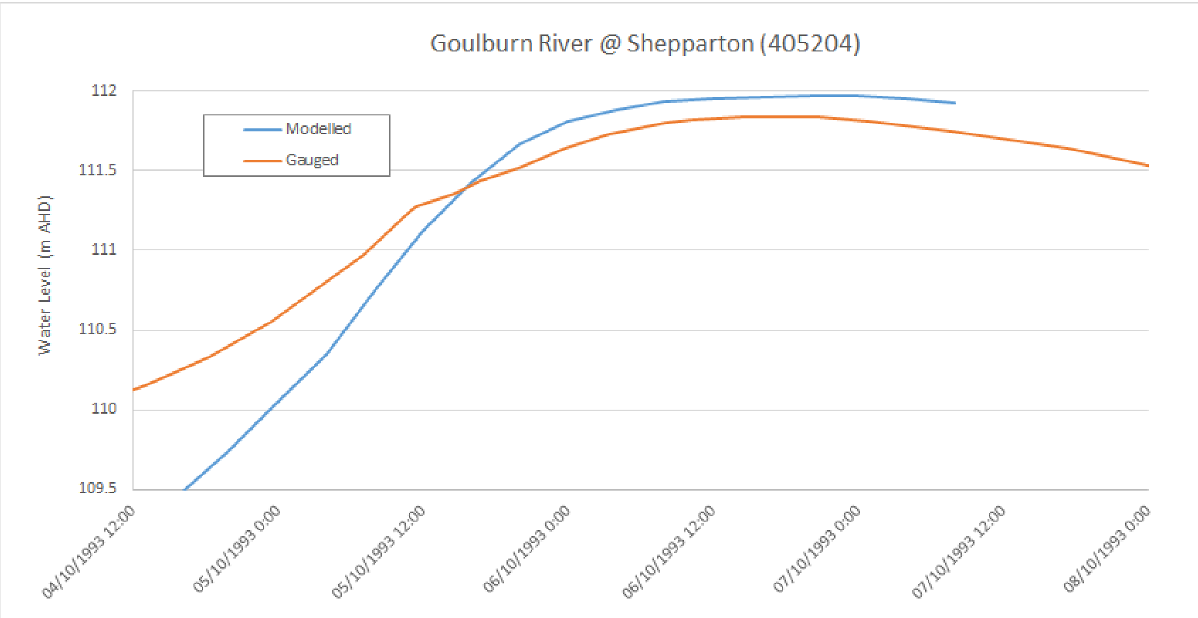

Figure 5-12 Comparison between modelled and gauged water levels for the Goulburn River at Shepparton during the October 1993 event

Figure 5-13 below shows a plot of the water level for the gauge on the Broken River at Orrvale comparing the model results to the gauged data. The graphs show that the rising limb of the modelled hydrograph arrives slightly later than the gauged data, the peak elevation is well represented in the model, despite overestimating the peak by 150 mm. There is limited streamflow data for Seven Creeks at Kialla West for the 1993 flood event. A 36-hour period which shows the peak passing the gauge was captured and is shown against the modelled flood levels in Figure 5-14. The modelled flood event has the peak slightly delayed, however appears to represent the gauged hydrograph well.

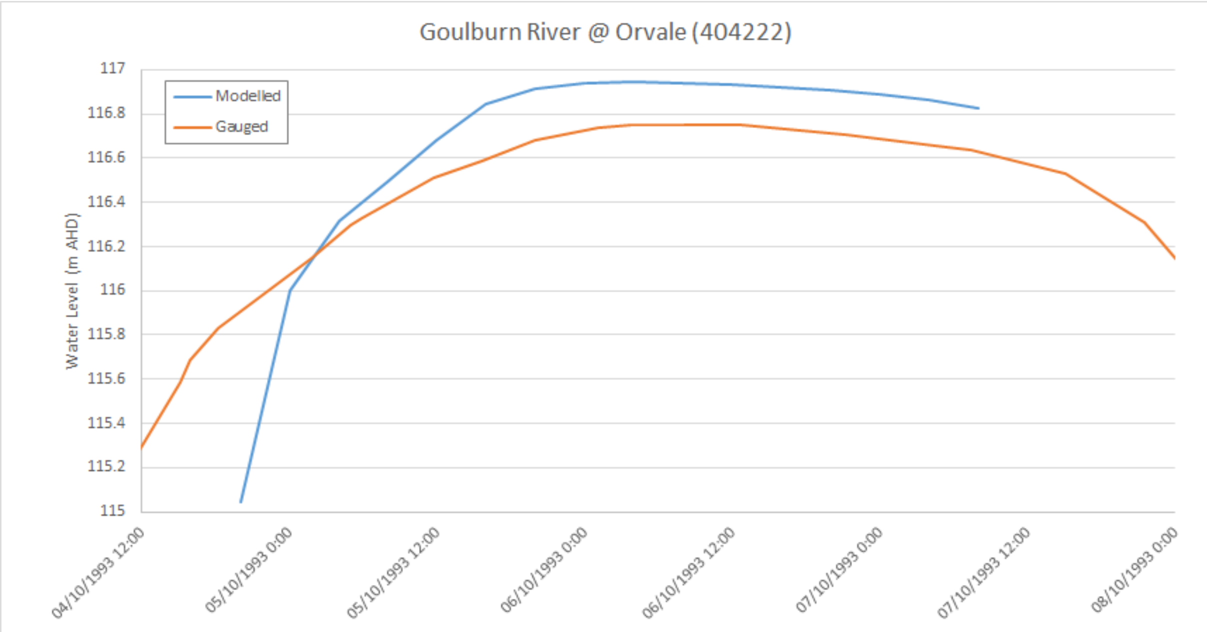

Figure 5-13 Comparison between modelled and gauged water levels for the Broken River at Orrvale during the October 1993 event

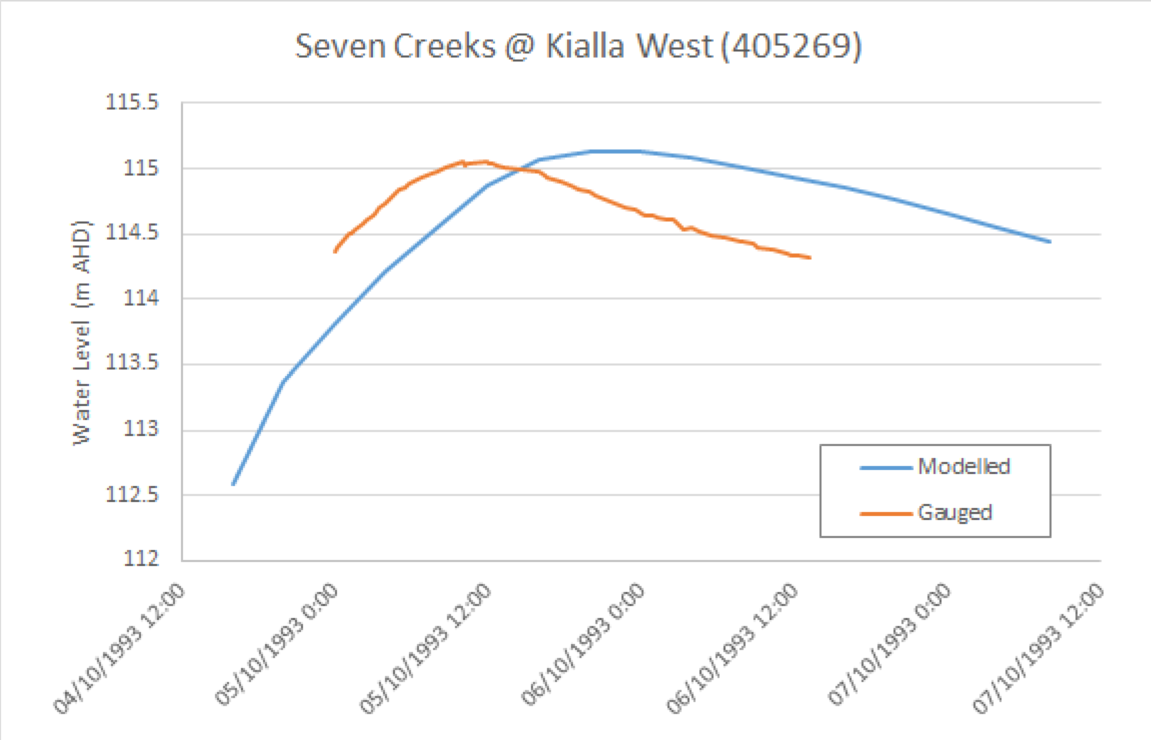

Figure 5-14 **Comparison between modelled and gauged water levels for the Seven Creeks at Kialla West during the October 1993 event**

The modelled flood extent matched very well with observations, gauged river heights and aerial photographs, and was deemed an acceptable calibration result. Figure 5-16 shows the water surface profiles along the three main waterways. These are plotted with the chainage distance of the waterway along the x – axis and the running distances (provided by the GBCMA) have also been included at key features along the waterways.

Flood Behaviour

In the lead up to the October 1993 flood, the Goulburn River had sustained high water levels for the majority of September. The Broken River and Seven Creeks during this time were relatively low until they both received a big inflow that started around 3rd October and lasted until early on the 9th October. The Goulburn River peaked again at the same time and stayed high until around the 16th October where it finally receded. Even though the peak in the Goulburn River wasn't as high upstream of Shepparton as it was in September, the combination of the three systems caused a peak water level of approximately 11.7 m at the Gauge in Shepparton on Wednesday 6th October.

Upstream of Shepparton on the Goulburn River most of the flow was contained within the lower floodplain. The flows in the Goulburn River, Broken River and Seven Creeks were all larger than 2010, particularly in Seven Creeks where significant overbank flood flow occurred in surrounding low lying areas. Parts of the town of Shepparton were inundated during the event and significant areas downstream of Shepparton were also inundated, particularly around the water treatment plant. The 1993 flood event is referred to as a 'Broken River and Seven Creeks dominant event'. This refers to the two systems mentioned being the dominant flooding mechanism and the flows recorded on these systems being of higher magnitude compared to the Goulburn River during the flood event.

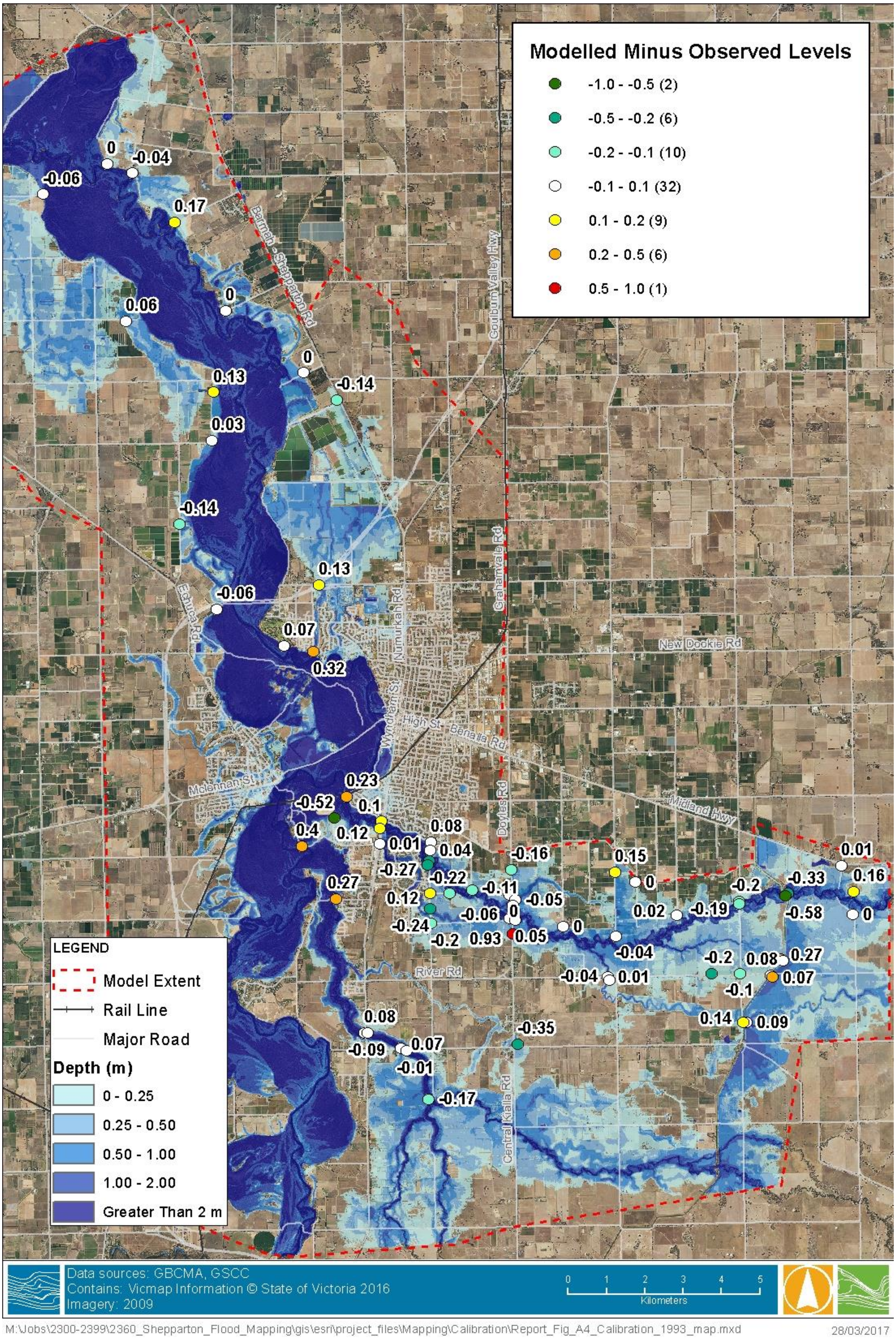

Figure 5-15 Hydraulic model calibration plot – October 1993 event

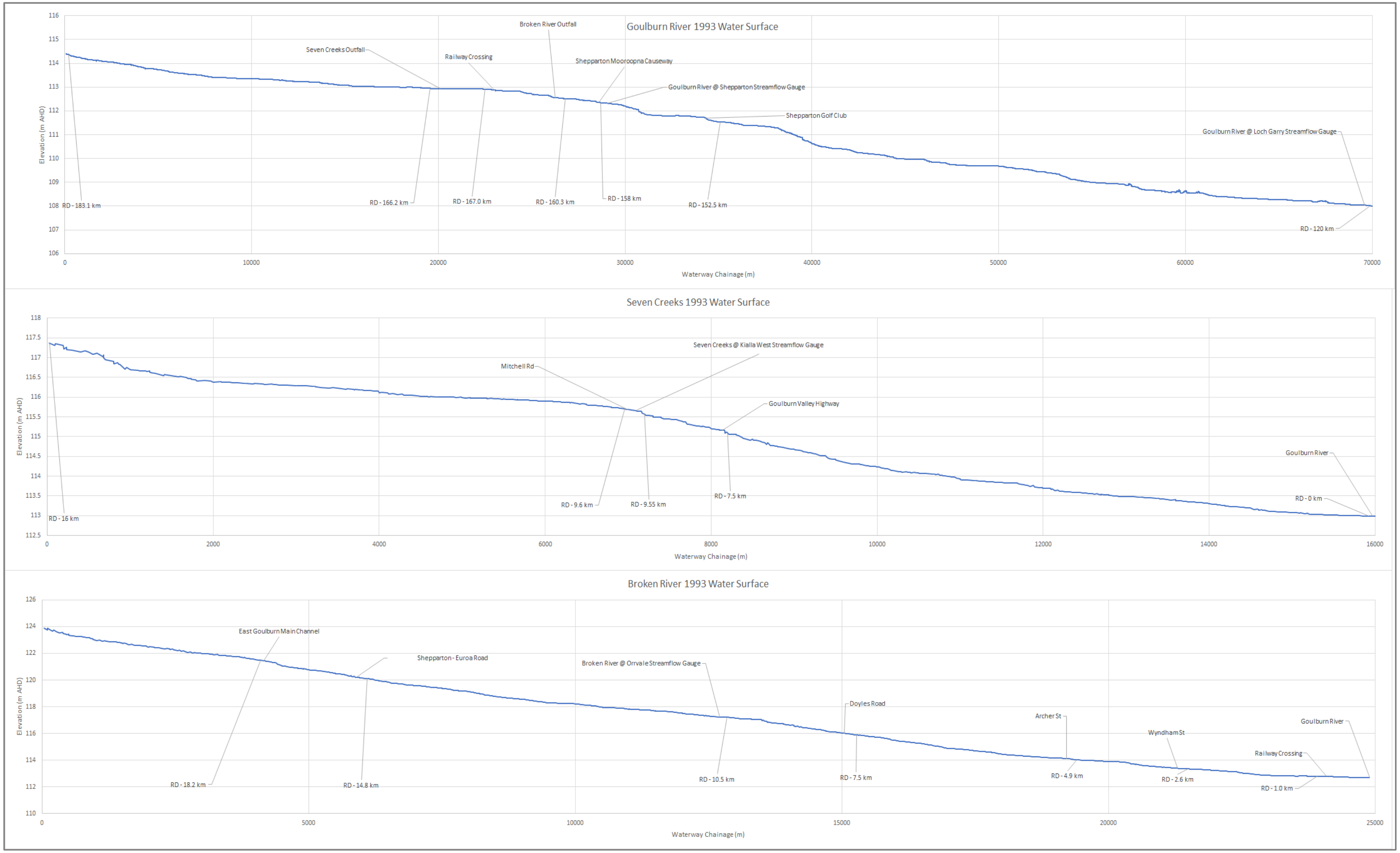

Figure 5-16 October 1993 - Water Surface Profiles

5.4.4 May 1974 Validation

Many survey flood marks were collected for the May 1974 flood event. In total there were 377 survey points available in the VFD, 114 of these points were classified with a reliability of 'Good' or 'High'. Only the surveyed points with these levels of reliability were used to validate the hydraulic model, giving confidence in the reliability of the reproduced flood behaviour. Validation plots of the May 1974 flood event are shown in Figure 5-18 and Figure 5-19. Of the 114 survey flood marks located within the study area:

- 40 (35%) points were within +/- 100 mm;
- 28 (25%) points were within +/- 100 - 200 mm;
- Approximately 60% of the modelled validation points were within 200 mm;
- 19 (17%) points were within +/- 200 300 mm;
- 3 (3%) points were below 300mm;
- 20 (18%) points were above 300mm.
- 4 (4%) points were not in the modelled flood extent; and
- On average the modelled water levels were 124 mm above the surveyed flood marks, with a standard deviation of 201 mm.

The overall trend showed that the modelled flood levels were slightly higher than the surveyed flood levels but the majority within the satisfactory error interval expected for flood modelling scenarios. Most of the modelled flood levels which were higher than observed levels were centred around the Mooroopna area. A comparison of aerial imagery from 1974 and present day shows extensive development to the north of the Midland Highway in Mooroopna. It would be expected that development through this area would have likely required earthworks to infill the floodplain which may have resulted in an increase in flood levels.

Figure 5-17 below shows a plot of the water level for the gauge on the Goulburn River at Shepparton comparing the model results to the gauged data. The graphs show that the rising limb of the modelled hydrograph arrives slightly later than the gauged data, the peak elevation is approximately 100 mm lower than the gauged data, and the falling limb receding slightly later than the gauged data as well.

Figure 5-20 shows the water surface profiles along the three main waterways. These are plotted with the chainage distance of the waterway along the x – axis and the running distances (provided by the GBCMA) have also been included at key features along the waterways.

Figure 5-21 provides a summary of the longitudinal section for the peak flood level each waterway during the three historical events modelled. This helps to provide context of the magnitude of the events on each of the waterways.

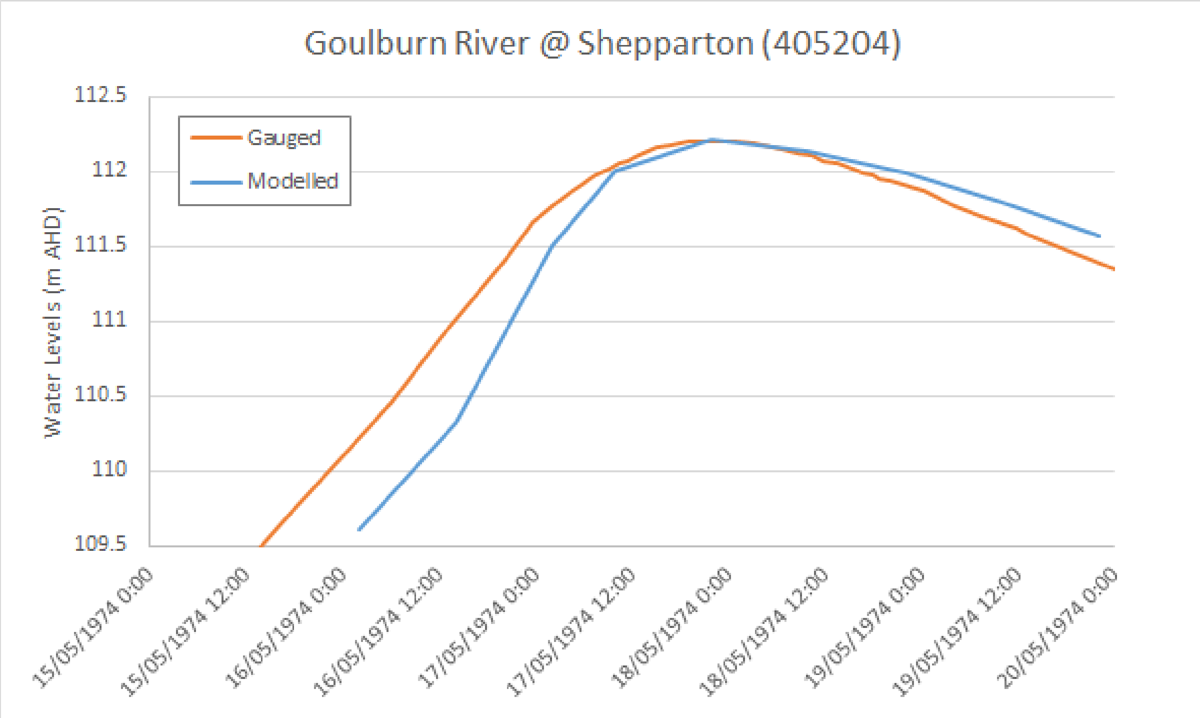

Figure 5-17 Comparison between modelled and gauged water levels for the Goulburn River at Shepparton during the May 1974 event

No streamflow data for the Broken River at Orrvale or the Seven Creeks at Kialla West gauge exists for the 1974 flood event. Both gauges were installed in 1977.

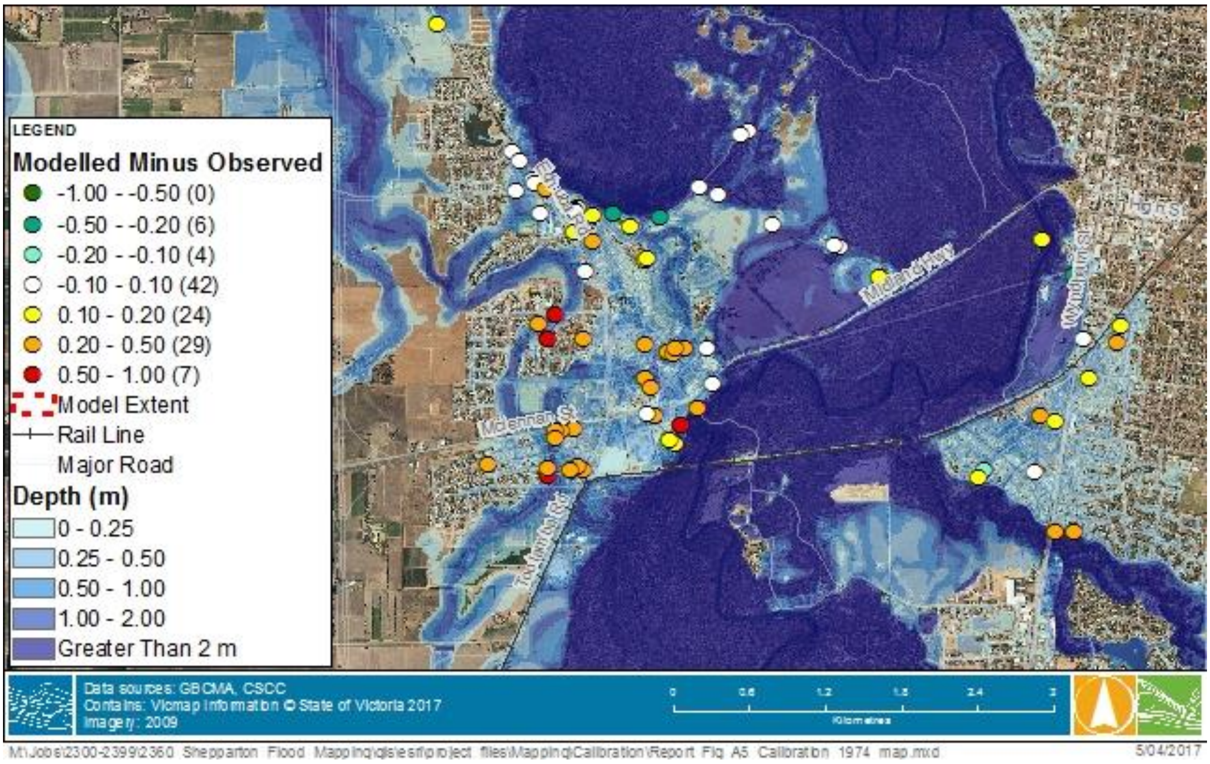

Figure 5-18 Hydraulic model validation plot – May 1974 event (township)

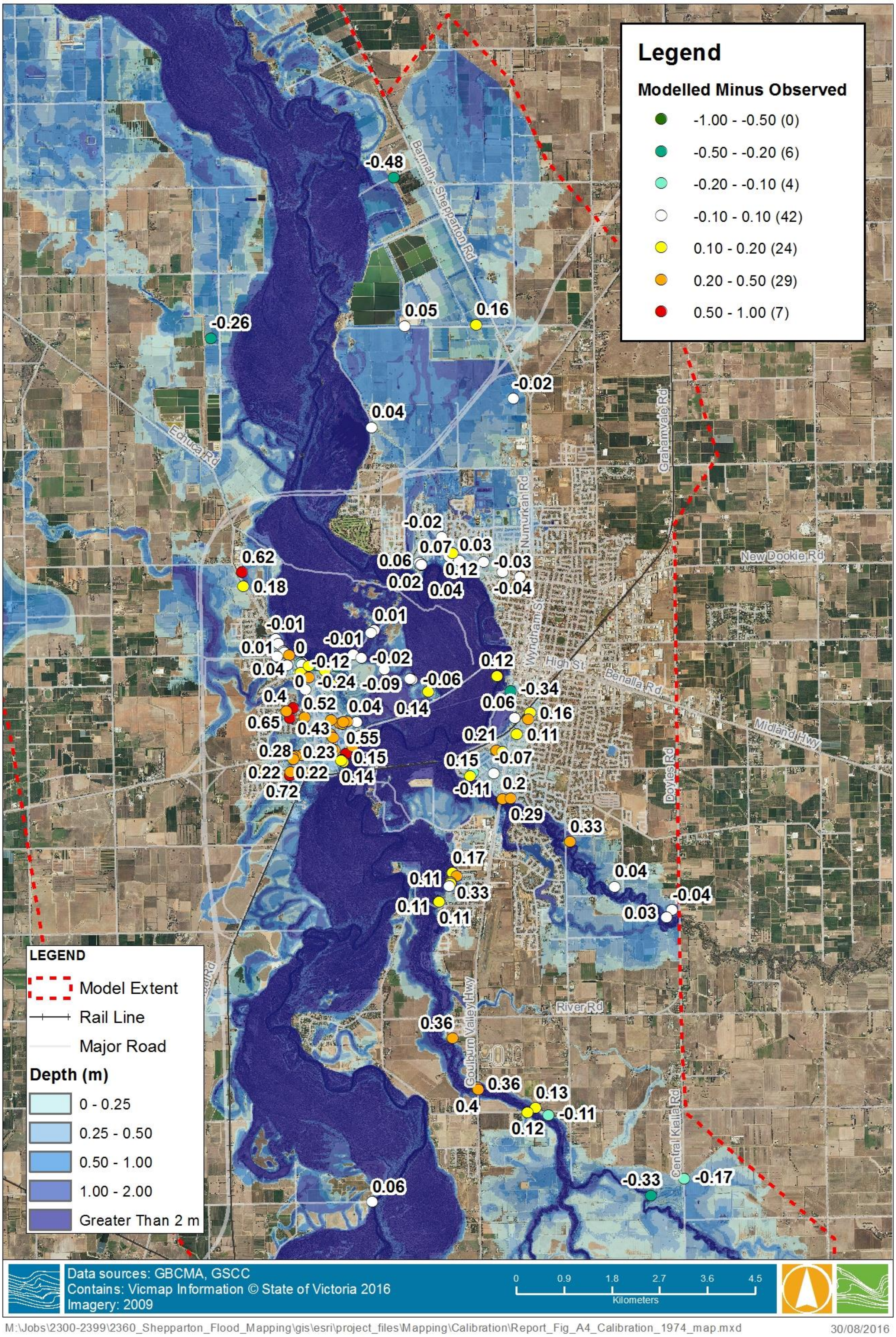

Figure 5-19 Hydraulic model validation plot – May 1974 event

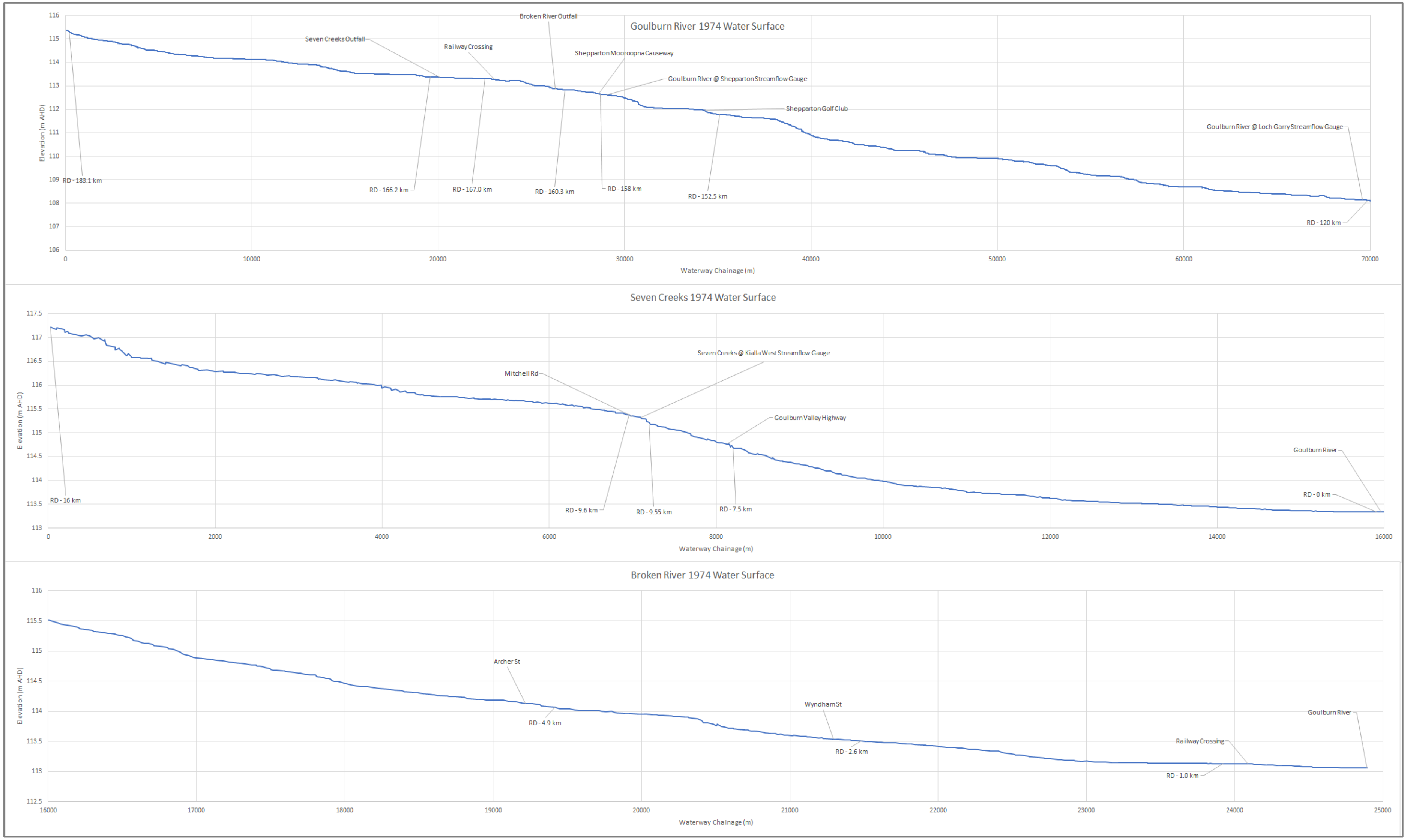

Figure 5-20 May 1974 - Water Surface Profiles

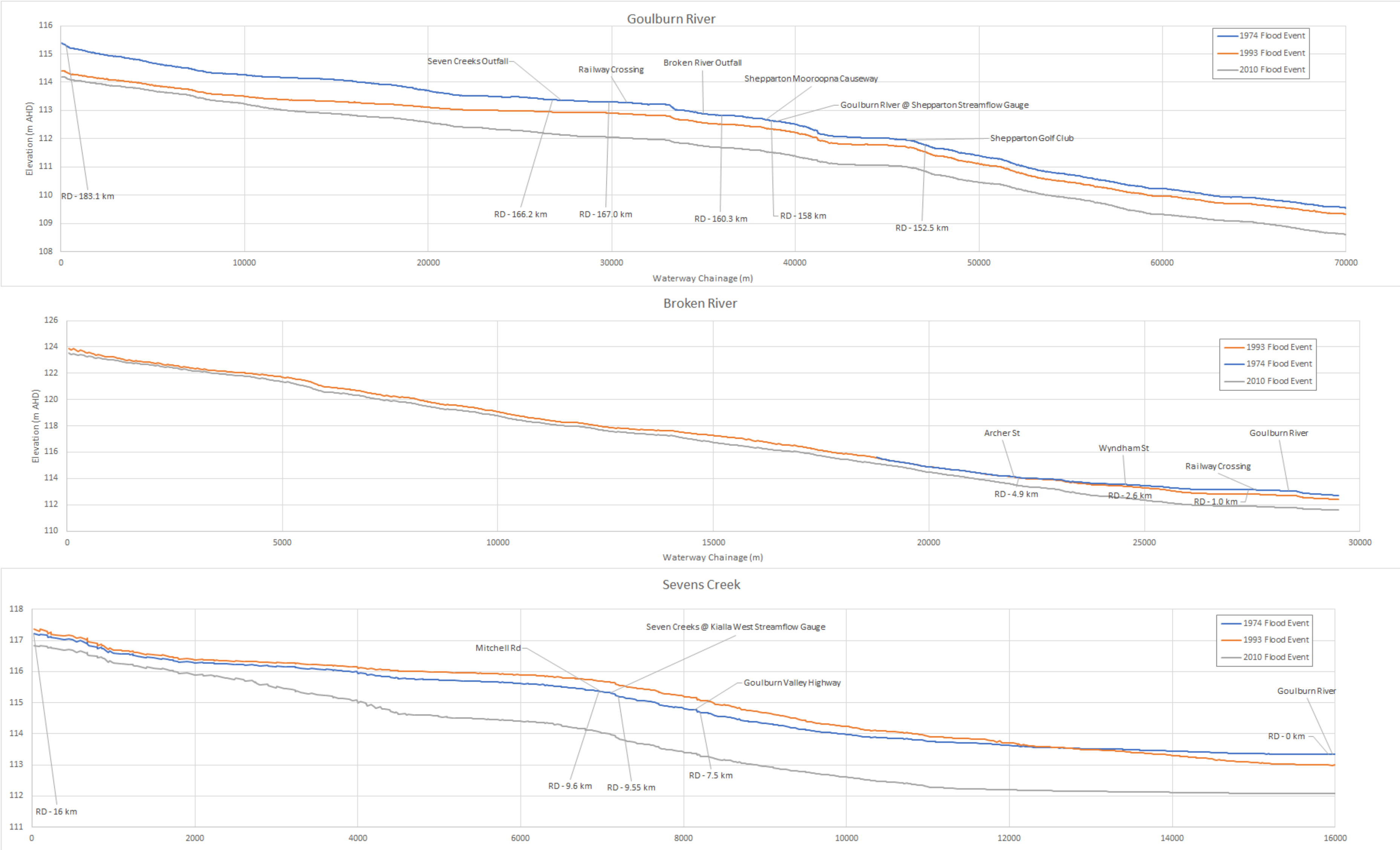

Figure 5-21 Calibration Event Summary - Water Surface Profiles

5.4.5 Hydraulic Model Calibration Summary

The hydraulic model calibration and validation results demonstrated the ability of the model to represent the flood behaviour for Shepparton and surrounding areas for the May 1974, October 1993 and September 2010 flood events. The modelling demonstrates that the events were quite different in nature with May 1974 being a Goulburn River dominated event whilst October 1993 and September 2010 were more neutral with significant contributing flows from the Broken River.

The October 1993 event and May 1974 event inundated approximately 30 and 600 residential and commercial buildings above floor respectively because of large breakouts from the Goulburn River, Broken River and Seven Creeks (SES, 2013). The September 2010 event resulted in damage to 13 houses and 31 structures. It is noted that recent development along the Broken River near Shepparton means that a flood of the magnitude of October 1993 would mean many more properties would be affected if it were to occur again.

The model results for the May 1974, October 1993 and September 2010 floods replicated the observed flood behaviour through Shepparton and surrounding areas quite accurately; this was confirmed by a comparison to observed flood marks, aerial images as well as gauged data. A summary of the peak flood levels at the Shepparton gauge is shown in Table 5-3. The model was considered appropriate for use for design event modelling.

Table 5-3 Comparison of peak flood levels at Shepparton gauge

Flood Level	1974	1993	2010
Observed	112.21 m AHD	111.84 m AHD	111.21 m AHD
Modelled	112.19 m AHD	111.95 m AHD	111.19 m AHD
Comparison	-0.02 m	+0.11 m	-0.02 m

Throughout the course of the lengthy calibration process, the sharing of information with Greater Shepparton City Council and the Goulburn Broken CMA has allowed for independent checking of results. This interrogation approach has led to successful calibration which is considered fit for purpose of design event modelling.

6. DESIGN FLOOD MODELLING

The design hydrographs were generated for 15 flood events ranging from a gauge height of 9.5 m up to 12.5 m at the Shepparton gauge Table 6-1. Each of these gauge increments was replicated across three scenarios;

- A Goulburn dominant event – where the Goulburn River was the dominant flooding mechanism at the Shepparton gauge like the 1974 flood event.
- A Broken/Seven dominant event – where the Broken River and Seven Creeks are the dominant flooding mechanism within Shepparton, like the 1993 flood event.
- A neutral flood event – where the flood magnitude of all events is approximately the same.

The approximate AEP of the 45 combinations as well as the PMF design flows are also shown in Table 4-26. Several preliminary design events focused around the 1% AEP design level at the Shepparton gauge have been modelled so far as well as some sensitivity modelling as outlined in 4.4.3.

Table 6-1 Goulburn River at Shepparton Design Levels to be modelled

Event	Goulburn River @ Shepparton Gauge Height (m AHD)	Goulburn River @ Shepparton Gauge Level (m)
Minor Flood	109.627	9.5
20-10%	110.227	10.1
Moderate Flood	110.827	10.7
10%	111.027	10.9
Major Flood 2010	111.127	11.0
10-5%	111.227	11.1
5%	111.427	11.3
5-2%	111.627	11.5
1993	111.827	11.7
2%	112.027	11.9
1974	112.227	12.1
1%	112.327	12.2
0.5%	112.427	12.3
0.2%	112.627	12.5
PMF		

7. CONCLUSION AND RECOMMENDATIONS

Water Technology believe that the hydrology as completed and the hydraulic model development and calibration is sufficient across the Shepparton-Mooroopna study area including the Goulburn River, Broken River and Seven Creeks. Given the level of uncertainty in some of the older 1974 observations and the significant changes to the topography within the floodplain, the model calibration is deemed fit for purpose and the model suitable for modelling design conditions.

Water Technology is currently completing the large design flow matrix and once complete the project will progress through to the flood intelligence stage.

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