Goulburn and Broken River Flood Study

Hydrology and Hydraulic Calibration Report

Draft A

15/11/2024

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Contents

1. Introduction

The Goulburn Broken Catchment Management Authority (GBCMA) commissioned HARC to undertake the Goulburn and Broken Rivers Flood Study. The study is being undertaken in conjunction with Venant Solutions.

The objective of this project is to produce flood mapping for the floodplain of the Goulburn River between Lake Eildon and Loch Garry and for the floodplain of the Broken River between Lake Nillahcootie and the Goulburn River.

This project forms part of the Victorian Government's Regional Flood Mapping Program. Outputs from this study will be used by GBCMA, Local Government Authorities (LGAs) and the Victoria State Emergency Service (VicSES) to meet a range of business requirements. Uses for the outputs include definition of flood related controls in municipal planning schemes, development of flood intelligence products and to inform emergency response planning as well as the preparation of community flood awareness and education products. Furthermore, outputs will also be used to support the assessment of flood risk for insurance purposes.

This report documents the hydrological and hydraulic investigations calibration undertaken as part of the study. This report is a combination of two previously provided separate reports, one on hydrology the other on the hydraulics. The previous reports were submitted prior to the October 2022 flood. The October of 2022 flood had a devastating impact on the community, with hundreds of properties inundated and livelihoods significantly affected. It was agreed with the GBCMA that the 2022 event should be included as part of the calibration events. Therefore, this report has been updated to include the October 2022 event.

2. Project context

2.1 Catchment overview

The Goulburn River is one of the major river systems within the Victorian component of the Murray-Darling Basin. Excluding the Murray River itself, the Goulburn is Victoria's largest and longer river. The headwaters are located in the Victorian Alps near Woods Point and it discharges into the Murray River at Echuca Village. The river generally flows in north westerly direction through central Victoria.

There are a number of significant tributaries of the Goulburn River, the largest of which is the Broken River. The Broken River discharges into the Goulburn River at Shepparton, although it should be noted that significant flow volumes break out of the Broken River between Benalla and Shepparton and enter the Broken Creek catchment. The catchment area of the Broken River is appropriately 2,550 km^2. The total catchment area of the Goulburn River at Shepperton is approximately 16,100 km^2.

The Goulburn River catchment contains the 3,300 GL capacity dam at Lake Eildon, which features a gated spillway capable of passing over 3,000 m^3/s. Goulburn-Murray Water (GMW) have a detailed flood operations plan for the dam, which incorporates the ability to surcharge the storage by up to 600 mm to mitigate floods downstream of the dam. This, coupled with the variability in storage level in the dam, results in the structure having an impact on floods in the Goulburn River particularly areas close to the dam. Similarly, Goulburn Weir near Nagambie also features a gated spillway whose operations are likely to influence the passage of relatively frequent floods at Murchison and Shepparton. The Broken River catchment contains the 40 GL Lake Nillahcootie. Although this storage has an ungated spillway, it exhibits significant variability in storage levels which can influence flood magnitude downstream of the dam.

Downstream of Eildon and Nillahcootie, design flood estimates transition from being primarily influenced by rainfall upstream of the dams (and released through the dam spillways), to being primarily influenced by rainfall which falls on the residual catchment downstream of the dams.

In general, the upper reaches of the catchment consist of rolling hills whilst the lower portion of the catchment is substantially flatter. For example, the catchment upstream of Lake Eildon rises from a level of approximately 250 m AHD at the dam to approximately 1,670 m AHD over a distance of approximately 60 kilometres at an average slope of 1 in 25. Whereas downstream of Lake Eildon the catchment falls from approximately 250 m AHD at the dam to approximately 110 m AHD at Shepparton over a distance of approximately 135 kilometres at an average slope of 1 in 1,000.

The average annual rainfall varies from approximately 1,370 mm/year upstream of Lake Eildon down to 495 mm/year at Shepparton. The catchment area is shown in Figure 3-1.

2.2 Previous studies

The GBCMA and Department of Energy, Environment and Climate Action (DEECA) provided several reports to provide background information for this project. The main reports of relevance are listed below:

- Benalla Flood Plain Management Study, 1984 (SR & WSC,1984)
- Benalla Floodplain Management Study, 2002 (Cardno, 2002)

- Broken River Catchment Floods October 1993 Volume 4, 1995 (HydroTechnology, 1995a)
- Lower Goulburn River Flood October 1993 Volume 5, 1995 (HydroTechnology, 1995b)
- Appendix F to Seymour – A Report on Flooding from Goulburn River, Lake Eildon Effect on Flood Frequencies at Eildon, 1981 (SR & WSC,1981)
- Seymour Floodplain Mapping Study, 2001 (WBM,2001)
- Shepparton Mooroopna Floodplain Management Study – Floodplain Management Plan – Stage 1 Technical Report, 2002 (SKM, 2002a)
- Shepparton Mooroopna Floodplain Management Study – Floodplain Management Plan – Stage 2 Technical Report, 2002 (SKM, 2002b)
- Shepparton Mooroopna Floodplain Management Study – Floodplain Management Plan – Executive Summary, 2002 (SKM, 2002c)
- Yea Flood Study, 2005 (Water Technology, 2005)
- Shepparton Mooroopna Flood Mapping and Flood Intelligence Study, 2018 (Water Technology, 2018)
- Lake Nillahcootie Flood Study – RM2179 Version 1.0 Final, 2008 (Cardo, 2008)
- Goulburn River Constraints Management – Environmental Flow Inundation Modelling and Mapping, 2016 (Water Technology, 2016)
- Murchison Flood Mapping Study Report, 2014 (Water Technology, 2014)

2.3 Existing flood models

GBCMA provided three existing hydraulic models to aid this study. These models include:

- Goulburn River Environmental Flows Constraints (Water Technology, 2016);
- Murchison Township Flood Mapping Study (Water Technology, 2014); and
- Shepparton-Mooroopna Flood Mapping and Flood Intelligence (Water Technology, 2017).

The Goulburn River Constraints models were developed using TUFLOW GPU by Water Technology in 2016. The various models extend from Lake Eildon and terminate at Echuca on the Victoria-NSW border. The purpose of the project was the assessment of environmental flows and as such the models were not calibrated for large flood events which are the focus of the current study. For environmental flows assessment the water levels of interest are typically below bank full, therefore there are no hydraulic structures in the supplied model. However, the model contains an array of layers of use for this study including land use material layers and bathymetric string lines.

The Murchison flood study is a detailed town study extending approximately 4 km upstream of the town and terminating approximately 10 km downstream. The model was developed using TUFLOW. The model includes an array of layers of use for this study including hydraulic structures, land use material layers and bathymetric string lines.

The Shepparton-Mooroopna model extends from Arcadia to the south on the Goulburn, from Kialla East on the Broken and covers the Shepparton-Mooroopna area terminating near Loch Garry to the north. The model was developed as a multi-domain TUFLOW (classic) model with a 10m grid upstream of Shepparton with a coarser 20m grid adopted downstream of the main populated areas. Similar to the other studies, the model contains input layers that could be utilised for this study including land use and structural information.

3. Hydrologic data review

A review of the available streamflow and rainfall data was undertaken to determine the events to be used for calibration of the hydrologic model (refer Section 5) and which streamflow gauges to include in the model verification process (refer Section 7).

3.1 Streamflow gauges

There is numerous streamflow gauges located throughout the Goulburn River and Broken River catchments. The streamflow gauge stations used in the hydrological investigation are shown in Table 3-1

There were a small number of gauges not used for calibration due to their location in the catchment, missing data or significant uncertainty associated with the rating curve at high flows. For example, the gauge, Broken River at Benalla, was not used as it was missing a number of the events that were used in the model calibration. However, it was referred to for the events that did have data. Broken River at Nillahcootie was not used, as is, in the RORB model as it records levels only and for calibration a flow estimate is required. However, for the historic events, flows were estimated downstream of Nillahcootie using the stage discharge relationship supplied by Goulburn Murray Water (GMW – refer to Section 4).

Table 3-1 Streamflow data used for calibration and verification

Station Number	Name	Date of Available Data	Max Gauged Level (m)	Max Gauged Flow (m^3/s)	Max Recorded Level (m)	Max Recorded Flow (m^3/s)	Date of Max Recorded Flow	Catchment Area (km^2)
405204	Goulburn River @ Shepparton	8/06/1921 to date	11.9	1,678	12.1	2,212	May 1974	16,074
405269	Seven Creeks @ Kialla West	28/06/1977 to date	6.7	228	6.7	228	Sep 2010	1505
405246	Castle Creek @ Arcadia	12/12/1973 to date	2.5	54	2.5	54	Oct 1993	164
404207	Holland Creek @ Kelfeera	9/05/1960 to date	4.7	346	6.1	700	Oct 1993	450
404218	Broken River @ Lake Nillahcootie (Head Gauge)	01/05/1967 to date	-	-	266.46	65	Oct 1993	422
404203	Broken River at Benalla	1/5/1955 to date	5.4	1,249	5.5	1294	Oct 1993	1,461
404242	Broken River @ Casey's Weir Combined Flow	01/01/1894 to date	1.9	197	4.2	1,216	Oct 1993	1,916
404224	Broken River @ Gowangardie	20/01/1928 to date	6.6	690	6.6	690	Oct 1993	2,305
404222	Broken River @ Orrvale	23/06/1977 to date	8.4	498	8.4	498	Oct 1993	2,385
405200	Goulburn River @ Murchison	14/06/1881 to date	10.9	1,450	12.5*	4,753*	Sep 1975	10,661
405253	Goulburn River @ Goulburn Weir	01/03/1967 to date	10.6	839	-	-	-	10,630
405202	Goulburn River @ Seymour	19/12/1957 to date	6.5	777	8.3	1780	Oct 2022	8,535
405201	Goulburn River @ Trawool	1/01/1908 to date	7.5	991	7.7	1030	Oct 2022	7,303
405209	Acheron river @ Taggerty	12/12/1945 to date	2.8	95	3.3	207	Oct 2022	545
405212	Sunday Creek @ Tallarook	21/11/1945 to date	4.8	161	5.5	291	Oct 2022	331
405226	Pranjip Creek @ Moorilim	10/12/1957 to date	4.9	96	6.0	202	Oct 1993	822
405228	Hughes Creek @ Tarcombe Road	16/09/1958 to date	3.3	100	4.8	540	Oct 2022	478
405240	Sugarloaf Creek @ Ash Bridge	13/10/1966 to date	5.3	412	7.0	836	Oct 2022	609
405274	Home Creek @ Yark	16/06/1977 to date	3.1	188	3.8	462	Dec 2017	187
405241	Rubicon River @ Rubicon	1/05/1922 to date	1.7	76	1.8	100	Sep 2010	129
405209	Acheron River @ Taggerty	19/12/1973 to date	3.1	163	3.3	208	Oct 2022	619
405248	Major Creek @ Graytown	19/04/1971 to date	3.3	52	4.1	250	Oct 1974	291

* This flow is reported to be the 1975 event. It is thought by HARC to be an error and is discussed further in Section 7.3.3

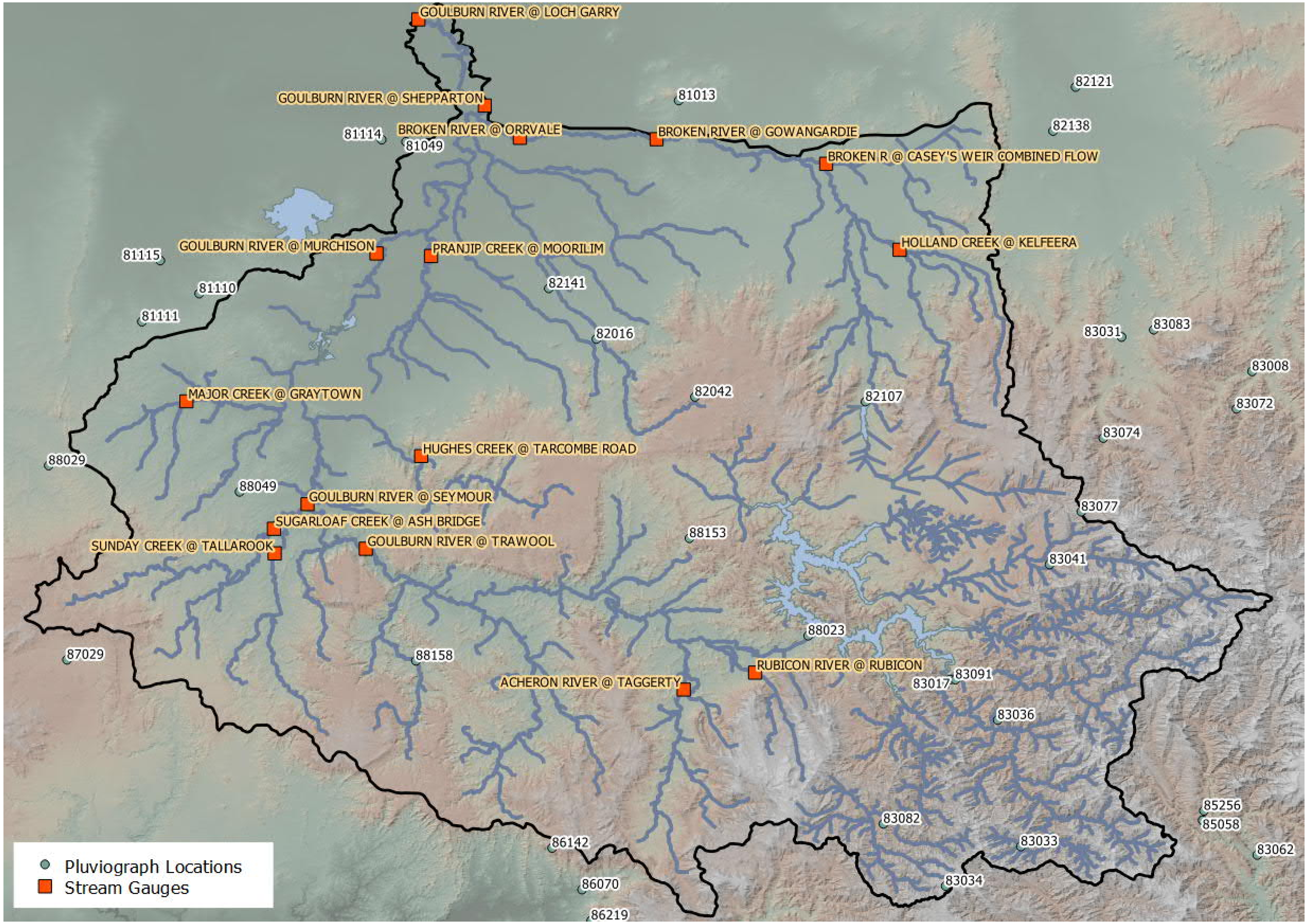


Figure 3-1 Gauge location map for the Goulburn River and Broken River catchments

3.2 Rating curve review

To assist in calibrating and verifying the hydrologic model, rating curve reviews were carried out on seven of the key gauging stations within the study area. Note, not all of the stream flow gauges within the catchment were reviewed – only selected key gauges within the domain of the study area two-dimensional hydraulic model were included. The stream gauges selected for the rating curve review are shown in Table 3-2.

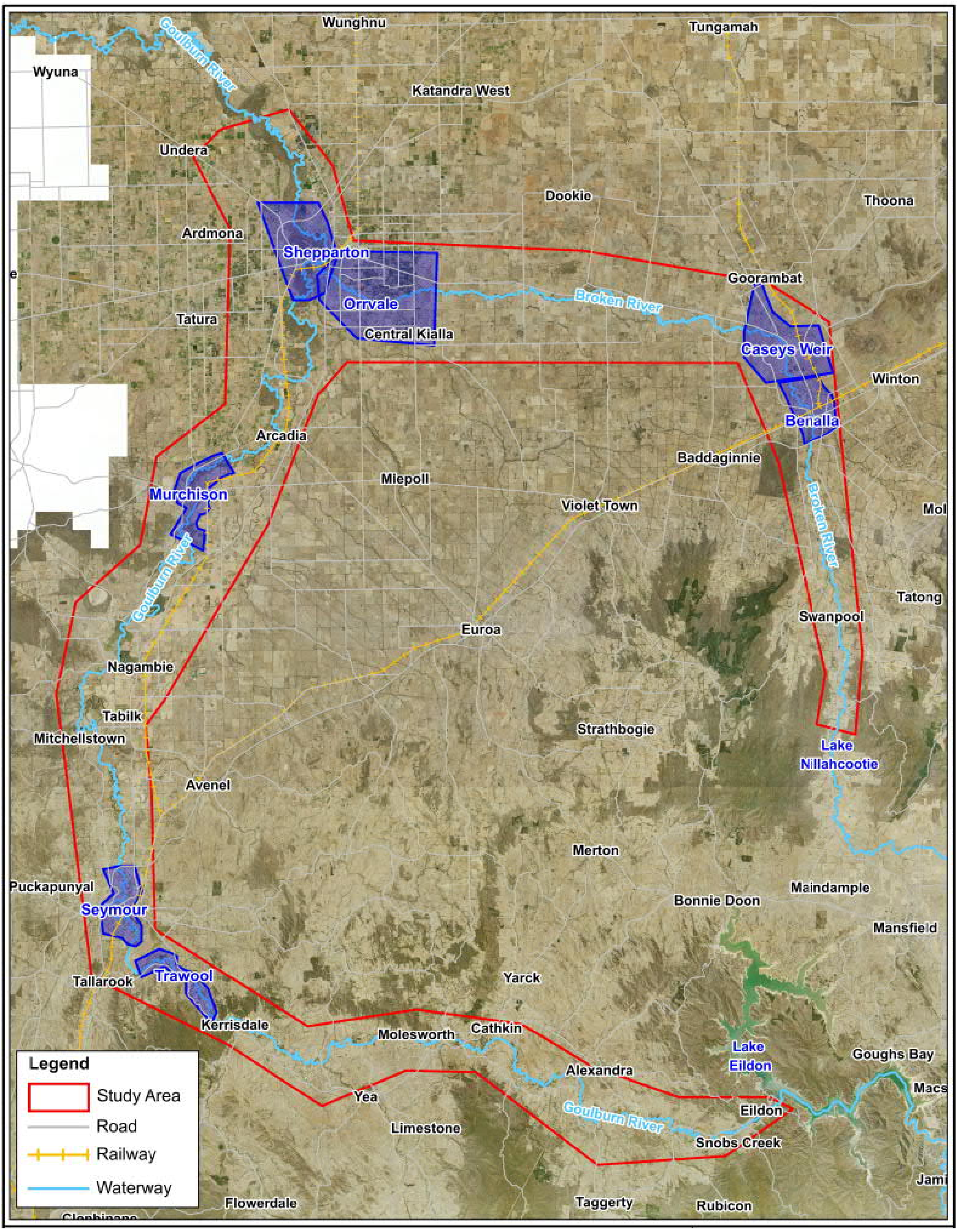
Table 3-2 Selected streamflow gauges for rating curve review

Gauge number	Gauge name
405201	Goulburn River at Trawool
405202	Goulburn River at Seymour
405200	Goulburn River at Murchison
405204	Goulburn River at Shepparton
404203	Broken River at Benalla
404216	Broken River at Casey's Weir
404222	Broken River at Orrvale

To assist in the review of the rating curves, hydraulic models were developed using TUFLOW from which a depth/height-flow relationship was developed. In total seven individual models were developed, the extents of each model can be seen in Figure 3-2. The key features of the model setups were:

- The model extents were approximately 5 km upstream and downstream of the gauge site;
- A grid cell size of 5 metres was adopted;
- Due to the staging of the study, the models were developed using typical industry parameters and no detailed calibration of the models was undertaken;
- Hydraulic structure losses were determined based on information gathered and documented above. Losses were based on those from Guide to Bridge Technology Part 8 Hydraulic Design of Waterway Structures (Austroads, 2018) and applied to the models as layered flow constrictions;
- For the Goulburn River gauges, the bathymetry from the Goulburn River Environmental Flows Constraints (WT, 2016) models were used. On the Broken River, due to the lack of bathymetry for all bar the Benalla gauge the 'zero gauge' level published for each gauge was used to set an approximate bed level within the river;
- The exact locations of the river gauges were found using a combination of DEECA data, aerial and street imagery as well as site visits;
- The hydraulic model flow output lines were extended across the full floodplain, not just bank-to-bank;
- Inflow hydrographs were applied as scaled historic floods, for each gauge the largest flood on record was applied to the model and scaled by 150% to allow for curves to be generated for greater than recorded floods; and
- Downstream boundaries were applied using the H-Q relationship automatic generated within TUFLOW – the downstream boundaries were located sufficiently far downstream so as to not influence the flood levels and flows at the gauge sites.

In each case, the official rating curve table published on the WMIS website was downloaded. Each of the gauge's rating tables were compared to the rating curve produced by the model. Accuracy of the modelled rating curve when compared to the official values was expected within the confines of the river, however deviation is expected when the flow spreads to the floodplain due to the difficulties involved in physically measuring flood discharges. Of note is that a number of the rating curves changed following the October 2022 event. The hydraulic model review was undertaken prior to the October 2022 event. Where the rating curve has been updated a comment has been added.

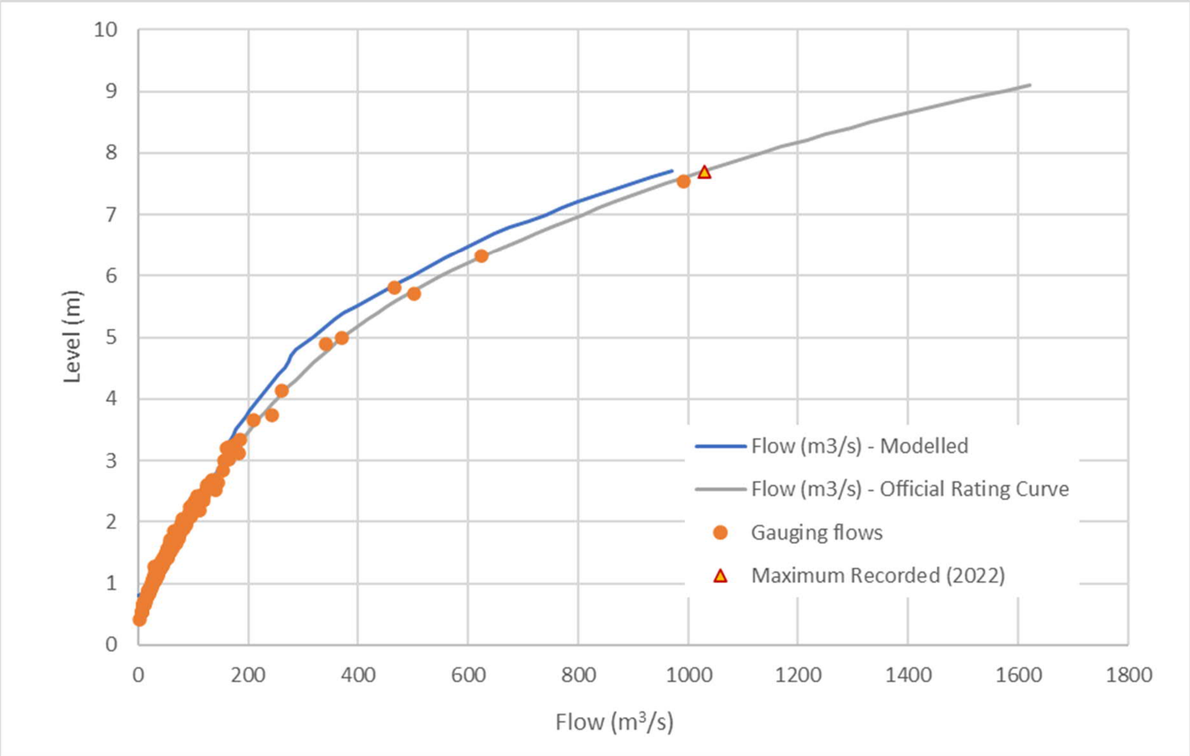


- Figure 3-2 Location and extent of the rating curve review hydraulic models

3.2.1 Goulburn River at Trawool

The Trawool gauge is located in the base of relatively confined section of the Goulburn River. The gauge is located immediately upstream of the Goulburn Valley Highway. The comparison between the published rating curve and the hydraulic model is presented in Figure 3-3. The hydraulic model was found to fairly accurately match the published rating curve, particularly up to the 3 metre gauge level. Above this level the hydraulic model was found to produce slightly lower flows for a given depth, although this discrepancy could easily be explained by the assumptions in the bathymetry.

For this investigation the published rating curve was adopted, as it matches the gaugings very well and for flows above the gauge flows the slope of the curves is similar.



- Figure 3-3 Rating curve comparison – Goulburn River at Trawool

3.2.2 Goulburn River at Seymour

The rating curve at Seymour changed a reasonable amount during the project. A couple of higher flows were gauged during the October 2022 event which appear to have altered the rating curve. The match between the modelled and published rating curves is very good for Seymour up to the 6.0 metre mark, as can be seen in Figure 3-4. At greater flow depths the curves diverge with the model predicting greater flows for a given depth once the floodplain is engaged. Of note is that if only flow west of Emily Street (refer to Figure 3-5) is considered then the modelled curve is very similar to the previous (2014 – 2017) rating and the current rating curve above 7.2 m.

The current rating curve has an inflection point at 6.7 metres which is approximately the top of bank. For this gauge the current rating curve was adopted. However, it was considered to be unreliable for larger flows, with the recorded 2022 event a lot higher than the largest gauged flow.

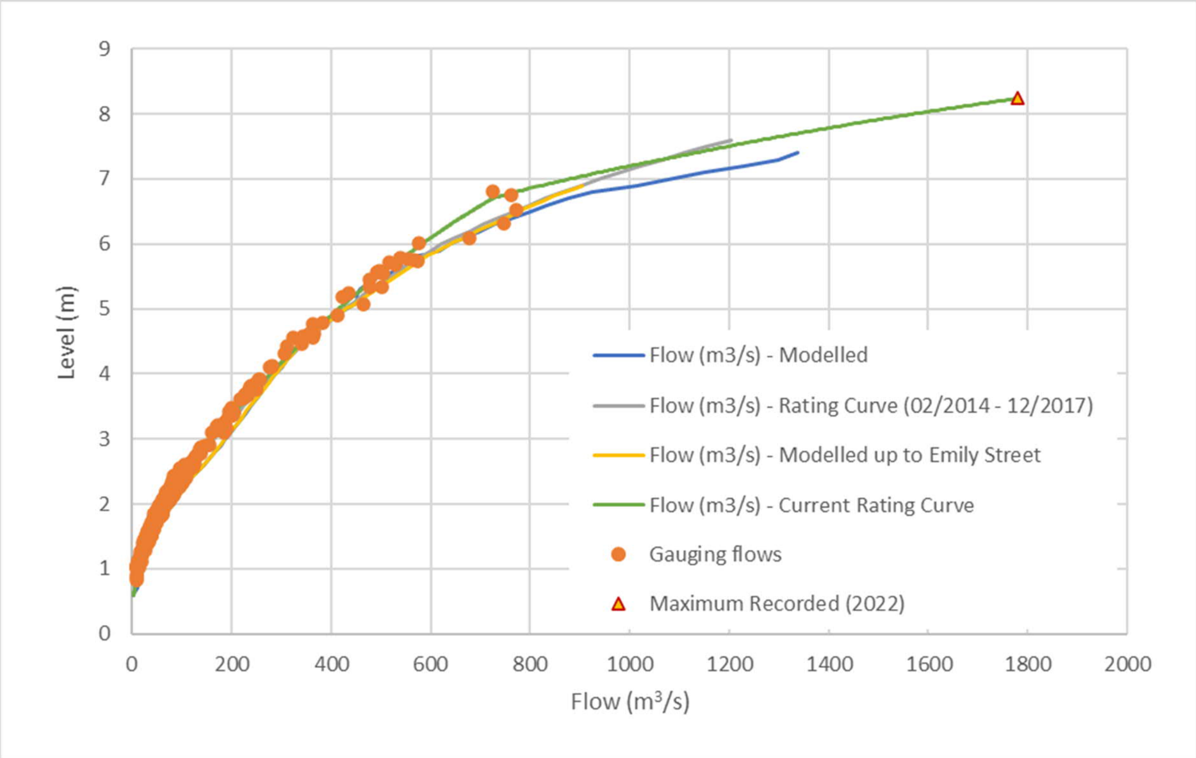


Figure 3-4 Rating curve comparison – Goulburn River at Seymour

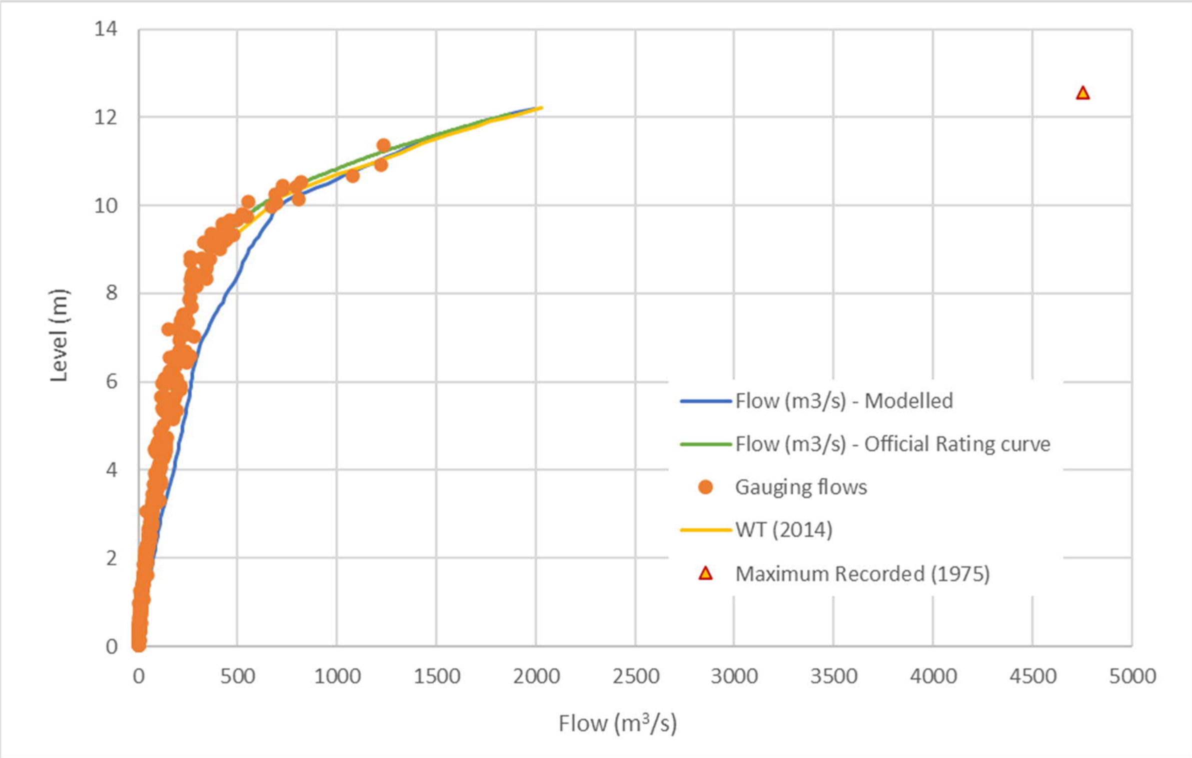
- Figure 3-5 Extent of Goulburn River at Seymour

3.2.3 Goulburn River at Murchison

The Murchison streamflow gauge is located on the Goulburn River immediately upstream of the Bendigo-Murchison-Violet Town Road which acts as a hydraulic control on the broader floodplain. To the east of the Goulburn River and west of Campbells Bend Road there are a considerable number of culverts and bridge structures that convey the overbank flows from the Goulburn River.

In 2021 the rating curve for this gauge was updated. For the higher flows the hydraulic model and the previous rating curve review undertaken at the site (Water Technology, 2014) are consistent with the current official rating curve. Therefore, the official rating curve has been adopted. However, there is still significant uncertainty in the flow at this gauge during large flood events.

The comparison between the modelled and published rating curve is presented in Figure 3-6. It is thought, by HARC, that the flow maximum recorded level in 1975 is an error. This is discussed further in Section 7.3.3.

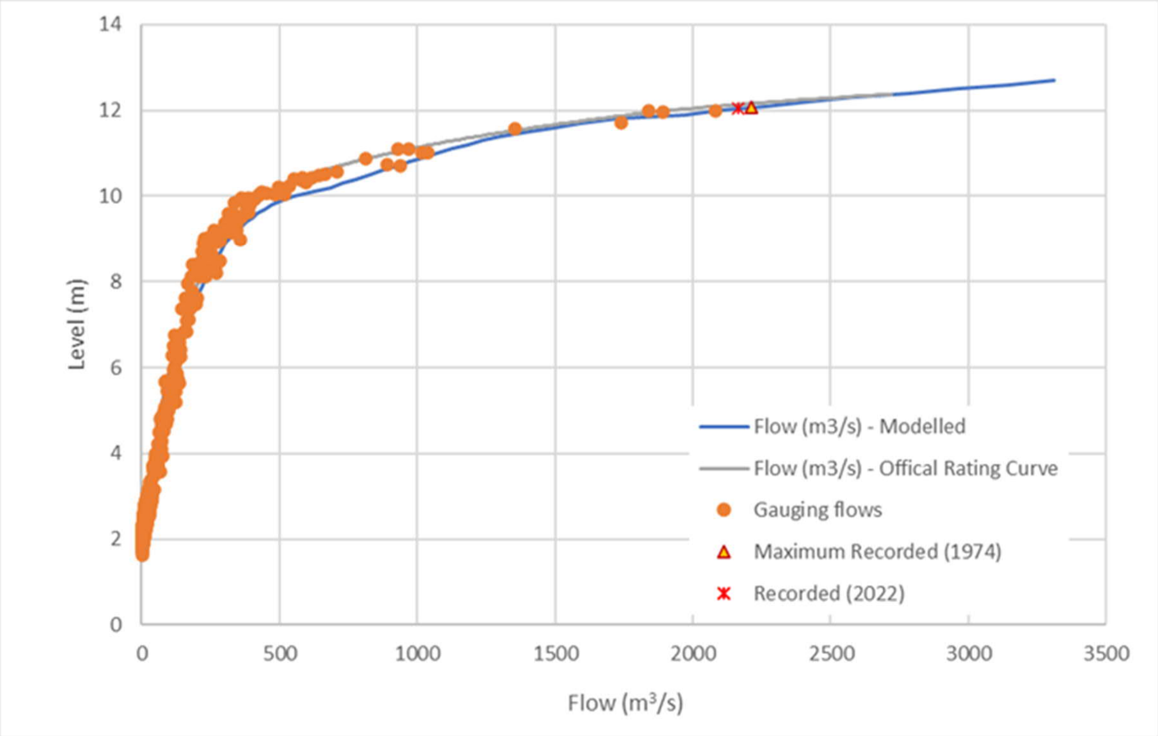


- Figure 3-6 Rating curve comparison – Goulburn River at Murchison

3.2.4 Goulburn River at Shepparton

The Goulburn River gauge at Shepparton is located on the river near the Midland Highway, which acts as a hydraulic control across the floodplain. Overall, a reasonable match was observed between the two rating curves, as shown in Figure 3-7. For reference the gauge is located 8 metres downstream of the Daintons Bridge on the Midland Highway. The threshold of flooding over the Midland Highway (also known as the Causeway) commences at a level similar to 1974 being 12.09 m.

In 2021 the rating curve was extended to include higher flows and is consistent with the modelled results. Therefore, the published rating curve has been adopted. Of note is that the October 2022 event was very close to the highest recorded event at this gauge in 1974.



▪ Figure 3-7 Rating curve comparison – Goulburn River at Shepparton

3.2.5 Broken River at Benalla

The Benalla river gauge is located on the Broken River and is sited at the Benalla Art Gallery. Once flooding overtops the river banks much of Benalla is flood prone. This results in a relatively chaotic and complex distribution of flow that passes through the various hydraulic structures in the town. Figure 3-8 compares the published rating curve with the modelled results. Overall there is a very good match between the published rating curve and the outputs from the hydraulic model up until the extremities of the rating at the 5.5 metre gauge level.

Overall the modelling indicates that the existing rating curve at Benalla is reasonable to adopt for the hydrological investigation.

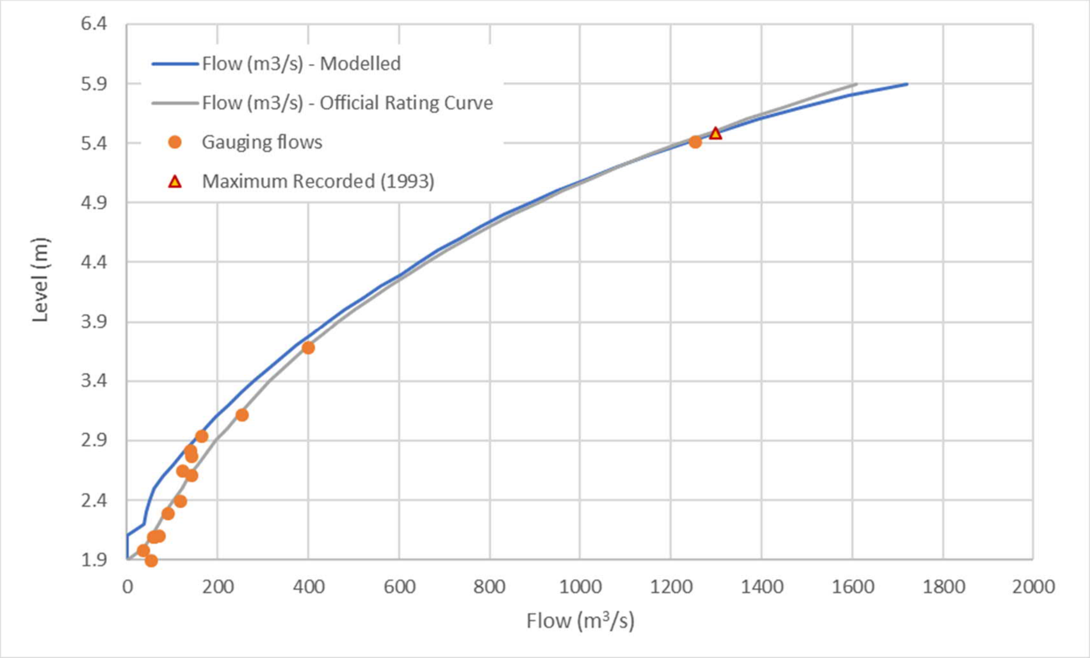


Figure 3-8 Rating curve comparison – Broken River at Benalla

3.2.6 Broken River at Casey's Weir

By far the greatest divergence of all the published rating curve and the modelled results was found at the Casey's Weir gauge on the Broken River. The comparison is presented in Figure 3-9. Based on the published notes with the rating table, the highest gauging was at the 1.9 metre mark and was undertaken in 1984.

In this instance a very good match between the published rating and the hydraulic model results was recorded up to the 2.1 metre mark. Beyond this level, the model showed greater linearity with increasing depth up to approximately the 3.5 metre mark, where the floodplain becomes engaged and the flow rate required to increase depth increases significantly.

As the modelled curve matches well up to the highest gauged flows this gives confidence that the model is representing the hydraulic conditions well. Therefore, for this project the current rating curve was adopted up to 2.0 metres. For higher levels the results from the hydraulic model were adopted.

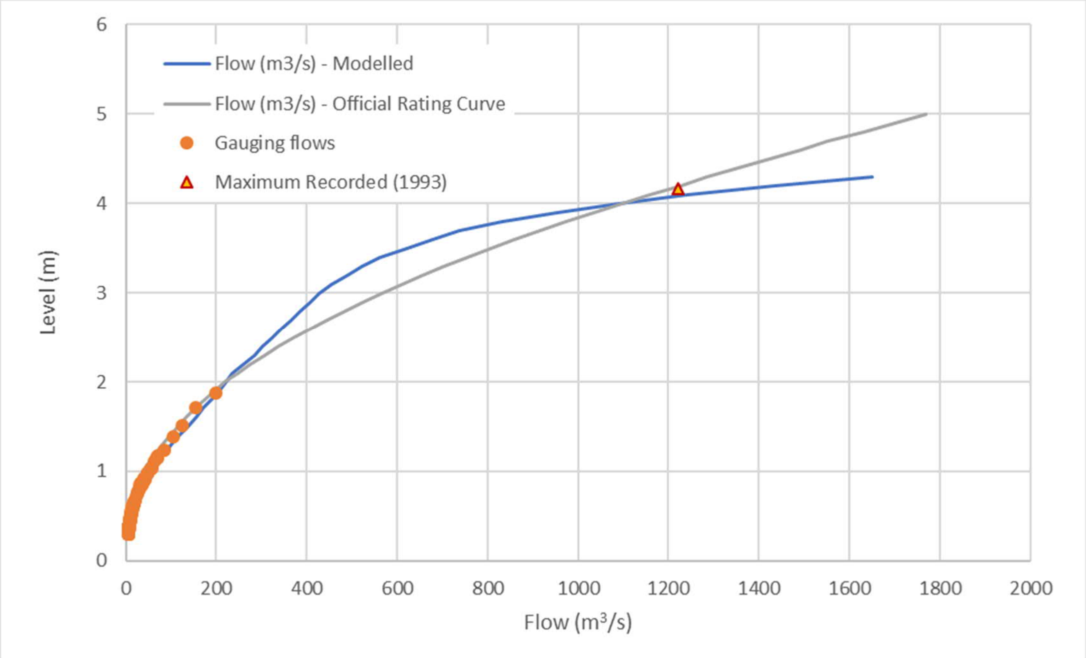
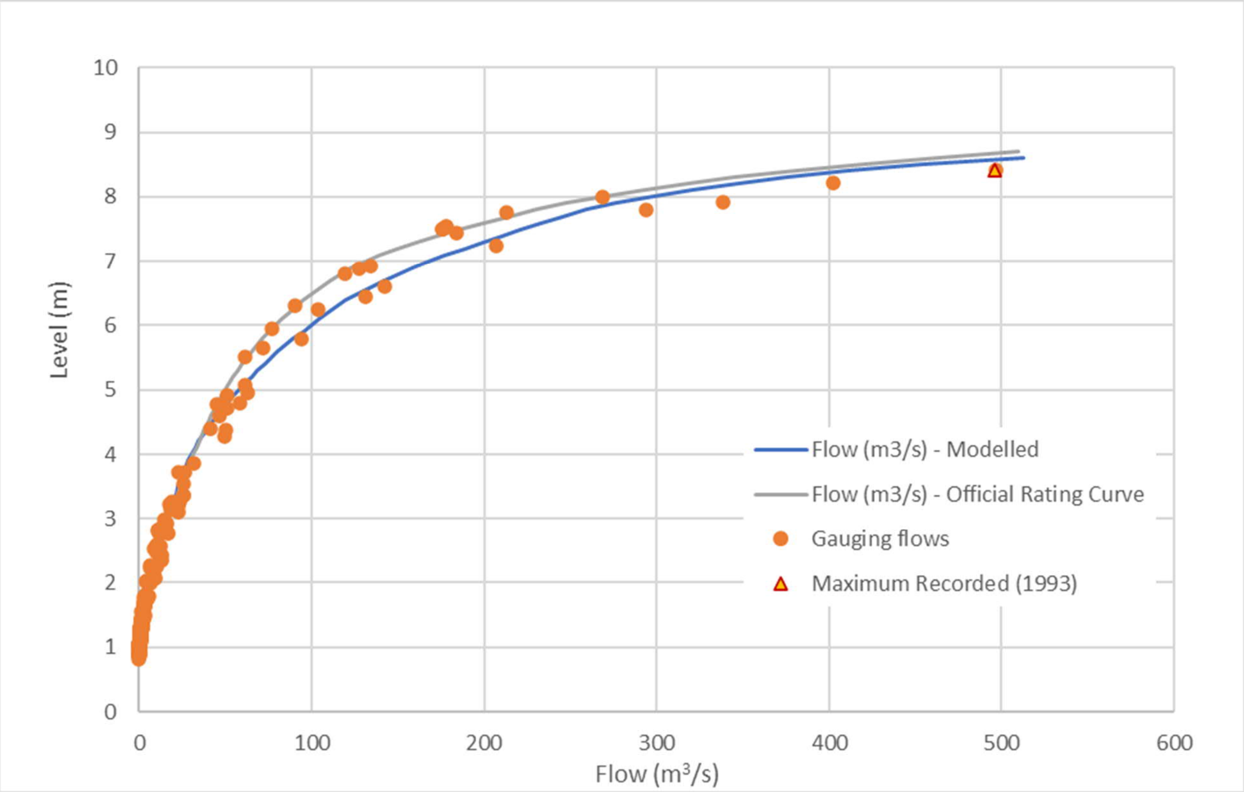


Figure 3-9 Rating curve comparison – Broken River at Casey's Weir

3.2.7 Broken River at Orrvale

The comparison between the modelled and published rating curve at Orrvale is presented in Figure 3-10. The comparison is largely favourable, particularly up to the 4.5 metre mark with top of bank at approximately the 6 metre mark. Once floodwaters have exceeded the bank there is still a reasonable match between the published rating curve and the modelled results with the rate of climb in the modelled results notably steeper than in the published curve.

Overall the modelling indicates that the existing rating curve at Orrvale is reasonable to adopt for the hydrological investigation.



▪ Figure 3-10 Rating curve comparison – Broken River at Orrvale

3.3 Daily rainfall data

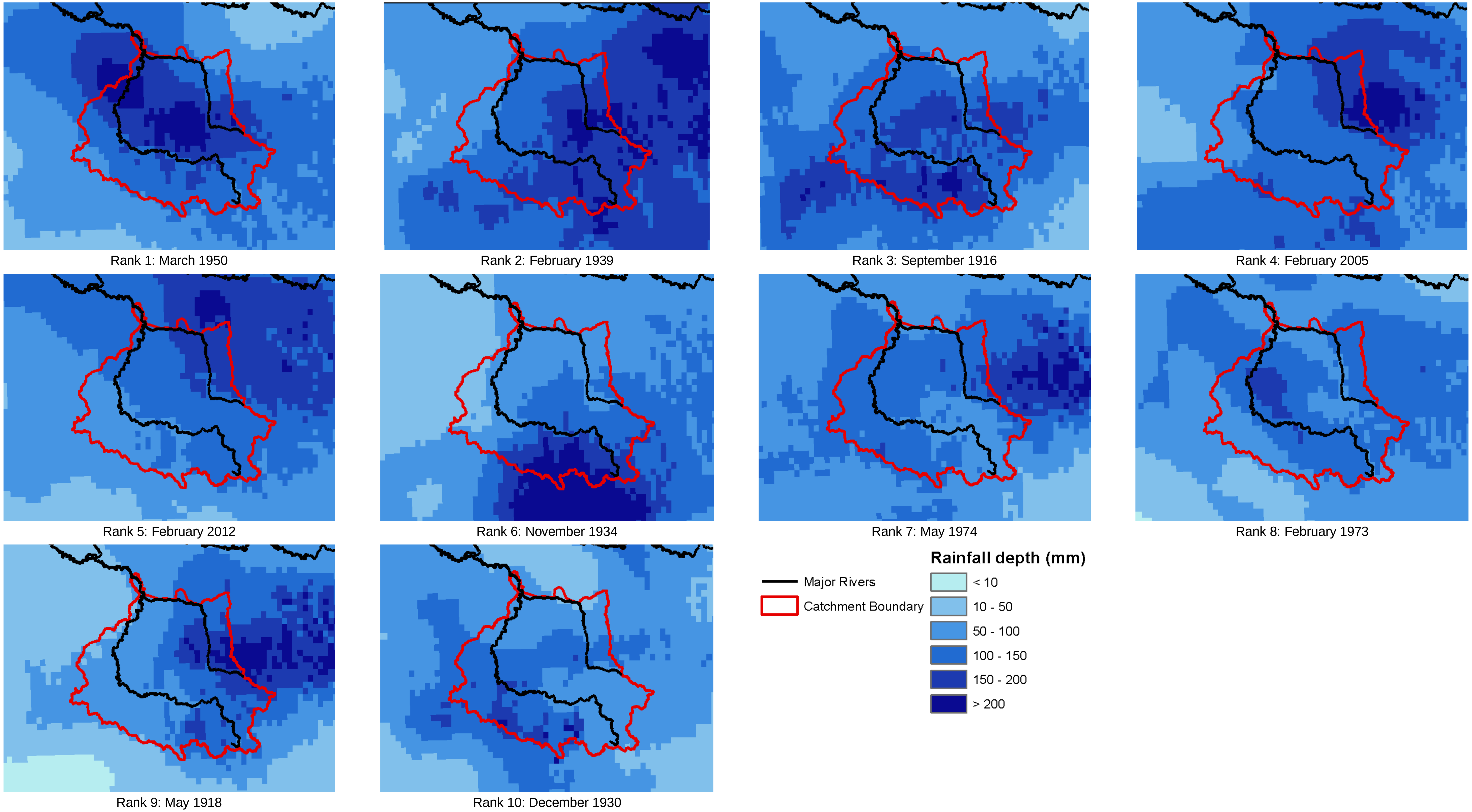
Three data sources were used to determine the rainfall depths and spatial patterns for each of the hydrologic model calibration events, namely:

- Australian Water Availability Project (AWAP) (Raupach et al., 2009);
- daily rainfall gauges; and
- pluviographs.

The primary source of data used to determine spatial patterns of rainfall were the AWAP daily gridded rainfalls. This dataset is available from 1900 to the current day and was downloaded from the Bureau of Meteorology website. The AWAP data provide a spatial (5 kilometre resolution) distribution of daily rainfall across the Australian continent. The AWAP data use model-data fusion methods to combine both measurements and modelling to estimate rainfall.

The AWAP data were checked for consistency with the rainfall recorded at daily rainfall gauges and the daily sum of rainfall recorded at the pluviographs.

Figure 3-11 shows the rainfall depths across the catchment for the largest historic 72 hour duration storm events over the study area catchment extracted from the AWAP data. Figure 3-11 shows the variability of the spatial patterns across the catchment, highlighting the importance of incorporating spatial variability into the hydrological analysis.



- Figure 3-11 Spatial rainfall patterns for the largest historic 72 hour duration storms over the study area catchment

3.4 Pluviograph data

A pluviograph is an instrument that records the amount of rainfall that has fallen over a sub-daily period of time, typically 6 minutes. The pluviographs were used to determine the temporal pattern of rainfall over the catchment for each of the calibration events (refer Section 5) and used to choose the temporal patterns for the design rainfall space-time patterns (refer Section 7). The details of the pluviographs used are shown in Table 3-3 and on the map in Figure 3-1. Rainfall data at each pluviograph was supplied by the Bureau of Meteorology.

- Table 3-3 Pluviograph data used for calibration and verification

Station No.	Name	Latitude	Longitude	Start Date	End Date
81013	Dookie Agricultural College	-36.37	145.70	Jan 1950	To date
81049	Tatura Inst. Sustainable Ag.	-36.44	145.27	Jul 1960	To date
81079	Youanmite	-36.15	145.70	Jun 1975	Jan 1976
81110	Wanalta Recorder Three	-36.68	144.94	Feb 1974	Aug 1980
81111	Wanalta Recorder Two	-36.73	144.84	Feb 1974	Aug 1980
81114	Tatura (Thiess services)	-36.43	145.23	Jan 1975	Jul 1999
81115	Wanalta Daen Station	-36.63	144.87	Jul 1974	To date
82016	Euroa	-36.75	145.57	Dec 1967	To date
82042	Strathbogie	-36.85	145.73	Jan 1974	To date
82107	Lima South (Lake Nillahcootie)	-36.86	146.00	Jul 1968	To date
82121	Ovens River (Wangaratta)	-36.35	146.34	Aug 1957	Nov 1993
82138	Wangaratta Aero	-36.42	146.31	May 1987	To date
82141	Euroa (Miepoll)	-36.67	145.49	May 1997	Jan 2001
83008	Dandongadale (Mountain View)	-36.81	146.63	Mar 1980	Jan 1985
83017	Jamieson	-37.30	146.14	Apr 1974	Feb 2004
83031	Whitfield	-36.75	146.42	Oct 1962	Nov 1991
83033	Woods Point	-37.57	146.25	Jan 1954	To date
83034	Thomson Upper	-37.63	146.13	Feb 1971	Mar 1977
83036	Kevington (Ten Mile)	-37.37	146.22	Jan 1931	Jan 1932
83041	Timbertop	-37.12	146.30	Jun 1974	Feb 1981
83062	Tamboritha	-37.58	146.63	Jan 1965	Oct 1966
83072	Dandongadale Upper Site No.1	-36.87	146.60	Apr 1975	Dec 1979
83074	Lake William Hovell Reservoir	-36.91	146.39	Jan 1988	Sep 2000
83077	Bald Hill	-37.03	146.35	Dec 2003	May 2004
83082	Big River (Stockmans Reward)	-37.53	146.03	Dec 1977	Feb 1981
83083	Edi Upper	-36.74	146.47	Jan 1991	To date
83091	Jamieson Licola Rd	-37.30	146.15	Mar 2004	To date
85000	Aberfeldy	-37.70	146.37	Oct 1969	Sep 1984
85058	Glencairn (Barkly River (Glenview))	-37.53	146.54	Apr 2004	To date

Station No.	Name	Latitude	Longitude	Start Date	End Date
85256	Barkly River (Glenlea)	-37.51	146.55	Apr 1974	Apr 2004
85278	Aberfeldy (Lily Creek (Larommi))	-37.72	146.39	Jul 1985	To date
86070	Maroondah Weir (Melbourne Water)	-37.63	145.55	Jun 1956	Dec 1975
86142	Toolangi (Mount St Leonard Dpi)	-37.57	145.50	Jan 1954	To date
86219	Coranderrk Badger Weir	-37.69	145.56	Dec 1955	Jan 1978
87029	Lancefield	-37.27	144.72	Jan 1929	Jul 1975
88023	Lake Eildon	-37.23	145.91	Oct 1957	To date
88029	Heathcote	-36.96	144.69	Apr 1968	To date
88049	Puckapunyal	-37.00	145.00	Apr 1968	Jan 1989
88153	Spring Creek Basin Two	-37.07	145.72	Dec 1973	Jun 1984
88158	Strath Creek	-37.27	145.28	Aug 1991	To date

4. Hydrologic model development

A rainfall runoff model (RORB) was set up for the entire Goulburn and Broken Rivers catchment upstream of Loch Garry. There were several existing models used to create the RORB model. These included the areas:

- Upstream of Lake Eildon (HARC, 2017a)
- Upstream of Lake Nillahcootie (HARC, 2017b)
- Downstream of Lake Nillahcootie to Casey's Weir on the Broken River (HARC, 2017c).
- Goulburn River downstream of Lake Eildon to Loch Garry (HARC, 2017c)

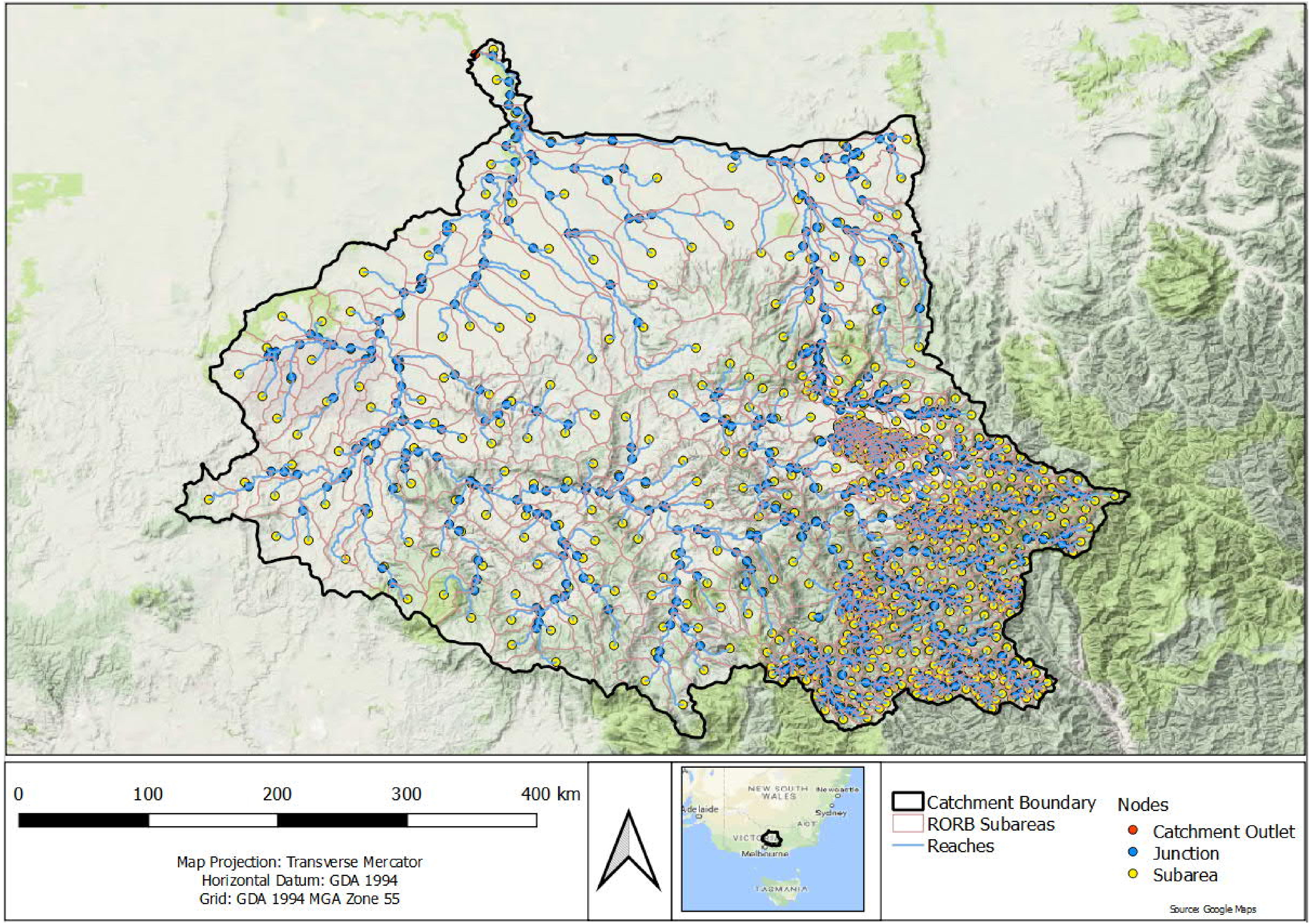
RORB (Laurenson, Mein and Nathan, 2010) is a general runoff and streamflow routing program that is used to calculate flood hydrographs from rainfall and other channel inputs. It subtracts losses from rainfall to determine rainfall excess and routes this through catchment storages to produce streamflow hydrographs at points of interest. The model is spatially distributed, non-linear, and applicable to both rural and urban catchments. It makes provision for both temporal and areal spatial distribution of rainfall as well as losses, and can model flows at any number of points throughout a catchment (including upstream and downstream of reservoirs). RORB also has the capacity to use a Monte-Carlo approach to produce design flood estimates that incorporate the joint probability of several factors that influence flood characteristics.

The combined RORB model layout is shown in Figure 4-1. In general terms, development of a RORB model entails sub-dividing the catchment into a series of subareas to suit the catchment topography and other features such as the location of gauging stations and storage locations.

Four different types of reaches can be defined in RORB, each having different properties and different relative delay times. The reach types are identified as natural, excavated but unlined, lined channel or pipe and drowned reaches. Drowned reaches were used within reservoir water bodies; natural reaches were used for all other reaches. Excavated and lined channel reaches are normally only applied in urbanised areas and hence were not used in this study.

Impervious fractions are required for each sub-area. For rural areas the impervious fraction is usually assumed to be zero. For areas that would be inundated by the reservoir created by the dams, an impervious fraction was calculated based on the percentage of the sub-area that would be inundated.

As noted previously, the catchment includes a number of large storages, diversions and cross-catchment flows. GMW supplied stage storage and stage discharge information for Lake Eildon, Lake Nillahcootie, Goulburn Weir and Lake Mokoan/Winton Wetlands. Each of these structures was placed into the RORB model and represented as a special storage. Appendix A shows the stage-discharge information for each structure.



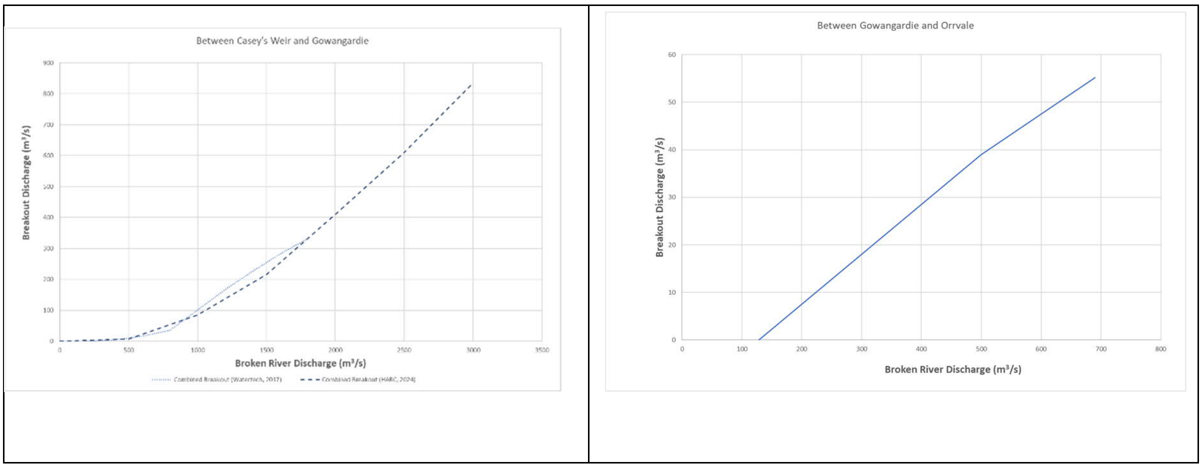
■ Figure 4-1 RORB model layout

For the calibration process recorded hydrographs were placed into the model downstream of Lake Eildon, therefore, obviating the need to model gate operations. For the verification gate operations were coded into the RORB model based on flood operation information provided by GMW.

At Goulburn Weir, the model was configured such that the Goulburn Weir gates were fully open during the flood. In discussion with GMW the operation of Goulburn Weir during a flood is such that water is diverted to Waranga Basin via the Stuart Murray and Cattanach Canals up to the maximum diversion capacity. Beyond this, water is released to the lower Goulburn River. The combined total capacity of the Stuart Murray and Cattanach Canals is 81 m^3/s. The only water that will pass into the East Goulburn Main Channel will be the volume needed to meet irrigation demand, which during a flood is likely to be minimal. The Stuart Murray and Cattanach Canals were modelled as a diversion in RORB using the maximum outflow of 81 m^3/s.

An investigation by Cardno Lawson Treloar in 2008 concluded that “decommissioning Lake Mokoan will make no practical difference to flood levels at Benalla”. As a result, for the verification process diversions were not modelled. For each of the calibration events diversions were considered and found to have little impact on the results at Casey’s Weir (which was used for calibration and verification).

On the Broken River, downstream of Casey’s Weir, floodplain flows exceeding the river channel capacity can move north into the Broken Creek catchment at two locations, between Casey’s Weir and Gowangardie and between Gowangardie and Orrvale. These cross-catchment flows were simulated in the RORB model using outflow relationships at these locations. The breakout relationships used were initial based on the investigation undertaken as part of the Numurkah Floodplain Management Study and Plan (WaterTechnology, 2017). These were then confirmed during the initial phases of the hydraulic modelling. HARC undertook the Boosey and Upper Broken Creek Regional Flood Study (HARC, 2024) and it that study the breakout relationship between Casey’s Weir and Gowangardie was modelled and adopted. For the relationship between Gowangardie and Orrvale the 2017 relationship was adopted. Figure 4-2 shows the relationship used in the RORB model.



■ Figure 4-2 Broken River cross-catchment flow relationships (left: Casey’s Weir and Gowangardie right: Gowangardie and Orrvale)

5. Hydrologic model calibration

This section describes the calibration of the hydrologic model used for this project, and presents results from the calibration process.

5.1 Calibration approach

RORB models are based on catchment geometry and topographic data, and the two principal routing parameters are k_c and m. The parameter m describes the degree of non-linearity of the catchment's response to rainfall excess, while the parameter k_c describes the delay in the catchment's response to rainfall excess.

A value of 0.8 was adopted for the non-linearity parameter, m, for this study, which is recommended by Laurenson et al. (2010) as well as Book 8 of Australian Rainfall and Runoff (Nathan and Weinmann, 2016) for modelling very large and extreme flood events. The value of the routing parameter, k_c, was selected by calibrating the RORB model to a number of historic flood events.

Note that the remaining RORB model parameters represent rainfall losses, using either an initial loss/continuing loss model, or an initial loss/proportional loss (i.e. runoff coefficient) model. An initial loss/continuing loss model was adopted for this study because it is more appropriate for modelling large floods. Selection of the loss values used for design was undertaken using the model verification process described in Section 7.

In general, the calibration approach was:

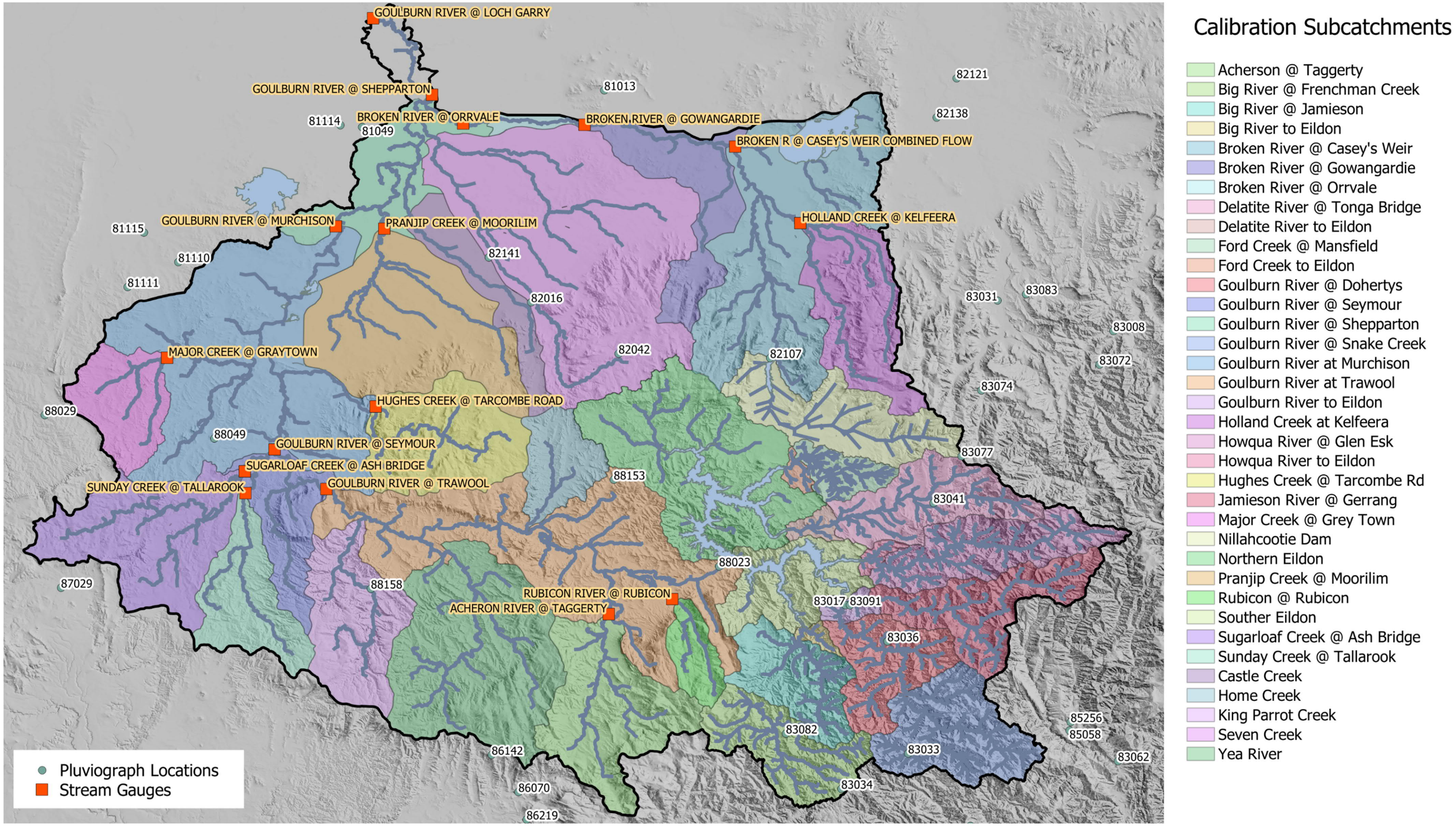
- Adjustment of the k_c to achieve a fit to the shape of the recorded hydrograph. The model was run interactively with various trial values of k_c, and the value giving best reproduction of the observed data was adopted.
- Initial loss directly affects the start of the hydrograph rise, but also affects the time distribution of rainfall excess and hence the hydrograph peak, especially for long storms with large variations of intensity. The continuing loss generally affects the hydrograph volume. The initial and continuing loss were adjusted in conjunction to attempt to match the start of the hydrograph rise and achieve a reasonable fit between the modelled and observed hydrograph volumes.

To calibrate the RORB model to the selected historic flood events, input storm files were derived from the rainfall and streamflow data described in Section 3. The model was then run with physically reasonable loss and routing parameter values to optimise the match between modelled and gauged flow hydrographs at key locations.

The model was broken up into different sub-catchments, based on the locations of the key gauges used in calibration. This allowed the values of k_c and losses to be varied spatially across the model domain. Therefore, a different k_c value was adopted for each sub-catchment. Figure 5-1 shows the different regions used for each k_c value. For each of the calibration events, reliable recorded estimates of outflow from Lake Eildon and Lake Nillahcootie were used in place of modelled outflows, to ensure that systemic bias was not introduced into the calibrated model downstream. Detailed hydrologic studies (including model calibration) has previously been completed upstream of these major storages (Jacobs, 2016 and HARC, 2017b) and so there was no need to further refine the k_c values there.

The focus of the calibration was to primarily ensure that the model could reproduce gauged flows along the Goulburn River and Broken River. There were a number of other gauges on significant tributaries (i.e. Rubicon River, Acheron River, Sugarloaf Creek, Sunday Creek, Hughes Creek, Major Creek, Pranjip Creek and Holland Creek) which were used to check that the contribution of flow from these catchments was in the correct order of magnitude.

Initially the RORB model was calibrated separately to the hydraulic model (TUFLOW). The initial results from RORB were placed into the hydraulic model for calibration of the hydraulic model. The process of calibrating the hydraulic model highlighted some differences between the two models e.g. the TUFLOW model showed less routing along the upper Broken River compared to the RORB model. The results from the TUFLOW model were then used to recalibrate the RORB model. This process was iterative and in this way a joint calibration between the hydrology and hydraulic model was undertaken. The results shown in this report are those that were used in the calibration of the hydraulic model.



■ Figure 5-1 Regions used for each k_c value

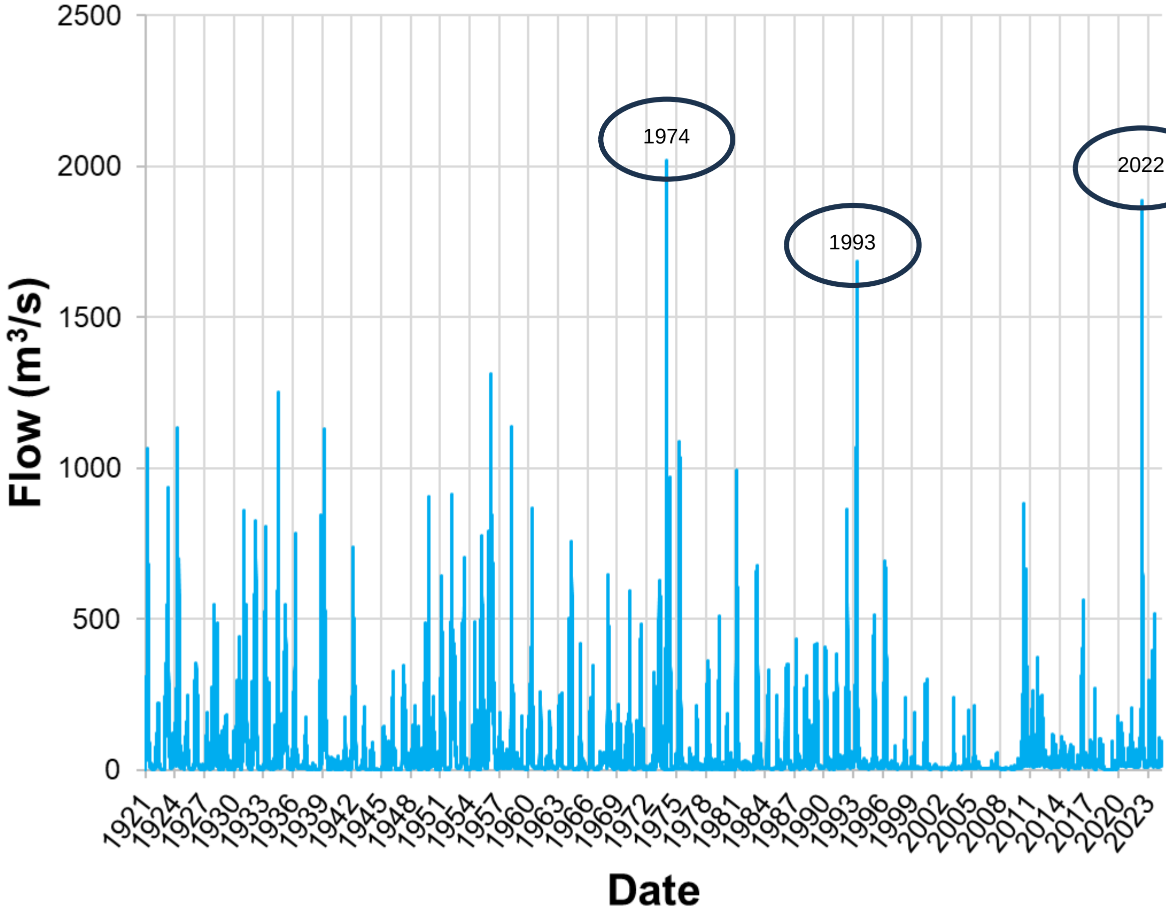
5.2 Selection of calibration events

The events chosen for calibration of the hydrological model were determined by examining streamflow data in conjunction with available pluviograph information and flood level information (which is required for the subsequent calibration of the hydraulic model). The events chosen were:

- October 2022;
- October 1993;
- May 1974;

Figure 5-2 shows the flows recorded at the Goulburn River at Shepparton gauge (405204) with the events chosen for calibration highlighted with a black circle. It is noted that the 1974 event is the largest on record, 2022 is the second largest and 1993 event the third largest. The 1916 event, is considered to be the largest known flood to have occurred on the catchment, but this event has very little data in which to calibrate to and was prior to significant changes in the catchment like the construction of Eildon Dam.

Due to the size of the study catchment other gauges exhibit a different sequence of peak events, but the 2022, 1993 and 1974 floods are consistently the largest events across the different gauges used for model calibration. At a high level 2022 and 1974 are the largest gauged flows on the Goulburn River and 1993 event is the largest gauge flows on the Broken River (refer to Table 3-1).



- Figure 5-2 Recorded streamflow data at Goulburn River at Shepparton (405204)

For the selected calibration events, streamflow data was obtained directly from the WMIS website, except for at Casey's Weir where the rating curve review (refer Section 3.2) indicated that a composite rating using hydraulic modelling results was appropriate. For this site, the streamflow hydrographs were obtained by converting raw recorded water level data from WMIS to streamflow using the composite rating curve.

As RORB does not simulate baseflow, this was manually separated out from the event streamflow hydrographs prior to model calibration. The separation was undertaken using the principles outlined in Chapter 4, Book 5 of Australian Rainfall and Runoff 2016 (Ball, 2016).

The calibration process was an iterative process between the rainfall runoff (RORB) model and the hydraulic (TUFLOW) model. As such the results shown in Section 5.3 for the 2022 and 1993 events are those adopted for input to the hydraulic model (Section 8). For some locations it was possible to get a better match in the RORB model but this compromised the results of the hydraulic model. The focus of the RORB calibration was to determine appropriate routing parameters for the Goulburn River.

5.3 Calibration results

5.3.1 October 2022

For the October 2022 event a significant distribution of pluviograph information was available to calibrate the RORB model for this event. Figure 5-3 shows the pluviograph temporal patterns available (refer to Figure 3-1 for the location of each pluviograph). The temporal pattern of rainfall in each subarea was defined using the closest pluviograph. Most of the rain fell over a 24 hour period.

Rainfall depths were estimated by extracting total rainfall depth for the event at each RORB sub-area centroid from the AWAP rainfall data. The extracted rainfall depths from AWAP were then checked against the total rainfall recorded at each pluviograph. Figure 5-7 shows the adopted rainfall depths across the catchment. This event was primarily centred around Seymour and the rivers along the Goulburn such as the Acheron and Yea Rivers.

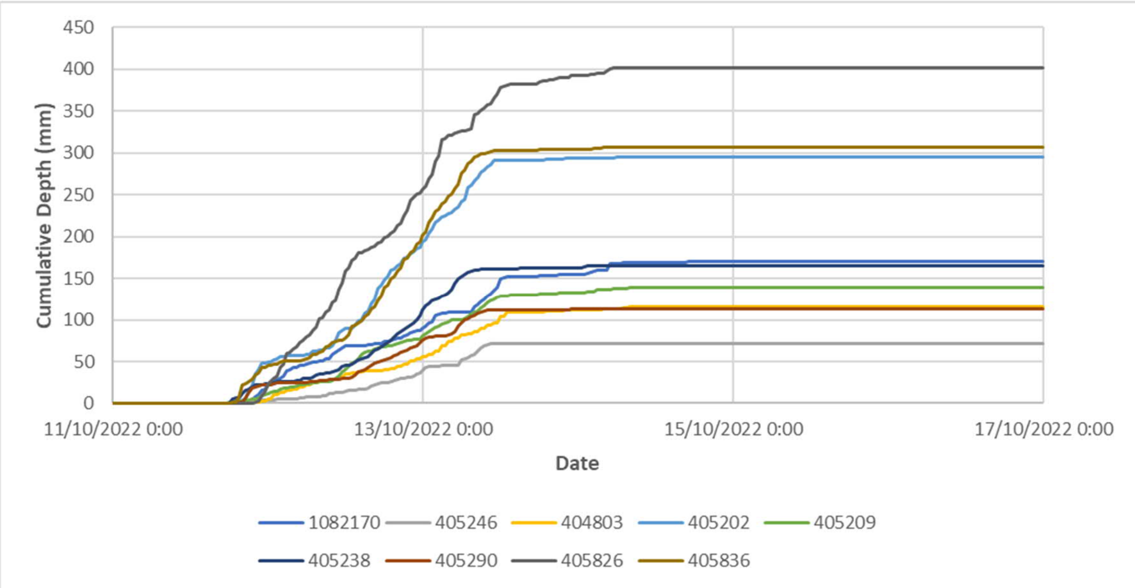


Figure 5-3 Pluviograph temporal patterns for the October 2022 event

For this event, Lake Eildon, Lake Nillahcootie and Goulburn Weir were all in operation. Goulburn Weir and Lake Mokoan were incorporated into the RORB model as special storages using the stage-storage and stage-discharge data described in Appendix A. Decommissioning has been completed at Lake Mokoan, and so the stage-storage and stage-discharge characteristics of the Winton Wetland were used to simulate the remaining storage. Gauged outflow hydrographs for Lake Eildon and Lake Nillahcootie were used as inputs to the model in place of explicitly modelling those dams.

Baseflow was manually separated from the total flow hydrographs at each of the streamflow gauges. Appendix B shows the baseflow that was removed.

A summary of the calibration results for the October 2022 event is shown in Table 5-3 with a select number of key hydrograph comparisons shown in Figure 5-8. A full set of hydrograph comparisons is shown in Appendix C. A summary of the adopted calibration parameter values for this event is shown in Table 5-4. Of note is the actual flow at Caseys Weir has been adjusted in accordance with the modelled rating curve shown in Section 3.2.6.

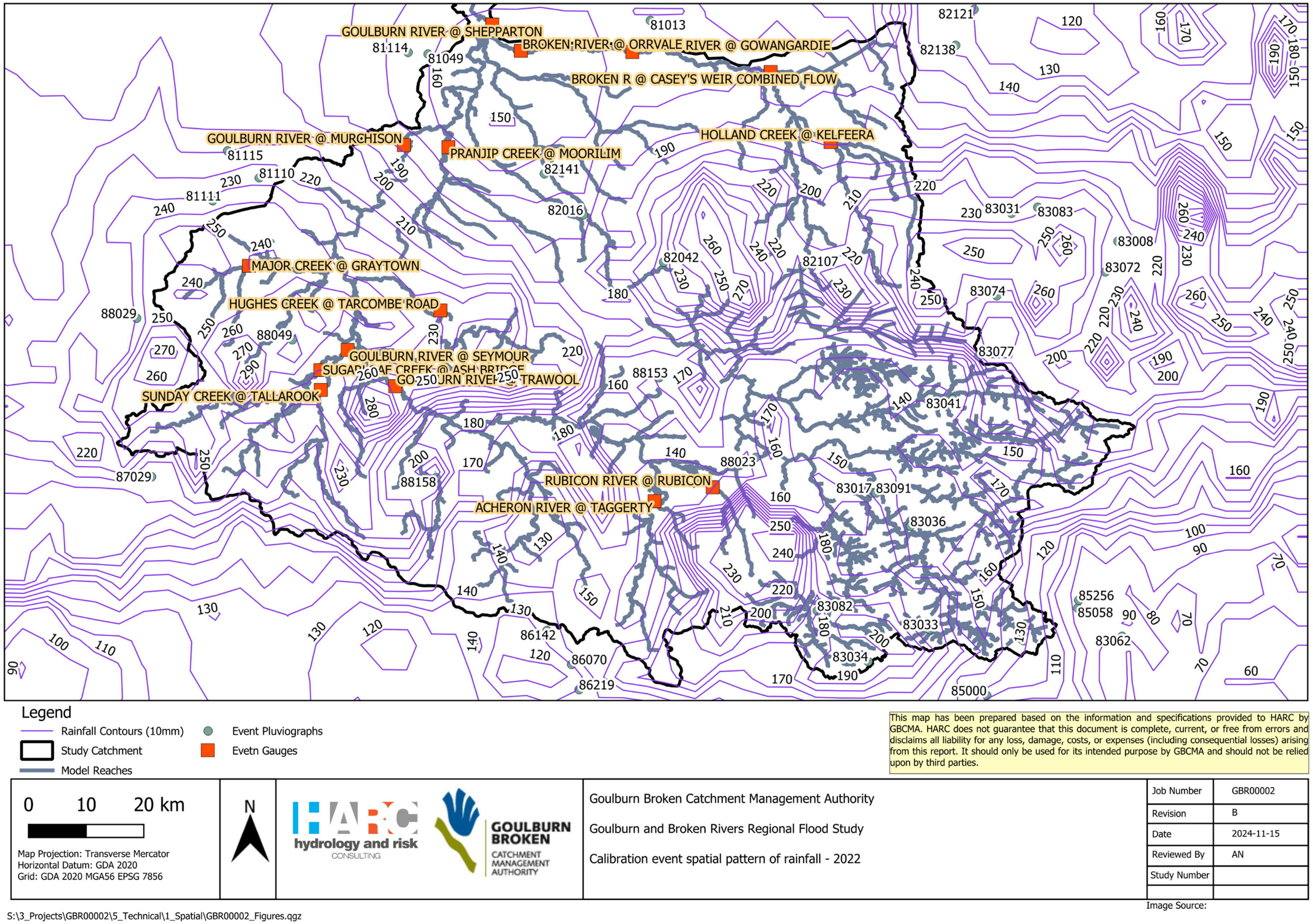


Figure 5-4 October 2022 event rainfall spatial pattern

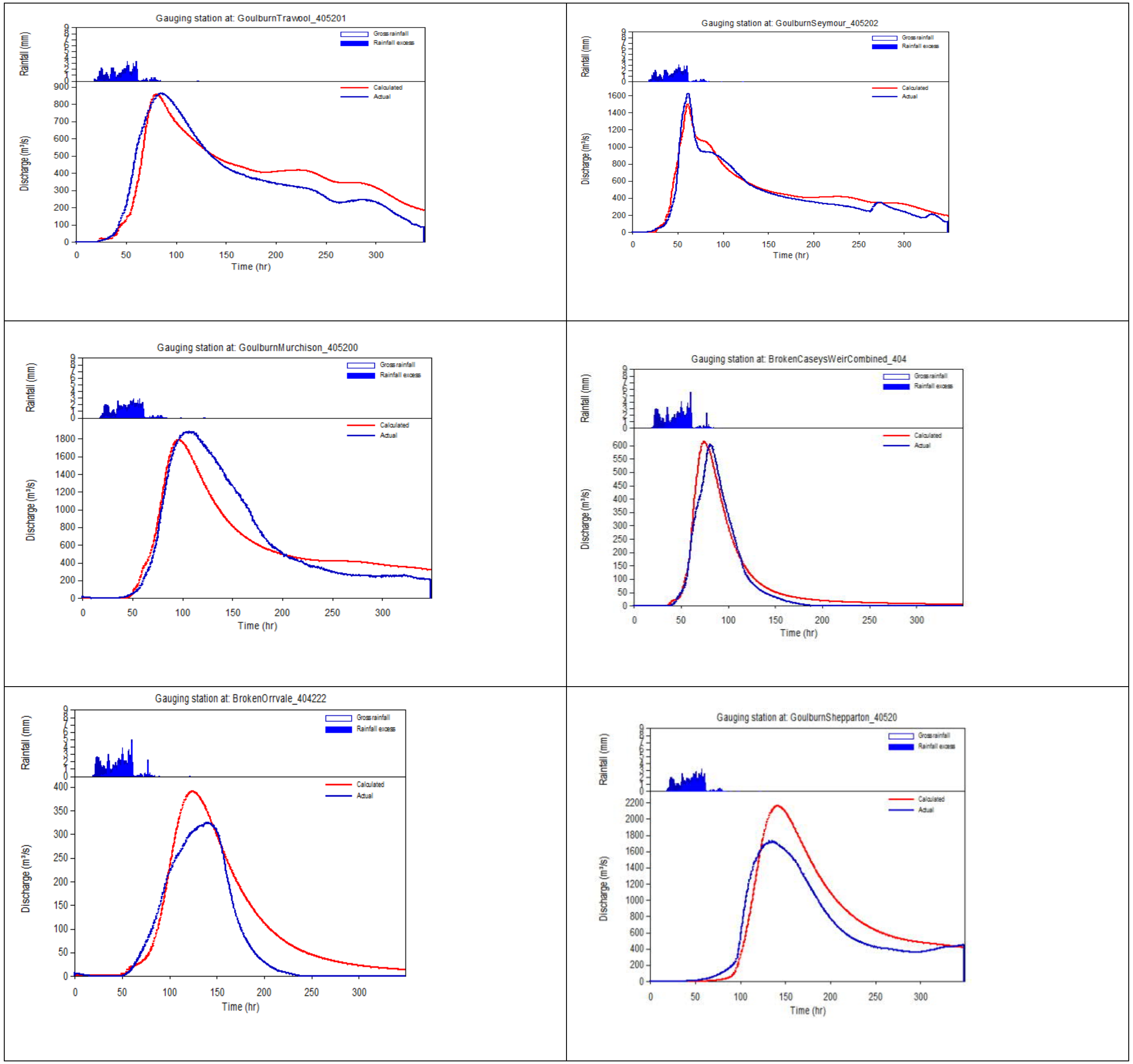


Figure 5-5 Selected hydrograph comparisons for the October 2022 event

Table 5-1 Calibration result summary for October 2022

Gauge	Peak flow (m^3/s)			Time to peak (hours)			Volume (m^3)	
	Calculated	Recorded	Difference (%)	Calculated	Recorded	Difference (%)	Calculated	Recorded
Rubicon River at Rubicon	53.0	52.5	1.0	62.5	59.0	5.9	5.5E+06	5.3E+06
Acheron River at Taggerty	291.1	285.0	2.1	62.0	61.0	1.6	3.1E+07	2.0E+07
Home Creek at Yark	183.7	215.5	-14.8	53.0	61.0	-13.1	1.0E+07	1.5E+07
Yea River at Goulburn Valley Water Pumping Station	381.3	-#	-	62.0	59.5	-	2.9E+07	-
King Parrot Creek at Fairview Road Bridge Kerrisdale`	122.2	113.7	7.4	64.5	65.0	-0.8	1.3E+07	1.6E+07
Goulburn River at Trawool	857.5	863.2	-0.7	80.5	85.5	-5.8	4.7E+08	4.3E+08
Sugarloaf Creek at Ashbridge	720.2	729.0	-1.2	58.5	55.5	5.4	6.5E+07	6.6E+07
Sunday Creek at Tallarook	290.3	290.7	-0.1	58.5	59.5	-1.7	2.7E+07	2.5E+07
Goulburn River at Seymour	1502.0	1626.6	-7.7	61.0	61.0	0.0	5.9E+08	5.4E+08
Hughes Creek at Tarcombe	512.3	536.8	-4.6	58.0	57.5	0.9	4.5E+07	4.5E+07
Major Creek at Greytown	181.3	171.0	6.0	56.0	54.5	2.8	1.2E+07	1.4E+07
Goulburn River at Murchison	1788.5	1882.7	-5.0	96.0	106.0	-9.4	7.2E+08	7.6E+08
Pranjip Creek at Moorlim	152.0	153.3	-0.9	94.5	103.5	-8.7	4.7E+07	4.3E+07
Castle Creek at Arcadia	38.2	38.3	-0.2	68.0	86.5	-21.4	9.7E+06	1.1E+07
Seven Creek at Kialla West	312.1*	327.2	-4.6	98.0	109.0	-10.1	7.2E+07	-
Holland Creek at Kelfeera	401.5	401.2	0.1	62.0	65.5	-5.3	2.5E+07	3.2E+07
Broken River at Casey's Weir	614.2	603.7^	1.7	74.5	81.0	-8.0	1.1E+08	9.4E+07
Broken River at Gowangardie	507.9	631.5	-19.6	96.5	101.5	-4.9	1.4E+08	1.3E+08
Broken River at Orrvale	390.7	325.5	20.2	124.0	141.0	-12.1	1.3E+08	9.3E+07
Goulburn River at Shepparton	2161.0	1733.6	24.7	141.0	135.5	4.1	8.9E+08	7.5E+08

Flood level only recorded in Yea. Used to match shape and timing of hydrograph

` No hydrograph recorded based on upstream gauge at Flowerdale (405231)

* Peak delayed by 24 hours and full hydrograph not available

^ Calculated peak based on modelled rating curve (Section 3.2.6)

Table 5-2 Adopted model calibration parameter values for October 1993

Sub-catchment	k_c	m	IL (mm)	CL (mm/h)
Rubicon River at Rubicon	25	0.8	35	2.2
Acheron River at Taggerty	32	0.8	20	0.2
Home Creek at Yark	6.0	0.8	45	0.6
Yea River at Goulburn Valley Water Pimping Station	30.0	0.8	15	2.0
King Parrot Creek at Fairview Road Bridge Kerrisdale	38	0.8	20	4.0
Goulburn River at Trawool	140	0.8	5	1.2
Sugarloaf Creek at Ashbridge	41	0.8	12	0.5
Sunday Creek at Tallarook	38	0.8	22	1.9
Goulburn River at Seymour	28	0.8	20	0.1
Hughes Creek at Tarcombe	27	0.8	40	0.3
Major Creek at Greytown	25	0.8	40	3.2
Goulburn River at Murchison	180	0.8	45	0.1
Pranjip Creek at Moorlim	180	0.8	45	0.8
Castle Creek at Arcadia	150	0.8	45	0.8
Seven Creek at Kialla West	110	0.8	45	0.7
Holland Creek at Kelfeera	17	0.8	30	2.7
Broken River at Casey's Weir	70	0.8	30	1.1
Broken River at Gowangardie	120	0.8	20	0.1
Broken River at Orrvale	45	0.8	25	5.5
Goulburn River at Shepparton	130	0.8	50	5.5

For the 2022 event a good calibration was achieved up to and including Seymour. For Murchison and Shepparton whilst a poor fit to the peak was achieved in the RORB model, the hydrographs achieved a reasonable match during the hydraulic model calibration (Section 8.2). In the upper reaches of the Goulburn River this event was dominated by the outflows from Lake Eildon (peak flow of approximately 400 m^3/s) along with flow from the Archeron, Yea and King Parrot Creeks. The peak at Seymour was dominated by flow from Sugarloaf (744 m^3/s) and Sunday Creek (292 m^3/s).

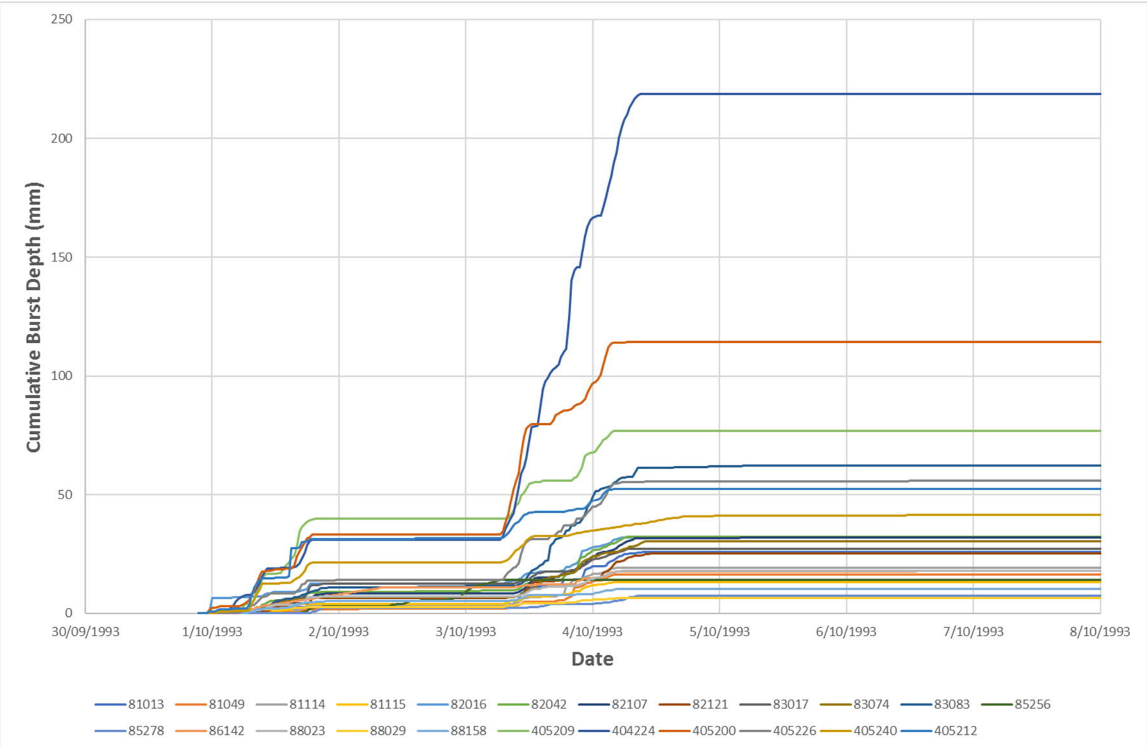
For the flow between Seymour and Murchison the only significant inflows are from Hughes Creek (540 m^3/s) and Major Creek (183 m^3/s) which match very well in the RORB model (refer to Appendix C). Therefore, to increase the flow at Murchison would require increasing the flow at Seymour, which is matching well. For the flow between Murchison and Shepparton the significant inflows are from Pranjip (156 m^3/s), Honeysuckle (estimate 180 m^3/s) and Seven Creeks (238 m^3/s) along with the Broken River (353 m^3/s) which match reasonably well (where gauge data is available) in the RORB model. The only way to significantly reduce the flow at Shepparton was to further reduce the flow at Seymour and Murchison (which is already lower than the recorded). As such, in conjunction with the hydraulic model calibration it was concluded that a reasonable match was achieved on the Goulburn River.

On the Broken River, reasonable results were also obtained at Casey's Weir, particularly when the peak flow was adjusted to account for the uncertainty in the rating curve. Reasonable results were also obtained at Orrvale. As with the Goulburn River a join calibration was undertaken between the RORB and hydraulic (TUFLOW) models.

5.3.2 October 1993

For the October 1993 event a significant distribution of pluviograph information was available to calibrate the RORB model for this event. Figure 5-6 shows the pluviograph temporal patterns available (refer to Figure 3-1 for the location of each pluviograph). The temporal pattern of rainfall in each subarea was defined using the closest pluviograph. The event features two distinct and separate bursts of rainfall, with the second burst carrying the majority of the rainfall depth.

Rainfall depths were estimated by extracting total rainfall depth for the event at each RORB sub-area centroid from the AWAP rainfall data. The extracted rainfall depths from AWAP were then checked against the total rainfall recorded at each pluviograph. Figure 5-7 shows the adopted rainfall depths across the catchment. This event was primarily centred around Lake Nillahcootie and in particular Hollands Creek, with significant rainfall depths also occurring on the Goulburn River tributaries south of Lake Eildon resulting in a significant flow from Eildon.

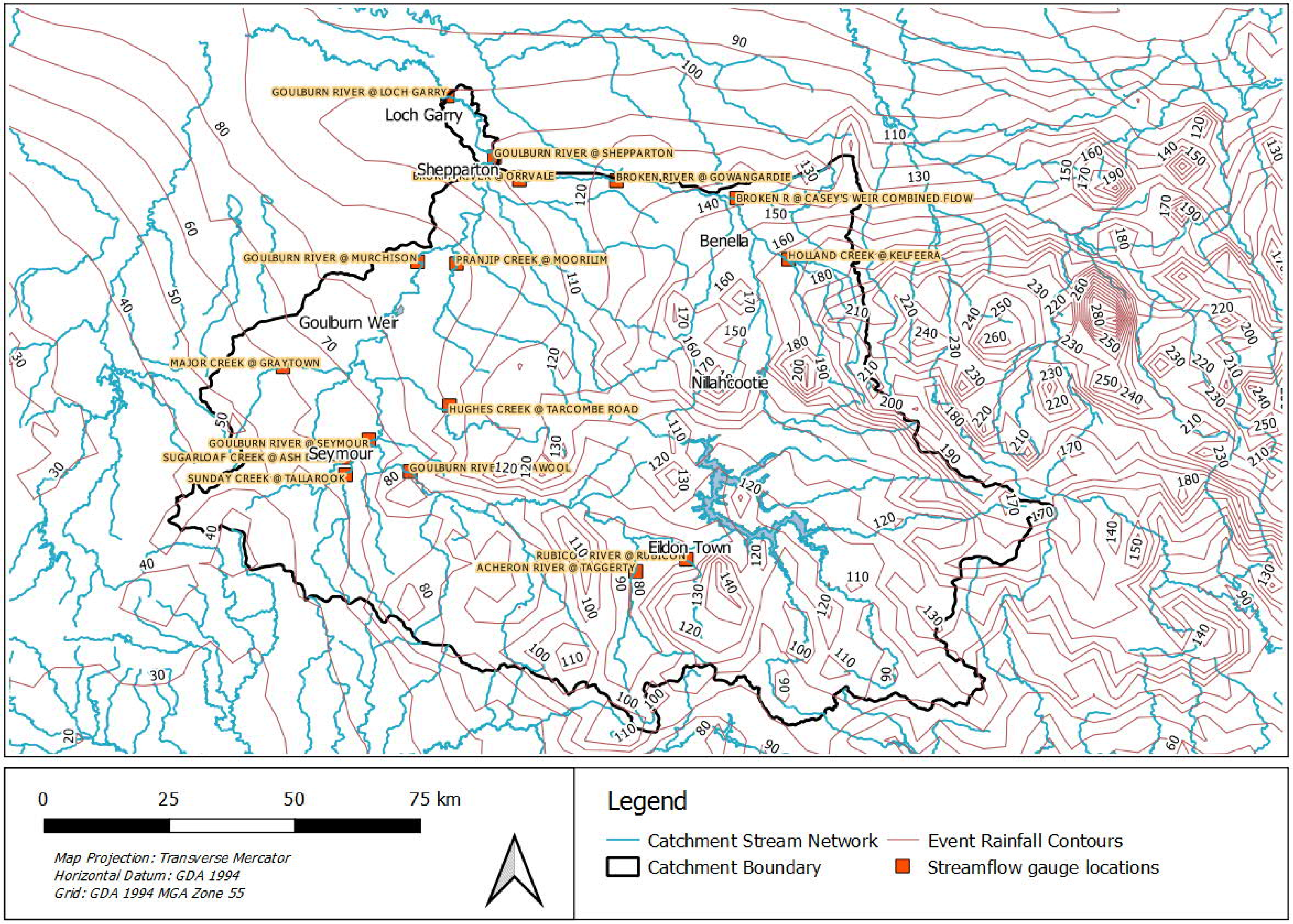


■ Figure 5-6 Pluviograph temporal patterns for the October 1993 event

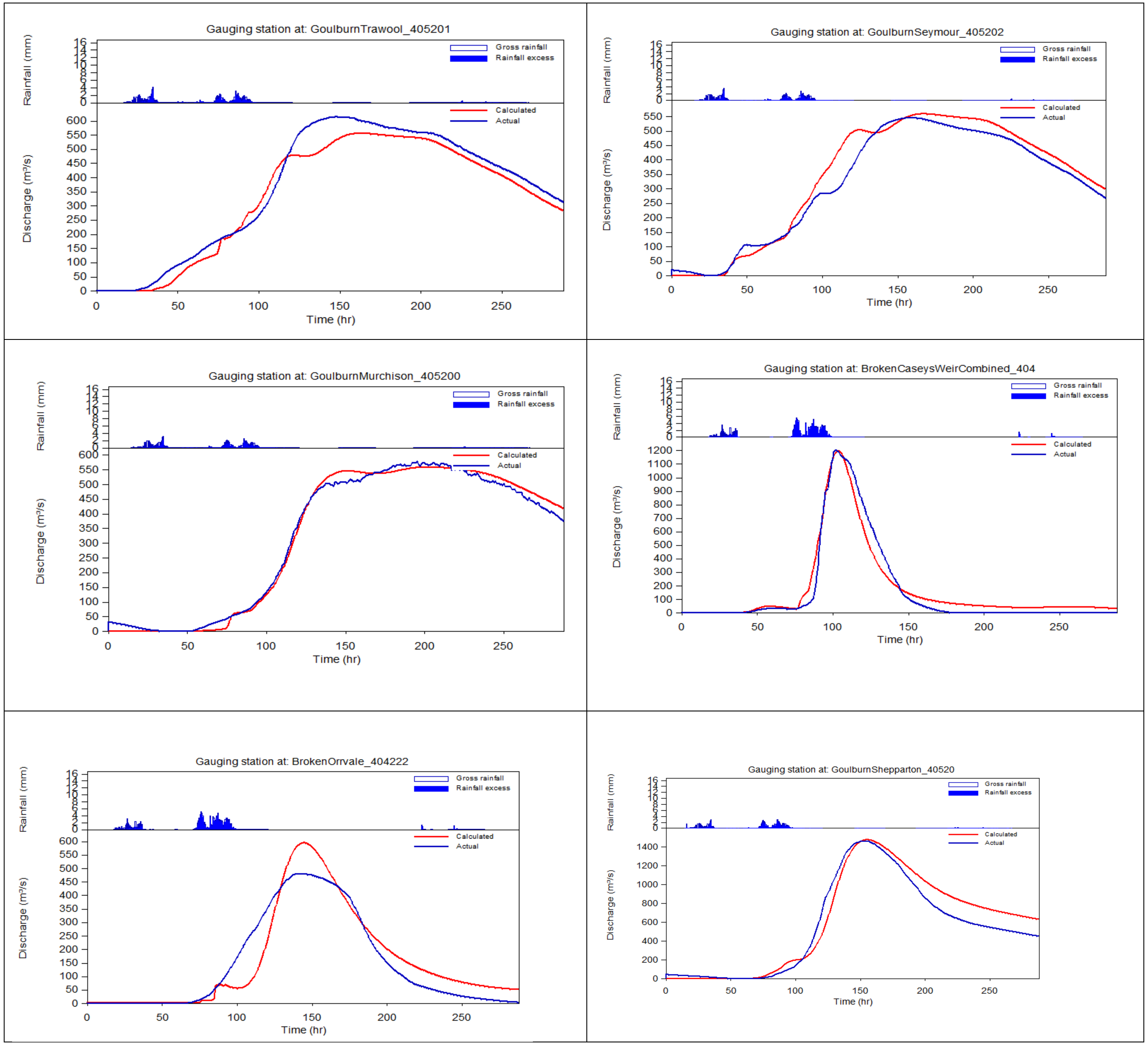
At the time of this event, Lake Eildon, Lake Nillahcootie, Goulburn Weir and Lake Mokoan were all in operation. Goulburn Weir and Lake Mokoan were incorporated into the RORB model as special storages using the stage-storage and stage-discharge data described in Appendix A. Gauged outflow hydrographs for Lake Eildon and Lake Nillahcootie were used as inputs to the model in place of explicitly modelling those dams.

Baseflow was manually separated from the total flow hydrographs at each of the streamflow gauges. Appendix B shows the baseflow that was removed.

A summary of the calibration results for the October 1993 event is shown in Table 5-3 with a select number of key hydrograph comparisons shown in Figure 5-8. A full set of hydrograph comparisons is shown in Appendix C. A summary of the adopted calibration parameter values for this event is shown in Table 5-4.



- Figure 5-7 October 1993 event rainfall spatial pattern



■ Figure 5-8 Selected hydrograph comparisons for the October 1993 event

Table 5-3 Calibration result summary for October 1993

Gauge	Peak flow (m^3/s)			Time to peak (hours)			Volume (m^3)	
	Calculated	Recorded	Difference (%)	Calculated	Recorded	Difference (%)	Calculated	Recorded
Rubicon River at Rubicon	12.8	12.3	3.8	41.0	30.5	34.4	1.9E+06	1.5E+06
Acheron River at Taggerty	32.4	33.8	-4.4	42.5	50.0	-16.0	6.3E+06	8.2E+06
Home Creek at Yark	220.6	197.6	11.7	87.5	95.0	-7.9	7.0E06	1.2E+07
Yea River at Goulburn Valley Water Pumping Station	101.6	-#	-	39.0	41.0	-4.9	1.3E+07	-
King Parrot Creek at Fairview Road Bridge Kerrisdale`	12.2	11.1	10.1	77.0	37.0	108.0	3.3E+06	2.0E+06
Goulburn River at Trawool	556.0	613.8	-9.4	163.0	148.0	10.1	3.5E+08	3.7E+08
Sugarloaf Creek at Ashbridge	57.4	57.7	-0.6	98.5	89.0	10.7	1.2E+07	6.8E+06
Sunday Creek at Tallarook	17.7	16.9	4.7	95.0	91.5	3.8	3.5E+06	2.9E+06
Goulburn River at Seymour	559.0	545.6	2.4	167.0	157.0	6.4	3.6E+08	3.3E+08
Hughes Creek at Tarcombe	308.7	323.6	-4.6	96.5	97.5	-1.0	2.7E+07	2.1E+07
Major Creek at Greytown	24.4	24.4	-0.2	93.5	89.0	5.1	2.5E+06	2.2E+06
Goulburn River at Murchison	559.2	577.4	-3.1	202.0	195	3.6	3.4E+08	3.3E+08
Pranjip Creek at Moorlim	176.0	173.1	1.7	118.0	114.5	3.1	5.0E+07	3.7E+07
Castle Creek at Arcadia	53.6	55.4	-3.2	99.5	108.0	-7.9	1.3E+07	9.4E+06
Seven Creek at Kialla West	707.4	719.6	-1.7	128.0	124.0	3.2	1.6E+09	-*
Holland Creek at Kelfeera	687.5	695.1	-1.1	90.0	91.0	-1.1	5.0E+07	4.9E+07
Broken River at Casey's Weir	1198.5	1200.0	-0.2	103.5	102.5	1.0	1.8E+08	1.6E+08
Broken River at Gowangardie	748.0	682.6	9.6	124.5	122.0	2.0	1.7E+08	1.8E+08
Broken River at Orrvale	595.4	480.5	26.9	144.5	140.0	3.2	1.6E+08	1.5E+08
Goulburn River at Shepparton	1478.0	1463.0	1.0	155.0	153.0	1.3	6.3E+08	5.7E+08

Flood level only recorded in Yea. Used to match shape and timing of hydrograph

` No hydrograph recorded based on upstream gauge at Flowerdale (405231)

* The full hydrograph was not recorded

Table 5-4 Adopted model calibration parameter values for October 1993

Sub-catchment	k_c	m	IL (mm)	CL (mm/h)
Rubicon River at Rubicon	25	0.8	35	6.5
Acheron River at Taggerty	38	0.8	12	4.0
Home Creek at Yark	10.0	0.8	20	3.8
Yea River at Goulburn Valley Water Pimping Station	30.0	0.8	2	3.8
King Parrot Creek at Fairview Road Bridge Kerrisdale	38	0.8	10	4.5
Goulburn River at Trawool	130	0.8	30	1.2
Sugarloaf Creek at Ashbridge	41	0.8	15	0.5
Sunday Creek at Tallarook	38	0.8	22	1.8
Goulburn River at Seymour	26	0.8	30	3.5
Hughes Creek at Tarcombe	24	0.8	50	0.1
Major Creek at Greytown	25	0.8	25	2.2
Goulburn River at Murchison	140	0.8	20	3.5
Pranjip Creek at Moorlim	160	0.8	20	0.5
Castle Creek at Arcadia	150	0.8	30	0.7
Seven Creek at Kialla West	110	0.8	10	0.2
Holland Creek at Kelfeera	17	0.8	20	2.0
Broken River at Casey's Weir	65	0.8	40	0.5
Broken River at Gowangardie	125	0.8	10	5.0
Broken River at Orrvale	35	0.8	10	5.0
Goulburn River at Shepparton	130	0.8	10	3.2

For the 1993 event a good calibration was achieved across the catchment. In the upper reaches of the Goulburn River this event was dominated by the outflows from Lake Eildon (peak flow of approximately 550 m^3/s). This is particularly the case for the hydrograph at Trawool, which is dominated by these releases from Eildon.

A slight overestimation of peak flow was recorded on the Goulburn River at Seymour, but it was found that attempting to match the gauged peak there caused significant ramifications for modelled flows at Murchison and Shepparton. As such, it was concluded that a reasonable match to hydrograph volume, peak and timing was obtained across all four gauges.

On the Broken River, a good results was obtained at Casey's Weir despite the significant uncertainty associated with the high flow rating curve there but a poor fit was obtained at Orrvale. A good match to the recorded hydrograph at the Shepparton gauge was achieved.

No recorded flow data was available for calibration at Benalla for this event. The estimated peak flow at Benalla is approximately 1,250 m^3/s (HydroTechnology, 1995a) which is consistent with the result from the RORB model (1,240 m^3/s). A hydrograph was available in the HydroTechnology, 1995a report, a copy of which is shown in Figure 5-9 compared to the output from RORB.

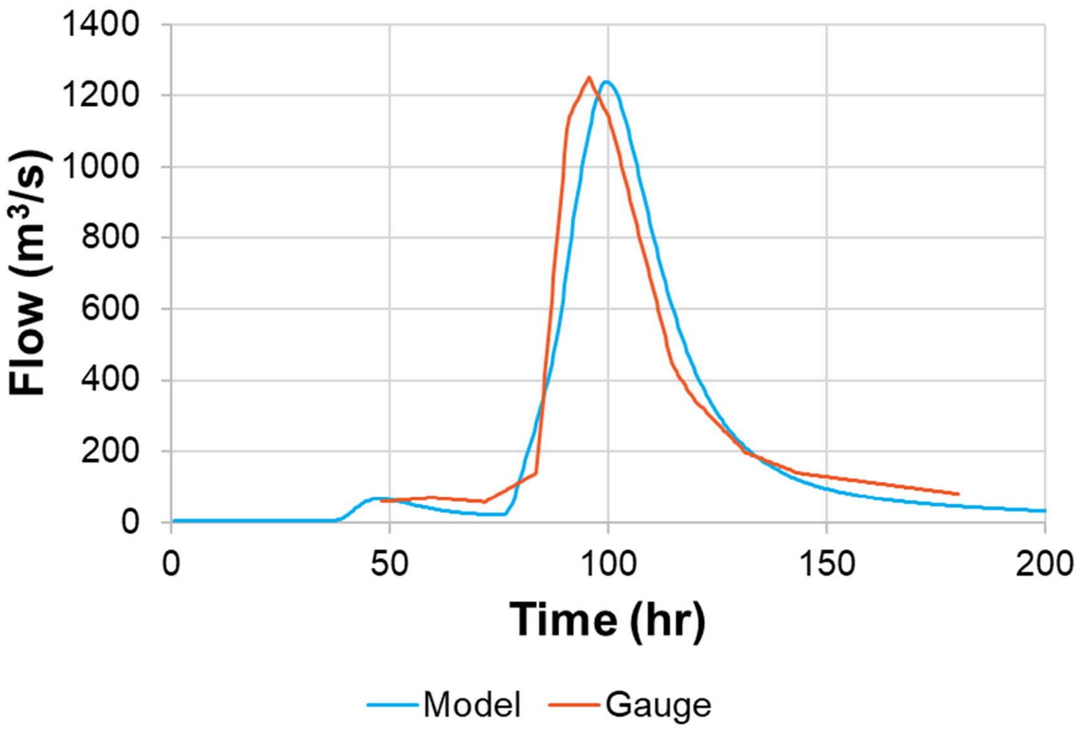


Figure 5-9 Hydrograph at Benalla for the October 1993 event

5.3.3 May 1974

For the May 1974 event a significant distribution of pluviograph information was available to calibrate the RORB model for this event. Figure 5-10 shows the pluviograph temporal patterns available (refer to Figure 3-1 for the location of each pluviograph). The temporal pattern of rainfall in each subarea was defined using the closest pluviograph. The event features two bursts of rainfall separated by a period of lighter falls, with the second burst carrying the majority of the rainfall depth.

Rainfall depths were estimated by extracting total rainfall depth for the event at each RORB sub-area centroid from the AWAP rainfall data. The extracted rainfall depths from AWAP were then checked against the total rainfall recorded at each pluviograph. Figure 5-11 shows the adopted rainfall depths across the catchment. It can be seen that this event was primarily to the south of Lake Eildon and featured significant rainfall depths over the middle reaches of the Goulburn River. Significant falls were also recorded around Lake Nillahcootie.

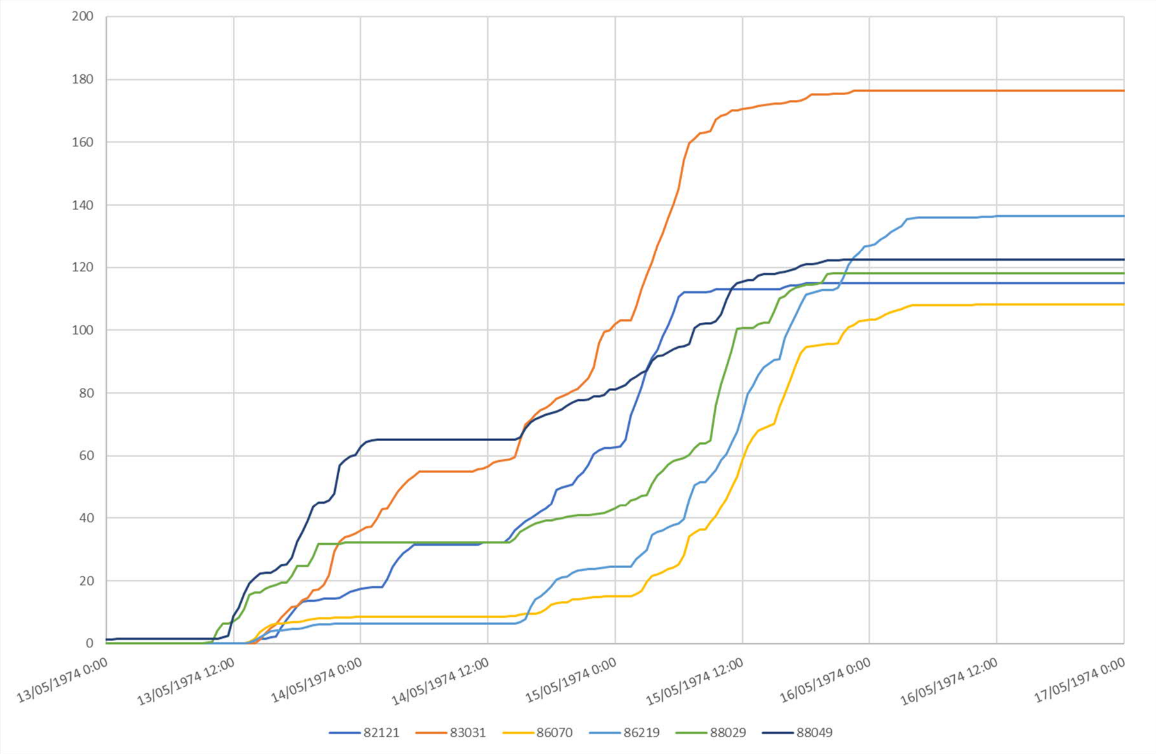
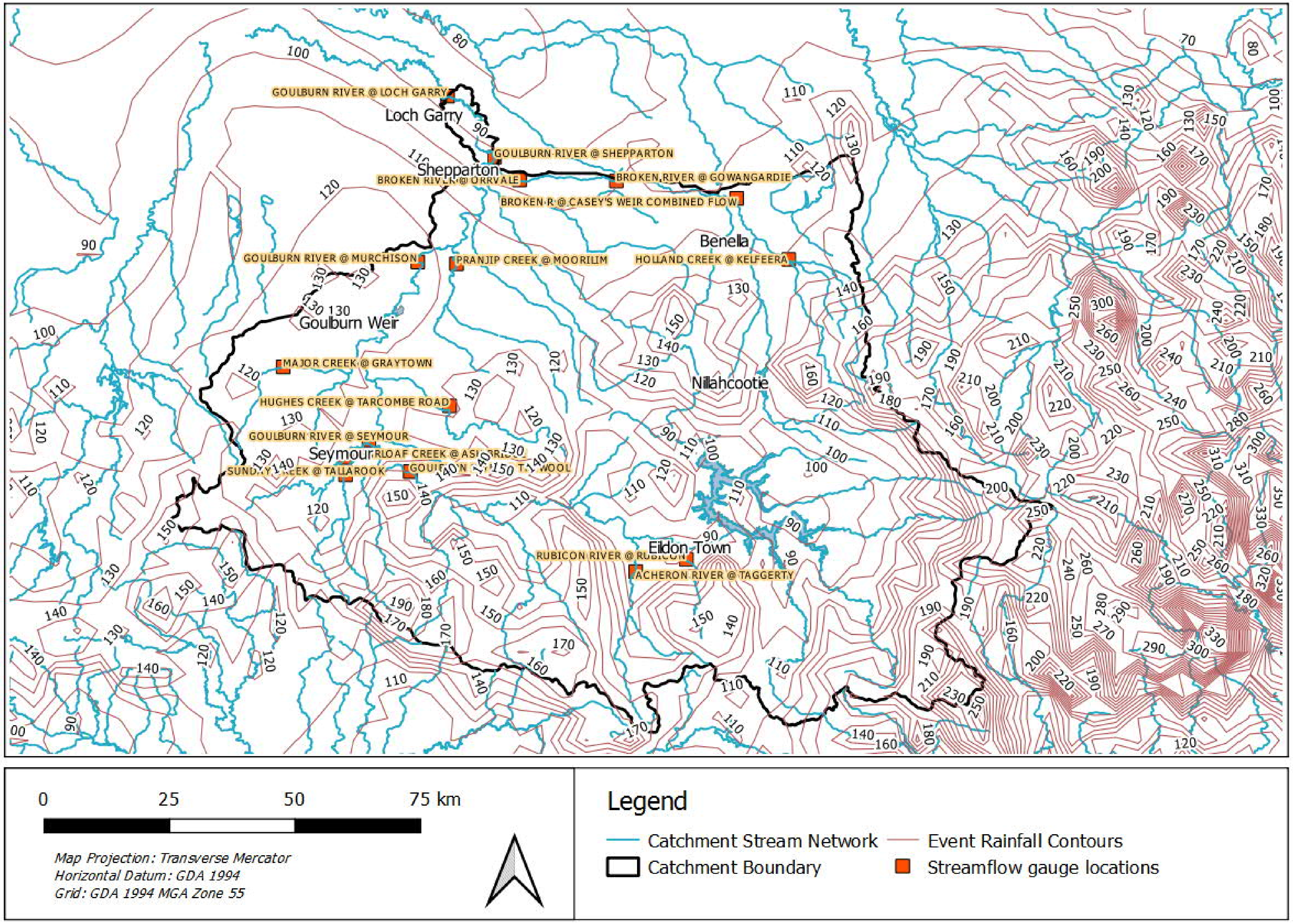


Figure 5-10 Pluviograph temporal patterns for the May 1974 event

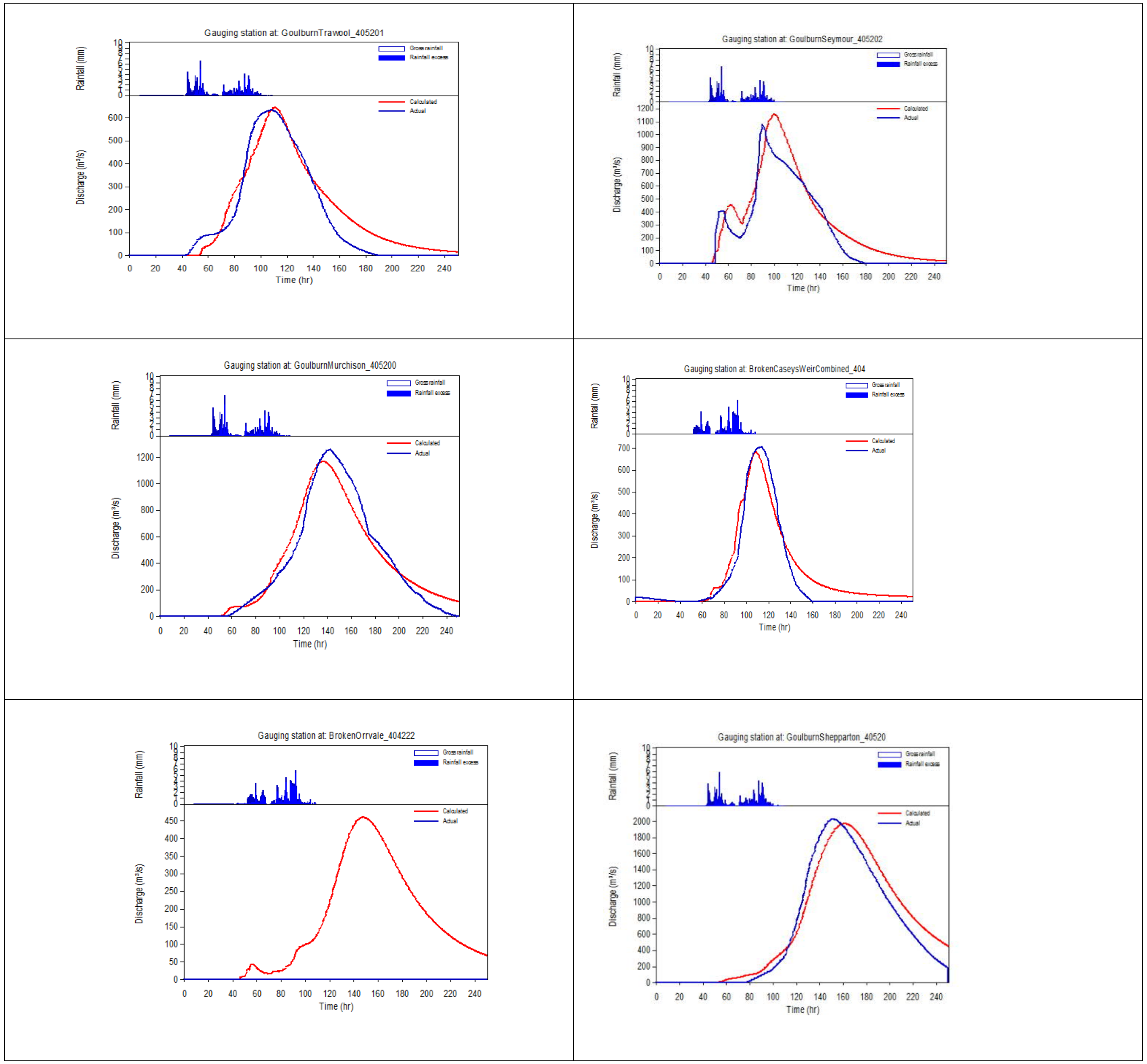
At the time of this event, Lake Eildon, Lake Nillahcootie, Goulburn Weir and Lake Mokoan were all in operation. Goulburn Weir and Lake Mokoan were incorporated into the RORB model as special storages using the stage-storage and stage-discharge data described in Appendix A. Gauged outflow hydrographs for Lake Eildon and Lake Nillahcootie were used as inputs to the model in place of explicitly modelling those dams.

Baseflow was manually separated from the total flow hydrographs at each of the streamflow gauges. Appendix B shows the baseflow that was removed.

A summary of the calibration results for the May 1974 event is shown in Table 5-5 with a select number of key hydrograph comparisons shown in Figure 5-12. A full set of hydrograph comparisons is shown in Appendix C. A summary of the adopted calibration parameter values for this event is shown in Table 5-6.



- Figure 5-11 May 1974 event rainfall spatial pattern



■ Figure 5-12 Selected hydrograph comparisons for the May 1974 event

Table 5-5 Calibration result summary for May 1974

Gauge	Peak flow (m^3/s)			Time to peak (hours)			Volume (m^3)	
	Calculated	Recorded	Difference (%)	Calculated	Recorded	Difference (%)	Calculated	Recorded
Rubicon River at Rubicon	34.7	32.6	6.4	96.0	93.5	2.7	4.2E+06	3.7E+06
Acheron at Taggerty	133.6	138.6	-3.7	99.5	101.0	-1.5	1.6E+07	1.6E+07
Home Creek at Yark	132.2	-	-	92.2	-	-	-	-
Yea River at Goulburn Valley Water Pumping Station	368.0*	-	-	96.2	-	-	-	-
King Parot Creek at Fairview Road Bridge Kerrisdale	156.8*	-	-	94.0	-	-	-	-
Goulburn River at Trawool	645.4	635.4	1.6	110.5	108.0	2.3	1.6E+0.8	1.4E+08
Sugarloaf Creek at Ashbridge	384.4*	-	-	98.1	-	-	-	-
Sunday Creek at Tallarook	208.6	296.7	-29.7	93.0	99.5	-6.5	2.8E+07	2.5E+07
Goulburn River at Seymour	1156.4	1078.3	7.2	100.0	90.0	11.1	2.6E+08	2.1E+08
Hughes Creek at Tarcombe	182.1	188.3	-3.3	96.0	105.5	-9.0	1.5E+07	2.6E+07
Major Creek at Greytown	182.5	182.9	-0.2	94.5	101.5	-6.9	1.4E+07	1.2E+07
Goulburn River at Murchison	1167.8	1259.0	-7.2	136.5	142.5	-4.2	3.3E+08	3.3E+08
Pranjip Creek at Moorlim	189.5	199.5	-5.0	116.0	115.5	0.4	5.8E+07	4.3E+07
Castle Creek at Acadia	54.4*	-	-	100.7	-	-	-	-
Seven Creek at Kialla West	493.2*	-	-	132.2	-	-	-	-
Holland Creek at Kelfeera	358.1	356.7	0.4	94.5	92.5	2.2	2.0E+07	3.4E+07
Broken River at Casey's Weir	682.1	705.4	-3.3	108.5	114.0	-4.8	1.2E+08	9.9E+07
Broken River at Gowangardie	554.3*	-	-	129.1	-	-	-	-
Broken River at Orrvale	464.4*	-	-	149.5	-	-	-	-
Goulburn River at Shepparton	1971.2	2023.6	-2.6	161.5	149.5	8.0	6.3E+08	5.8E+08

* No recorded hydrograph

Table 5-6 Adopted model calibration parameter values for May 1974

Sub-catchment	k_c	m	IL (mm)	CL (mm/h)
Rubicon River at Rubicon	25	0.8	45	2.0
Acheron River at Taggerty	42	0.8	48	1.4
Home Creek at Yark	6	0.8	48	1.4
Yea River at Goulburn Valley Water Pumping Station	30	0.8	48	1.4
King Parrot Creek at Fairview Road Bridge Kerrisdale	38	0.8	48	1.4
Goulburn River at Trawool	140	0.8	40	1.0
Sugarloaf Creek at Ashbridge	35	0.8	35	0.1
Sunday Creek at Tallarook	33	0.8	35	0.1
Goulburn River at Seymour	20	0.8	10	0.1
Hughes Creek at Tarcombe	27	0.8	60	1.4
Major Creek at Greytown	25	0.8	20	2.3
Goulburn River at Murchison	170	0.8	20	2.0
Pranjip Creek at Moorlim	170	0.8	20	0.5
Castle Creek at Arcadia	150	0.8	20	0.5
Seven Creek at Kialla West	110	0.8	20	0.5
Holland Creek at Kelfeera	20	0.8	20	3.5
Broken River at Casey's Weir	65	0.8	10	0.5
Broken River at Gowangardie	120	0.8	10	0.5
Broken River at Orrvale	35	0.8	10	0.5
Goulburn River at Shepparton	110	0.8	10	0.5

For the May 1974 event a reasonable calibration was achieved where gauged data was available. On the WMIS website there was no recorded data available on the Goulburn River at Trawool or the Broken River at Orrvale and on the Goulburn River at Seymour only mean daily data was available. In addition, there are a number of creeks, such as Home Creek etc where there was no gauge information available. The State River and Water Supply Commission undertook an investigation into flooding at Seymour in 1981. In the State River and Water Supply Commission report a hydrograph at both Trawool and Seymour is shown. Estimates of the peak flow for this event are also found at Trawool and Seymour in the Victorian Surface Water Information to 1987, Volume 3 (RWC, 1987). The estimated mean flow at Trawool is 754 m^3/s and estimated instantaneous peak flow is 785 m^3/s. The estimated mean flow at Seymour is 899 m^3/s and instantaneous peak flow is 1,215 m^3/s. The same hydrograph shown in the State River and Water Supply Commission report also was used in the Seymour Floodplain Mapping Study (WBM, 2001). The WBM report has no detail on the origin of this data so it is assumed that it was from the State River and Water Supply Commission report. Also, there is no discussion on baseflow removal in WBM, 2001 study and as the peaks shown are the same as the State River and Water Supply Commission report is it assumed that baseflow was not removed in the WBM, 2001 study. In Figure 5-12 the Trawool and Seymour hydrographs are as shown in the State River and Water Supply Commission report as the actual hydrograph but have had baseflow removed.

There is also some uncertainty of the peak flow at Murchison for this event. The WMIS website has the instantaneous peak as 1,647 m^3/s with a peak level of 11.29 m. This level indicates a flow of approximately 1400 m^3/s based on the current rating curve. For this event there is an instantaneous flow estimate of approximately 1400m^3/s available at Goulburn Weir just upstream of Murchison (this estimate was confirmed during the site visit based on manual records). Therefore, based on the rating curve and the estimate at Goulburn Weir a flow of approximately 1400m^3/s was adopted for this event at Murchison.

For this event, most of the flow contribution to the Goulburn River was from tributaries downstream of Lake Eildon, with only approximately 80 m^3/s being released from the dam. The catchment area of the Goulburn River at Trawool is approximately 7,335 km^2 and at Seymour it is 8,601 km^2, which is an interstation catchment area of 1,266 km^2. If the estimated peak flow at Trawool from RWC (1987) is correct then there was a significant flow contribution (approximately an additional 430 m^3/s) from the tributary catchments between Trawool and Seymour. The major tributary between Trawool and Seymour is Sunday Creek, which has a catchment area of approximately 986 km^2 (approximately 80% of the total interstation catchment between Trawool and Seymour). There is a gauge on Sunday Creek at Tallarook which recorded a peak flow of approximately 300 m^3/s for this event. There is no data available for the gauge on Sugarloaf Creek at Ash Bridge however the RORB model results suggest that the total flow into the Goulburn River from Sunday Creek is approximately 600 m^3/s, which is significantly more than the additional 430 m^3/s required.

However, the timing of the hydrographs on the Goulburn River and Sunday Creek is very important. The recorded peak flow on Sunday Creek at Tallarook happened approximately 24 hours before the peak flow of the Goulburn River arrives at Seymour. By the time the Goulburn River peaks at Seymour, there is relatively little flow being contributed from Sunday Creek. The recorded hydrograph at Seymour does indicate that flow from Sunday Creek comes through earlier, as evidenced by the double peak at this location. This indicates either that the estimate of peak flow at Trawool is too low or alternatively the recorded peak flow at Seymour is too high. It does not appear possible to reconcile these peak flows given the timing of the contribution from Sunday Creek and the interstation area.

For this study, more confidence was placed in the estimate of the peak flow at Seymour, as this flow rate is consistent with the recorded peak flows at Murchison (approximately 1400 m^3/s) and Shepparton (approximately 2000 m^3/s). In addition, the calibration results on the major tributaries between Seymour and Shepparton (Hughes Creek, Major Creek, Pranjip Creek and Holland Creek) are reasonable.

On the Broken River, reasonable results were also obtained at Casey's Weir despite the significant uncertainty associated with the high flow rating curve at this location.

5.4 Summary of adopted routing parameters

In general, a reasonable calibration was achieved for all events at each of the streamflow gauge locations, with a few notable exceptions as highlighted above.

As with all hydrological modelling, the observed variations between the recorded and modelled hydrographs can be the result of a number of uncertainties, including factors such as historic changes in catchment conditions, recorded rainfall and streamflow data errors, baseflow separation uncertainties, and the lack of adequate rainfall gauges to represent the temporal and spatial variability of the storms across the catchment. It must be noted that RORB (and all hydrologic models) are only a representation of a variable and complex rainfall runoff process. Notwithstanding this, at a high level the quality of the calibrations obtained across this large and complex river system are considered more than sufficient to warrant use of this model for design flood estimation.

Table 5-7 summarises the calibrated k_c values adopted for each sub-catchment from the calibration process. The results of the calibration were used to select a best estimate of the k_c value for use in design. The average flow distance (d_{av}, or the average distance between the sub-area centroids and the gauge locations) for each sub-catchment are also summarised, together with the ratio of k_c to d_{av}. McMahon and Muller (1983) showed that k_c is directly proportional to d_{av}. The relationship is given by Equation 3.

$$k_c = C\, d_{av} \qquad (3)$$

Where C is a physical characteristic of the catchment independent of the scale or size of the catchment.

Table 5-7 Summary of adopted routing parameter values

Sub-catchment	Calibration events			Adopted k_c	d_{av}	k_c/d_{av}
	2022	1993	1974			
Rubicon River at Rubicon	25	25	25	25	14.6	1.7
Acheron River at Taggerty	32	38	42	37	30.6	1.2
Home Creek at Yark	6	10	6	8	13.5	0.6
Yea River at Goulburn Valley Water Pumping Station	30	30	30	30	39.1	0.8
King Parot Creek at Fairview Road Bridge Kerrisdale	38	38	38	38	30.6	1.2
Goulburn River at Trawool	140	130	140	135	70.5	1.9
Sugarloaf Creek at Ashbridge	41	41	35	40	40.6	1.0
Sunday Creek at Tallarook	38	38	33	37	31.5	1.2
Goulburn River at Seymour	28	26	20	25	23.9	1.1
Hughes Creek at Tarcombe	27	24	27	25	31.5	0.8
Major Creek at Greytown	25	25	25	25	19.9	1.3
Goulburn River at Murchison	180	140	170	158	57.3	2.8
Pranjip Creek at Moorlim	180	160	170	168	35.0	4.8
Castle Creek at Arcadia	150	150	150	150	30.2	5.0

Sub-catchment	Calibration events			Adopted k_c	d_{av}	k_c/d_{av}
	2022	1993	1974			
Seven Creek at Kialla West	110	110	110	110	47.2	2.3
Holland Creek at Kelfeera	17	17	20	18	29.7	0.6
Broken River at Casey's Weir	70	65	65	67	34.6	1.9
Broken River at Gowangardie	120	125	120	123	39.3	3.1
Broken River at Orrvale	45	35	35	38	11.6	3.3
Goulburn River at Shepparton	150	130	110	132	40.1	3.3

The adopted k_c values for design was based on the calibration results, however comparison was also made to a number of regional approaches commonly used to estimate k_c.

For Victorian regions with a mean annual rainfall of greater than 800 mm, k_c can be estimated using Equation 1. For regions with a mean annual rainfall of less than 800 mm, k_c can be estimated using Equation 2. Both equations are documented in ARR2016 (Ball et. al., 2016).

$$k_c = 2.57\,A^{0.45} \qquad (1)$$

$$k_c = 0.49\,A^{0.65} \qquad (2)$$

Where A is the sub-catchment area in km^2.

Equation 1 is applicable to those catchments on the Goulburn River upstream of Trawool and on the Broken River upstream of Benalla. Equation 2 applies to all other sub-catchments.

The k_c value from calibration was also compared to another regional estimate based on the ratio of k_c and d_{av} (Pearse et. al, 2002). This approach is based on analysis of a large database of calibrated routing parameters, and derived a prediction equation applicable to Victoria. The d_{av} for each calibrated model in the database was regressed with the k_c value to result in Equation 3.

Pearse et al (2002) provide an expected value for k_c based on d_{av}, and also high and low estimates one standard deviation from the expected values.

Table 5-8 provides a summary of the regional estimates. The numbers that are in bold indicate which region the sub-catchment falls into based on mean annual rainfall. Those that are not highlighted have a portion of the catchment in both regions. Table 5-8 shows the k_c values adopted from the calibration events are generally in line with the regional estimates, although there are a number of exceptions to this. These include Home Creek, Goulburn River at Murchison, Pranjip Creek, Castle Creek, Seven Creek, Holland Creek, Broken River at Orrvale and Gowangardie and Goulburn River at Shepparton. The model calibration process indicated that for the lower catchment these sub-catchments exhibit greater attenuation when compared to the upstream areas, which is consistent with the floodplain in these areas (i.e. flat and exhibiting dispersed, shallow flows). A number of the sub-catchments are also interstation areas rather than complete catchments in their own right. As such, the regional approaches are likely to be less useful for k_c estimation.

Based on the comparisons shown in Table 5-8, the calibrated k_c values were adopted for design with some confidence.

Table 5-8 Regional routing parameter estimates

Sub-catchment	Total Area (km^2)	Interstation Area (km^2)	k_c (Equation 1)	k_c (Equation 2)	k_c (Equation 3)			k_c (calibration)
					Expected	High	Low	
Rubicon River at Rubicon	129	129	**22.9**	11.5	18.3	29.2	10.2	25.0
Acheron River at Taggerty	545	545	**43.8**	29.4	38.3	61.3	21.5	37.0
Home Creek	213	213	**28.7**	16.0	16.9	27.0	9.4	8.0
Yea at Yea	885	885	**54.5**	40.3	48.9	78.2	27.4	30.0
King Parrot Creek	434	434	**39.5**	25.4	38.3	61.2	21.4	38.1
Goulburn River at Trawool	7303	1225	**63.0**	49.8	88.1	140.9	49.3	135.0
Sugarloaf Creek at Ashbridge	609	609	46.0	**31.6**	50.7	81.1	28.4	40.0
Sunday Creek at Tallrook	331	331	35.0	**21.3**	39.3	62.9	22.0	37.0
Goulburn River at Seymour	8535	291.5	33.0	19.6	29.9	47.9	16.8	25.0
Hughes Creek at Tarcombe	478	478	41.3	**27.0**	39.4	63.0	22.0	25.0
Major Creek at Greytown	291	291	33.0	**19.6**	24.9	39.8	13.9	25.0
Goulburn River at Murchison	10661	1357	66.0	53.3	71.4	114.2	40.0	158.0
Pranjip Creek at Moorlim	822	822	52.7	**38.4**	43.8	70.0	24.5	168.0
Castle Creek	249	249	30.8	**17.7**	37.8	60.4	21.1	150.0
Seven Creek at Kialla	1539	1539	69.9	**57.8**	59.0	94.3	33.0	110.0
Holland Creek at Kelfeera	450	450	**40.2**	26.0	37.1	59.4	20.8	17.6
Broken River at Casey's Weir	1916	1050	58.8	45.1	43.1	69.0	24.1	67.0
Broken River at Gowangardie	2305	389.5	37.6	23.7	48.3	77.3	27.0	123.0
Broken River at Orrvale	2385	160	25.2	13.3	14.5	23.2	8.1	38.0

Sub-catchment	Total Area (km^2)	Interstation Area (km^2)	k_c (Equation 1)	k_c (Equation 2)	k_c (Equation 3)			k_c (calibration)
					Expected	High	Low	
Goulburn River at Shepparton	16074	338	35.3.	21.6	50.2	80.2	28.1	132.0

6. Design hydrology approach and inputs

6.1 Overview of adopted design flood approach

The estimation of design floods has traditionally been based on the 'design event' approach, in which all parameters other than rainfall are input as fixed, single values. This concept is illustrated in Figure 6-1 for the case where a distribution of design rainfalls is combined with fixed values of losses, rainfall temporal patterns and spatial patterns. Considerable effort is made to ensure that the single values of the adopted parameters are 'AEP-neutral', that is, they are selected with the objective of ensuring that the resulting flood has the same annual exceedance probability as its causative rainfall.

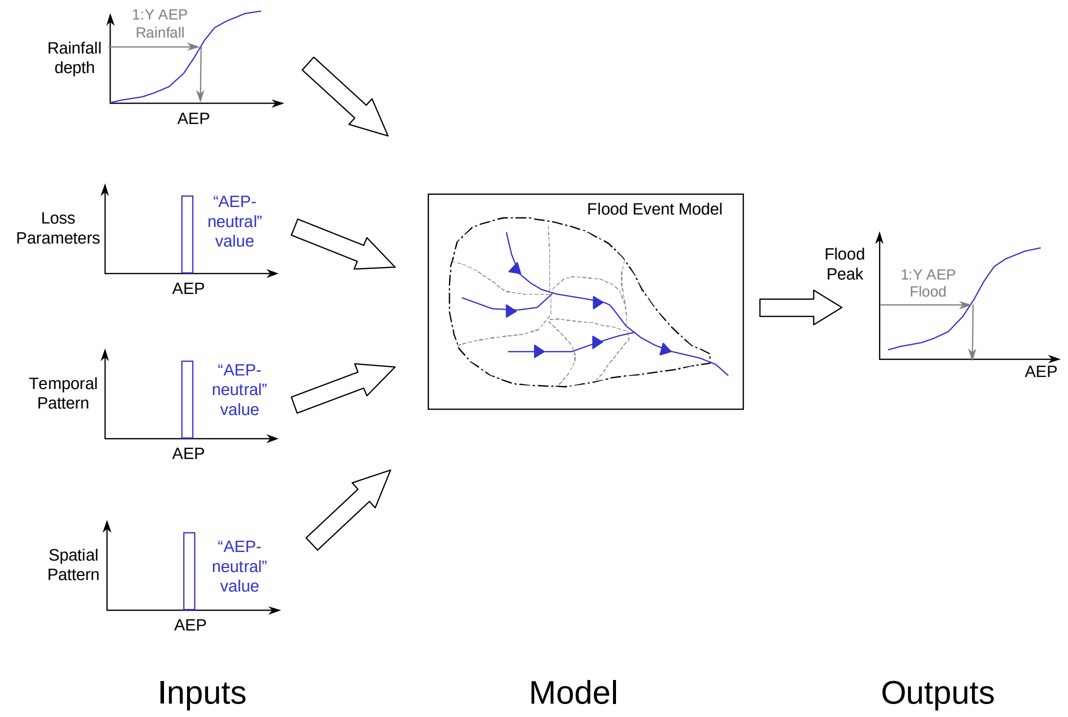
This approach suffers from the limitations that:

- the AEP-neutrality of some inputs can only be tested on frequent events for which independent estimates are available;
- for more extreme events, the adopted values of AEP-neutral inputs must be conditioned by physical and theoretical reasoning; and
- the treatment of more complex interactions (such as the variability in rainfall spatial and temporal pattern) becomes rapidly more complex and less easy to defend.

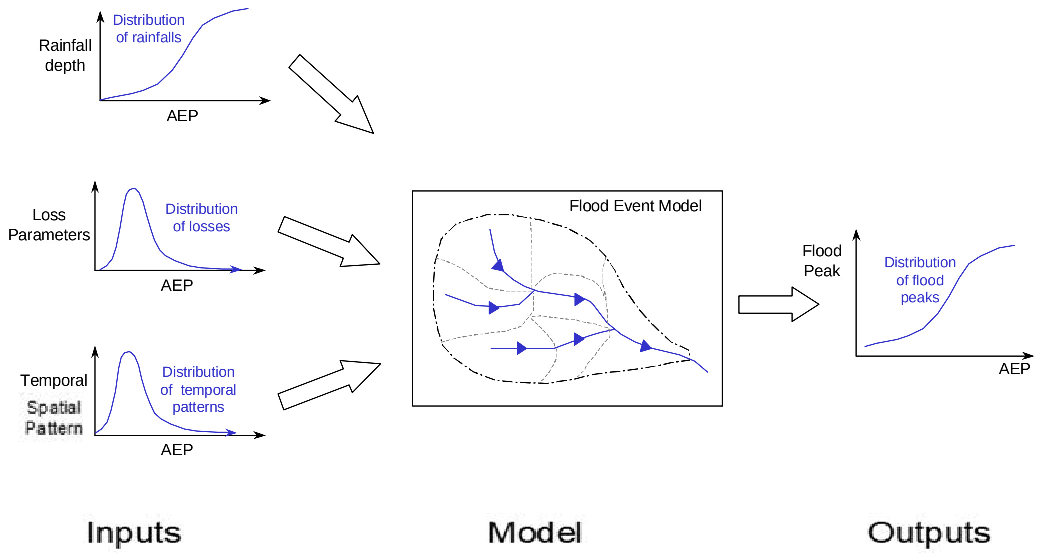
Joint probability techniques offer an improvement to the traditional design event method. These techniques recognise that any design flood characteristics (e.g. peak flow) could result from a variety of combinations of flood producing factors, rather than from a single combination. For example, the same peak flood could result from a moderate storm on a saturated catchment, or a large storm on a dry catchment. In probabilistic terms, a 1 in 100 AEP flood could be the result of a 1 in 50 AEP rainfall on a very wet catchment, or a 1 in 200 AEP rainfall on a dry catchment. Joint probability approaches attempt to consider the influence of the key probability distributed inputs, thereby providing a more realistic representation of the flood generation processes.

The method was adapted to focus on the aspects that are most relevant to the problem. For the Goulburn and Broken Rivers catchment, linked rainfall spatial and temporal patterns (space – time patterns) are important along with losses. The space – time patterns are particularly important on a large catchment to capture the variability in rainfall patterns across the catchment. Therefore, full distributions for each of the main inputs was entered into the model. Figure 6-2 illustrates the method adopted for this project.

The application of joint probability approaches to flood estimation is widely acknowledged to be a more thorough and defensible approach to design flood estimation than the design event approach in Australian practice, and has been incorporated in the 2016 version of Australian Rainfall and Runoff (Ball et al., 2016).

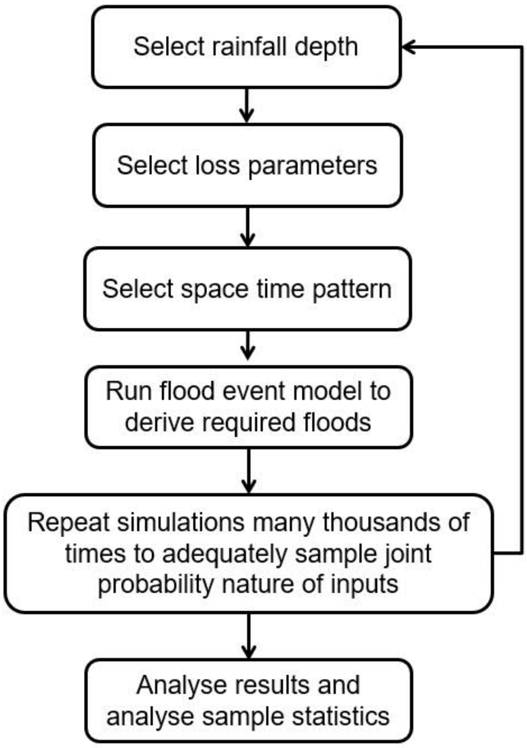


- Figure 6-1 Schematic illustration of the design event approach



- Figure 6-2 Schematic illustration of the joint probability approach

The joint probability framework adopted for the study was developed by Nathan et al (2002, 2003) and is summarised in Figure 6-3. In essence the approach involves undertaking numerous model simulations, where the model inputs are sampled from non -parametric distributions that are based either on readily available design information or on the results of recent research.



- Figure 6-3 Overview of adopted joint probability framework

6.2 Overview of design flood hydrology inputs

In developing the joint probability framework, particular attention was given to ensuring that the model inputs and the manner in which they were incorporated was consistent with ARR2016 (Ball et al., 2016). The following briefly describes the main inputs, and how they relate to established design information.

Select rainfall depth. Rainfall depths were stochastically sampled from the cumulative distribution of rainfall depths described in Section 6.3.

Select storm losses. Storm initial losses were stochastically sampled from a nonparametric distribution that was determined from the analysis of a large number of catchments across Australia (Hill et al., 2014). The limited number of investigations that have explored the correlation between initial and continuing loss values have concluded that there is little systematic dependence between the two. Current practice is for initial losses to be sampled from a distribution (Section 6.6), while the continuing loss is held constant; this approach was used for the design flood modelling.

Select space-time pattern. Sets of linked rainfall space-time patterns were randomly selected from a sample of patterns derived from historic rainfall data over the catchment (Section 6.4).

Monte-Carlo simulation. Simulations were undertaken using a stratified sampling approach in which the sampling procedure focuses selectively on the probabilistic range of interest. Thus, rather than undertake many millions of simulations in order to estimate an event with, say, a 1 in 100 probability of exceedance, a reduced number of simulations were undertaken over a specified number of probability intervals. In this study, the rainfall frequency curve was divided into 100 intervals uniformly spaced over the standardised normal probability domain, and 250 simulations were taken within each division. Thus, a total of 25,000 simulations were undertaken to derive the frequency curve corresponding to each storm duration considered.

6.3 Design rainfall burst depths

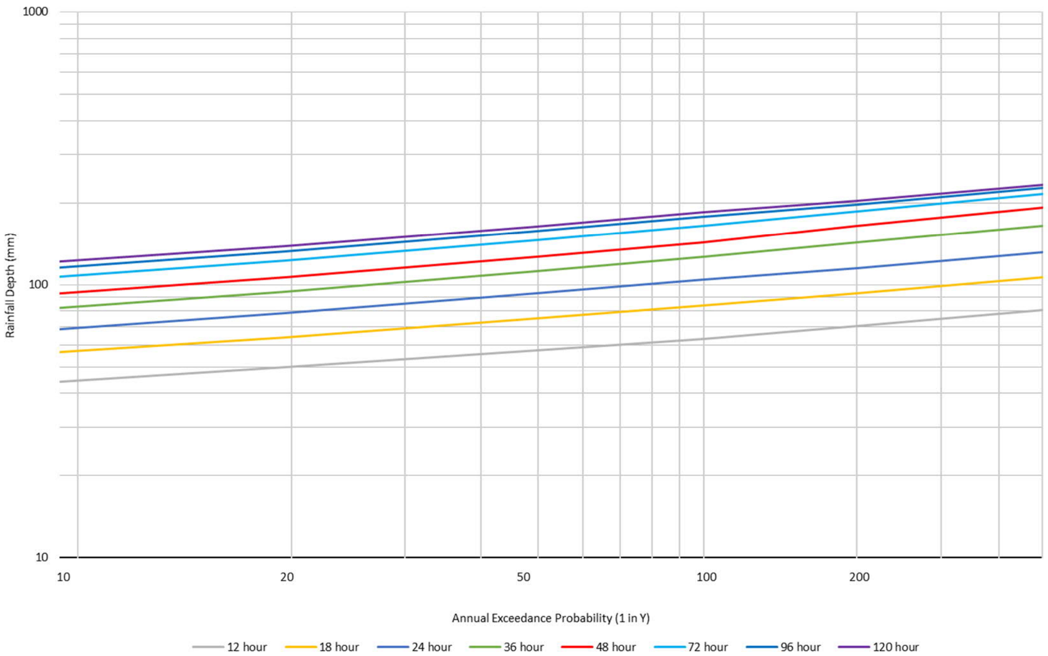
Design rainfall depths were derived for total area of each sub-catchment used in the verification process (refer to Section 7). To ensure that a set of consistent, representative hydrographs could be developed for use in hydraulic modelling, separate sets of design rainfalls were developed for each sub-catchment. This allows for the effects of changing rainfall intensity-frequency-duration (IFD) data and areal reduction factors (ARFs) to be incorporated moving from upstream to downstream.

Design rainfall burst depths up to 1 in 2,000 AEP. Catchment average point design rainfall depths for burst durations between 24 and 120 hours, and AEPs from 1 in 2 to 1 in 2,000, were calculated using the Bureau of Meteorology's IFD2016 product.

Areal reduction factors. The point rainfall estimates were converted to areal values using the ARR2016 areal reduction factors (Jordan et al 2016) extracted from the ARR Data Hub. Conceptually, these factors account for the fact that larger catchments are less likely to experience high intensity storms over the whole catchment. Importantly for this study, as the focus sub-catchment moves from upstream to downstream these ARFs increase significantly.

Probable Maximum Precipitation (PMP) estimates. PMP estimates for the study area where required to enable an indicative Probable Maximum Flood (PMF) to be determined. PMP estimates for burst durations between 24 and 120 hours were obtained using the Generalised Southeast Australia Method (Bureau of Meteorology, 2006). No attempt was made to assign an AEP to the PMP, given the considerable uncertainty associated with this over such a large catchment.

A summary of the complete, catchment average areally reduced design rainfall depths adopted for the Goulburn River at Shepparton sub-catchment are shown in Figure 6-4 and Table 6-1. Summaries of the adopted design rainfall depths for the other sub-catchments are shown in Appendix D.



- Figure 6-4 Adopted design rainfall depths for the Goulburn River at Shepparton
- Table 6-1 Adopted design rainfall depths for the Goulburn River at Shepparton

AEP (1 in Y)	24 hour	36 hour	48 hour	72 hour	96 hour	120 hour
2	45	53	60	69	75	80
5	59	70	79	91	99	105
10	69	82	93	107	116	122
20	79	94	106	123	132	138
50	93	112	127	146	157	164
100	104	127	143	165	177	185
200	115	143	165	186	197	203
500	131	165	192	216	227	233
1000	144	182	215	241	252	258
2000	158	201	239	267	279	285

6.4 Rainfall space-time patterns

Figure 3-11 shows the spatial variability of the largest historical 72 hour duration storm events recorded over the study area catchment. For a catchment of this size it is important to capture the significant variability in rainfall spatial pattern observed from the historic data in the design flood estimation process. To do this, sets of rainfall space-time patterns were developed for each sub-catchment, for a range of storm durations. Each individual space-time pattern consists of a rainfall spatial pattern linked to a number of rainfall temporal patterns derived from pluviograph data. Each temporal pattern is then linked to a particular RORB model sub-area. While complex, this approach avoids any possible introduction of bias from unlinked samples of spatial patterns and temporal patterns, and importantly allows for realistic simulation of the passage of large frontal rainfall systems across the catchment.

The spatial component of each space-time pattern was derived by extracting the total rainfall depth at the centroid of each RORB model sub-area for the particular historic event from AWAP data. These values were then normalized using the catchment average rainfall depth to yield a percentage of catchment average rainfall for each sub-area.

Pluviograph data were then used to disaggregate the total rainfall depths over each sub-area into a temporal pattern. Pluviographs were assigned to each subarea on the basis of geographic proximity (i.e. Thiessen polygons).

Selected space time patterns relevant to each storm duration over the Goulburn River at Shepparton sub-catchment are presented in Appendix E.

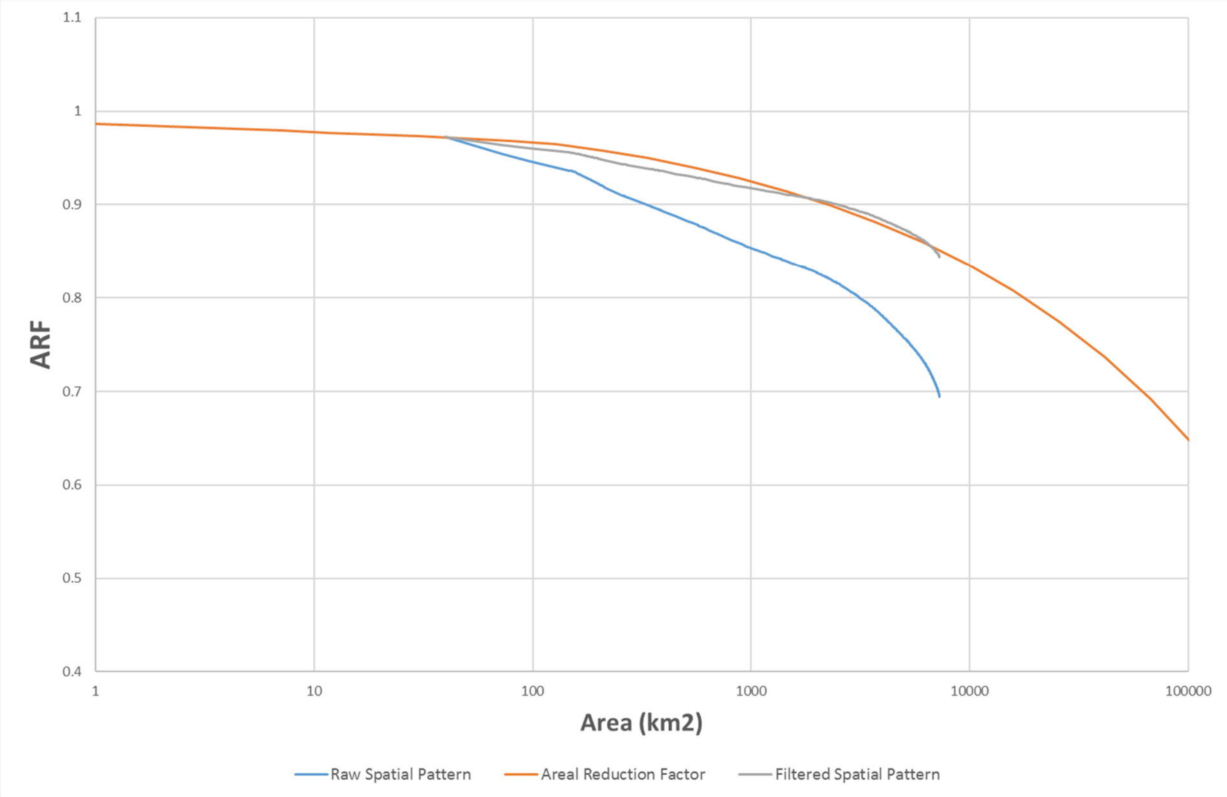
In order to scale the space-time patterns to the randomly sampled rainfall depth for any given individual model run within the Monte Carlo simulation, an areal average temporal pattern was first calculated. This pattern was derived by weighting each temporal pattern by the total area of subareas assigned to it. For each individual model run the areal average temporal pattern is checked and corrected so that no temporal embedded bursts exceed their corresponding rainfalls from the areal rainfall frequency curves. This embedded burst correction procedure is detailed in Section 6.5.

As there is a large spatial variability across the catchment, ten space-time patterns were assembled for each standard storm duration between 24 and 120 hours. For each duration the top ten largest historic rainfall patterns were chosen. Selection of the top ten historic events was limited to the period after 1950 to ensure that there was sufficient availability of pluviograph data to fully describe each pattern.

6.5 Embedded burst filtering

Currently there are only a limited number of examples of the derivation of complete rainfall space-time patterns for use in design flood estimation, and this is an area of further research. One particular area of concern associated with these patterns is the presence of temporal and spatial embedded rainfall bursts within each space-time pattern. Scorah et al (2016) showed the impact of embedded bursts within a temporal pattern. These embedded bursts occur when a sub-set of the rainfall within a given temporal pattern has a rarer AEP than the design storm. This has been shown to cause substantially higher peak flows, if they are not treated. For this project, the rainfall space-time patterns were filtered to remove embedded bursts using the process outlined in Scorah et al. (2016). As noted previously, this filtering was undertaken using a derived catchment average temporal pattern.

Embedded bursts can also occur within the spatial component of the space-time pattern. This occurs when there is large rainfall spatial variability in the historic storm used to derive the space-time pattern. The rainfall depths associated with localised, intense areas of rainfall occurring over certain parts of the sub-catchment become inconsistent with the areal reduction factors applied to determine the design rainfall depths. This results in a much higher rainfall depth being applied to some sub-areas than the corresponding areally reduced rainfall of that particular sub-area. Figure 6-5 shows the effective areal reduction factor of the raw spatial pattern, which is then filtered to minimise the presence of spatial embedded bursts.



- Figure 6-5 Example of inconsistency between areal reduction factors and rainfall spatial pattern

As a practical example of the issue of spatial embedded bursts, one of the rainfall space-time patterns derived for the Goulburn River at Shepparton catchment (10,000 km^2) indicated that one particular 200 km^2 RORB subarea was being assigned 200% of the overall design rainfall depth. This resulted in a much rarer storm being modelled on that portion of the model than the assigned AEP.

To alleviate this problem, in addition to the temporal pattern filtering, each spatial pattern was filtered so that no sub-areas were assigned rainfall depths higher than the corresponding areally reduced rainfall derived for that sub-catchment. This filtering was done using the 1 in 100 AEP design rainfall for each catchment.

For all constituent areas within a sub-catchment, filtering of spatial embedded bursts ensures that the design rainfall applied to any area in the sub-catchment is not greater than the appropriate areally reduced design rainfall for that sub catchment. An example of a filtered and unfiltered rainfall spatial pattern is shown in Figure 6-6.

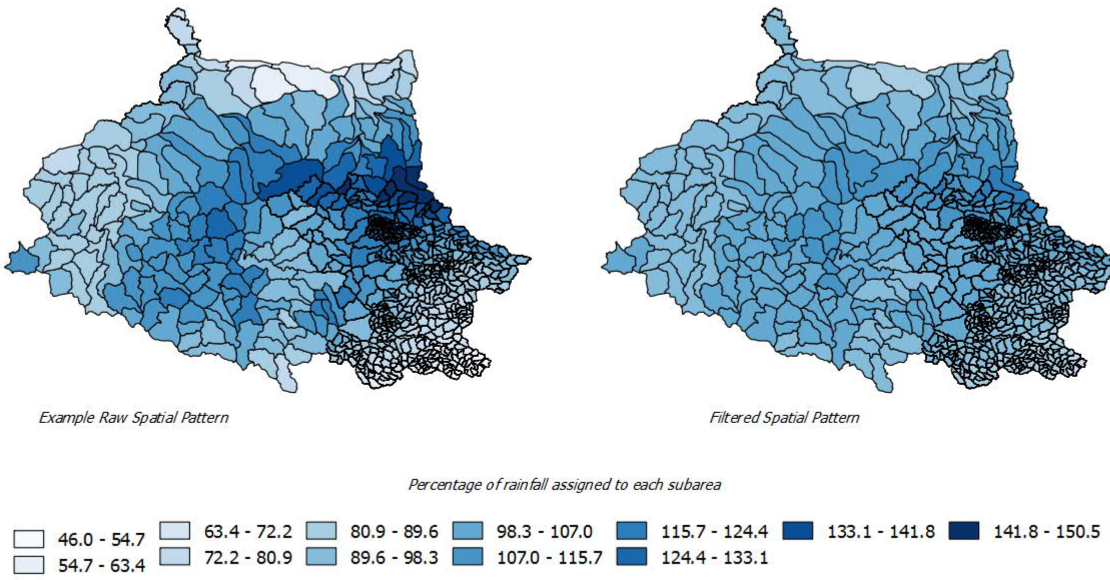


Figure 6-6 Example of filtered and unfiltered rainfall spatial pattern

6.6 Losses

There are two key types of loss models that are typically adopted when modelling design floods:

- Initial loss/continuing loss
- Initial loss/proportional loss

Investigations by Hill et al. (2014) as part of the ARR 2016 revision were inconclusive as to which loss model works best. Even for catchments where one of the loss models performed better for a majority of events, there were still some events for which the other approach was better. Similarly, there was no obvious relationship between the relative performance of the loss models and hydro-climatic or catchment characteristics.

The advice in ARR is that the initial loss/continuing loss model is most suitable for design flood modelling, because it can be used to estimate flood peaks and volumes for all AEPs. In contrast, it is often difficult to derive unbiased estimates of flood quantiles using the initial loss/proportional loss model over the same range of AEPs. The initial loss/proportional loss model underestimates peak flows for extreme floods if the proportional loss is not varied appropriately with AEP; and to date there is little evidence about how proportional loss varies with AEP. Therefore, for this study an initial loss/continuing loss model was adopted.

The shape of the initial loss distribution used in the design flood modelling was derived by Hill et al. (2014) from flood modelling results for a large number of catchments across Australia. Hill et al. (2014) developed a non-dimensional distribution of initial loss values for each catchment, by representing initial losses as a proportion of the median loss. This allowed the distributions of initial losses across different catchments to be directly compared. The standardised distributions exhibited a high degree of consistency, and suggested that while the magnitude of initial losses may vary between different catchments, the shape of the distribution does not. That is, while it may be expected that typical loss rates vary from one catchment to another, the likelihood of a catchment being in a relatively dry or wet state is similar for all catchments. The adopted distribution of initial loss is shown in Figure 6-7.

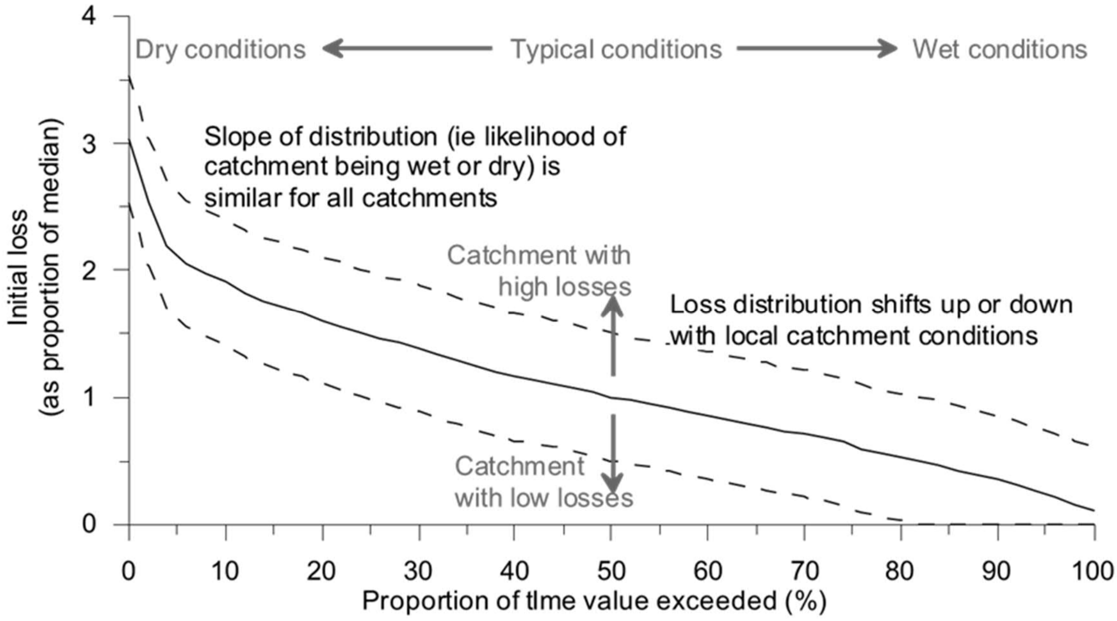


Figure 6-7 Cumulative probability distribution of initial loss

The correlation between initial losses and continuing losses is not well understood. Current practice is for initial losses to be sampled from a distribution, while the continuing loss is held constant; this approach was used for this study.

Values for the median initial loss and constant continuing loss rate for each of the catchments were estimated by verifying the flood quantiles produced by RORB to at-site flood frequency analyses based on observed flood peaks. Details of the verification process and the adopted losses are provided in Section 7.

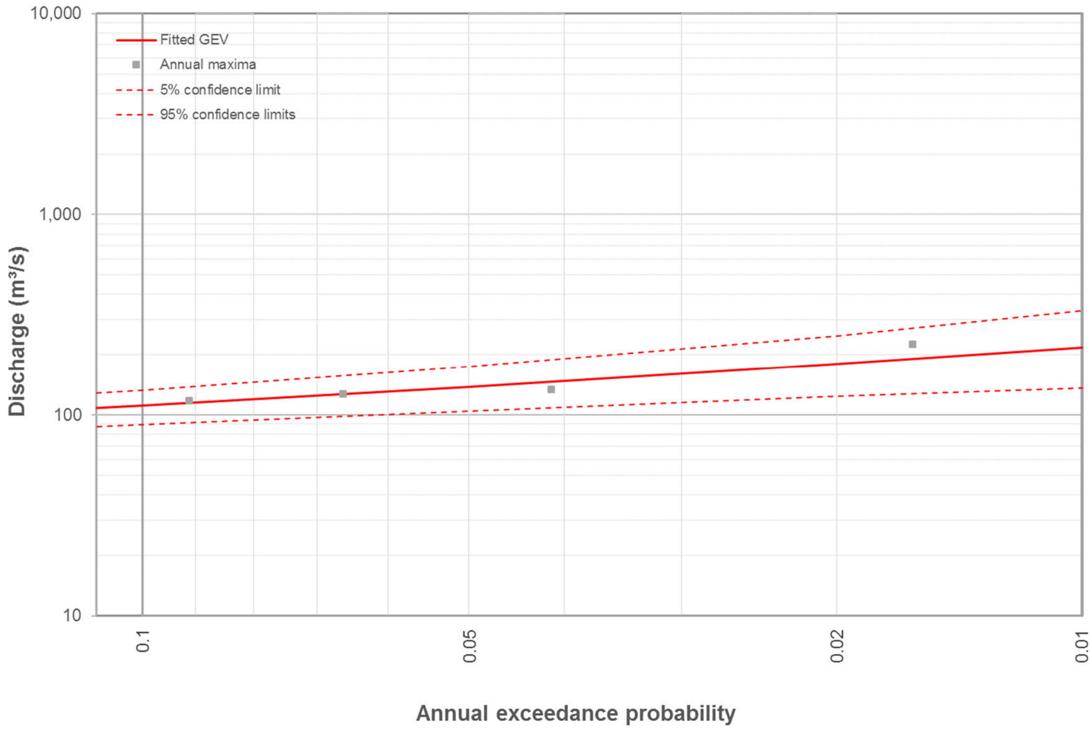
6.7 Baseflow

For each study area sub-catchment, a procedure for incorporating design baseflow estimates, based upon flood frequency analysis of baseflow annual maxima, was adopted. These design baseflow estimates were added to the model at the downstream end of each sub-catchment (typically associated with a key streamflow gauge location).

The Lyne and Hollick (1979) digital baseflow filter was applied to estimate the daily time series of baseflow at each of the sub-catchment outlet streamflow gauges. The filter was applied to the daily time series of total streamflow. The filter parameter was adjusted until the baseflow associated with the peak from the digital filter matched, on average, the manually separated estimates of baseflow under the peak for the calibration flood events (described in Section 5.1). The filter parameter adopted was 0.925 and 3 passes were applied.

A frequency distribution of baseflow with AEP was then estimated by fitting a GEV distribution to the annual maxima series of baseflow associated with peak flood events, using L-Moments. This provided the frequency distribution for baseflow under the peak of the annual maxima flood events. Figure 6-8 shows an example of the GEV distribution at the Goulburn River at Trawool streamflow gauge. Appendix F shows the GEV distributions of baseflow adopted for each study area sub-catchment. These distributions were added to the model at the sub-catchment outlets for both verification and design.

Chapter 6, Book 8 of ARR2016 (Nathan and Weinmann, 2016) states that the proportion of baseflow is typically small compared with direct runoff for very rare to extreme flood events. Therefore, ARR2016 recommends that baseflow for extreme events should be between 20% and 50% greater than the baseflow in observed floods. It was therefore assumed for this study that the baseflow adopted for the indicative PMF event was 50% greater than the estimate of the baseflow for the 1 in 100 AEP design flood.



- Figure 6-8 Fitted flood frequency analysis to baseflow annual maxima for Goulburn River at Trawool

7. Hydrologic model verification

7.1 Method

The initial and continuing losses estimated for the calibration events described in Section 5 may be biased towards low values because large floods are more likely to be produced for catchments with wet antecedent conditions. Therefore, while a small sample of historic events provides useful data for the selection of RORB model routing parameters, these events provide less information about the appropriate losses to use in design flood modelling because the manner in which losses vary with rainfall depends on chance. Therefore, suitable initial and continuing loss values were estimated through a verification process, which involved using the design inputs described in Section 6 and varying the losses until there was an acceptable match between the RORB model results and flood frequency analyses of historic peak flows.

Current practice in design flood estimation includes verification of the results from rainfall runoff modelling (such as RORB) against at-site flood frequency analysis from observed streamflows. Suitable median initial loss and continuing loss values for use in design were estimated using this process, which involved run the model in design mode and varying the losses until there was an acceptable match between the RORB flood frequency quantiles and the gauged flood frequency curves.

There are several streamflow gauges in the study area suitable for development of at-site gauged flood frequency curves. As the focus of this study is the Goulburn River between Lake Eildon and Loch Garry and the Broken River between Lake Nillahcootie and Shepparton, the following gauges were selected for verification:

- Goulburn River at Trawool (405201)
- Goulburn River at Seymour (405202)
- Goulburn River at Murchison (405200)
- Broken River at Casey's Weir (404242)
- Goulburn River at Shepparton (405204)

7.2 At site gauged flood frequency curves

Available gauged water level annual maxima for the verification sites were extracted from WMIS and converted to flow using the rating curves described in Section 3.2. Recorded streamflow data for the Goulburn River at Murchison, which has the longest period of record in the study area, indicates that the two largest flood events prior to the enlargement of Lake Eildon were in 1916 (peak flow 3,600 m^3/s) and in 1934 (peak flow 2,050 m^3/s). After the increase in capacity at Eildon, the four largest events have been 2022 (1863 m^3/s), 1956 (1,782 m^3/s), 1974 (1650 m^3/s) and 1993 (930 m^3/s).

A significant effort was undertaken to establish a uniform flood record on the Goulburn River from Eildon to Shepparton in order to have a consistent estimate of flows from gauge site to gauge site from 1916 onwards. For each of the gauges a flow record was established assuming that Lake Eildon and Lake Nillahcootie were in place in their current configurations and operated under current conditions.

To facilitate the development of stationary flood records at each gauge site, DEECA provided a long term, monthly water resources (REALM) model of the Goulburn and Broken River system. The

REALM model supplied assumed current storage configuration and operating rules applied to the historic climate sequence from 1891 to 2012. As the REALM model is only an estimate of what would have happened historically with the current configuration of Lake Eildon in place the results were used to extend the gauged annual maxima record for years between 1900 and 1955 on the Goulburn River, excluding the annual maxima for 1916 and 1934 which are known to have been large flood events. As these large events will have the biggest influence on the flood frequency curve a different approach was adopted for these events.

The only gauged flow record of the 1916 event on the Goulburn River is at Murchison. Therefore, to estimate the flow for this event at other locations a rainfall storm file was developed for the 1916 event and placed into the calibrated RORB model assuming that Lake Eildon was not there. The model was calibrated to match the recorded historical flow at Murchison. The RORB model was then re-run with Lake Eildon in place to simulate the effect that the dam would have on the downstream gauges. The starting level of Eildon at the time of the storm was taken from the REALM model. The same approach was undertaken for the 1934 event however, an estimate of peak flow for this event was available at Trawool, Murchison and Shepparton. Table 7-1 shows the estimated flows at various locations along the Goulburn River assuming Eildon is in place.

- Table 7-1 Summary of estimated Goulburn River peak flows assuming Eildon is in place

Site	Flow (m^3/s)	
	1916 Event	1934 Event
Goulburn River at Trawool (405201)	1,660	1,160
Goulburn River at Seymour (405202)	1,750	1,150
Goulburn River at Murchison (405200)	1,810	965
Goulburn River at Shepparton (405204)	1,940	815

For flows on the Broken River at Casey's Weir there was no estimate of the 1916 event available at the gauge. Therefore, the flow estimated at Benalla was used for this event, noting that this is uncertain. More detail on the at site gauged flood frequency curve on the Broken River at Casey's weir is given in Section 7.3.4.

Having established a consistent set of annual maxima from 1916 onwards at each verification streamflow gauge, the flood frequency analysis was completed by fitting a Generalised Extreme Value (GEV) distribution to the annual maxima. This was done using the technique of weighted L-Moments.

The RORB model was then run in Monte Carlo simulation mode using the design inputs described in Section 6 to estimate flood frequency quantiles for flood events with AEPs of 10%, 5%, 2% and 1%. The verification process concentrated upon modifying the values of the median initial loss and continuing loss rate parameters to achieve a match between the RORB model and the flood frequency analysis of gauged data.

The verification process effectively anchors the estimates of design flood peaks from the RORB model across the range between 10% and 1% AEP. The verification process improved confidence in design flood estimates from the RORB model, in addition to calibration to selected flood events, because:

- calibration was restricted only to a sub-set of flood events for which there were adequate pluviograph rainfall data to model the flood in RORB, whereas all gauged flood peaks may be incorporated in the verification process, permitting the use of a longer data set of floods and often resulting in the consideration of larger flood events; and
- loss parameters typically vary across a wide range between individual flood events, due to variations in antecedent climatic conditions. Calibration to a small number of flood events provides an unreliable basis for estimating initial and continuing loss parameters, whereas the verification process fits the loss parameters to a much larger sample of flood events and therefore provides a considerably more reliable basis for estimation of loss parameters for design flood event modelling.

To model outflows from Lake Eildon, gate operations were coded into the RORB model based on flood operation information provided by GMW. In addition to the gate operations RORB was also run sampling the initial drawdown in the reservoir from a distribution based on the historic recorded reservoir storage levels from 1956 to date. These aspects are documented in some detail in GMW's Lake Eildon flood hydrology study (HARC, 2017a). The Lake Eildon drawdown curve is shown in Figure 7-1.

The following assumptions were made for the other main storages within the catchment.

- At Goulburn Weir it was assumed that the gates were opened.
- At Lake Mokoan it was assumed that the structure had not been decommissioned. This assumption would only affect the recorded flows at Casey's Weir from 2009 onwards. An investigation by GHD in 2005 indicated that decommissioning Lake Mokoan would have little impact on flooding at Casey's Weir.
- At Lake Nillahcootie the RORB model was also run sampling the initial drawdown in the reservoir from a distribution based on the historic recorded reservoir storage levels from 1967 to date. This is discussed in more detail in GMW's flood hydrology study for the dam (HARC 2017b). The drawdown curve is shown in Figure 7-2.

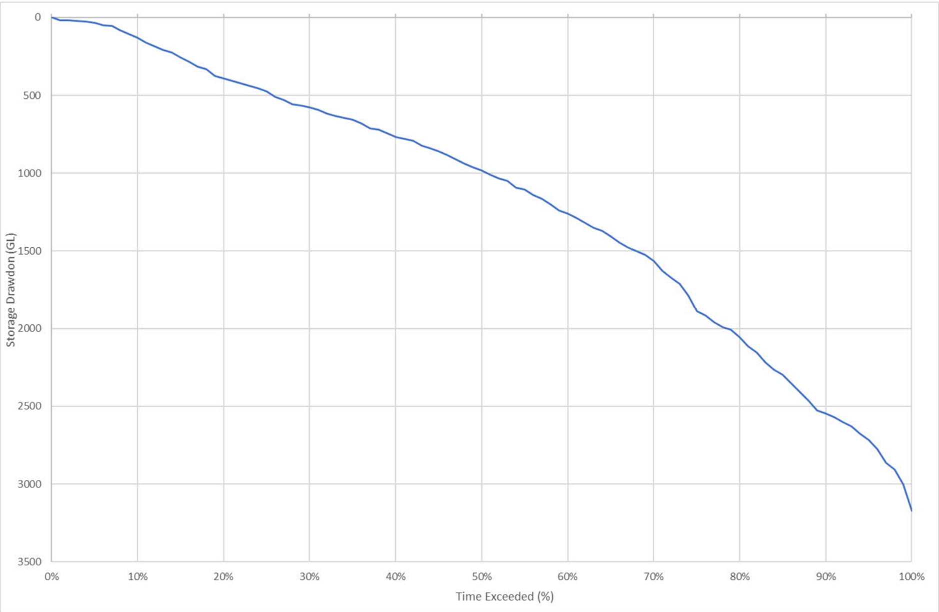
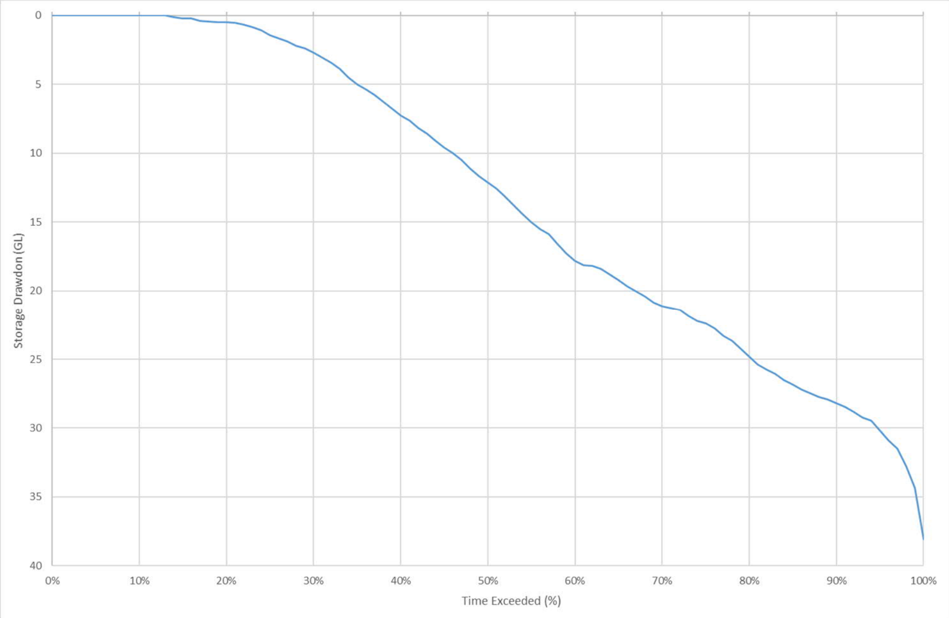
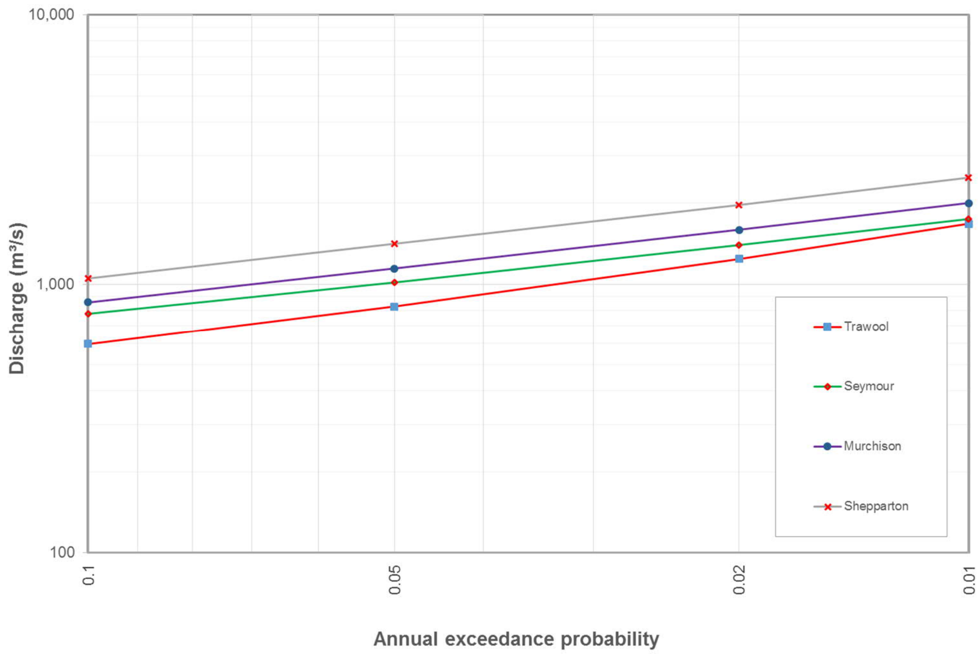


Figure 7-1 Initial drawdown exceedance curve for Lake Eildon



■ Figure 7-2 Initial drawdown exceedance curve for Lake Nillahcootie

The estimation of peak flows along the Goulburn River has in the past been undertaken sporadically as part of separate, town specific flood studies. As such, flow estimates at each gauge location have been prepared at different times with different methods. Given that the aim of the current study was to have a consistent set of design flood estimates from Trawool to Shepparton, some care was taken to ensure the as-site frequency curves derived for each gauge were comparable. Figure 6 8 shows the flood frequency curves for each of the key locations along the Goulburn River. It can be seen that the curves are consistent, with Trawool and Seymour converging for less frequent events due to tributaries, such as the Acheron, Yea and King Parrot Creek etc having more of an influence on flows in the Goulburn River.



■ Figure 7-3 Summary of adopted Goulburn River at-site flood frequency curves

7.3 Verification results

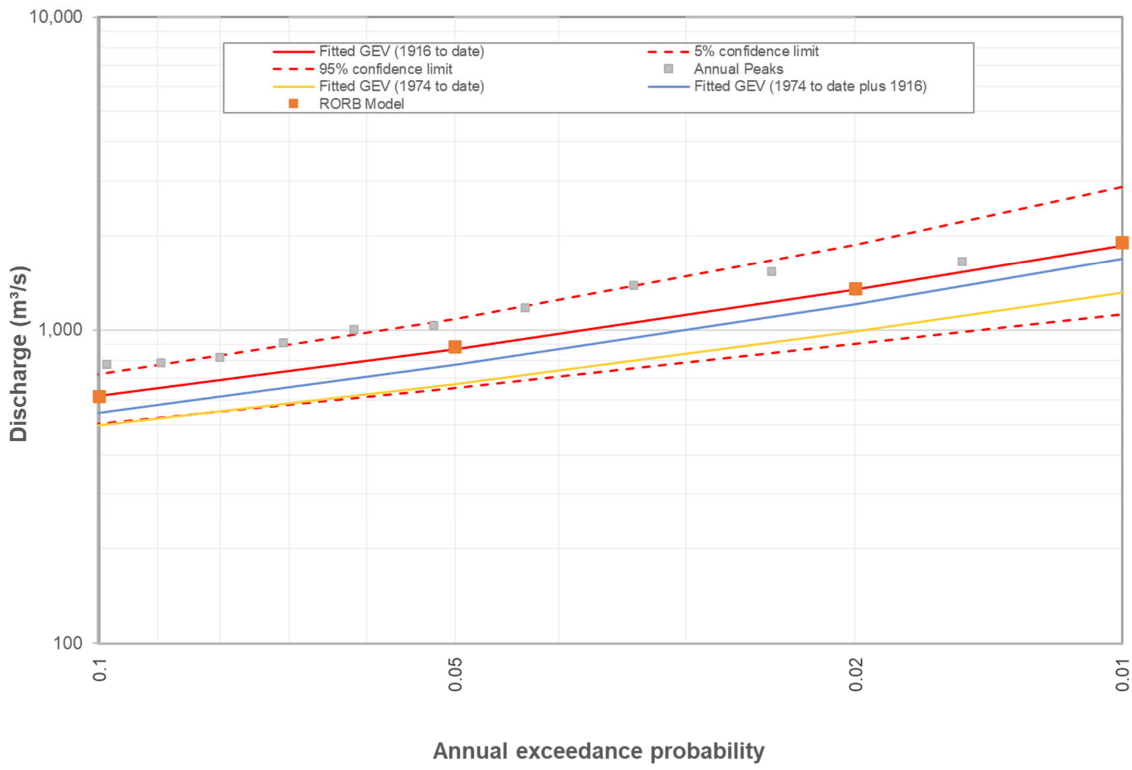
7.3.1 Goulburn River at Trawool

This gauge is located approximately 100 kilometres downstream of Lake Eildon. The gauge is listed as starting in 1908 however, the first full year of data available from WMIS is 1975.

The methodology for establishing a flow series at Trawool was as described in Section 7.2. The only event that was modified from the described methodology was the 1974 event. For the 1974 event the peak flow estimate from the Lower Goulburn River Flood October 1993 Volume 5 (HydroTechnology, 1995b) report of 785 m^3/s was adopted.

Figure 7-4 shows GEV distributions fitted to a range of annual maxima, including the period 1974 to date, 1974 to date plus the 1916 peak and the full composite series from 1916 to date. This demonstrates the importance of considering the full composite period of record, as the flood quantiles are significantly higher when the period prior to 1974 is taken into consideration. The RORB model results shown in Figure 7-4 demonstrate that a good match was achieved between the model and the distribution fitted to annual maxima from 1916 to date for the 10%, 5%, 2% and 1% AEP flood quantiles.

For the Goulburn River at Trawool sub-catchment (and all interstation sub-catchments upstream to Lake Eildon), a median initial loss value of 20 mm and a continuing loss value of 2.9 mm/h were adopted for use in design.



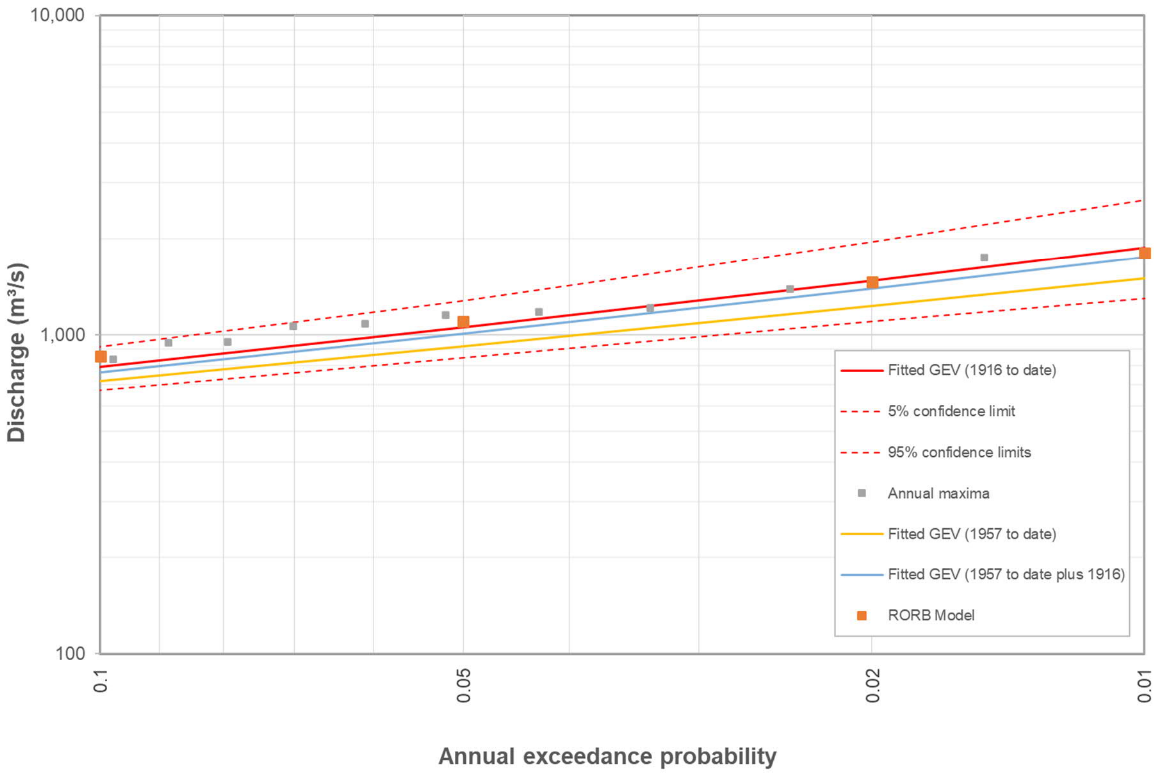
▪ Figure 7-4 RORB model verification results – Goulburn River at Trawool

7.3.2 Goulburn River at Seymour

Recorded flows are available at this gauge from 1957 onwards, which post-dates the major upgrade to Lake Eildon. The methodology for establishing a flow series at Seymour was as described in Section 7.2. The only event that was modified from the described methodology was the 1974 event. For the 1974 event the peak flow estimate from the Lower Goulburn River Flood October 1993 Volume 5 (HydroTechnology, 1995b) report of 1,215 m³/s was adopted.

Figure 7-5 shows GEV distributions fitted to a range of annual maxima, including the period 1974 to date, 1974 to date plus the 1916 peak and the full composite series from 1916 to date. This demonstrates the importance of considering the full composite period of record, as the flood quantiles are significantly higher when the period prior to 1974 is taken into consideration. The RORB model results shown in Figure 7-5 demonstrate that a good match was achieved between the model and the distribution fitted to annual maxima from 1916 to date for the 10%, 5%, 2% and 1% AEP flood quantiles.

For the Goulburn River at Seymour sub-catchment (and all interstation sub-catchments upstream to Trawool), a median initial loss value of 15 mm and a continuing loss value of 0.5 mm/h were adopted for use in design.



- Figure 7-5 RORB model verification results – Goulburn River at Seymour

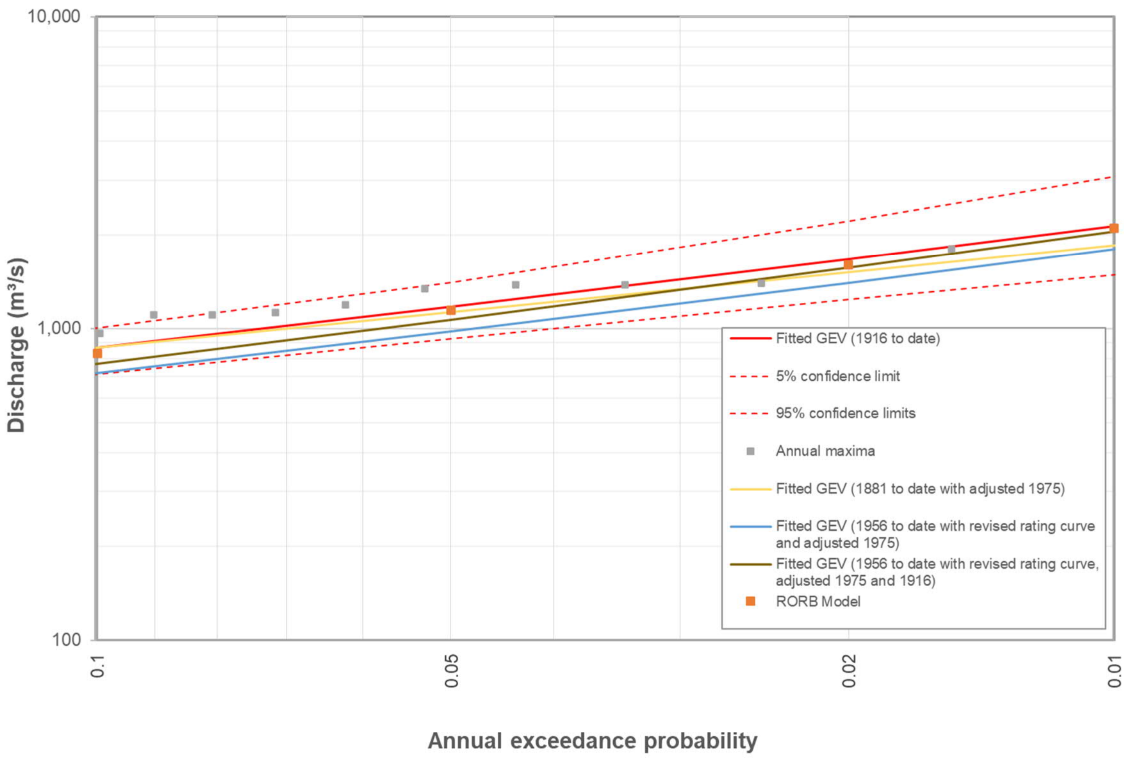
7.3.3 Goulburn River at Murchison

This gauge has the longest period of record on the Goulburn River, with data available from 1881. The methodology for establishing a flow series at Murchison was as described in Section 7.2. For this gauge the streamflow data review, as documented in Section 3.1, indicated that the 1975 estimate of

4,750 m^3/s was in error. For the 1975 event the flow recorded at Goulburn Weir of 800 m^3/s was adopted for all flood frequency curves.

Figure 7-6 shows GEV distributions fitted to a range of annual maxima, including the period 1916 to date, 1881 to date, 1956 to date with flows adjusted based on the revised rating curve plus 1956 to date with flows adjusted based on the revised rating curve and 1916 peak. The RORB model results shown in Figure 7-6 demonstrate that a good match was achieved between the model and the distribution fitted to annual maxima from 1916 to date for the 10%, 5%, 2% and 1% AEP flood quantiles.

For the Goulburn River at Murchison sub-catchment (and all interstation sub-catchments upstream to Seymour), a median initial loss value of 40 mm and a continuing loss value of 3.5 mm/h were adopted for use in design.



- Figure 7-6 RORB model verification results – Goulburn River at Murchison

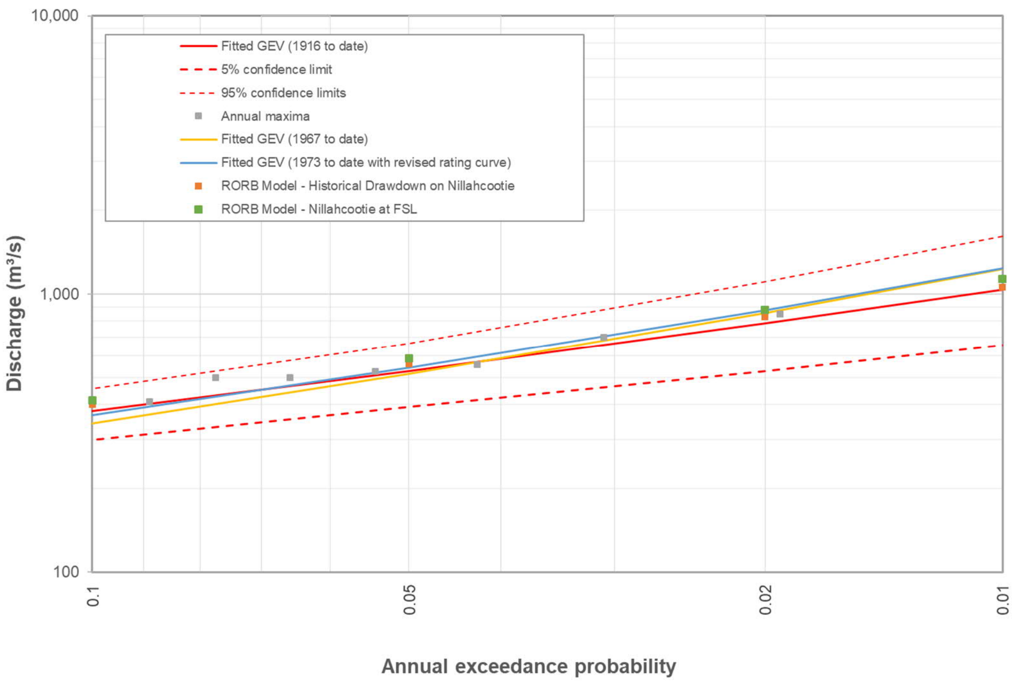
7.3.4 Broken River at Casey's Weir

At this location there are two gauges of interest, Broken River at Goorambat (Casey Weir Head Gauge - 404216) and Broken River at Goorambat (Casey Weir Tail Gauge - 404200). The tail gauge has data from 1888 to June 1916 then from 1973 to date. The head gauge has data from July 1916 to 1972.

The final flow series was a combination of flow estimates from historical estimates at Benalla and gauged information. Level data is only available from 1972 onwards, therefore from 1972 onwards the flow was adjusted based on the rating curve from the hydraulic model. The Benalla Flood Plain Management Study (SRWSCV, 1984), which has then been adopted by subsequent investigations, estimated the flow at Benalla in 1916 as 850 m^3/s. The next largest events at Benalla were in 1921, 1958, 1954 and 1966 with 700 m^3/s, 650 m^3/s, 500 m^3/s and 500 m^3/s respectively. The flows reported in the Benalla Flood Plain Management Study (SRWSCV, 1984) were adjusted to account for Lake Nillahcootie noting that it was assumed that Lake Nillahcootie was full at the start of each flood. Another consideration is that the flow estimates at Benalla be adjusted for use at Casey's weir. Plotting the concurrent annual peak recorded flows at Benalla and Casey's weir indicates that the flows are similar and no adjustment was required.

Figure 7-7 shows that the GEV distribution fitted to the annual maxima from 1916 to date (to remain consistent with the Goulburn River flood frequency analysis). It also shows the GEV distributions fitted to the annual maxima from 1967 (when Lake Nillahcootie was completed) to date and the annual maxima from 1972 to date based on the adjusted flows from the revised rating curve. The RORB model results shown in Figure 7-7 demonstrate that a good match was achieved between the model and the distribution fitted to annual maxima from 1916 to date for the 10%, 5%, 2% and 1% AEP flood quantiles. To test the impact of the assumption that the storage was full at the start of the flood the RORB model was run assuming the storage was full at the start of the storm and with the drawdown curve shown in Figure 7-2. Figure 7-7 shows that the results are not sensitive to the assumption on drawdown.

For the Broken River at Casey's Weir sub-catchment (and all interstation sub-catchments upstream to Lake Nillahcootie), a median initial loss value of 35 mm and a continuing loss value of 1.4 mm/h were adopted for use in design.



- Figure 7-7 RORB model verification results – Broken River at Casey's Weir

7.3.5 Goulburn River at Shepparton

This gauge is located on the Goulburn River immediately upstream of the Midland Highway bridge. Data is available from 1921 onwards, and the methodology for establishing a composite flow series was as described in Section 7.2.

Figure 7-8 shows GEV distributions fitted to a range of annual maxima, including the period 1956 to date, 1956 to date plus the 1916 peak, 1921 to date, 1921 to date plus the 1916 peak and the full composite series from 1916 to date. This demonstrates the importance of considering the full composite period of record, as the flood quantiles are significantly higher when the period prior to 1974 is taken into consideration. The RORB model results shown in Figure 7-8 demonstrate that a good match was achieved between the model and the distribution fitted to annual maxima from 1916 to date for the 10%, 5%, 2% and 1% AEP flood quantiles.

For the Goulburn River at Shepparton sub-catchment (and all interstation sub-catchments upstream to Murchison and Casey's Weir), a median initial loss value of 20 mm and a continuing loss value of 2 mm/h were adopted for use in design.

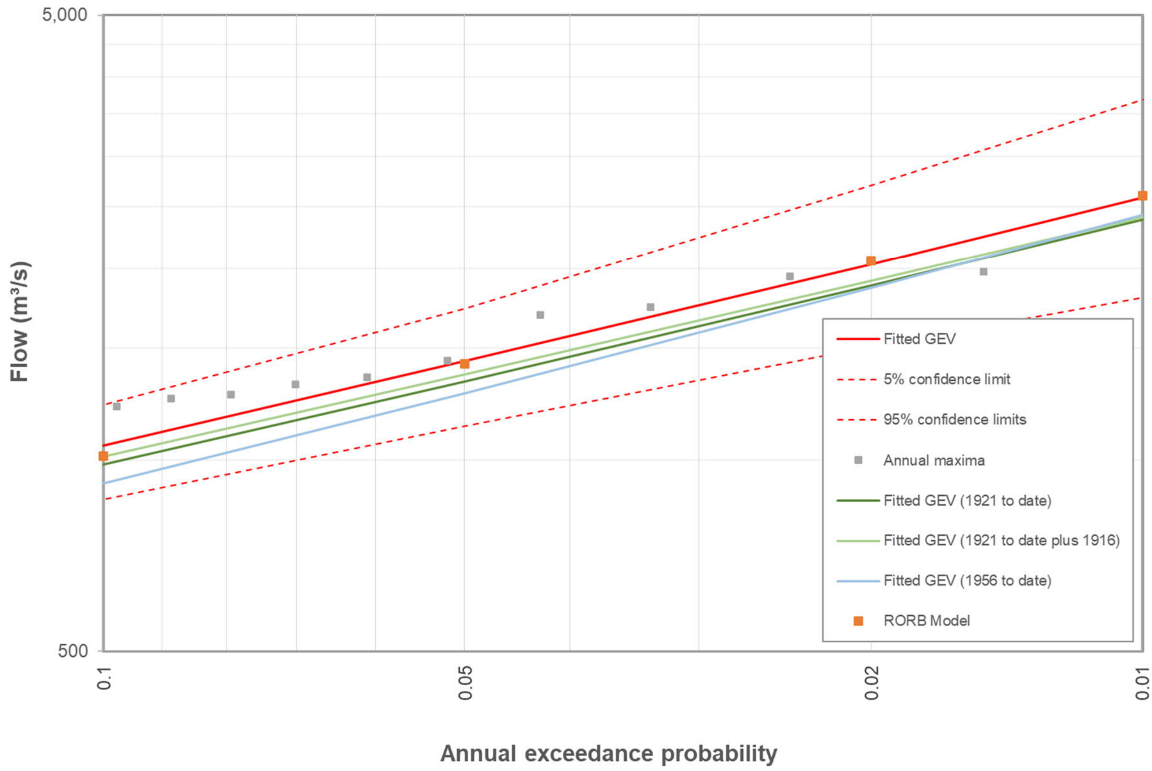


Figure 7-8 RORB model verification results – Goulburn River at Shepparton

7.4 Summary of verification results and adopted loss parameter values

Table 7-2 summarises the adopted routing (from calibration) and loss (from verification) model parameter values adopted for design.

Table 7-2 Summary of adopted RORB model parameter values

Sub-catchment	k_c	m	IL (mm)	CL (mm/hr)
Rubicon River at Rubicon	25	0.8	20	2.9
Acheron River at Taggerty	37			
Home Creek at Yark	8			
Yea River at Goulburn Valley Water Pumping Station	30			

<table>
<tr><th>Sub-catchment</th><th>k_c</th><th>m</th><th>IL (mm)</th><th>CL (mm/hr)</th></tr>
<tr><td>King Parot Creek at Fairview Road Bridge Kerrisdale</td><td>38</td><td rowspan="16"></td><td rowspan="2"></td><td rowspan="2"></td></tr>
<tr><td>Goulburn River at Trawool</td><td>135</td></tr>
<tr><td>Sugarloaf Creek at Ashbridge</td><td>40</td><td rowspan="3">15</td><td rowspan="3">0.5</td></tr>
<tr><td>Sunday Creek at Tallarook</td><td>37</td></tr>
<tr><td>Goulburn River at Seymour</td><td>25</td></tr>
<tr><td>Hughes Creek at Tarcombe</td><td>25</td><td rowspan="3">40</td><td rowspan="3">3.5</td></tr>
<tr><td>Major Creek at Greytown</td><td>25</td></tr>
<tr><td>Goulburn River at Murchison</td><td>158</td></tr>
<tr><td>Pranjip Creek at Moorlim</td><td>168</td><td rowspan="3">20</td><td rowspan="3">2</td></tr>
<tr><td>Castle Creek at Arcadia</td><td>150</td></tr>
<tr><td>Seven Creek at Kialla West</td><td>110</td></tr>
<tr><td>Holland Creek at Kelfeera</td><td>18</td><td rowspan="4">35</td><td rowspan="4">1.4</td></tr>
<tr><td>Broken River at Casey's Weir</td><td>67</td></tr>
<tr><td>Broken River at Gowangardie</td><td>123</td></tr>
<tr><td>Broken River at Orrvale</td><td>38</td></tr>
<tr><td>Goulburn River at Shepparton</td><td>132</td><td>20</td><td>2</td></tr>
</table>

8. Hydraulic Model

This section documents the hydraulic model development and calibration process undertaken for the study. The model has been developed to provide the flood mapping component of the study. Specifically, this sections documents:

- The hydraulic model development; and
- Calibration and validation of the hydraulic model.

8.1 Hydraulic Model Development

To produce the various mapping outputs required for the study, specifically flood extent, flood depth, velocity, hazard and other hydraulic properties, a two-dimensional (2D) hydraulic model was developed. For the Goulburn and Broken Rivers Flood Study a linked 1D/2D hydraulic model was developed using TUFLOW Highly Parallelised Computing (HPC). TUFLOW HPC solver uses an explicit finite volume solution scheme.

Within the TUFLOW HPC model the waterway and floodplain were represented in the 2D domain, with culverts and weir control structures represented as embedded 1D elements. The benefits of modelling the waterways and floodplain in the 2D domain include:

- Explicitly represents the spilling and remerging of flows between the waterway and the floodplain;
- Explicit modelling of bend losses;
- Accounts for contraction and expansion losses through constrictions; and
- Better representation of velocity across the waterway by providing cell-by-cell velocities across the waterways rather than limited to a horizontally averaged velocity.

Model inflows were extracted from the hydrologic model (RORB) developed for the catchment.

8.1.1 TUFLOW Model Version

Model runs were performed with the latest version (at time of assessment) build of TUFLOW HPC, specifically, 2020-10-AE-iSP-w64.

8.1.2 Modelling Events

The hydraulic model was run for the calibration and validation events. The October 2022 and October 1993 events were adopted for the calibration and validation events respectively.

8.1.3 Model Extent

The upstream extent of the hydraulic model is the outflow from Lake Eildon on the Goulburn River and the outflow from Lake Nillahcootie on the Broken River. The model follows both rivers to their confluence just south (upstream) of Shepparton and the downstream boundary of the model is approximately 26 km north (downstream) of Shepparton.

The width of the model around the river varies with the width of the floodplain and availability of high-quality digital elevation model (DEM) data. The model extent is shown in Figure 8-9.

8.1.4 Topography

The geometry of the 2D floodplain and watercourses were established by reading in a uniform grid of square elements from the DEM. This grid (or zpt layer) forms the basis of the hydraulic model. The DEM used in the hydraulic model was based on digital elevation datasets (LiDAR) as supplied by DEECA and GBCMA. The LiDAR was supplied as processed regularly spaced grids rather than the raw irregular data. This pre-processed LiDAR data was used to generate a Digital Elevation Model (DEM) of the study area and surrounds. Several LiDAR sources were supplied which were captured at different times, to different levels accuracies and different coverage areas.

8.1.4.1 LIDAR Datasets

This section details the various data sources used for this study as well as the quality assurance checks undertaken to provide confidence in the accuracy and use of the data and to determine the appropriate hierarchy in the creation of the DEM.

DEECA and GBCMA supplied LiDAR from six different sources:

- 2007 Fugro;
- 2011 ISC Rivers;
- 2011 Floodplains;
- 2013-14 North East Towns Elevation;
- 2016-17 North East (Broken Ck and Kialla LiDAR); and
- 2016 Granite Creek.

In addition to the above the hydrologically reinforced Shuttle Radar Topography Mission (SRTM-H) DEM was sourced from Geoscience Australia.

With each of the datasets, metadata was provided, detailing the accuracy of each of the datasets. The reported accuracy for each dataset is summarised in Table 8-1.

Table 8-1 Summary of Reported LIDAR Accuracy

Data Source	Capture Date	Vertical Accuracy	Horizontal Accuracy	Resolution
Fugro	2007	unknown	unknown	1 m
ISC Rivers	2011	unknown	unknown	1 m
Floodplains	2011	unknown	unknown	1 m
North East Towns*	2013 – 2014	± 0.13 m	± 27 cm	1 m
Broken Ck and Kialla LiDAR	2016 – 2017	± 10 m**	± 15 cm	1 m
Granite Creek	2016	± 0.1 m	± 0.18 m	1 m
Hydrologically reinforced Shuttle Radar Topography Mission (SRTM-H)	2011	unknown	unknown	1 second ~30m

* Different accuracy reported in different sections as this is a merged dataset
** Although this is the stated accuracy in the metadata, it is believed to be a typographical error, 10 cm is considered the more likely value

The extent of each LIDAR dataset is shown in Figure 8-1 except the SRTM-H DEM which covers the full study area. To generate a DEM of the study area where multiple LiDAR datasets exist it is good practice to review each to determine the hierarchy for their use within the model. Typically, the most reliable data is used as a priority with lower performing datasets used to infill areas where better data doesn't exist.

Once the accuracy of each LiDAR dataset is determined the best performing datasets are stamped on one another to form a single cohesive DEM of the study area. Through this process the best possible outcome for the study was achieved as the final DEM will be used in the hydraulic modelling to represent the existing ground surface.

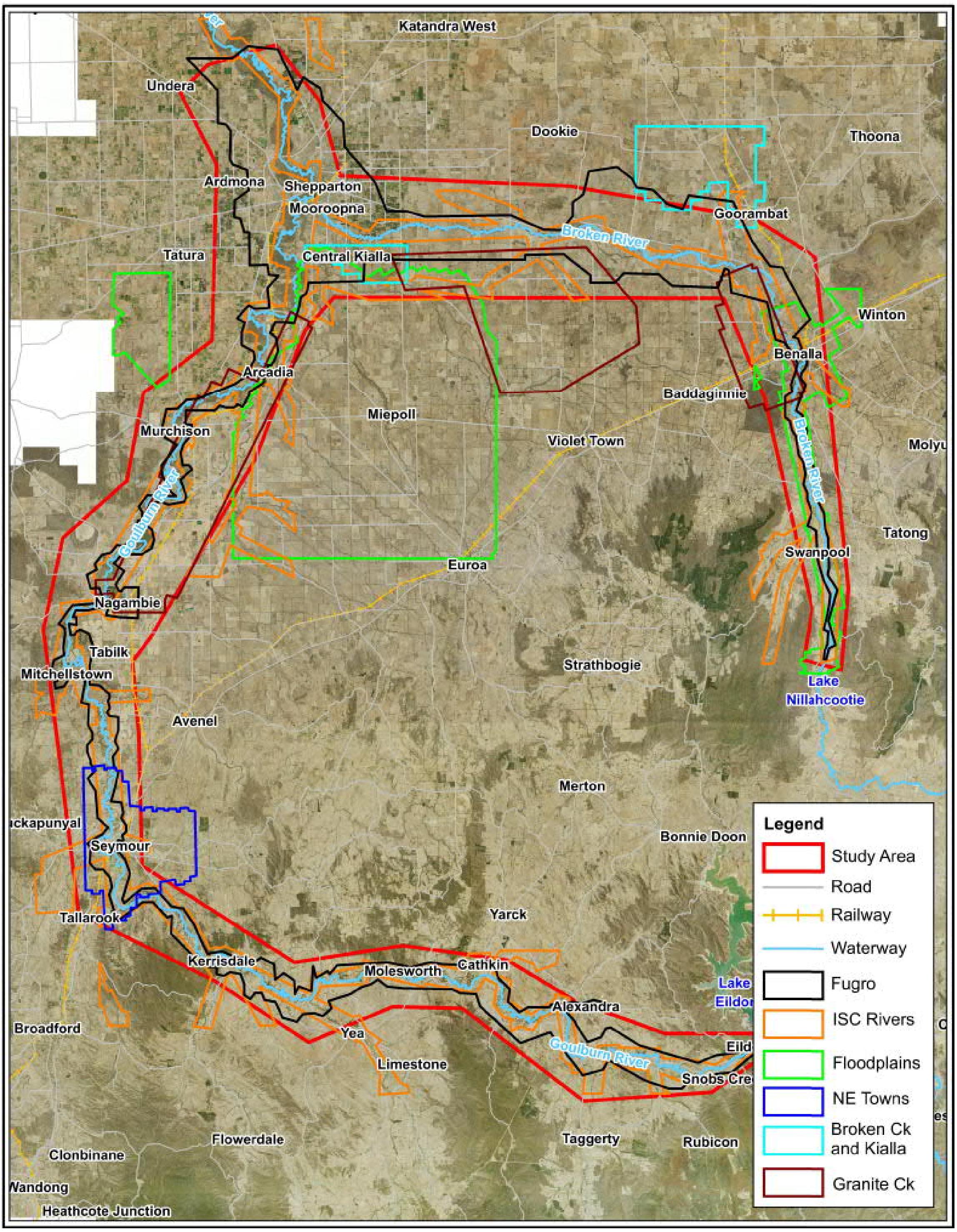


Figure 8-1 LiDAR Extents

8.1.4.2 Permanent Survey Marks Comparison

Permanent Survey Marks (PSMs)[1] were sourced from DEECA who are the controlling agency and manage the survey mark network in Victoria. Specifically, the PSMs are managed by the State Government by the Surveyor-General for the purpose of aiding land surveying, protection of cadastral information, land administration and position of infrastructure. In total 2,095 PSMs with a vertical order (accuracy) of 3 (nearest mm) or better were found to intersect the various LiDAR data sources. An initial comparison of the DEMs to the complete dataset showed considerable differences between the DEMs and the levels in the PSMs to the point that any statistical comparison was rendered meaningless. A review of a number of PSMs spread throughout the study area with the largest differences was undertaken. This review indicated the likelihood of considerable horizontal error in the location of the PSMs with many marks being located in the middle of paddocks and even within waterways. This is due to the majority of the PSMs horizontal coordinates being originally scaled from 1:100,000 maps. Whilst it is possible to download the original PSM information to manually attempt to rectify a PSM it is not practical or feasible for a study area of this size. Therefore, it was decided to limit the comparison of survey marks to only those with surveyed horizontal coordinates. By selecting only those PSMs with a high degree of confidence in their accuracy in terms of vertical and horizontal coordinates resulted in 230 marks that were usable for the assessment. That isn't to say that many of the culled marks were not were reliable, however their reliability could not reasonably be determined, and likewise the adopted marks are not necessarily all perfect, however the approach adopted is reasonable given the available information and size of the study area.

Each PSM was compared to each LiDAR dataset and the difference calculated. From this a number of statistical assessments were undertaken to determine the vertical accuracy of each dataset. Due to the different coverage areas of each dataset, not all PSMs intersected with every LiDAR data source. The quoted and statistical assessments of the vertical accuracy of each of the LiDAR data sets is presented below in Table 8-2. Histogram plots have been provided for the Fugro, ISC Rivers, Floodplains, Granite Creek, and the Broken/Kialla DEMs in Figure 8-2 through Figure 8-6.

Of the seven datasets assessed, the Fugro followed by the ISC Rivers datasets were found to most accurately represent the ground surface based on the assessment of the PSMs within the study area. Both of these datasets cover the largest area, therefore having many more PSMs to assess, have low means, medians and quartiles. The North East Towns dataset was found to have a bias low of around 10 cm, whilst the other datasets tended to be biased high. Unsurprisingly the SRTM- H DEM is the most inaccurate, with a significant high bias.

A spatial comparison of the each PSM and the Fugro and ISC Rivers DEMs is shown in Figure 8-7 and Figure 8-8 respectively.

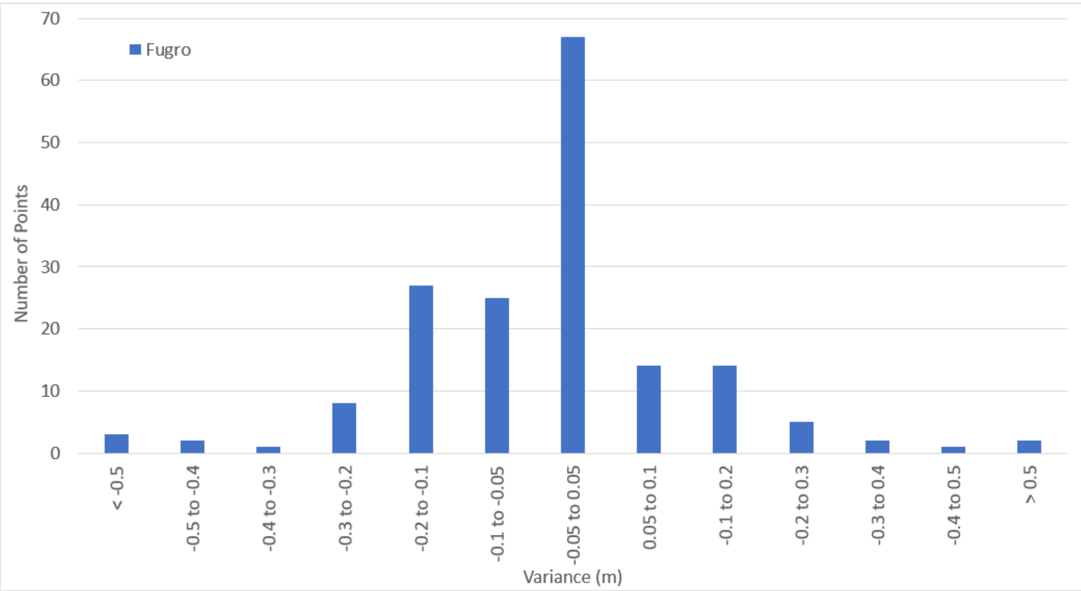
▪ Table 8-2 Summary of Reported LIDAR Accuracy – PSM Comparison

Data Source	Fugro	ISC Rivers	DEPI Floodplains	North East Towns	Broken / Kialla	Granite Creek	SRTM-H
Quoted Accuracy	Unknown	Unknown	Unknown	± 0.13 m	± 10 m*	± 0.18 m	n/a
No. of PSM points	171	51	21	4	8	42	397

[1] PSMs are objects placed to mark key survey points on the Earth's surface.

Data Source	Fugro	ISC Rivers	DEPI Floodplains	North East Towns	Broken / Kialla	Granite Creek	SRTM-H
Mean (m)	-0.02	-0.01	0.07	-0.08	0.17	0.18	0.80
Median (m)	-0.02	0.03	0.11	-0.10	0.10	0.07	0.76
Standard Deviation (m)	0.17	0.40	0.11	0.04	0.19	0.38	2.97
Lower Quartile (m)	-0.10	-0.01	0.00	-0.11	0.05	0.03	-0.02
Upper Quartile (m)	0.04	0.13	0.14	-0.03	0.39	0.21	1.67

* Although this is the stated accuracy in the metadata, it is believed to be a typographical error, 10 cm is considered the more likely value



■ Figure 8-2 PSM – LiDAR Comparison – Fugro

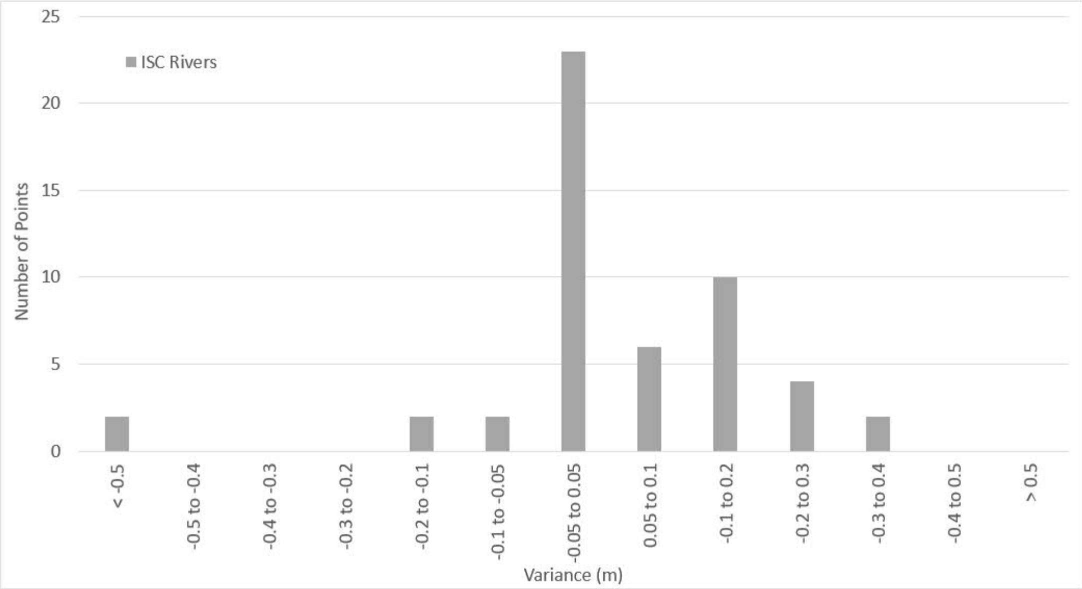


Figure 8-3 PSM – LiDAR Comparison – ISC Rivers

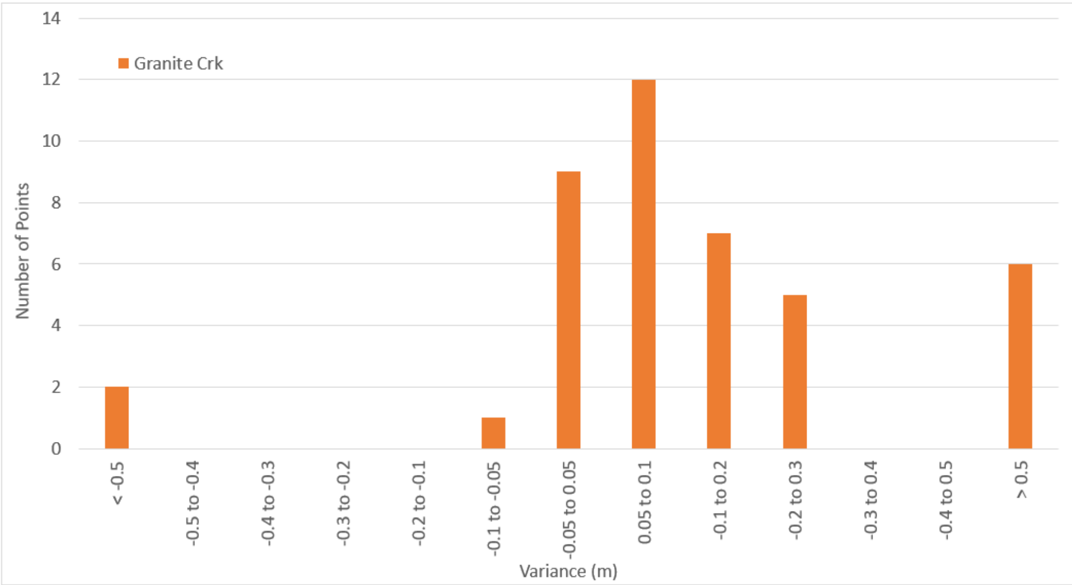


Figure 8-4 PSM – LiDAR Comparison – Granite Creek

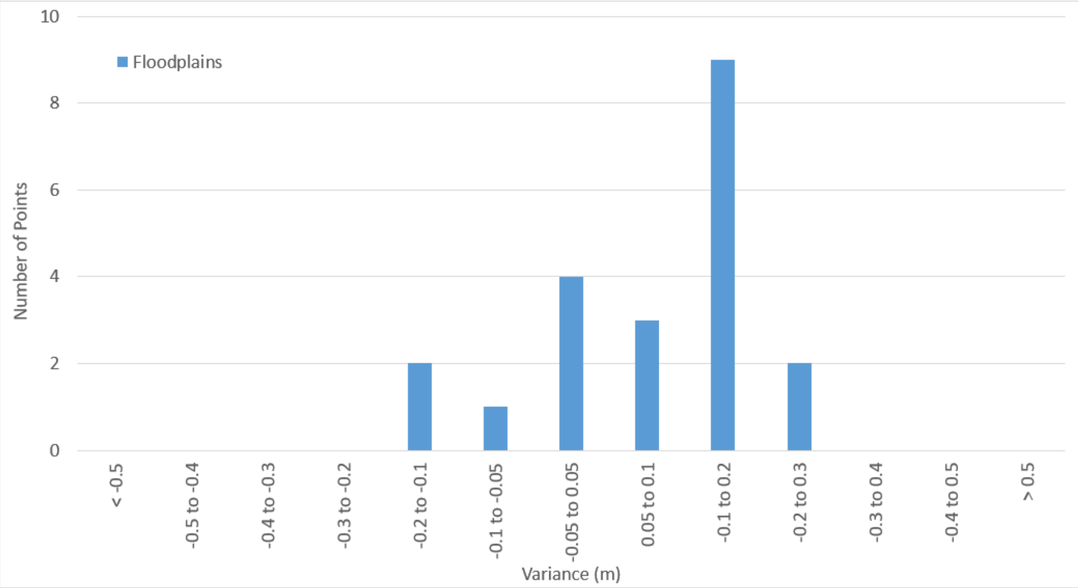


Figure 8-5 PSM – LiDAR Comparison – Floodplains

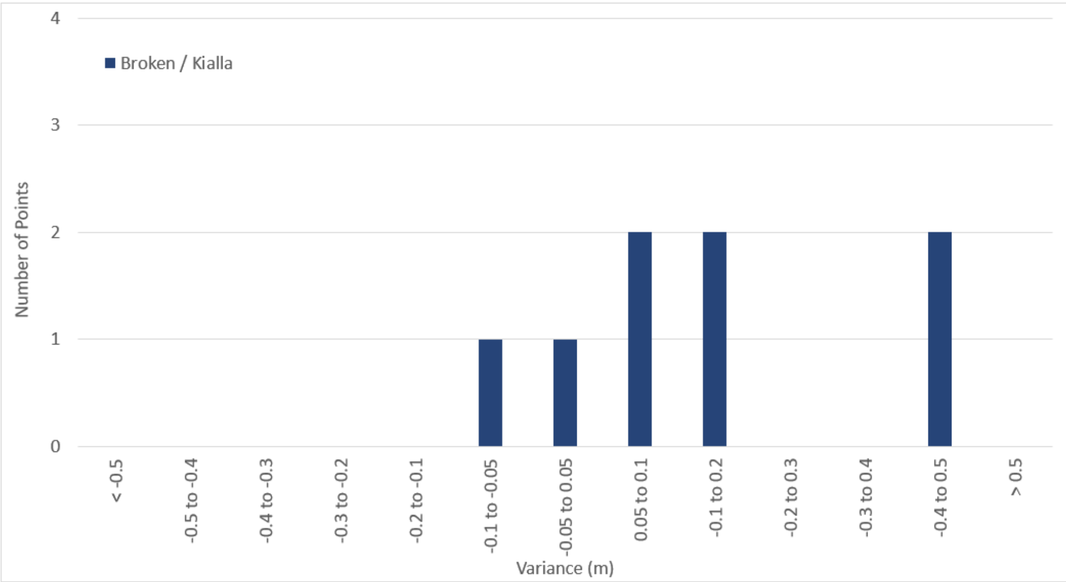
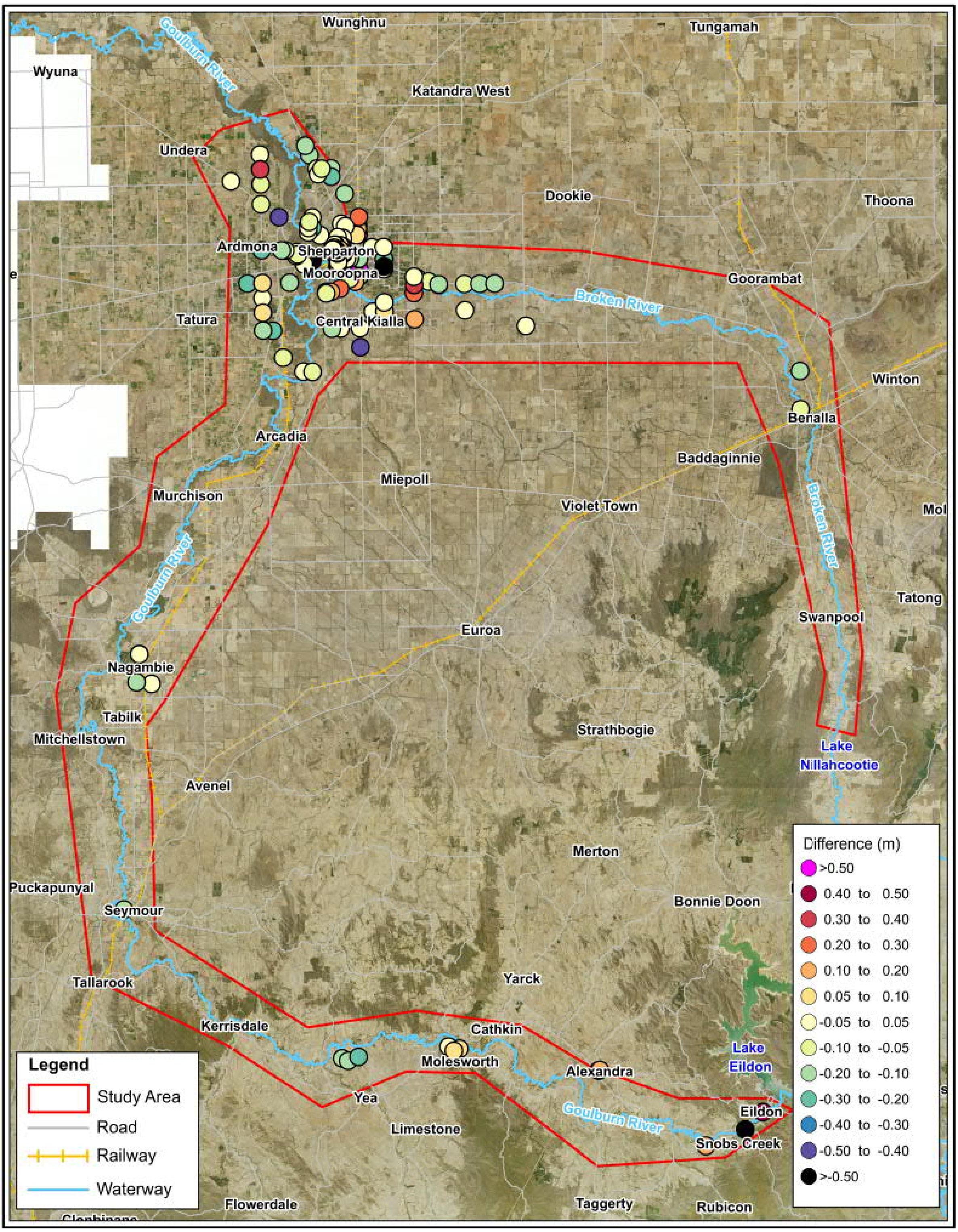


Figure 8-6 PSM – LiDAR Comparison – Broken / Kialla



■ Figure 8-7 Fugro and PSM Comparison

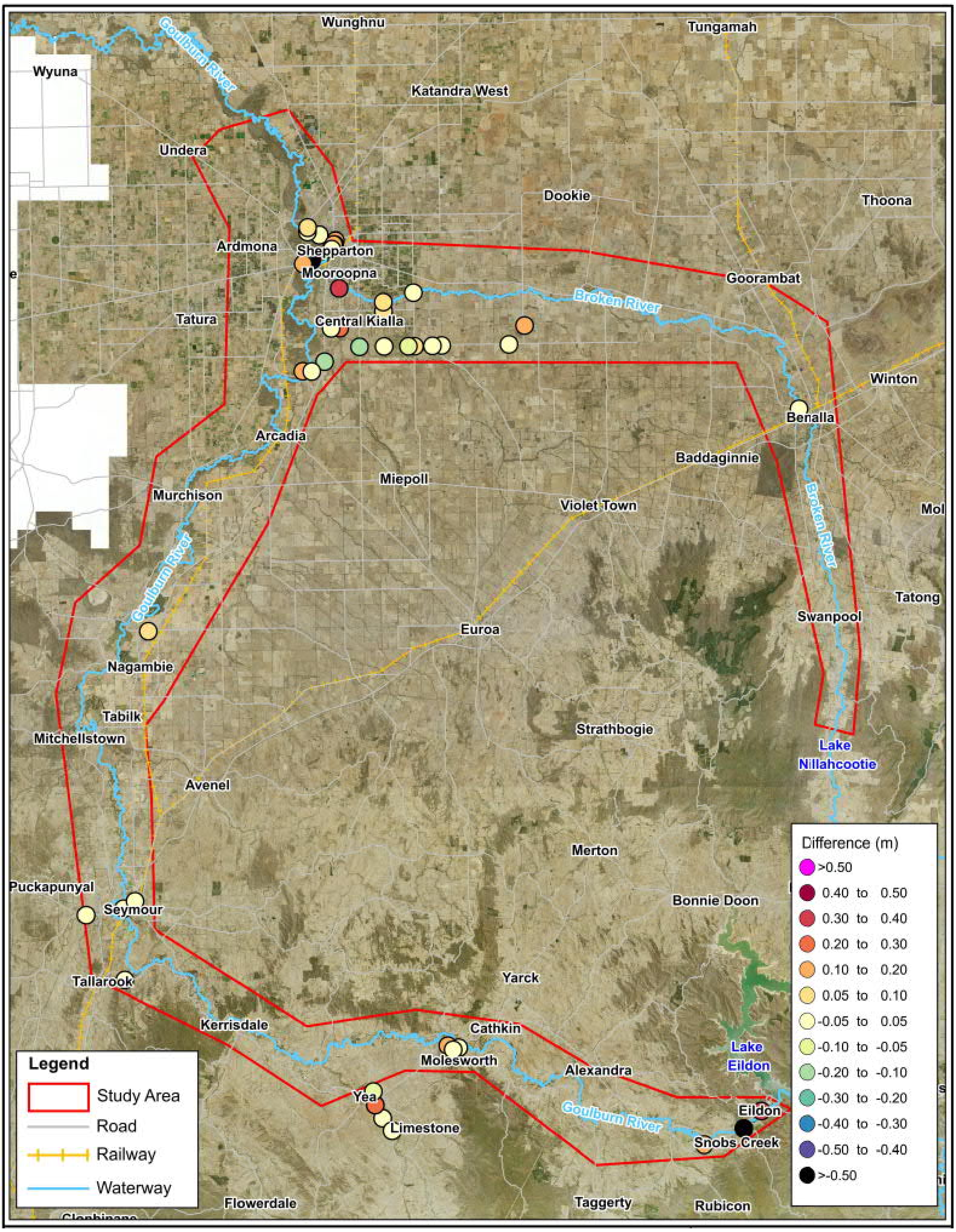


Figure 8-8 ISC Rivers and PSM Comparison

With the exception of the SRTM-H DEM, all LiDAR datasets were found to reasonably accurately represent the vertical topography within the study area based on the comparisons to each dataset, the State PSM network data. However, based on the review some datasets were determined to be more accurate than others. In this instance the Fugro and ISC Rivers LiDAR were the best performing, with the other LiDAR having slight high or low biases.

Based on the findings of this analysis the final DEM used for the hydraulic model is (in order of highest priority to lowest):

1) Bathymetry layers
2) Fugro
3) 2010 ISC Rivers
4) Granite Creek
5) 2011 Floodplains
6) NE Towns
7) Broken / Kialla
8) SRTM-H

The Fugro and ISC Rivers combined cover the largest portion of the catchment, therefore the remaining DEMs will typically be used only once flood levels have reached significant levels and have inundated the broader floodplain.

8.1.4.3 Channel Reinforcing

DEM data along rivers is generally filtered to the water surface. This means that the full depth of permanent or controlled rivers is not well represented in a TUFLOW model that does not use z-shapes to reinforce river channels. This is less of an issue in transient or low flow rivers, although it does depend on when the LiDAR that the DEM data is based on was flown.

The GBCMA supplied three sources of bathymetry data:

- Lake Benalla Bathymetry;
- Gowangardie weir pool; and
- Various topography modifiers in the supplied hydraulic models.

Due to the lack of supporting information the bathymetry could not be independently verified. The levels within Lake Benalla are controlled by the weir immediately upstream of the railway bridge and therefore the accuracy of the bathymetry will not significantly alter the outcomes of the flood study. However, it will be included for completeness and to aid in illustrating depths.

The Gowangardie weir pool covers a 4.7 km reach of the Broken River downstream of Nalinga. There is no way for the accuracy of this to be independently verified without detailed bathymetric survey.

The topography modifiers developed by Water Technology for the flows assessment cover Goulburn River downstream of the Goulburn Weir. The DEM developed by Water Technology was adopted for this study.

As part of the Victorian Constraints Measure Program (Sequna, 2023) bathymetric data was captured from upstream of the Goulburn Weir pool to immediately downstream of the Eildon pondage. This data included a thalweg survey along the entire river reach and thirty cross sectional surveys (bank to bank) at various locations between the Goulburn Wier pool and the Eildon pondage. The Goulburn River channel geometry for the reach from Eildon pondage to Goulburn Weir was incorporated into the DEM by interpolating information between the surveyed cross sections[2]. To do this, a left and right bank line – with levels taken from the LIDAR data – were created and used as tin lines. The channel centreline level was taken from the thalweg survey (except at the cross sections[3]). The channel geometry between these known points was estimated using linear interpolation.

Around Seymour the cross sections from the Seymour Floodplain Mapping Study (WBM,2001) were digitised and used to further develop a DEM in Seymour.

For the Goulburn Weir storage a bed level was adopted from engineering drawings for the construction of the Goulburn Weir structure.

No river bathymetry data was available for the Broken River except for Lake Benalla. It appears that the Fugro DEM along the Broken River was flown when the water level was relatively low and hence there is a reasonable representation of the river capacity. Importantly though the capacity of the river is relatively small compared with the floodplain, particularly in larger events. Sensitivity analysis of the bed level undertaken during the calibration process demonstrated that the flood levels were not significantly influenced by the bed level along the Broken River.

8.1.5 Grid Resolution

One of the key considerations in hydraulic modelling is the selection of an appropriate grid element size. Grid element size affects the resolution, or degree of accuracy, of the representation of the physical properties of the study area as well as the size of the computer model and its resulting run times. Selecting a smaller grid size will result in both higher resolution and longer model run times.

Sub-Grid Sampling

TUFLOW 2020-01-AA introduced a new method of representing topography in TUFLOW. Sub-grid Sampling (SGS) uses curves representing the sub 2D cell terrain data to construct the model instead of each 2D cell and each 2D cell face having one elevation. The curves are made from heights sampled at set intervals (for example 1 metre) along the side of a 2D cell. SGS is only available in a model that is using TUFLOW HPC.

There are two main benefits to using SGS, both ameliorating the difficulties inherent in modelling on a grid with constant size and orientation. Firstly, SGS mitigates the effects of grid resolution. In sampling multiple points along the face of each cell variations in terrain can be represented in a single cell that would have previously taken several cells to accurately represent. Secondly, SGS is adept at representing a defined channel running at an angle to the grid. In the traditional method of topographical representation this would produce a pronounced 'saw-tooth' effect that incurred significant additional losses as the flow was forced at right angles. With SGS these losses are eliminated as the jagged edges to the mesh are effectively smoothed out. Whilst SGS is an important tool in improving the resolution of the representation of the terrain, it is important to recognise that the two-dimensional computations still occur at the grid resolution not the SGS resolution. Therefore, in areas of complex two-dimension flow it is still important to adopt an appropriate grid resolution.

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[2] Several pre-existing cross section surveys around the town of Seymour and immediately downstream of Eildon pondage were also available for this task

[3] It was assumed that the cross section was more accurate than the thalweg so the cross section level was adopted

When compared to TUFLOW HPC, which treats each square grid element in the same way as TUFLOW Classic, SGS differs only in the topography, sampling from the DEM many times across the width of the cell. This allows SGS to have even more defined control of hydrodynamic elements, such as flood depth, volume and spilling at a sub-grid scale. This contrasts dramatically with Mike21 and the superseded TUFLOW GPU engine which provide only one element at the centre of each cell which controls all hydrodynamic elements. The surface resistance to flow (Manning's 'n' value) is still sampled at half the grid resolution.

More information on the processes involved in the selection of cell heights and the benchmarking of SGS can be found in the TUFLOW 2020-01 Release Notes (BMT WBM, 2020).

While SGS does have many advantages, models that use SGS have a slower run time than equivalent (same grid resolution) standard HPC models and a considerably higher RAM draw on start up.

To ensure accurate representation of flooding within the catchment a grid resolution of 14 metres was adopted for the model. SGS was used, with heights sampled at 1 metre intervals along the side of each cell. When SGS was not used a grid resolution of 10 metres was able to be run. Overall the results produced by the 14m SGS and the 10 m non-SGS models were similar, however the SGS models had better calibration results in the upper portions of both the Goulburn and Broken Rivers.

Watercourses

The waterways within the system are variable. Much of the river system can be categorised as highly sinuous with significant anabranches and waterholes consistent with a river system that has migrated around the floodplain. However, sections of river system, in particular downstream of reservoirs, can be categorised as uniform channels. Typically, the Goulburn River is 60-80 m wide. On the 14 metre grid adopted for this study the waterways were represented by a 4-5 grid cells, which is consistent with the minimum number of cells recommended in the TUFLOW manual for a model without SGS. The Broken River is a much narrower channel and may only be represented by 2 to 3 grid cells. While this is less than what is recommended for models without SGS, the addition of SGS means that the channel is well represented with a reduction in saw-tooth edge effects typical of a coarsely represented channel and a reliable representation of the channel conveyance capacity. On this the TUFLOW manual states "*open channels can now be accurately modelled using TUFLOW HPC using coarse cell sizes at any orientation to the channel*". Therefore with a 1 m SGS sampling a good representation of the channel of the Broken River is achieved.

Hydraulic Controls

Levees, irrigation channels, railway embankments and roads form hydraulic controls within the study area. Control levels on select levees, irrigation channels, rail and roads crests were reinforced within TUFLOW using z-lines. The hydraulic controls that were reinforced were those that were narrower than the 14 m grid and hence the model may not have sufficiently defined control levels of the feature. This included the majority of paved roads, levees and railway embankments.

Viscosity

TUFLOW 2020-01-AA introduced a new approach to modelling sub grid scale turbulence (eddy viscosity) for HPC. Eddy viscosity is the turbulence that occurs at a scale that is impractical to model. The losses caused by this turbulence must be represented in hydraulic modelling in some manner.

Previously, TUFLOW used the Smagorinski approach to determine these losses. While this is fine on large scale models with coarse grid sizes, as cell sizes are reduced and the cell size to flood depth ratio decreases it becomes less accurate. This is because the Smagorinski approach is proportional to cell size. The result of this is that the Smagorinski coefficients can end up becoming important calibration parameters. The latest TUFLOW release introduced the Wu eddy viscosity formulation as the default. This approach differs from that proposed by Smagorinski as it takes in to account both 2D and 3D effects and is not dependent on cell size.

In calibration testing an extensive amount of time was dedicated to calibrating the Smagorinski coefficients in combination with Manning's roughness values. Different coefficients were required with different grid sizes. The use of the new Wu formulation has both increased the accuracy of the calibration and allowed easier switches between grid sizes for testing of certain model features.

8.1.6 2D Hydraulic Structures

Large hydraulic structures for this study consist of the bridges over the various waterways. It is important to ensure that these impediments and constrictions to flow are accurately represented. Due to their large size these structures are modelled in the 2D model domain. The location of these bridges is shown in Figure 8-9.

Bridge structures were modelled as layered 2D flow constructions with the appropriate losses derived from Waterway Design: A Guide to the Hydraulic Design of Bridges, Culverts and Floodways (Austroads, 1994). The layered flow constrictions used to model these bridges allows for typical bridge characteristics such as deck height and thickness, pier shape and width and blockages associated with guard or handrails to be directly incorporated into the 2D domain.

The details of these were extracted from supplied plans for the major bridges owned and managed by VicRoads, Goulburn-Murray Water, GBCMA and Benalla Rural City, Murrindindi Shire Council, Greater Shepparton City Council and Strathbogie Shire Council. Mitchell Shire Council directly provided TUFLOW flow constriction layers. For some smaller bridges, plans were unavailable. Where plans were not available bridge lengths estimated from aerial imagery and DEM data and the losses and structure element dimensions were based on typical bridge configurations and loss parameters.

From the enquires the following was received:

- VicRoads supplied details of 28 significant bridges within the study area. These details were supplied in the form of engineering drawings, the majority of which are hand-drawn.
- The GBCMA supplied plans from the State Rivers and Water Supply Commission for two bridges near Benalla.
- Goulburn-Murray Water supplied details of five weirs in the study area.
- Benalla Rural City supplied a spreadsheet of bridges and culverts within their council area.
- Greater Shepparton City Council provided a spreadsheet of bridges and culverts within their jurisdiction with locality plans and some engineering drawings.
- Mitchell Shire Council provided a hydraulic model which contained information on (TUFLOW lfcsh layers) for five bridges near Seymour. Additionally, engineering drawings and newspaper clippings were supplied for verification of the information in the hydraulic model.

- Murrindindi Shire Council supplied a spreadsheet and corresponding locality plan for the bridges and culverts in their council area. Additionally, engineering drawings were supplied for the three major bridges within the study area within their municipality.
- Strathbogie Shire Council supplied an extensive spreadsheet of bridges and culverts.

Often information gleaned from supplied engineering drawings or spreadsheets was incomplete with regards to the needs of hydraulic modelling and certain minor bridges had no information available. To infill missing details and verify information in the supplied plans, approximately 50 structures including bridges, weirs and culverts within the study area were measured during the site visit by the study team. The measurements were taken using a high precision laser measure.

Table 8-3 provides a summary of the bridges entered into the model

Table 8-3 Details of bridge structures

Creek/Channel	Structure Reference	Structure Location	Structure Configuration
Goulburn River	1	Back Elidon Rd	6 span bridge
Goulburn River	2	Goulburn Valley Hwy	12 span bridge
Goulburn River	3	Breakaway Rd	9 span bridge
Goulburn River	4	Maroondah Hwy	5 span bridge
Goulburn River Tributary	5	Goulburn Valley Hwy	29 span bridge
Goulburn River	6	Goulburn Valley Hwy	13 span bridge
Goulburn River	7	Ghin Ghin Rd	4 span bridge
Goulburn River	8	Terangaville Rd	Plans unavailable. Assumed 85 m bridge from aerial
Goulburn River	9	Goulburn Valley Hwy	5 span bridge
Goulburn River	10	Railway	166 m bridge Flow constriction supplied by Council
Goulburn River	11	Emily St	96 m flow constriction Flow constriction supplied by Council
Goulburn River Overflow	12	Emily St	108 m flow constriction Flow constriction supplied by Council
Goulburn River Overflow	13	Emily St	39 m flow constriction Flow constriction supplied by Council
Goulburn River Overflow	14	Emily St	27 m flow constriction Flow constriction supplied by Council
Sunday Creek	15	Hume Fwy	5 span bridges
Sunday Creek	16	Manse Hill Rd	3 span bridge
Goulburn River	17	Old Goulburn Bridge	Disused 7 span bridge – deck removed 118 m flow constriction
Goulburn River Tributary	18	Hume Fwy	7 span bridges
Goulburn River	19	Hume Fwy	9 span bridges

Creek/Channel	Structure Reference	Structure Location	Structure Configuration
Goulburn River Overflow	20	Hume Fwy	5 span bridges
Goulburn River	21	Mitchellstown Rd	9 span bridge
Tablik Lagoon	22	Mulberry Dr	11 span bridge
Goulburn River	23	Nagambie-Rushworth Rd	5 span bridge
Lake Nagambie	24	Glencairn Ln	3 span bridge
Lake Nagambie	25	Footbridge	Plans unavailable Assumed 36 m bridge from aerial
Goulburn River	26	Kirwans Bridge Rd	55 span bridge
Goulburn River	27	Railway	14 span bridge
Goulburn River	28	Murchison-Violet Town Rd	6 span bridge
Goulburn River Overflow	29	Murchison-Violet Town Rd	4 span bridge
Goulburn River Overflow	30	Railway	Plans unavailable Assumed 63 m bridge from aerial
Goulburn River	31	Railway	9 span bridge
Goulburn River	32	Bridge Rd	8 span bridge
Goulburn River	33	Railway	4 span bridge
Goulburn River	34	Watt Rd	5 span bridge
Goulburn River Overflow	35	Midland Hwy	3 span bridge
Goulburn River Overflow	36	Midland Hwy	4 span bridge
Goulburn River Overflow	37	Midland Hwy	6 span bridge
Goulburn River Overflow	38	Midland Hwy	5 span bridge
Goulburn River Overflow	39	Midland Hwy	8 span bridge
Goulburn River	40	Goulburn Suspension Bridge	3 span bridge
Goulburn River	41	Midland Hwy	14 span bridge
Broken River	42	William Rd	3 span bridge
Broken River	43	Pritchard Trk	Plans unavailable Assumed 30 m bridge from aerial
Broken River	44	Swanpool Rd	3 span bridge
Broken River	45	Farm Track	Plans unavailable Assumed 19 m bridge from aerial
Broken River	46	Farm Track	Plans unavailable Assumed 27 m bridge from aerial
Broken River	47	Farm Track	Plans unavailable Assumed 23 m bridge from aerial
Broken River	48	Farm Track	Plans unavailable Assumed 24 m bridge from aerial
Broken River	49	Hume Fwy	5 span bridges
Broken River	50	Hume Fwy	7 span bridges

Creek/Channel	Structure Reference	Structure Location	Structure Configuration
Holland Creek	51	Hume Fwy	11 span bridges
Holland Creek	52	Benalla-Tatong Rd	20 span bridge
Broken River	53	Midland Hwy	4 span bridge
Drain	54	Railway	Single span bridge
Broken River	55	Railway	17 span bridges
Broken River	56	Ackerly Ave	4 span bridge
Broken River	57	Tarnook Rd	4 span bridge
Broken River Overflow	58	Burnells Rd	3 span bridge
Broken River	59	Dookie-Violet Town Rd	4 span bridge
Broken River	60	Bridge Rd	6 span bridge
Broken River	61	Shepparton-Eurora Rd	6 span bridge
Broken River	62	Doyles Rd	5 span bridge
Broken River	63	Archer Rd	5 span bridge
Broken River	64	Footbridge	Plans unavailable Assumed 70 m bridge from aerial
Broken River	65	Goulburn Valley Hwy	22 span bridge
Broken River	66	Railway	12 span bridge

8.1.7 1D Hydraulic Structures

Small, sub-grid sized, hydraulic structures such as culverts were modelled as 1D elements dynamically linked to the 2D domain. The details these culverts were provided by the various local councils (as listed in Section 8.1.6). Those culverts that did not have details available for them were estimated from street view and aerial imagery if they had a notable impact on the modelling. However, those that would not affect the model, due to their size in comparison to the volumes of water in the floodplain, were excluded. The location of each of the culverts is shown in Figure 8-9. Table 8-4 shows the details of the culverts entered into the model. Table 8-5 shows the details of the weir/control structures.

Table 8-4 Details of culverts

Creek/Channel	Structure Reference	Structure Location	Structure Configuration
Goulburn River Overflow	67	Maroondah Hwy	Assumed 4 No. 2100x600 RCBC from aerial
Goulburn River Overflow	68	Nagambie-Rushworth Rd	3 No. 3050x2440 RCBC
Goulburn River Overflow	69	Murchison-Violet Town Rd	3 No. 2440x1820 RCBC
Goulburn River Overflow	70	Murchison-Violet Town Rd	8 No. 1820 RCP
East Goulburn Main Channel	71	Pranjip Ck	3 No. 900 RCP
East Goulburn Main Channel	72	Castle Ck	3 No. 900 RCP

Creek/Channel	Structure Reference	Structure Location	Structure Configuration
Seven Creek Tributary	73	Archer Rd	Assumed 2 No. 1500x300 RCBC from aerial and DEM
Seven Creek Tributary	74	Shepparton-Seymour Rd	Assumed 3 No. 3000x900 RCBC from aerial and DEM
Broken River Overflow	75	Railway	1050 RCP
Broken River Overflow	76	Doyles Rd	Assumed 1500 RCP from aerial and DEM
Broken River Overflow	77	Doyles Rd	Assumed 600 RCP from aerial
Broken River Overflow	78	Archer Rd	Assumed 3 No. 3000x900 RCBC from aerial and DEM
Broken River Overflow	79	Kialla Lakes Dr	15 No. 2400x1000 RCBC

Table 8-5 Details of weirs/flow control structures

Creek/Channel	Structure Reference	Structure Location	Structure Configuration
Goulburn River	80	Cattanach Canal Offtake	3 no. 4.20x3.65 m radial gates Refer Section 8.1.7.1 for details on modelling approach
Goulburn River	81	Stuart Murray Canal Offtake	4 no. 3.00x2.10 m radial gates Refer Section 8.1.7.1 for details on modelling approach
Goulburn River	82	Goulburn Weir	9 no. 12.87x3.65 m radial gates Refer Section 8.1.7.1 for details on modelling approach
Goulburn River	83	East Goulburn Main Channel Offtake	Plans unavailable Refer Section 8.1.7.1 for details on modelling approach
Broken River	84	Casey's Weir	70 m weir reinforced with z-shape

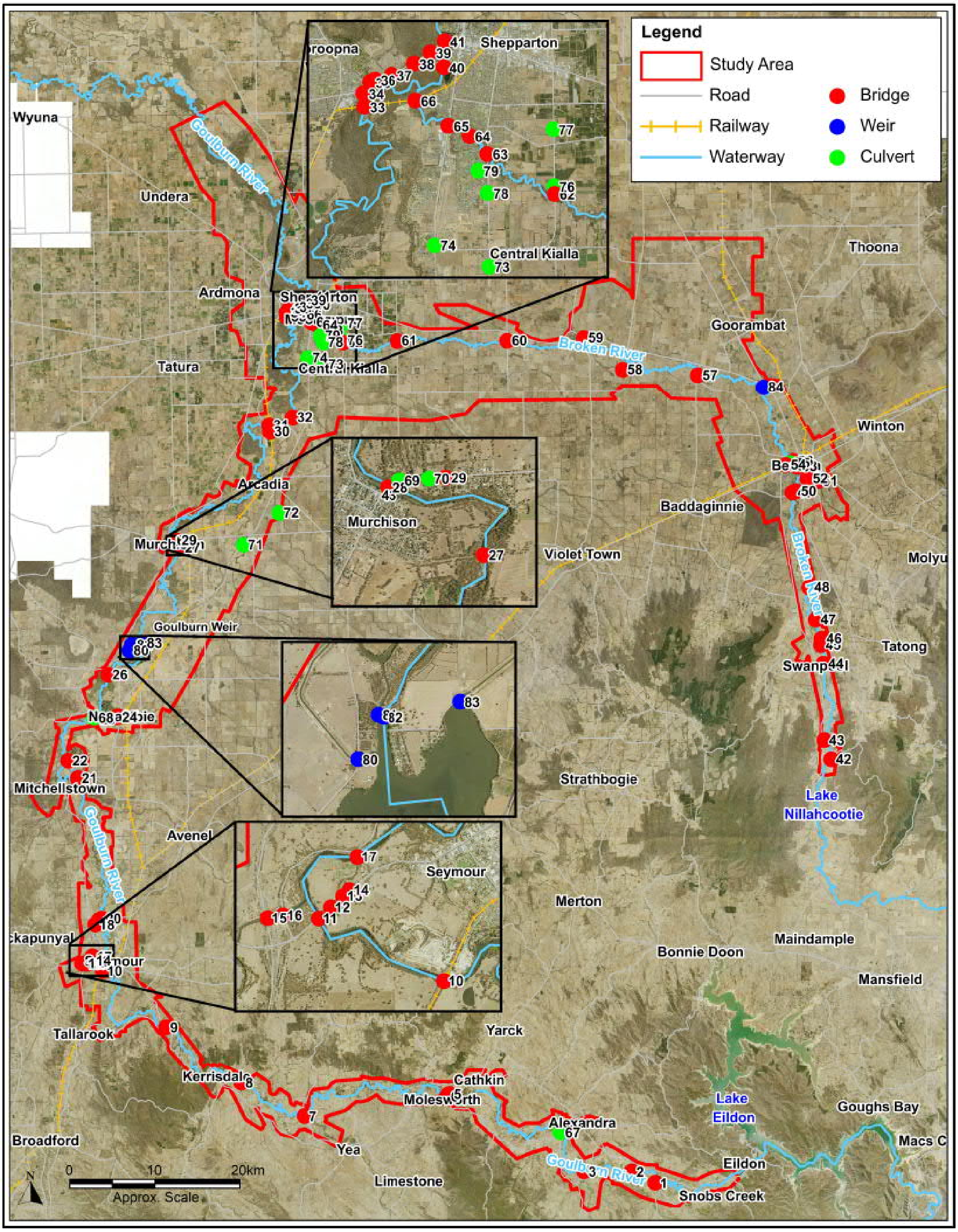


Figure 8-9 Hydraulic Model Structure Locations

8.1.7.1 Goulburn Weir

The Goulburn Weir and associated structures were also modelled as 1D elements linked to the 2D domain. The Goulburn Weir is located some 7 km north of Nagambie and was constructed in 1890 and upgraded in the mid-1980s. The dam was originally constructed to provide surety of water supply for irrigation against droughts and performs that function to this day, having a total storage capacity of

25,500 ML at the adopted full supply level (FSL) of 124.243 m AHD (GMW, 2017). Three irrigation canal offtakes begin at Goulburn Weir: the Stuart Murray Canal, to the immediate west of the weir, the Cattanach Canal, to the south-west of the weir and the East Goulburn Main Channel Offtake (EGMCO), to the east of the weir.

The Goulburn Weir Operational Manual (GMW, 2017) states that “as a general rule, until inflows have peaked and FSL or maximum storage level has been reached, releases should not exceed inflows on the rising limb.” Additionally, the maximum permissible flow into the Stuart Murray Canal is 3,520 ML/d (40.7 m^3/s) and the maximum permissible flow into the Cattanach Canal is 3,670 ML/d (42.5 m^3/s). The East Goulburn Main Channel Offtake is not used as an outlet during flood events, with the upstream gate structures used to “isolate the East Goulburn Main channel during non-irrigation season or to retain floods” (URS, 2014).

The Goulburn Weir itself has the following characteristics (SKM, 2012):

- Spillway crest level = 121.195 m AHD
- Full supply level (FSL) = 124.243 m AHD
- Spillway crest length = 115.83 m clear length
- Gate configuration = 9 no. 12.87x3.65 m radial

For the design runs and the 1993 calibration event the following operating rules were enacted:

- If the lake level above the weir is greater than 124.1 m AHD then
    - Flow from the lake to the Stuart Murray Canal = 40.7 m^3/s
    - Flow from the lake to the Cattanach Canal = 42.5 m^3/s
- The Weir was modelled as an operational spillway with the following properties:
    - Length = 115.83 m
    - Height = 4.105 m (Bottom height of radial gate in open position minus spillway crest level: 125.3 m AHD – 121.195 m AHD)
    - Crest elevation = 121.195 m AHD
    - Gate seat elevation = 120.79 m AHD
    - If the lake level is less than 124.243 m AHD (FSL) then the sluice gates were 1% open
    - If the lake level is equal to or greater than 124.243 m AHD (FSL) then the gates would open to 100%
    - The opening period of the gates is 30 minutes

The rationale behind the canal operating rules is that the dam operators, knowing that a flood event is imminent, would divert as much flow as possible into the irrigation channels so as to minimise the riverine flooding downstream although it is acknowledged that the canal flows are comparatively small and hence will not significantly influence the calibration. The minimum lake level is to stop the irrigation channels draining the lake before the high flows from the flood event arrives.

The sluice gate opening of 1% at a minimum is intended to represent low flows through the weir. An opening of 1% was selected through trial and error; if the opening was too high then the weir drained before the flood arrived. If the water level then reaches the FSL of the weir, the gates are slowly raised (over 30 minutes) until they are fully open. The speed of opening is adopted to stop significant fluctuations in flow from the weir causing instabilities in the model. While in extreme events (e.g. the PMF) the gates might remain fully open for some time, in smaller events the gates will remain partially open, continually opening and closing to keep the water level in the lake at 124.243 m AHD. While this fluctuation of the gate opening is not how the weir would be operated in reality, it does effectively mirror the effects of opening a small number of gates fully and achieve the aim of keeping the water level at 124.243 m AHD.

Although the weir was upgraded in the 1980s, details of the weir's specifications at the time of the 1974 flood or the operating procedures in place at that time were not available. Therefore, in the 1974 validation event the Goulburn Weir was operated as described above except the outflow from the Cattanach and Stuart Murray Canals was disabled. However, the East Goulburn Main Channel Offtake, to the east of the weir, was operating during the 1974 event. The EGMCO was modelled as an operational weir with the following properties:

The EGMCO was modelled as an operational sluice gate with the following properties:

- Length = 10 m
- Height = 3.6 m
- Crest elevation = 121.31 m AHD
- If the lake level is less than 124.243 m AHD then the sluice gates were 2% open
- If the lake level is equal to or greater than 124.243 m AHD then the gates would open to 100%
- The opening period of the gates is 10 minutes

To ensure the accuracy of the modelling of the weir, the equations governing sluice gate operation in TUFLOW were compared to the rating curves shown in SKM's 2012 Review of Storage Spillway Rating Tables (SKM, 2012). In TUFLOW, the flow through sluice gates is controlled by the equations outlined below, switching dynamically between upstream and downstream controlled flow.

For a flow over the spillway unaffected by a gate, the flow is calculated using:

$$Q = \frac{2}{3} C_d W H \sqrt{2gH}$$

Where

Q = Discharge
C_d = Discharge coefficient upstream controlled flow (default = 0.75)
W = Width
H = Upstream energy level – Crest level

For flow over the spillway that is affected by the gate, the ratio of the gated discharge to the ungated discharge is calculated using:

$$\frac{Q_G}{Q} = \frac{C_G}{C_d}\left(\frac{H_2^{\frac{3}{2}} - H_1^{\frac{3}{2}}}{H^{\frac{3}{2}}}\right)$$

Where

Q_G = Gated Discharge

C_G = Discharge coefficient (default = 0.75)
H_1 = Upstream water level – Gate lip elevation

H_2 = Upstream water level – Gate seat elevation

If the gated discharge is less than the ungated discharge, then the gated discharged is used as the flow through the spillway.

The coefficients of discharge were modified to match the rating curves shown in SKM (2012). The ungated discharge was compared to the rating curve shown in Figure 7-13. The coefficient of discharge was reduced from the default of 0.75 to 0.56. The results from the SKM rating curve and the discharge given by the TUFLOW spillway equation are shown in Figure 8-10. The water elevation is shown up to 126.635 m AHD, the level of the dam crest. Iterations of the discharge coefficient were undertaken to provide the best match to the SKM rating across the range of water elevations. At FSL the difference between the discharges is less than 4% and at the dam crest level the difference is less than 7%. Whilst a slightly closer match would have been preferable, it should be considered in light of the much greater uncertainty around the operational rules during the calibration events.

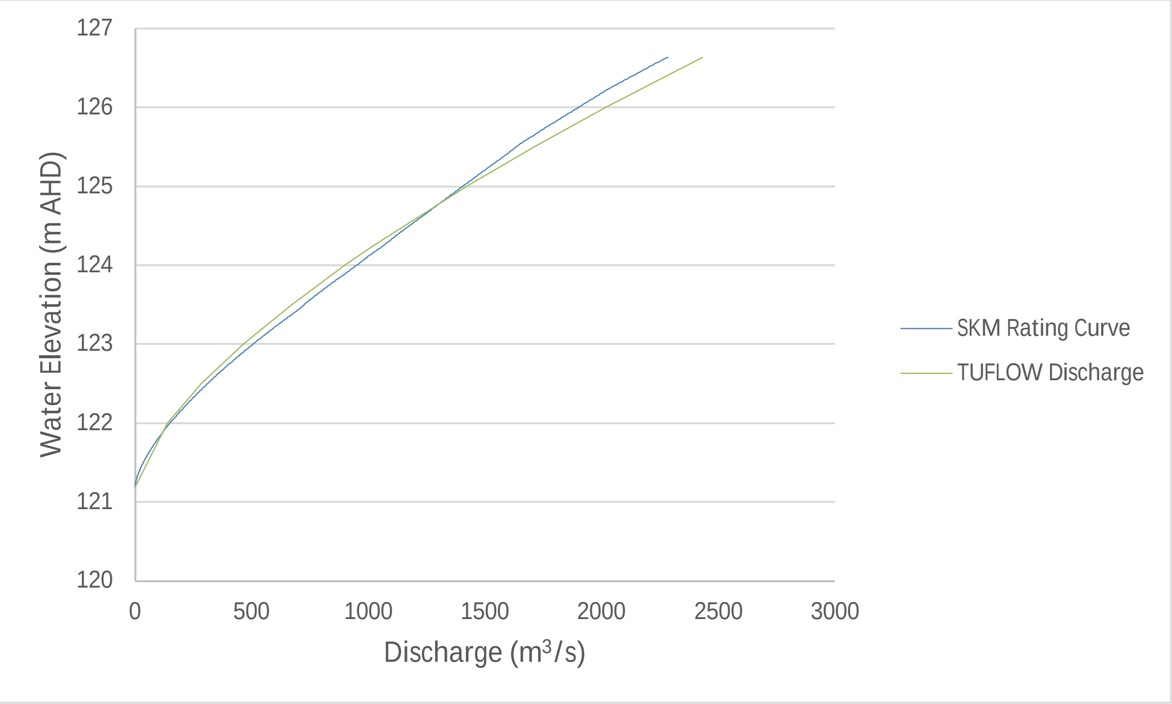


Figure 8-10 Goulburn Weir rating Curve for all Gates Fully Open (Ungated Weir)

The TUFLOW gated discharge equation was compared to the rating curve shown in Figure 7-9 of SKM (2012) for 9 gates with varying openings from 0 m to 4.5 m and a water level fixed at FSL (124.243 m AHD). The coefficient of discharge in TUFLOW was reduced from the default of 0.75 to 0.6 to match the SKM rating as shown in Figure 8-11. The TUFLOW data in this figure stops at a gate opening of 2.175 m because above this level the ungated discharge equation gives a smaller discharge than the gated equation and hence TUFLOW uses the ungated result as per Figure 8-10. SKM (2012) does not discuss which of their curves to adopt in this situation, but the approach adopted in TUFLOW is consistent with hydraulic principles.

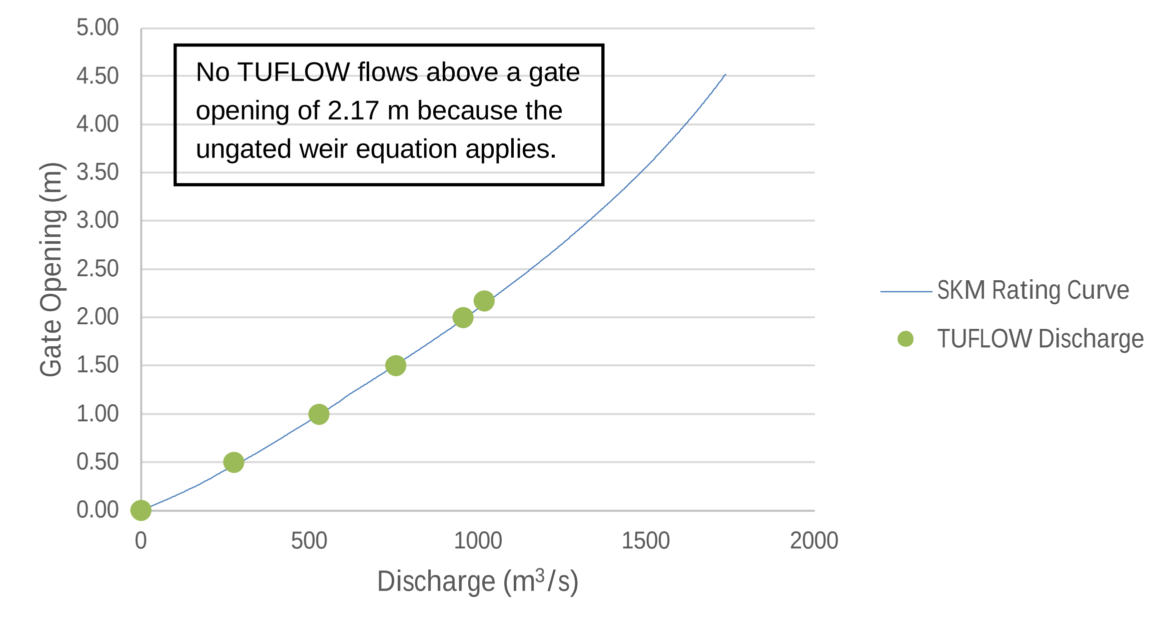


Figure 8-11 Goulburn Gated Weir Rating Curve for Water Level at FSL (124.243 m AHD)

8.1.8 Boundary Conditions

A typical hydraulic model incorporates three different boundary conditions. These are:

- Inflow boundaries;
- Outflow boundaries; and
- Linking boundaries.

The boundary locations are shown on Figure 8-12 to Figure 8-15 and they are discussed in detail below.

Inflow Boundaries

Inflow boundaries consist of two types, external and lateral inflows. External inflows are those occurring at the extremities of the hydraulic model, and lateral inflows are the runoff inflows from contributing areas that occur within the 2D model domain. These inflow boundaries were determined from the hydrologic model developed for this study. These inflow boundaries were applied as unsteady (flow vs. time) boundaries directly to the individual watercourses in the 2D domain. The external inflows were applied at the edge of the domain using the 2d_bc boundary option, or at some locations a 2d_SA boundary a short distance downstream of the domain edge was preferred. The lateral inflows were applied into the watercourses using the streamlines feature.

For the Goulburn-Broken study there were 39 external inflow boundaries. These start with the flows from the reservoirs at Elidon and Nilahcootie at the upstream extents of the Goulburn and Broken Rivers and continue down the length of those rivers as smaller catchments feed in.

The hydraulic model included an additional five inflow boundaries representing the riverine baseflow and 30 lateral inflow boundaries. These were applied as distributed (flow over area) inflows using the streamlines feature.

Riverine baseflows were added at the gauges at Trawool, Seymour, Murchison, Casey's Weir and Shepparton. These were applied as distributed inflows using SA lines in the centre of the channel. The baseflows were calculated based on techniques described in ARR2019.

Outflow Boundaries

The downstream boundary was located across the Goulburn River approximately 26 km downstream of Shepparton. This outflow boundary was represented as a stage-discharge boundary with a slope of 0.0001. This slope was adopted following sensitivity testing of a range of slopes through the calibration process; steeper slopes resulted in the water level in the model falling away from historical levels for up to about 5 km from the downstream boundary.

Other outflow boundaries (see Figure 8-9) were located where flow breaks out into distributary channels outside the model domain. These outflow boundaries were represented as stage-discharge boundaries with slopes of between 0.00135 and 0.0005 depending on the hydraulic gradient from initial runs. Sensitivity testing of the boundary configurations showed that the boundaries were sufficiently far from the main channel as to not influence the flow rates leaving the Goulburn and Broken Rivers.

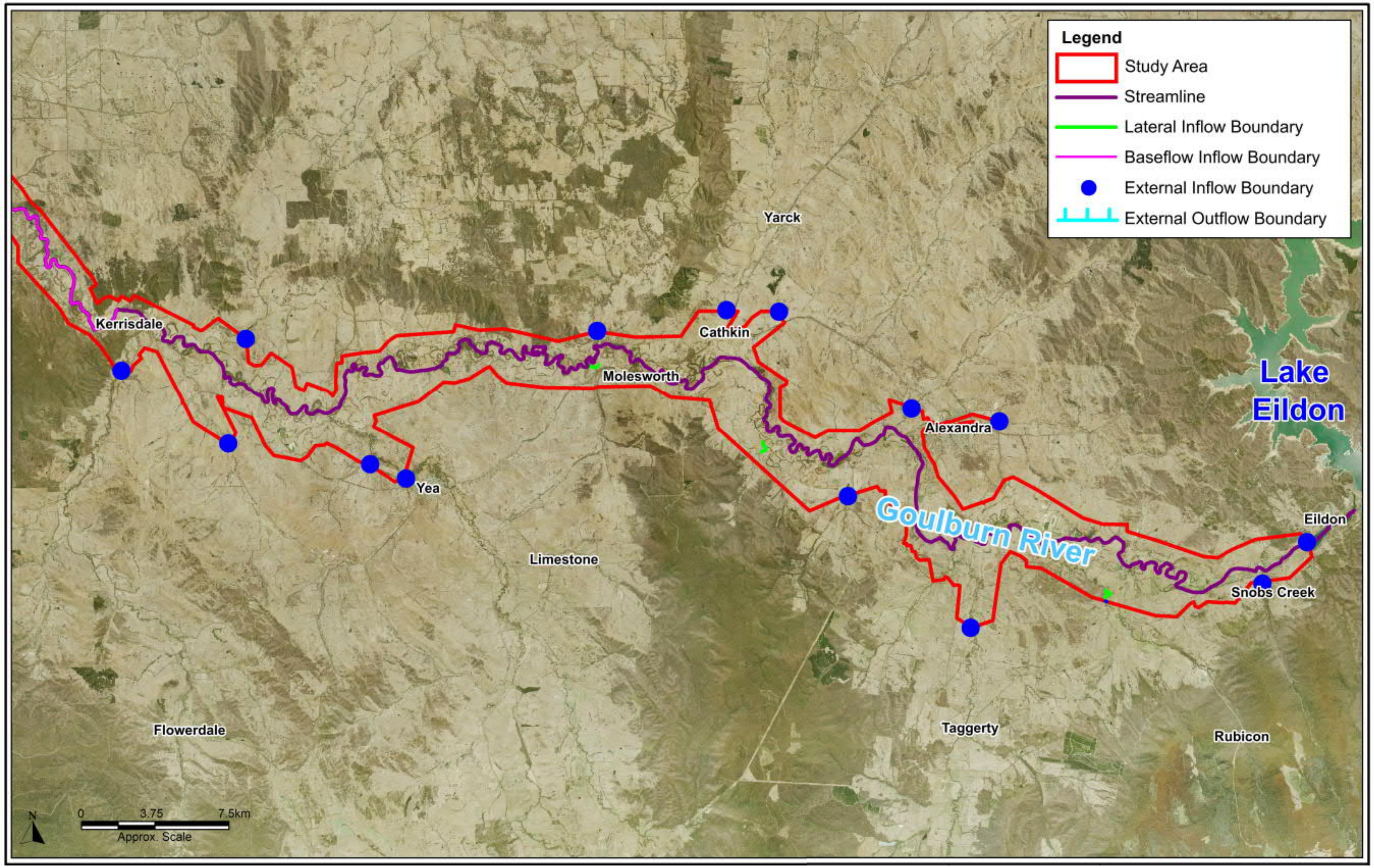


Figure 8-12 TUFLOW Boundaries – Upper Goulburn River

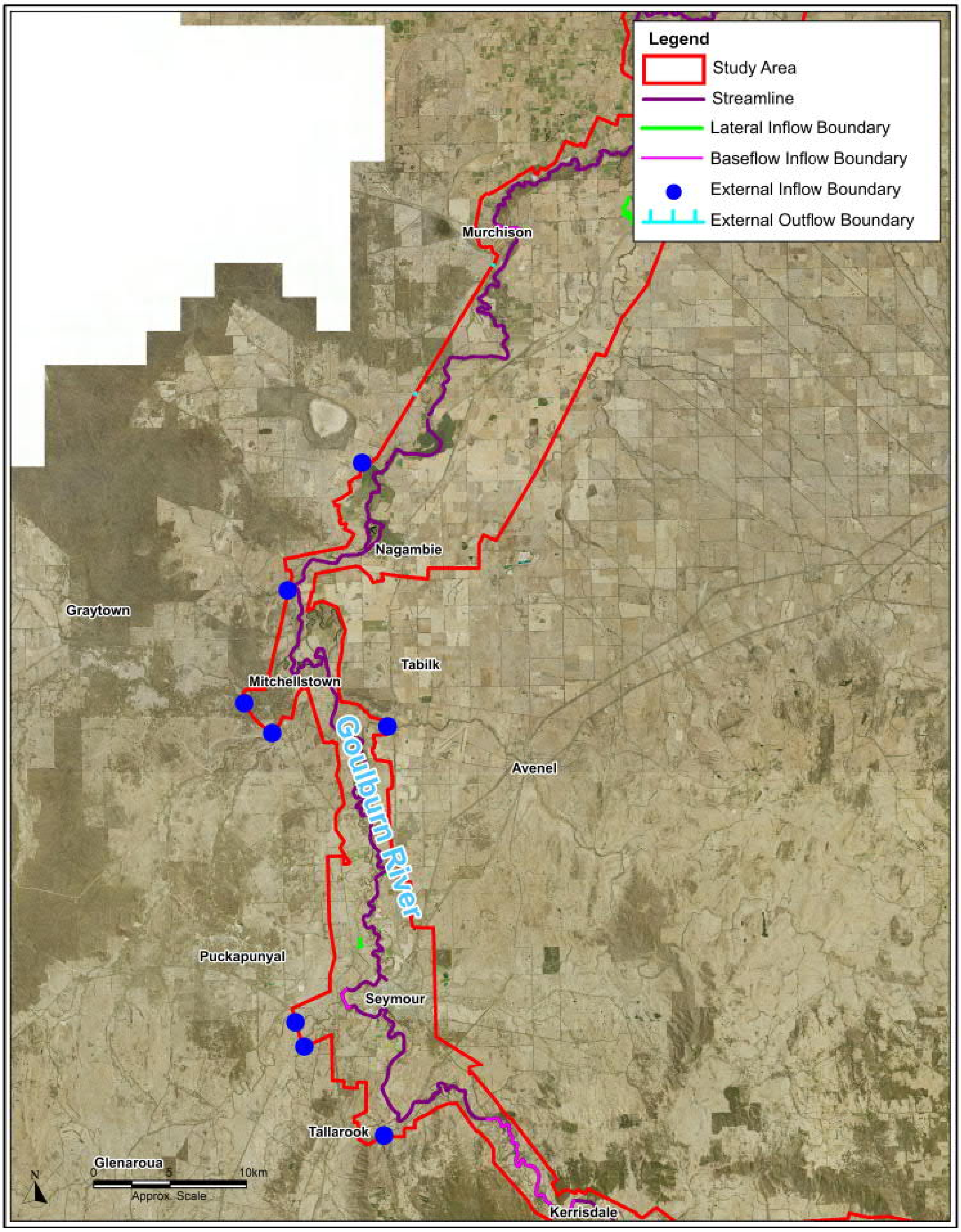


Figure 8-13 TUFLOW Boundaries – Central Goulburn River

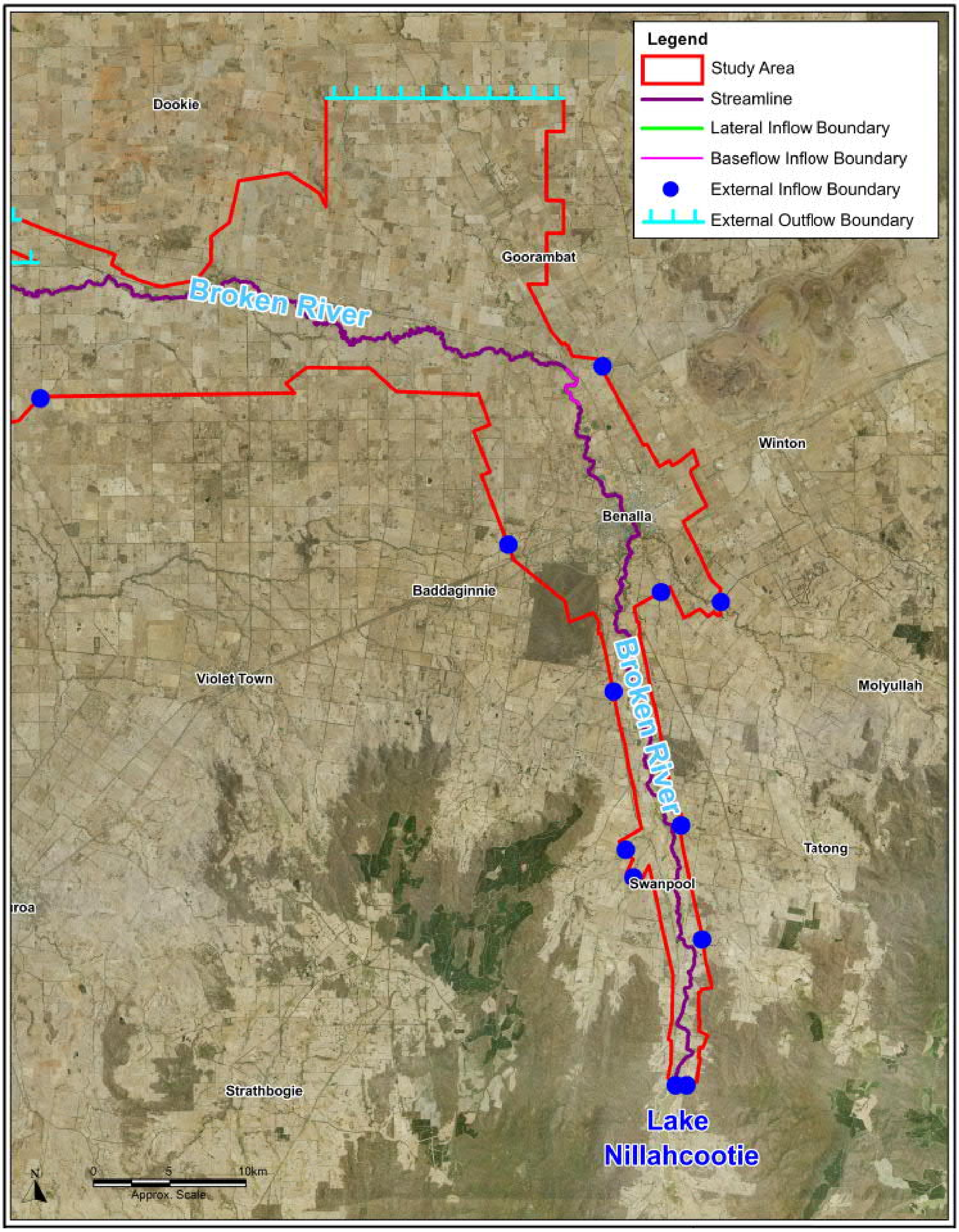


Figure 8-14 TUFLOW Boundaries – Upper Broken River

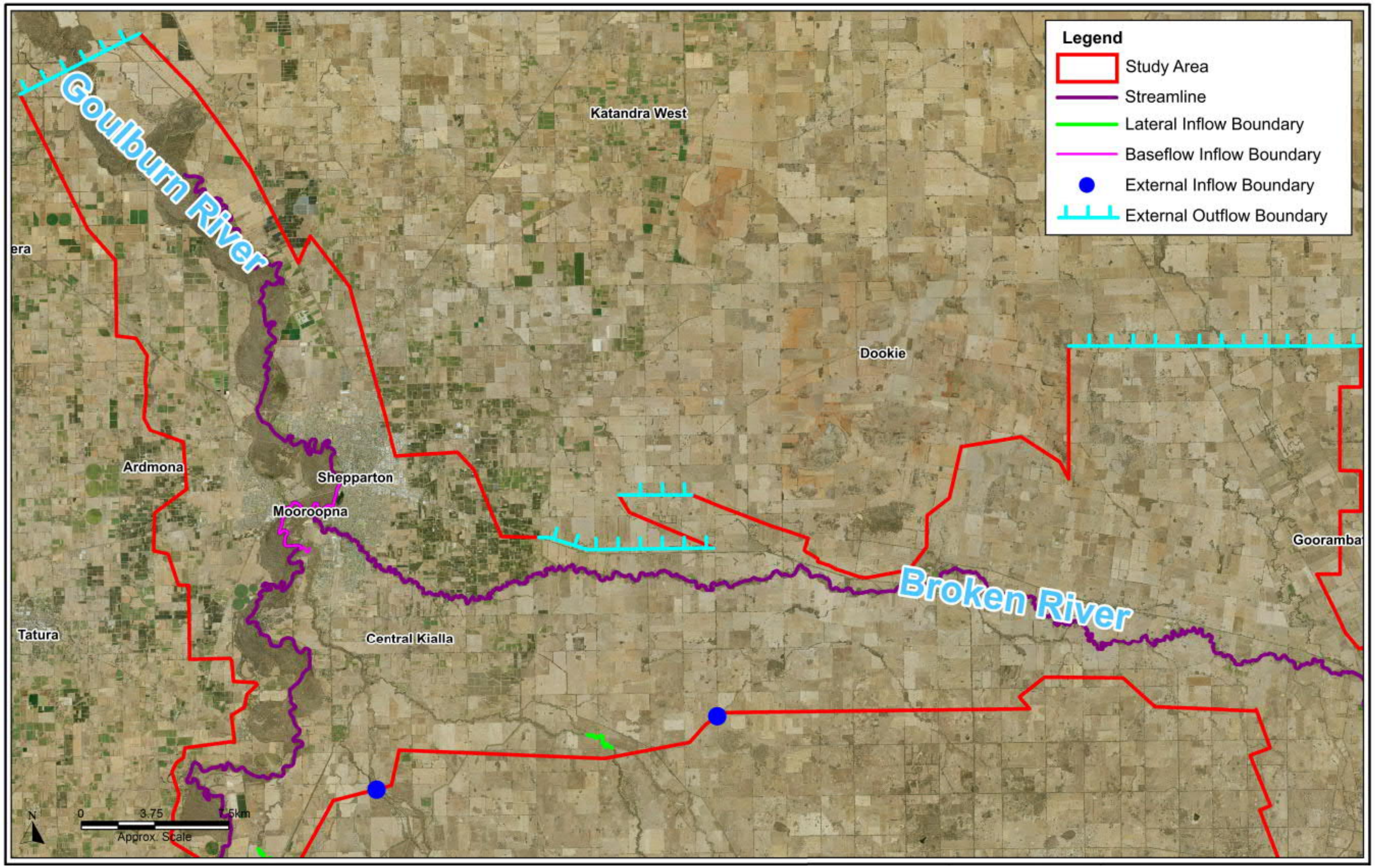


Figure 8-15 TUFLOW Boundaries – Lower Goulburn and Broken Rivers

During a large flood on the Broken River, like the 1993 event, flow breakouts from the Broken River into the Broken Creek system, just north of the Caseys Weir gauge as can be seen in Figure 8-16 which shows depth mapping at the peak of the 1993 event. An outflow boundary was included in the model and located more than 12 km downstream of the breakout to avoid any boundary effects influencing the breakout.

Not all the breakout from the Broken River flows into the Broken Creek system. Only flow that crosses the green line in Figure 8-16 labelled *Broken Creek Breakout Flow* enters Broken Creek. South of this line there is a large flow path to the west which ultimately joins back with the Broken River.

The TUFLOW model indicates that breakout from the Broken River commences when the flows is approximately 520 m^3/s in the Broken River. This is when water starts to flow over the Midland Highway (there is flow in the Broken Creek channel before this time). The relationship in RORB (refer to Section 4) was developed using a number of steady state flows so the volume of the hydrograph was not taken into account. This was done as the relationship was to be generic rather than for a particular event. At the peak of the 1993 event the flow in the Broken River at Caseys Weir is about 1200 m^3/s of which about 150 m^3/s reaches the Broken Creek. Figure 8-17 shows the relationship between the flow in the Broken River and the breakout flow used in the RORB model compared to the results from the peak of the 1993 event.

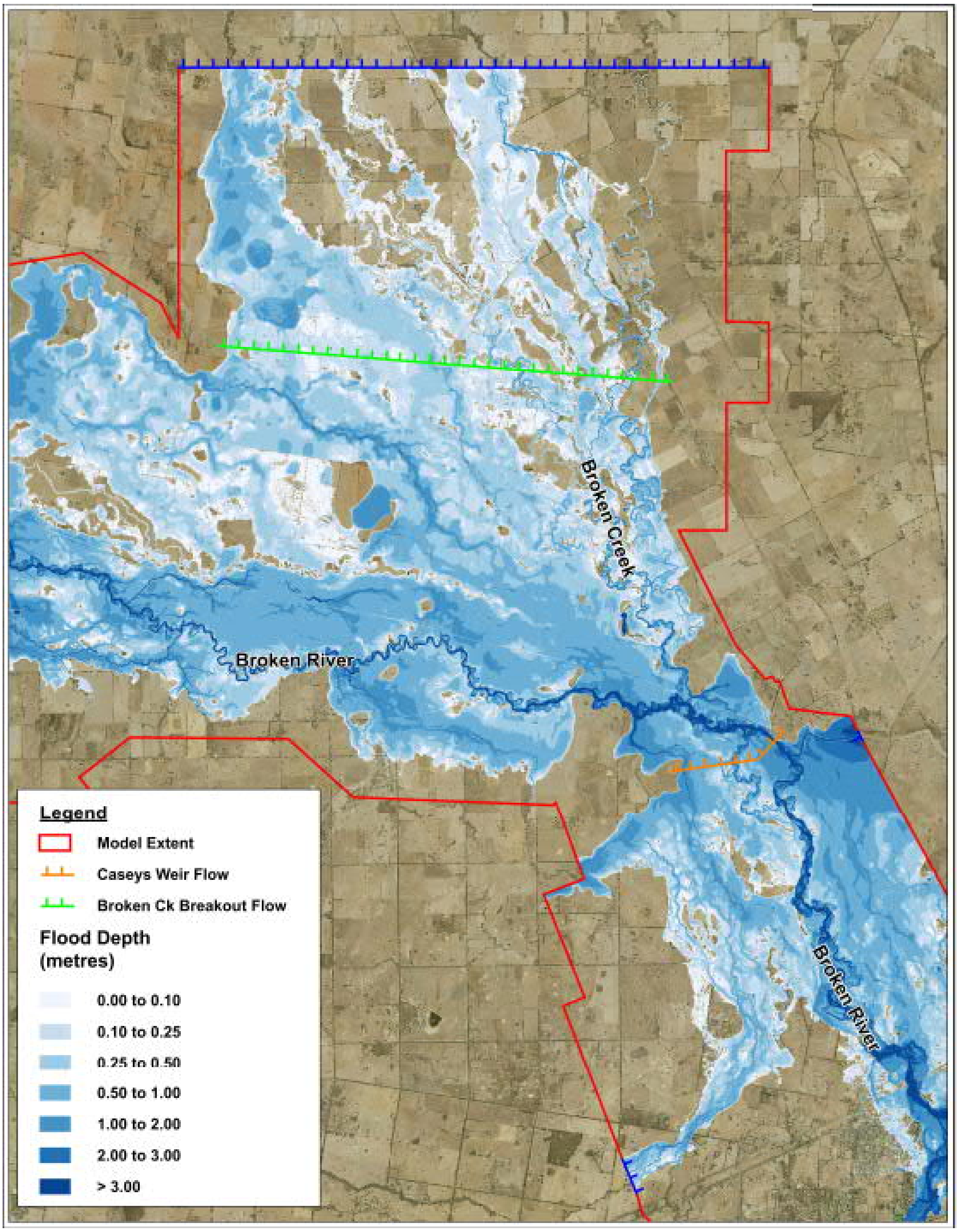


Figure 8-16 Location of Broken Ck Breakout Flow Measurements

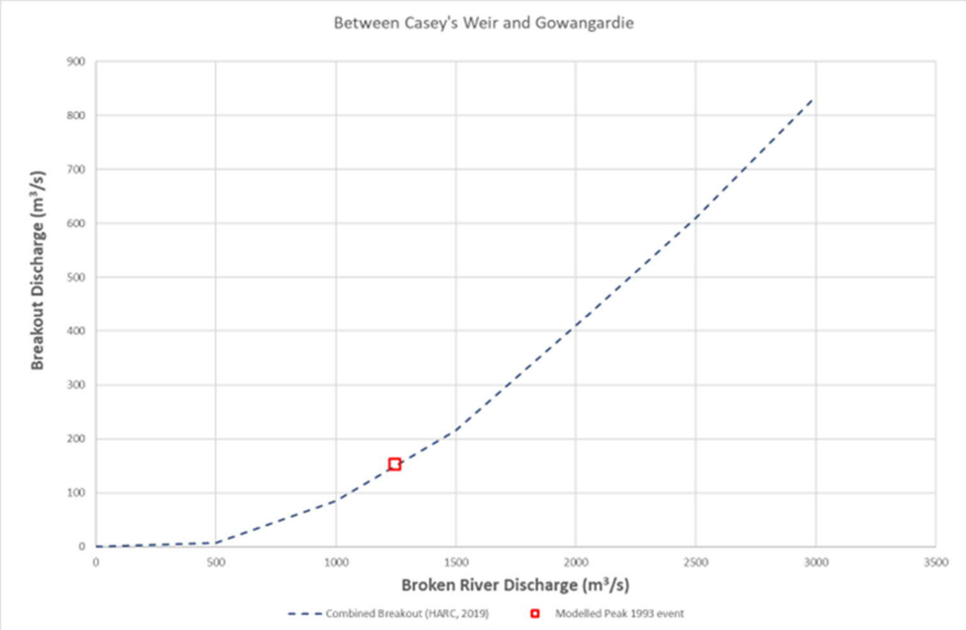


Figure 8-17 Comparison of Caseys Weir Breakout Flow Relationships and TUFLOW for 1993 event

Linking Boundaries

As described above in Section 8.1.7, the hydraulic model incorporated a number of 1D elements that were dynamically linked to the 2D domain. These links were located at the upstream and downstream of each of the culverts to allow flows to interchange water between the two domains. Each culvert had entrance loss of 0.5 and an exit loss of 1 applied. The number of grid cells for each 1D/2D link was selected to ensure that the links were not reducing or adding to the entrance or exit losses.

8.1.9 Surface Roughness

The surface roughness layer, or Manning's 'n' layer, for the floodplain were based on areas of different land-use type as indicated in the planning scheme, aerial photography and during the site inspections. Initially these values were based on standard texts such as Open Channel Hydraulics (Chow 1959), but they were refined during the calibration and validation process (refer Section 8.2). The adopted manning's values are shown in Table 8-6 and the surface roughness layer is shown in Figure 8-18 and Figure 8-19.

Table 8-6 Surface Roughness Values

Land Use	Manning's 'n'
Farms, Residential & Industry	
Roadways (including reserve)	0.025
Low Density Residential Areas	0.100
Residential Areas	0.200
Rural Living Zone	0.060
Town Zone	0.080
Commercial and Business Areas	0.300

Land Use	Manning's 'n'
Services and Utilities	0.040
Railway	0.040
Cemetery	0.080
Schools	0.060
Farm Zones	0.050
Orchard and Dense Cropping	0.100
Parks & Waterways	
Open Channel	0.035
Lakes and Straight Channels	0.018
Parklands and Reserves	0.040
Conservation and Resource Zones	0.060
Scattered Vegetation, Grassy or Shrubby Channel	0.070
Moderate Vegetation	0.080
Dense Vegetation	0.100

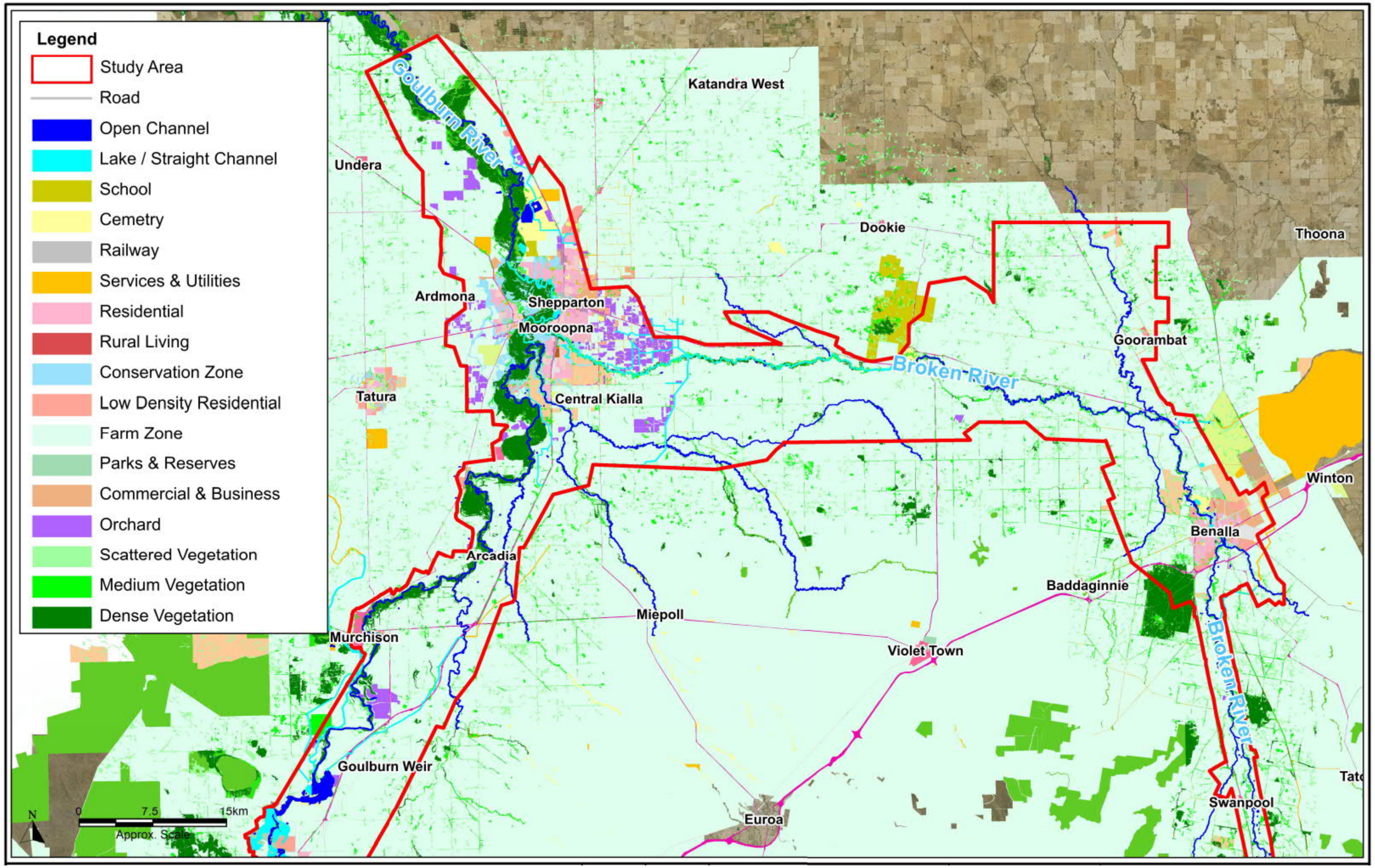


Figure 8-18 Surface Roughness Distribution - North

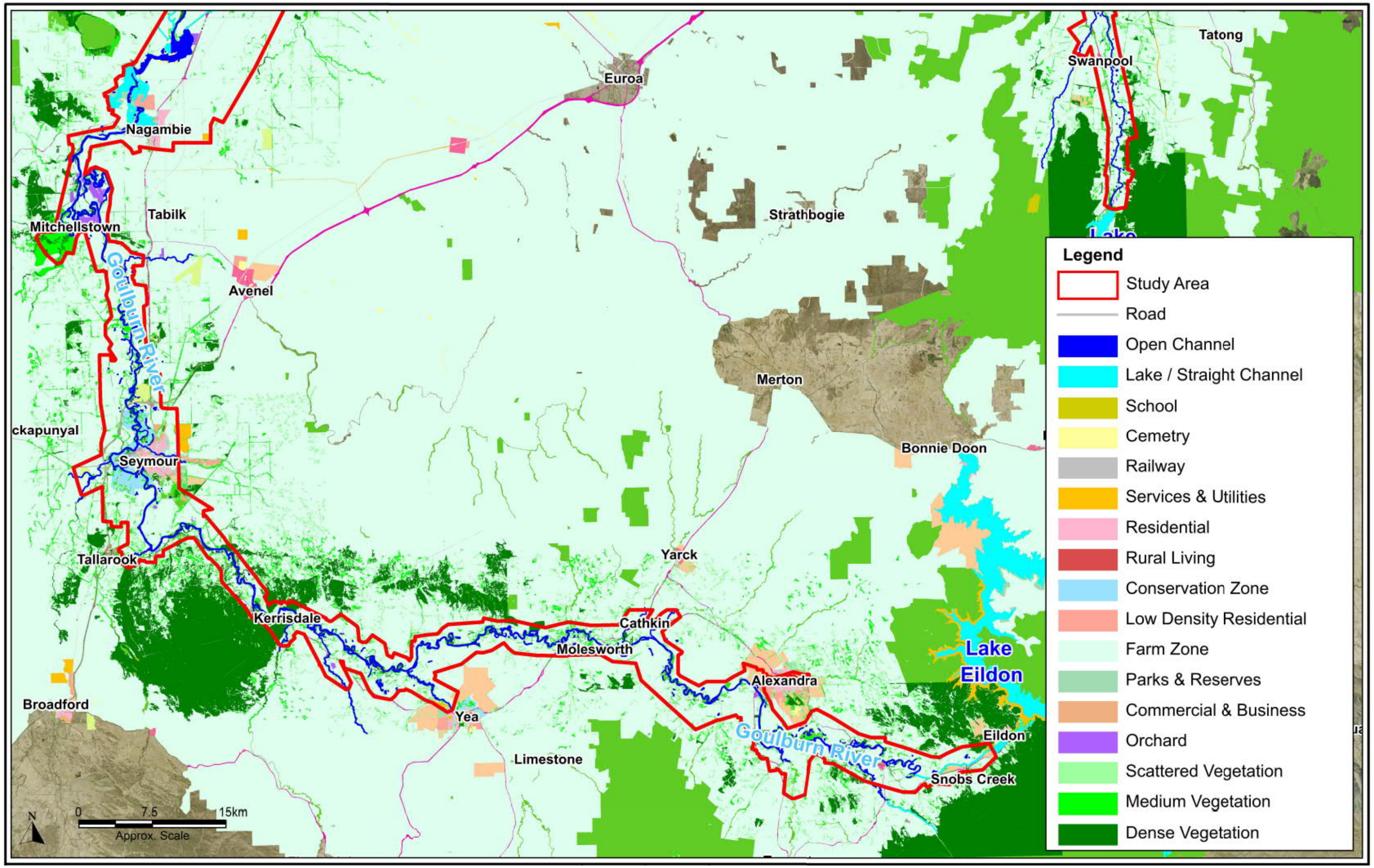


Figure 8-19 Surface Roughness Distribution – South

8.2 Hydraulic Model Calibration & Validation

8.2.1 Observed Flood Levels and Extents

Following a flood of significant magnitude it is common practice to record and document the behaviour of the flood. This information can then be used for a variety of purposes from community engagement, flood emergency response and flood studies such as this. Following the flood responsible agencies conduct community engagement which provides them with information on the maximum flood level and extent. These are then surveyed to provide a record in a GIS environment. Often the responsible agency will estimate and document the flood extent based on contour information or aerial photography. For this study there is a considerable spread of historic flood marks, digitised flood extents and aerial photography. Figure 8-20 shows the volume of available flood level and extents for the study area.

Surveyed historic flood levels marks were downloaded from Vicmap data (http://services.land.vic.gov.au/landchannel/content/productCatalogue). Within the study area the predominately surveyed flood levels are for the 1974 and the 1993 events. There are a number of other historical flood levels (i.e. 1916, 1917, 1934, 1956, 1958, 1966, 1979, 1981 and 2010) but these are isolated points and do not provide the same coverage as the 1974 and 1993 events. The surveyed flood levels for the 2022 event were provided by the GBCMA. There is a good spread of flood marks down both the Goulburn and Broken Rivers which aided calibration and confidence that the natural system throughout the study area is being reliably replicated.

A number of digitised flood extents are also available, namely; 1973, 1974, 1975, 1993, 2012 and 2022. The GBCMA also provided aerial photos for the 1958, 1971, 1974, 1975, 1981 and 1993 events and georeferenced aerial photos of the 2010 event.

Aerial photography captured during flood events was provided by GBCMA. These aerial photographs cover the 1958, 1971, 1974, 1975, 1981, 1993, 2010 and 2022 flood events. Due to the age of many of the events only the 2010 flood photography is georeferenced, and only the 1993, 2010 and 2022 flood events were captured in colour with the remainder in black and white.

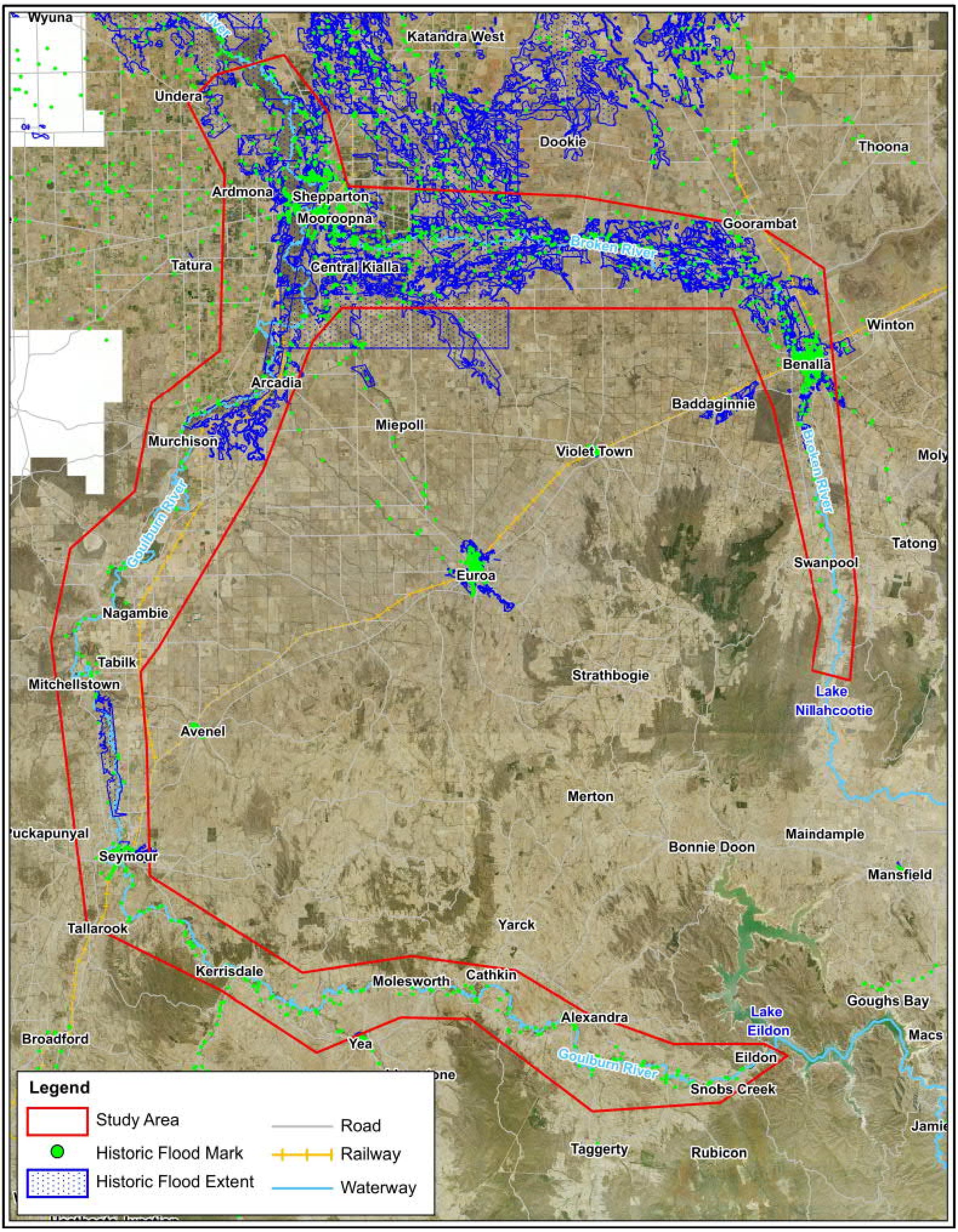


Figure 8-20 Available Flood Levels and Extents

8.2.2 Existing Flood Models

GBCMA provided three existing flood models to aid this study. These models include:

- Goulburn River Environmental Flows Constraints, Water Technology 2015;
- Murchison Township Flood Mapping Study, Water Technology 2014; and
- Shepparton-Mooroopna Flood Mapping and Flood Intelligence, Water Technology 2017.

The Goulburn River Constraints models were developed using TUFLOW GPU by Water Technology in 2015. The various models extend from Lake Eildon and terminate at Echuca on the Victoria-NSW border. The purpose of the model was the assessment of environmental flows and as such was not calibrated for large flood events which are the focus of this investigation. For environmental flows assessment the water levels of interest are typically below bank full, therefore there are no hydraulic structures in the supplied model. However, the model contains an array of layers of use for this study including land use material layers and bathymetric string lines.

The Murchison flood study is a detailed town study extending approximately 4 km upstream of the town and terminating approximately 10 km downstream. The model was developed using TUFLOW. The model includes an array of layers of use for this study including hydraulic structures, land use material layers and bathymetric string lines.

The Shepparton-Mooroopna model extends from Arcadia to the south on the Goulburn, from Kialla East on the Broken and covers the Shepparton-Mooroopna area terminating near Loch Garry to the north. The model was developed as a multi-domain TUFLOW (classic) model with a 10m grid upstream of Shepparton with a coarser 20m grid adopted downstream of the main populated areas. Similar to the other studies, the model contains input layers that could be utilised for this study including land use and structural information.

8.2.3 Calibration runs

The calibration of the hydraulic model is a critical process of any model development. Best practice in model calibration considers all available historic information, which typically would include stream gauge, historic flood extents and flood marks. The available data showed that for the purposes of model calibration the best events were 2022 and 1993. Prior to the 2022 event the 1974 was the largest event on the Goulburn with a reasonable amount of data to calibrate against. However, the 1974 event was replaced with the 2022 event which had more up to date data, with more reliable flow estimates (refer to Section 5.3.3).

For this study a model calibration and validation process was undertaken. Specifically, the hydraulic model was calibrated to the 2022 event, which has the most available data. Through the calibration process parameters were adjusted within typical bounds until a generally acceptable fit to the historic information was achieved. The model was then validated by running the 1993 flood event through the model and comparing to the historic gauge data and flood marks.

The calibration and validation process undertaken for this study are outlined below:

1. Collect and verify if possible relevant historic data including flood marks, flood extents and stream gauge information;
2. Event selection;
3. Hydrologic model calibration, as described in Section 5;
4. Optimise the TUFLOW model parameters for the calibration events within typical bounds;
5. Jointly iterate the hydrologic and hydraulic modelling through a feedback process; and
6. Validate the TUFLOW model against an independent flood event.

The following section document the results of the calibration and validation process whilst the adopted parameters are documented above in Section 8.1.

8.2.4 Hydraulic Model Calibration Event – 2022

The October 2022 flood event was a major event on the Goulburn River, particularly at and downstream of Trawool. It was the largest historic flood on record at Trawool and Seymour. It was a significantly smaller event on the Broken River. The following sections presents the calibration to the gauges and flood marks. The gauge data is the most reliable data available for calibration and it provides additional data in terms of timing of peak and the shape of the hydrograph. Flows above the maximum field gauging levels are derived from an extrapolated rating curve and hence are less reliable than flows from gauged levels but nonetheless are still considered a reliable data source for calibration, particular when a rating review is undertaken (refer to Section 3.2).

The least reliable data are the surveyed flood marks because of uncertainty in whether the mark itself represents the peak flood level. This is evident in the flood mark dataset on this project where there can be considerable discrepancy between marks in close proximity. For gauged flood levels generally a match in flood level within 100 mm and agreement on timing is considered a good calibration, but consideration is also given to the percentage error as a depth of flow. For flood marks a match within 200 mm is considered a good match, but it is recognised that this cannot be achieved with all marks because of uncertainty in the marks. In assessing the fit against flood marks it is also important to look for trends where the marks may be within the 200 mm tolerance but predominantly high or low. If the results are within 200 mm but skewed to the positive or negative, this would indicate an issue with the model rather than uncertainty in the flood marks.

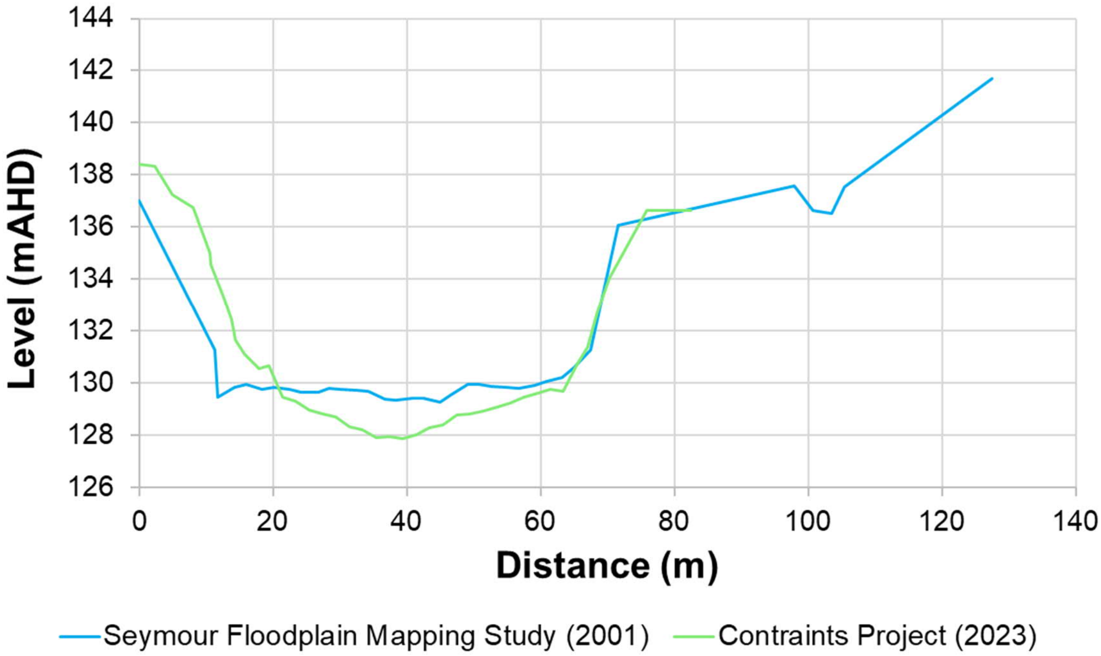
8.2.4.1 Calibration to Gauges

The TUFLOW model was updated with the RORB inflow hydrographs. Figure 8-22 to Figure 8-29 show the flood flow and level gauge comparisons along the Goulburn River. Table 8-7 and Table 8-8 summarise the peak flows and maximum flood heights at the gauges.

At the Trawool gauge (Figure 8-22 and Figure 8-23), TUFLOW matches well with the historical flow data with the peaks matching but the TUFLOW peak is approximately 0.19 m above the recorded level at the gauge. This site control for this gauge is natural. Therefore, the zero gauge may have changed since it was measured which could in part explain the difference. There is also a reasonable agreement in the timing of the modelled and recorded hydrographs.

At the Seymour gauge (Figure 8-24 and Figure 8-25), the TUFLOW matches well with the historical flow in terms of shape and timing with the peak of the recorded hydrograph being approximately 46 m^3/s lower (~3%). The TUFLOW flood level at the peak is about 0.15 m (~9% based on depth) below the recorded level at the gauge. This site control for this gauge is natural. Therefore, the zero gauge may have changed since it was measured which could in part explain the difference. There were several sources of bathymetry data available throughout Seymour to check against. As discussed in Section 8.1.4.2) this included survey that was gathered during the Victorian Constraints Measure Program (Sequna, 2023) and cross sections surveyed as part of the Seymour Floodplain Mapping Study (WBM,2001). Figure 8-21 shows the cross section at the gauge station from the two studies. The survey in 2023 was located at the gauge boards and in 2001 at the gauge, which are approximately 150 metres apart. Figure 8-21 highlights the uncertainty with the bathymetry data with the two cross sections have very different zero points. For this study the survey from 2001 was considered to represent where the levels are recorded acknowledging that the 2001 information was extracted manually from the Appendix in the report. Also, the cross section from 2001 looks like a better control location with a flatter base. The thalweg survey captured for the 2023 study indicates the river bed varies from 130.29 to 129.29 mAHD approximately 20 metres upstream and downstream of the gauge. For this study an average value of 129.7 mAHD was adopted based on the 2001 survey. During calibration and validation, it was found that the results are relatively sensitive to the bathymetry data. Therefore, it is recommended that in any future studies bathymetry data is captured.

There is also a good agreement in the timing of the modelled and recorded hydrographs.



■ Figure 8-21 Surveyed cross sections at Seymour gauge

At Murchison (Figure 8-26 and Figure 8-27) the TUFLOW peak flow is 1630 m^3/s well below (~20%) the recorded flow of 2060 m^3/s. However, the TUFLOW flood levels match very well (within 50 mm). The focus of the calibration was on level rather than flow, as the gauge records level and through a rating curve estimates the flow. The rating curve introduces uncertainty; therefore, the level is generally more reliable. Also, the site control for this gauge is natural. Therefore, the zero gauge may have changed since it was measured which could in part explain the difference. There is also a good agreement in the timing of the modelled and recorded hydrographs.

At Shepparton (Figure 8-28 and Figure 8-29) the TUFLOW peak flow is 2170 m^3/s above (~10%) the recorded flow of 1970 m^3/s. However, the TUFLOW flood levels match very well (within 10mm). The very start of the event does not match but this is due to the way the baseflow has been entered into the model. The baseflow hydrographs are to match the flow at the peak rather than the entire event. As the baseflow is cumulative if the correct baseflow was placed into the model at the start then by the time the upstream baseflow flowed to Shepparton there more be too much baseflow. For this site the control is listed as article rock. There is also a good agreement in the timing of the modelled and recorded hydrographs.

At Caseys Weir (Figure 8-30 and Figure 8-31) the TUFLOW peak flow is 660 m^3/s below (~20%) the recorded flow of 820 m^3/s. However, as discussed in Section 3.2 there is uncertainty in the rating curve above flows of approximately 300 m^3/s. For the adjusted flow the TUFLOW peak flow is above (~5%) the adjusted recorded flow of 630 m^3/s. The TUFLOW flood peak is approximately 70 mm below the recorded level at the gauge which indicates that the revised flow is appropriate. The TUFLOW hydrograph is peaking earlier than the recorded hydrograph in line with the RORB model results.

Table 8-7 2022 Calibration Summary – Peak Flow

Gauge	TUFLOW (m^3/s)	Historical (m^3/s)	Difference (m^3/s)	Difference (%)
Trawool	1020	1022	2	-0.1
Seymour	1740	1780	40	-2
Murchison	1640	2070	430	-20
Shepparton	1795	2212	1795	-19
Caseys Weir	660	814*	154	-20

* Compared to 630 m^3/s based on the revised rating curve

Table 8-8 2022 Calibration Summary – Maximum Flood Height

Gauge	TUFLOW (m AHD)	Historical (m AHD)	Difference (m)
Trawool	146.80	146.61	+0.19
Seymour	137.77	137.91	-0.14
Murchison	120.70	120.75	-0.05
Shepparton	112.16	112.16	0.00
Caseys Weir	160.35	160.43	-0.08

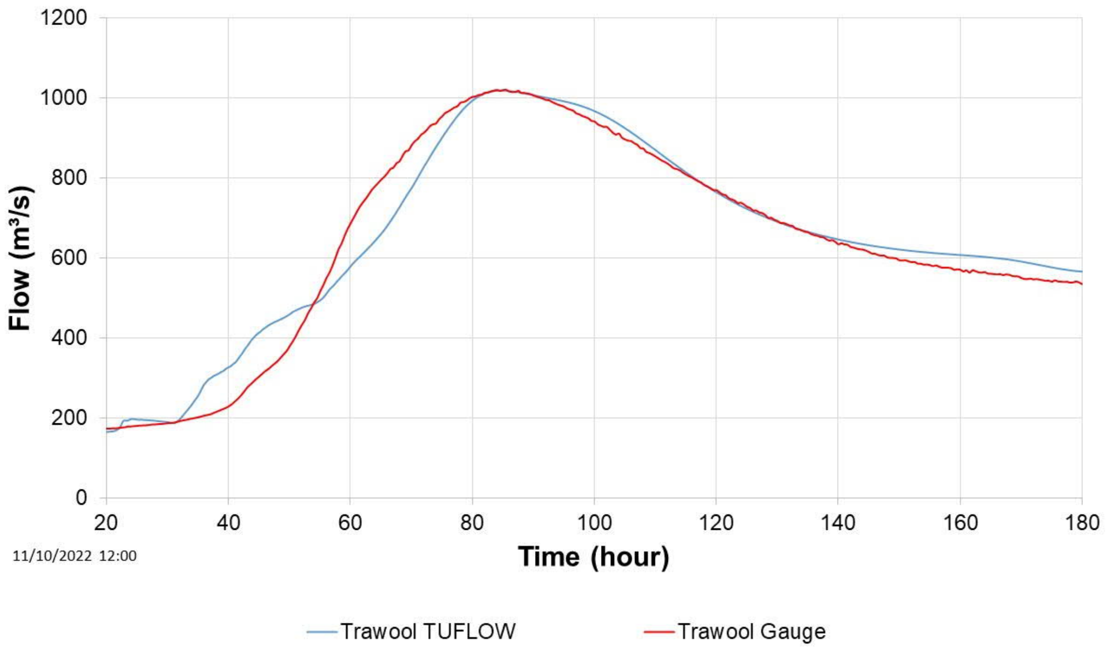


Figure 8-22 Flow Calibration Results 2022 Event – Trawool Gauge

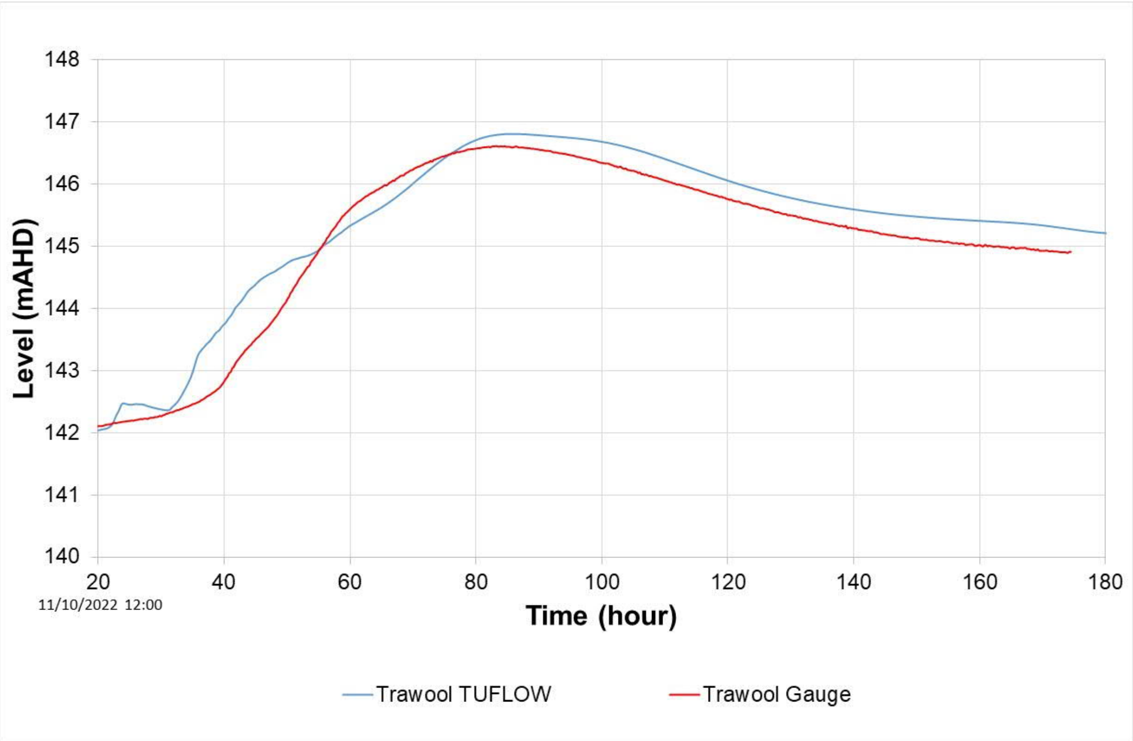


Figure 8-23 Height Calibration Results 2022 Event - Trawool Gauge

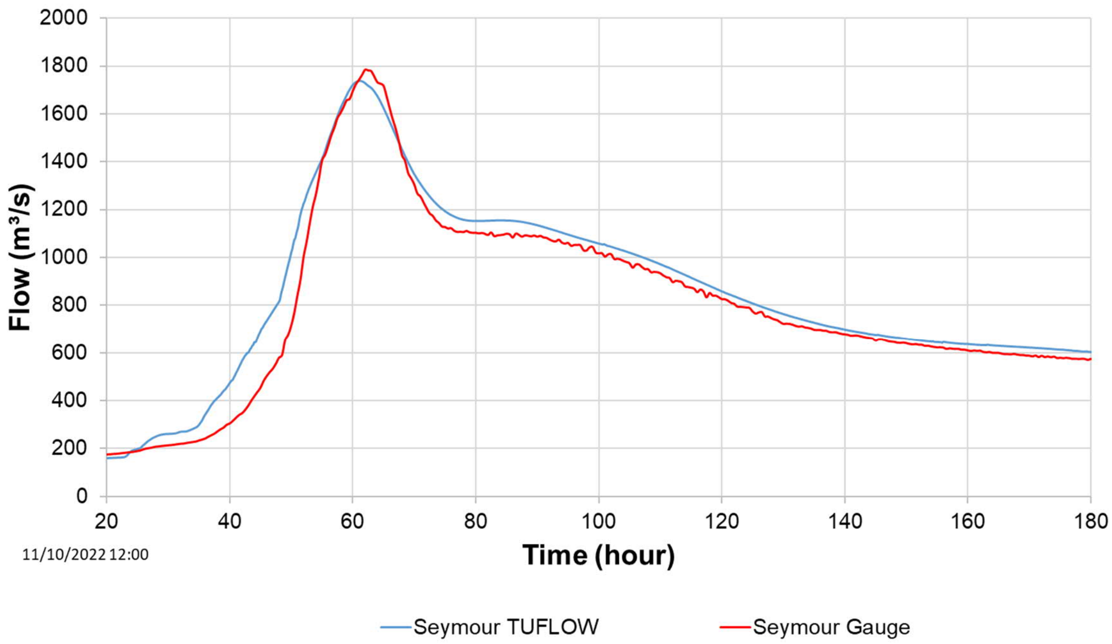


Figure 8-24 Flow Calibration Results 2022 Event - Seymour Gauge

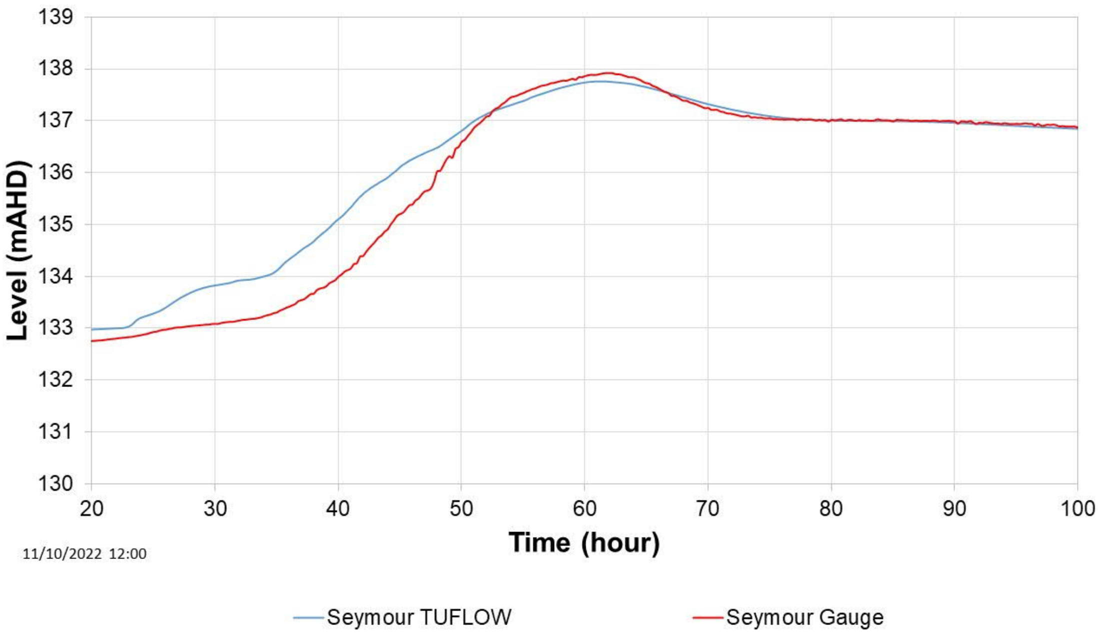


Figure 8-25 Height Calibration Results 2022 Event - Seymour Gauge

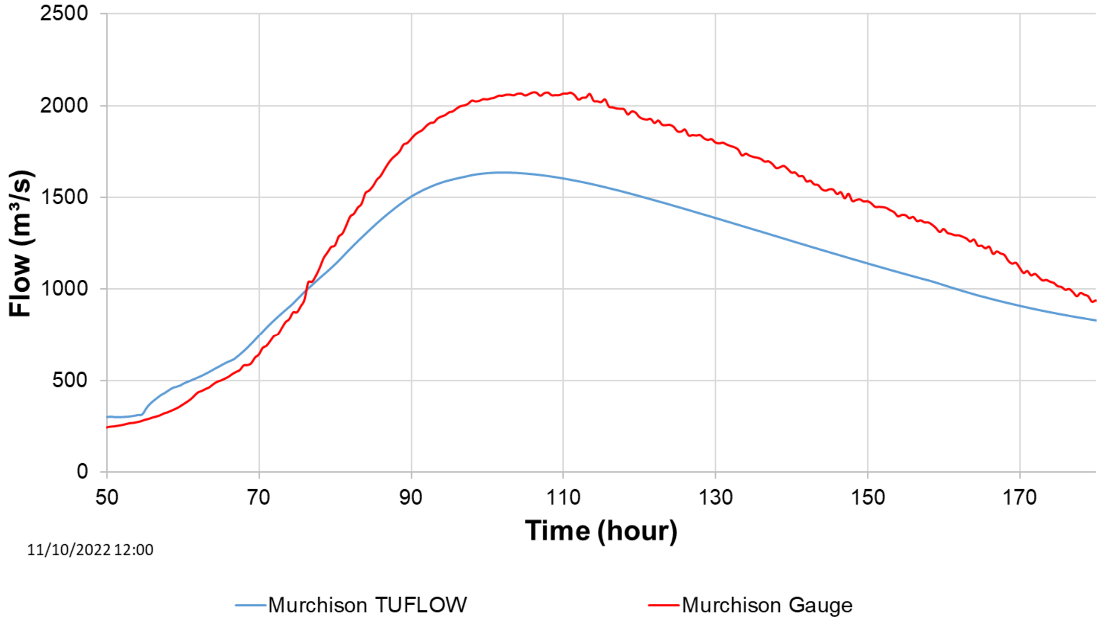


Figure 8-26 Flow Calibration Results 2022 Event - Murchison Gauge

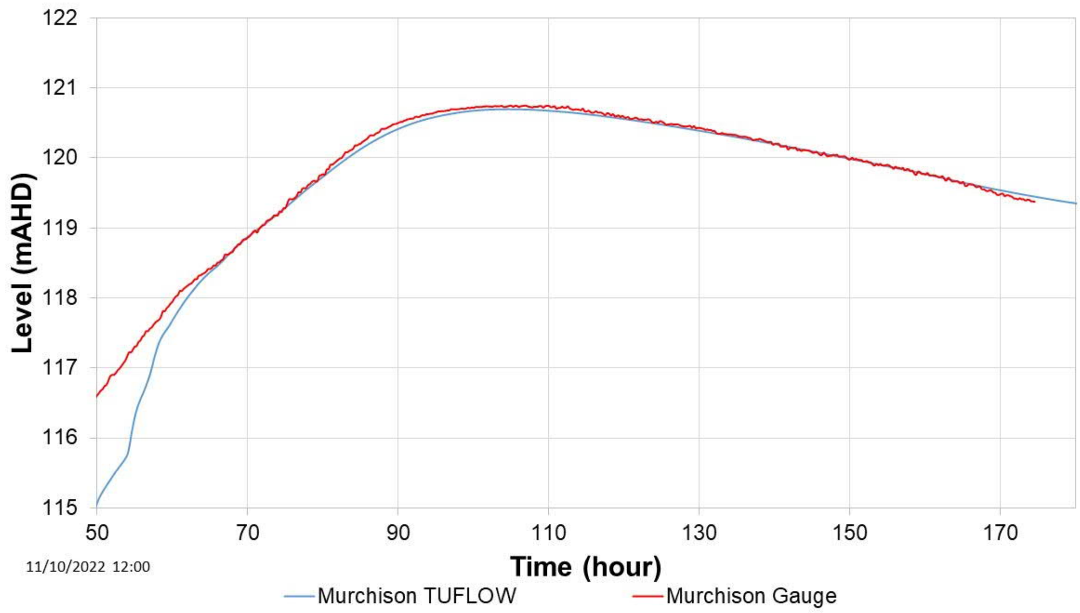


Figure 8-27 Height Calibration Results 2022 Event - Murchison Gauge

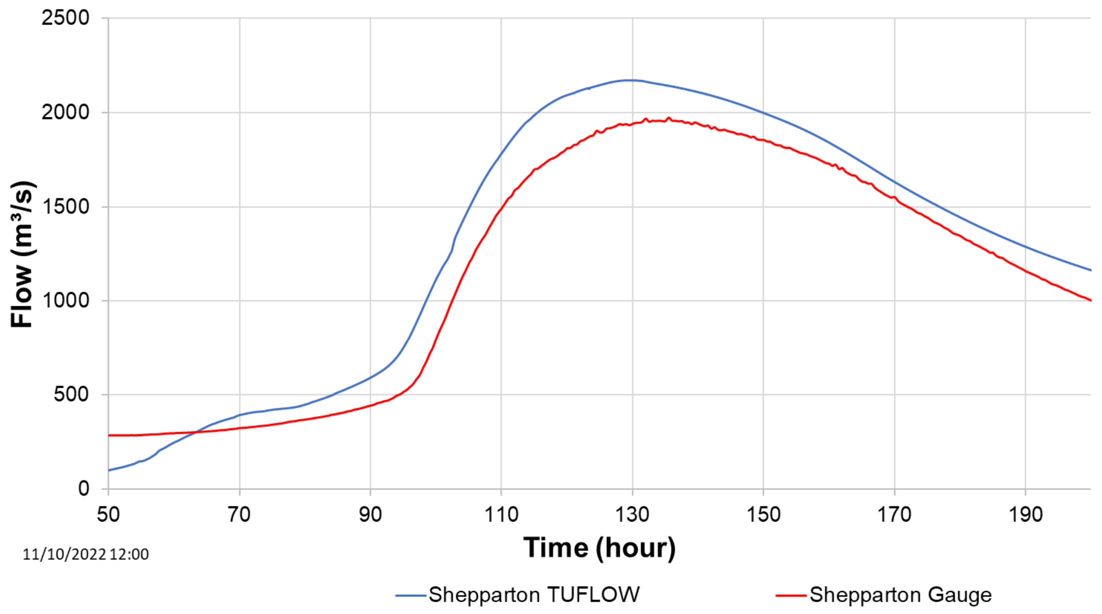


Figure 8-28 Flow Calibration Results 2022 Event - Shepparton Gauge

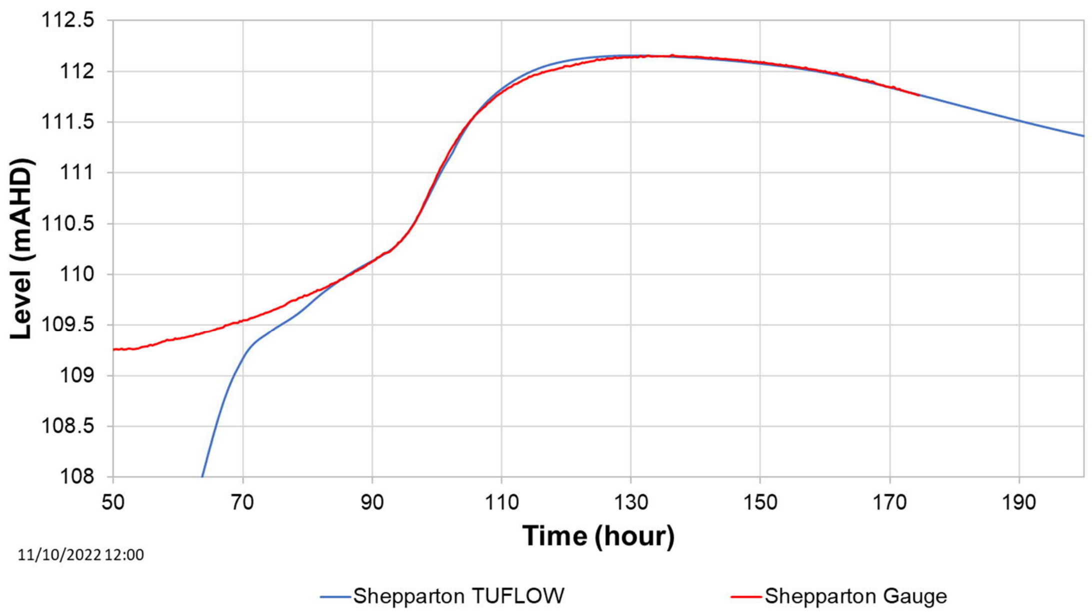


Figure 8-29 Height Calibration Results 2022 Event - Shepparton Gauge

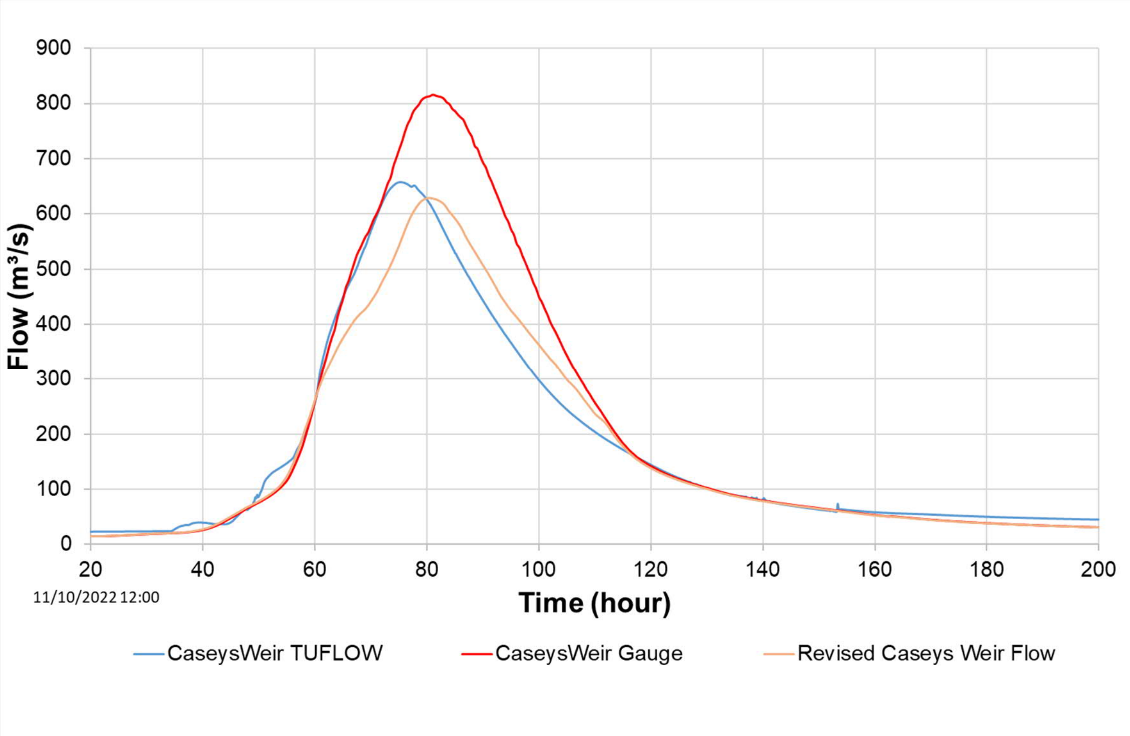


Figure 8-30 Flow Calibration Results 2022 Event – Caseys Weir Gauge

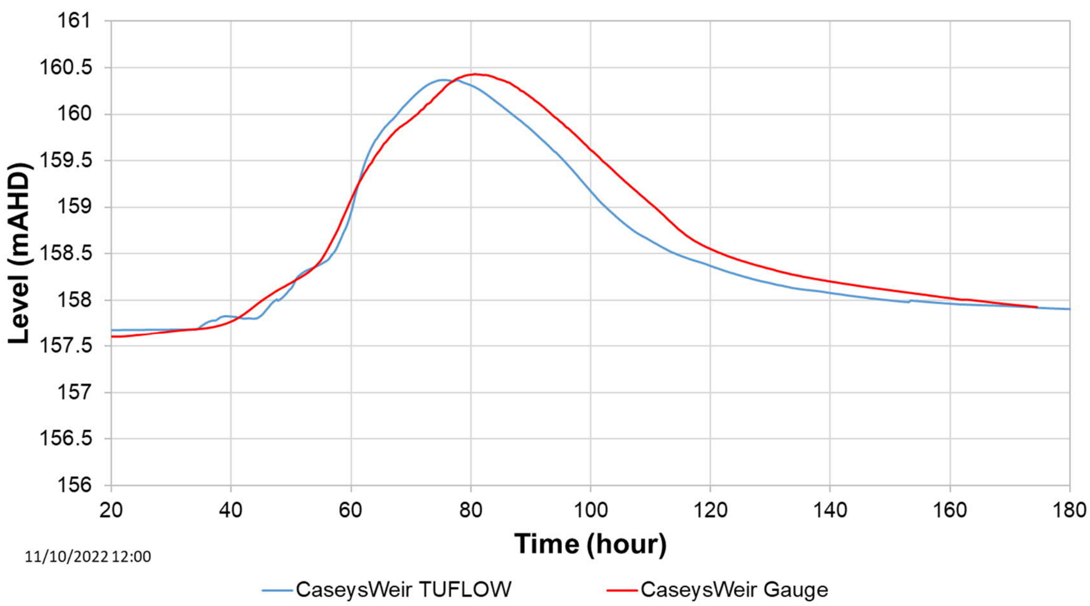


Figure 8-31 Height Calibration Results 2022 Event - Caseys Weir Gauge

8.2.4.2 Flood Mark Calibration

The historical flood marks for the 2022 event are well distributed around the study area. In total there are 298 marks along the Broken and 352 marks along the Goulburn downstream of Shepparton.

Inspection of the flood marks raised concern over the reliability of many of the marks because of inconsistent flood levels down the waterway. Many locations were identified where significant variance was found within a local area. An example of this is demonstrated in Benalla where there is nearly 1 m vertical differences in flood marks in close proximity as shown in Figure 8-32. Another example is near Kialla West as shown in Figure 8-33. In general, the model was much higher than the flood marks in and around Kialla and Kialla West.

A statistical summary comparison of the observed and modelled flood levels is presented in Table 8-9 and visually in Figure 8-34 which indicates the model is generally overestimating flood levels (skewed to the positive). Of note is that this result is being biased by the results around Kialla and Kialla West. There is some doubt about these flood marks as most are recorded as a high water mark on a tree or post. With the model reasonably replicating levels at Murchison, Broken and Shepparton it is not possible to reduce levels in this area without affected the other areas.

Figure 8-35 to Figure 8-39 shows the modelled flood depths along with a spatial comparison of the modelled and observed flood level at the flood marks. There are a number of trends evident:

- At Seymour the calibration is reasonable with an even spread of flood marks that are high and low;
- At Murchison the calibration is reasonable with a spread of flood marks high and low but tending towards high.
- In and around Kialla and Kialla West the flood marks are a lot lower than the model;
- Through Shepparton the calibration is reasonable to high;
- On the Broken, at Benalla the calibration is reasonable to high. Downstream of Benalla the calibration is good.

Figure 8-32 Example inconsistent vertical levels of 2022 flood marks - Benalla

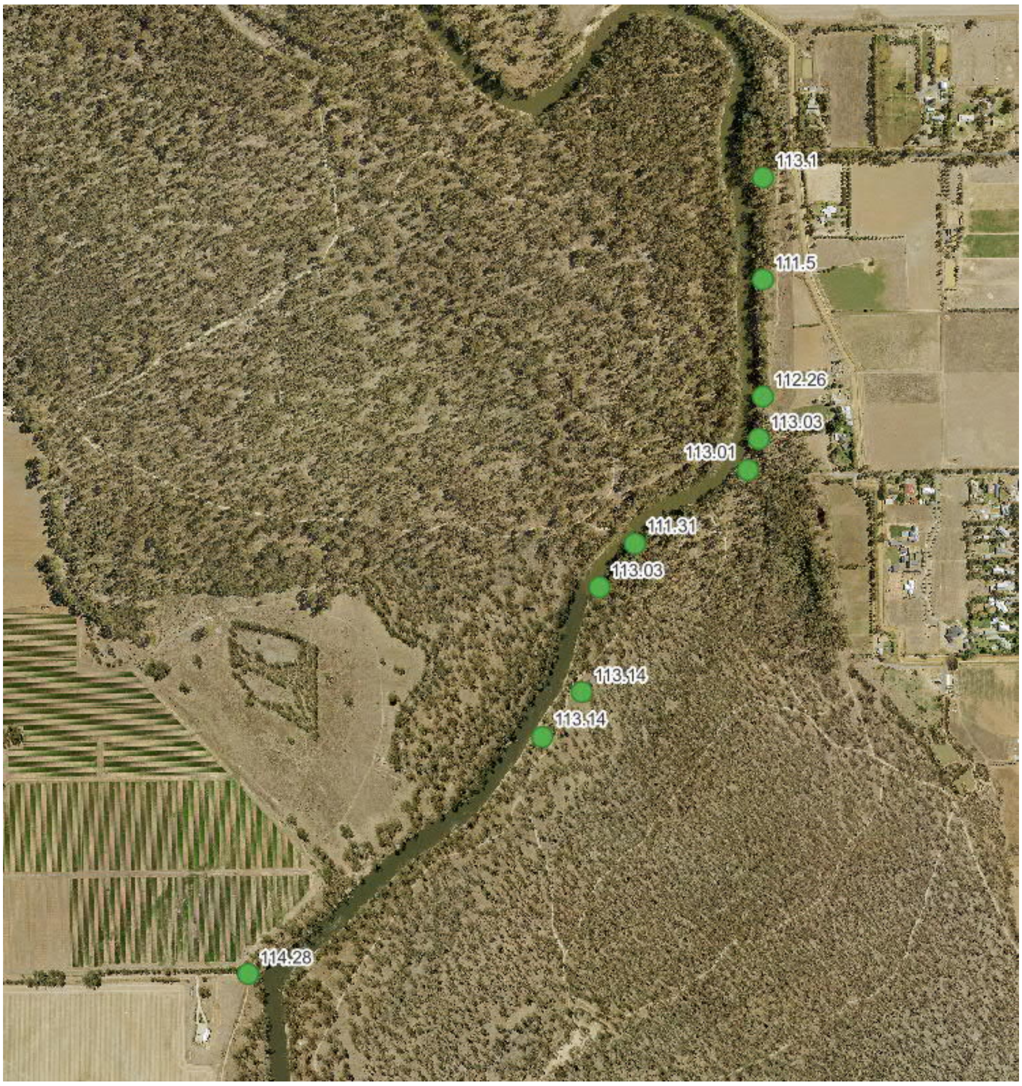


Figure 8-33 Example inconsistent vertical levels of 2022 flood marks – Kialla West

Table 8-9 Statistical Summary of Comparison of Observed and Modelled Flood Levels - 2022 Flood Event

Data Source	Historic Flood Heights
No. of Points Interrogated	650
Mean	0.12
Median	0.01
Standard Deviation	0.60
Lower Quartile	-0.13
Upper Quartile	0.22

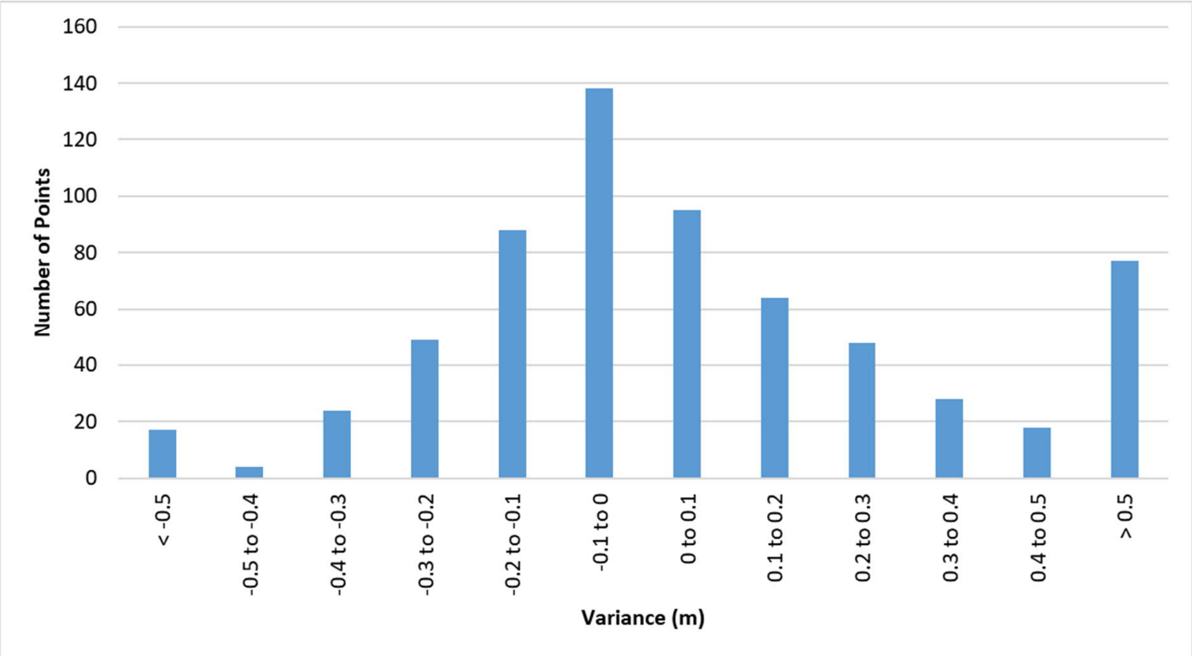


Figure 8-34 Statistical Summary of Comparison of Observed and Modelled Flood Levels - 2022 Flood Event

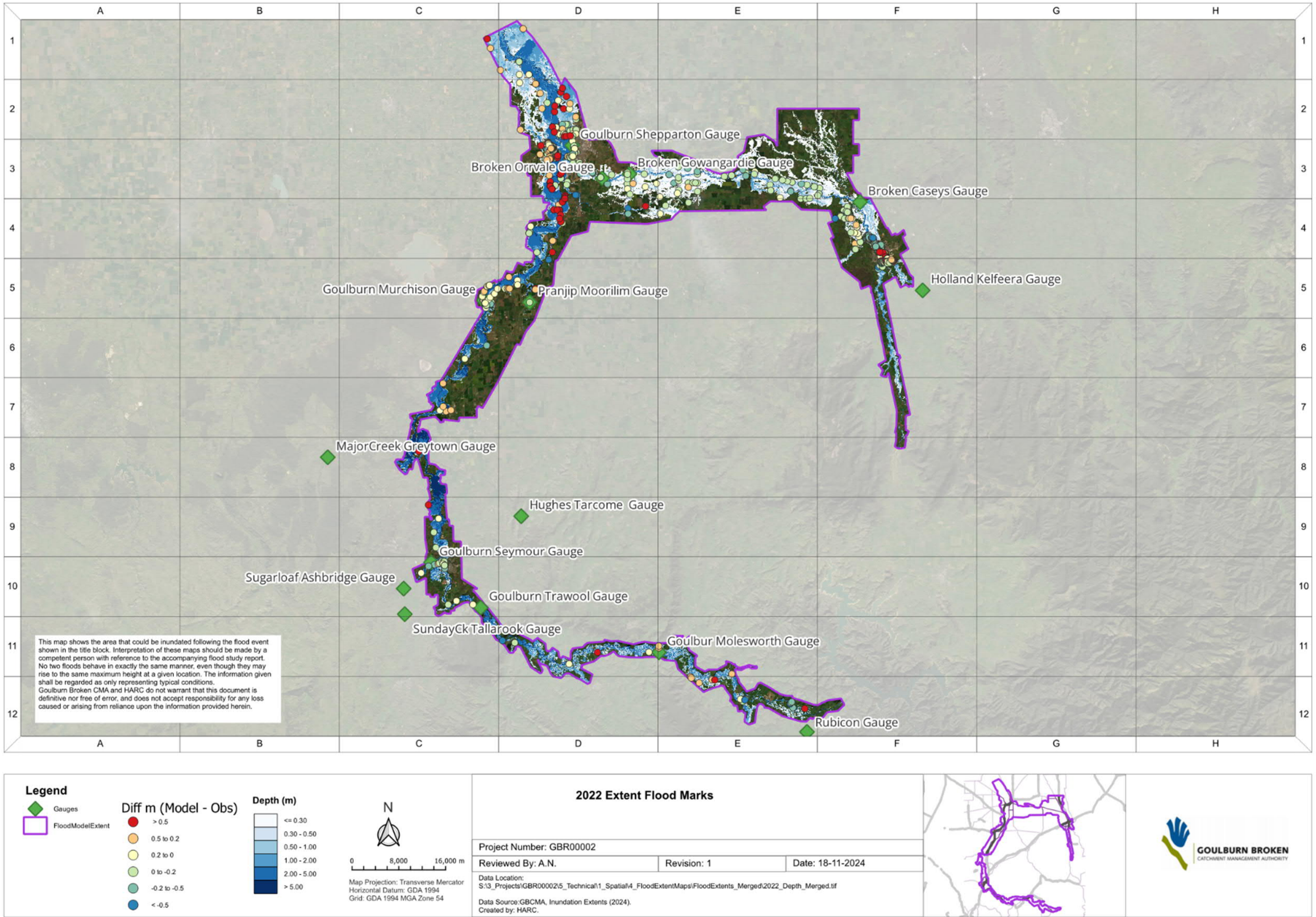


Figure 8-35 Hydraulic Model Validation Results 2022 Event

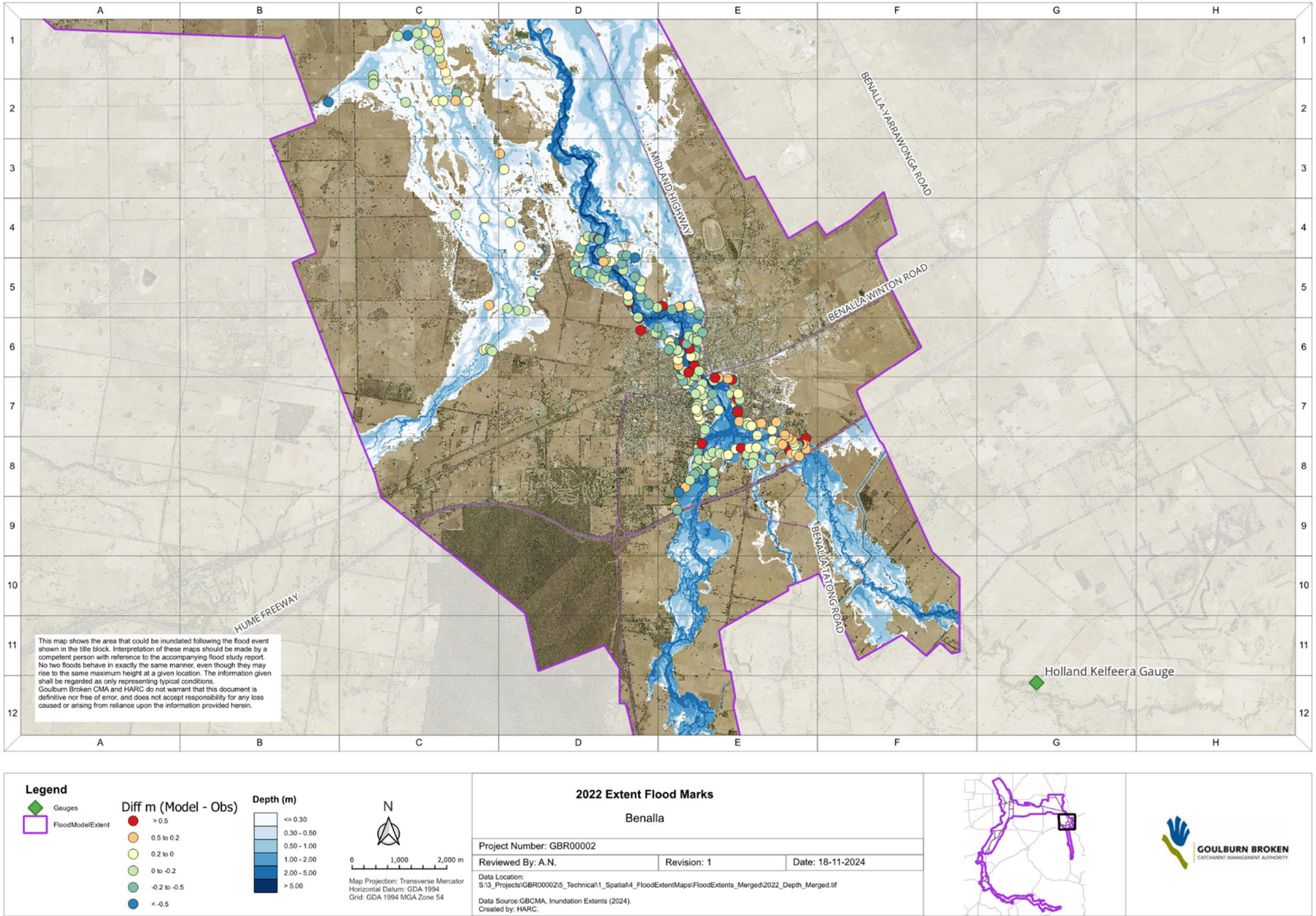


Figure 8-36 Hydraulic Model Validation Results 2022 Event (Benalla)

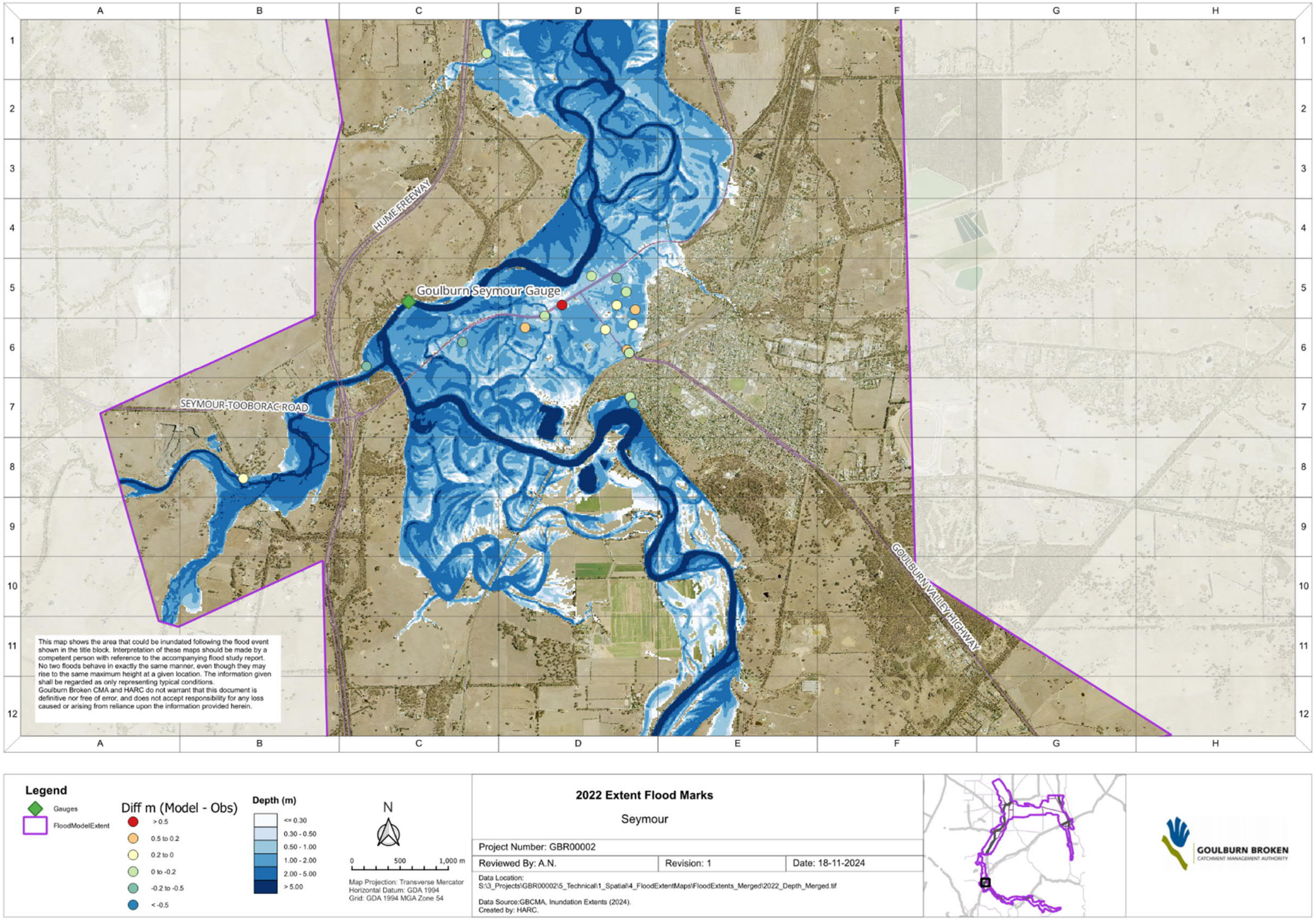


Figure 8-37 Hydraulic Model Validation Results 2022 Event (Seymour)

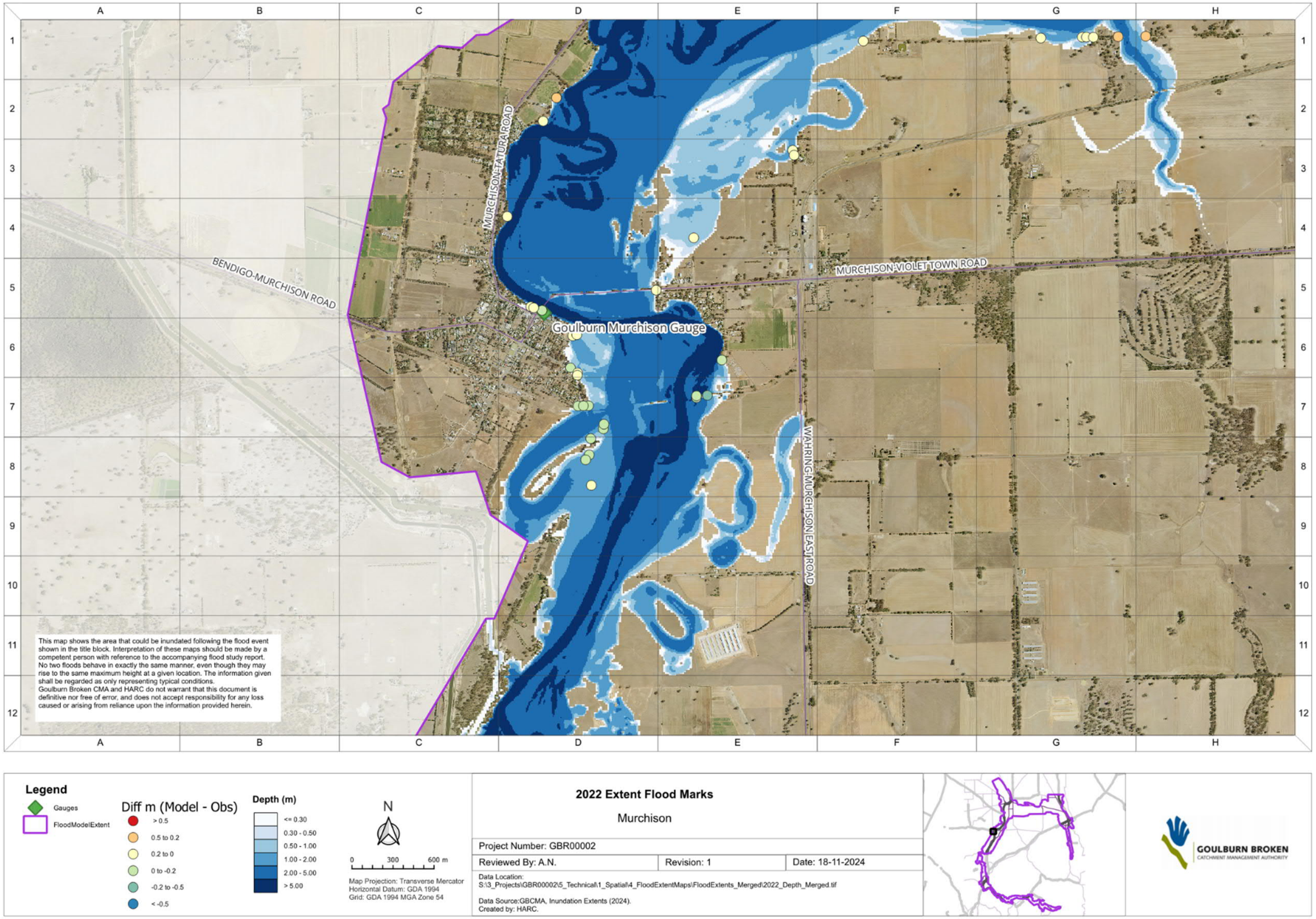


Figure 8-38 Hydraulic Model Validation Results 2022 Event (Murchison)

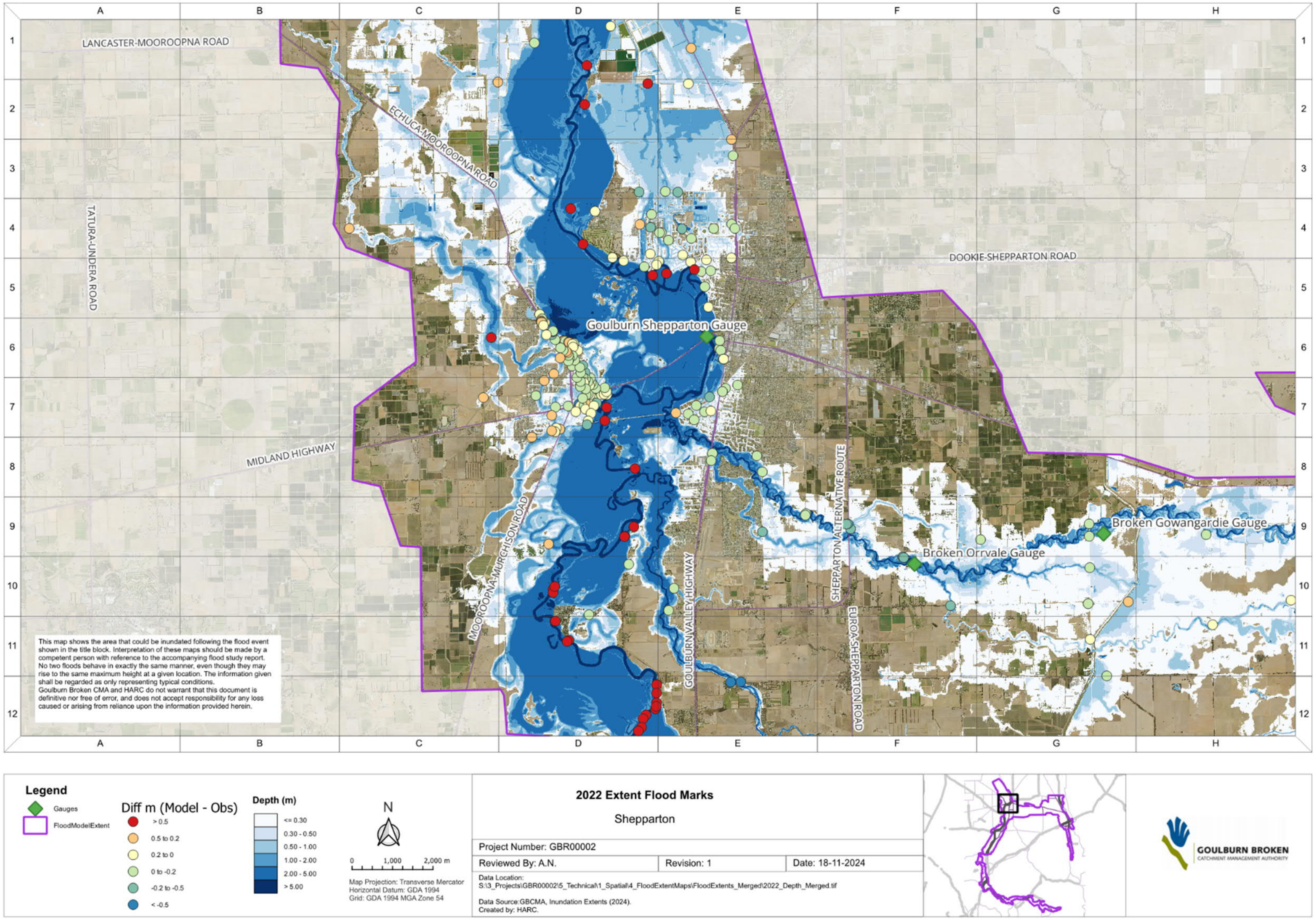


Figure 8-39 Hydraulic Model Validation Results 2022 Event (Shepparton

8.2.5 Hydraulic Model Validation Event – 1993

Being a validation event the same model setup was used as for the 2022 calibration event. Therefore, changes in, for example topography and surface roughness, that may have occurred are not reflected in the model and can lead to anomalous results in the validation event.

The 1993 flood event is the largest recorded event on the Broken River system. However, for the Goulburn River it was a significantly smaller event. This meant that a significant amount of data, in the form of flood marks, was gathered along the length of the Broken River and for the Goulburn River below the confluence. The upper reaches of the Goulburn River also have a significant number of flood marks, however, there were no flood marks recorded between Seymour and the confluence with the Broken River. In total there are 876 marks along the Broken, 129 marks along the Goulburn and another 298 marks around or downstream of Shepparton.

A summary of the validation outcomes at the gauges and peak flood marks is provided below, followed by a detailed discussion on the calibration process.

8.2.5.1 Validation to Gauges

Figure 8-40 to Figure 8-49 present the modelled and observed flood level and flow comparisons for the gauges on the Goulburn and Broken River. Table 8-10 and Table 8-11 summarise the peak flows and maximum flood heights at the gauges.

At the Trawool gauge (Figure 8-40 and Figure 8-41), the TUFLOW peak is 615 m^3/s below (~5%) the recorded flow of 650 m^3/s. However, the TUFLOW peak flood level matches well (within 70 mm) with the historical peak. The timing of the hydrograph is also reasonable, with the modelled hydrograph peaking latter than the recorded.

At the Seymour gauge (Figure 8-42 and Figure 8-43), the TUFLOW peak is 680 m^3/s higher (~5%) than the recorded flow of 650 m^3/s. The TUFLOW peak flood level is approximately 0.28 m above the recorded level at the gauge which is a poor fit. As mentioned in Section 8.2.4.1 at this gauge the site control for is natural. Therefore, the zero gauge may have changed since it was measured which could in part explain the difference. The timing of the hydrograph is also reasonable, with the exception of the rising limb. The poor timing of the rising limb is also reflected in the RORB model (refer to Section 5.3.1).

At Murchison (Figure 8-44 and Figure 8-45) the TUFLOW peak flow is 690 m^3/s below (~10%) the recorded flow of 730 m^3/s. The TUFLOW flood levels match very well (within 5 mm). The timing is good with the gauge flow rising quicker than the modelled.

At Shepparton (Figure 8-46 and Figure 8-47) the TUFLOW peak flow is 1790 m^3/s above (~3%) the recorded flow of 1730 m^3/s. The TUFLOW flood levels match very well (within 100mm). There is also a good agreement in the timing of the modelled and recorded hydrographs.

At Caseys Weir (Figure 8-48 and Figure 8-49) the TUFLOW peak flow is 1240 m^3/s above (~2%) the recorded flow of 1220 m^3/s. The TUFLOW flood peak is approximately 0.20 m below the recorded level at the gauge which is a reasonable match. The timing of the hydrograph is good.

Table 8-10 1993 Calibration Summary – Peak Flow

Gauge	TUFLOW (m^3/s)	Historical (m^3/s)	Difference (m^3/s)	Difference (%)
Trawool	615	650	35	-5
Seymour	680	650	30	5
Murchison	690	730	40	-10
Shepparton	1790	1740	50	3
Caseys Weir	1240	1220	20	2

Table 8-11 1993 Calibration Summary – Maximum Flood Height

Gauge	TUFLOW (m AHD)	Historical (m AHD)	Difference (m)
Trawool	145.43	145.36	0.07
Seymour	136.05	135.77	0.28
Murchison	118.97	118.94	0.03
Shepparton	111.93	111.84	0.09
Caseys Weir	160.9	161.09	0.19

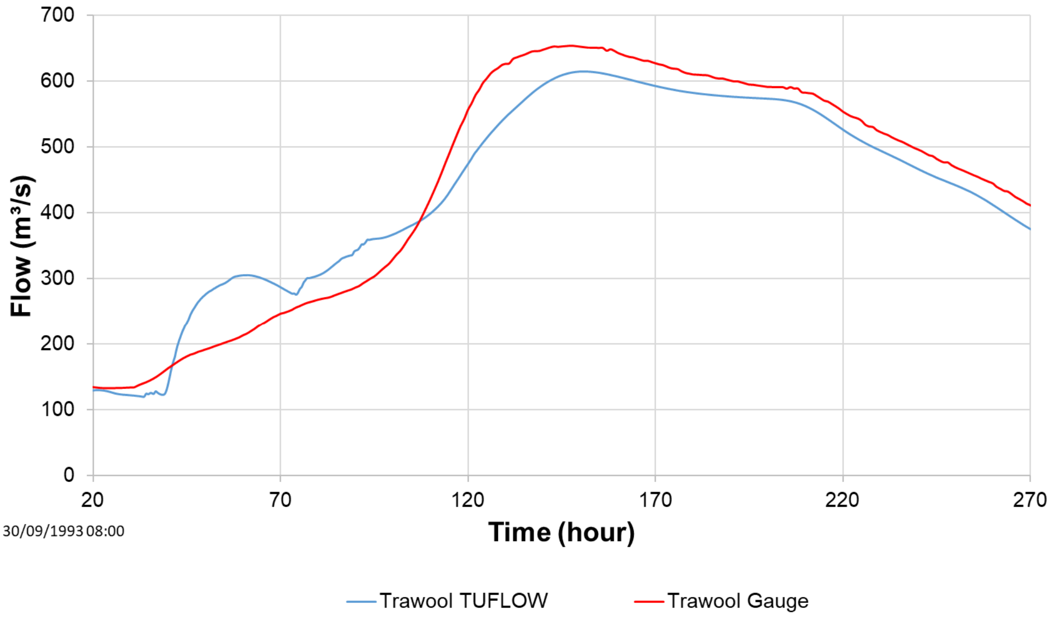


Figure 8-40 Flow Validation Results 1993 Event - Trawool Gauge

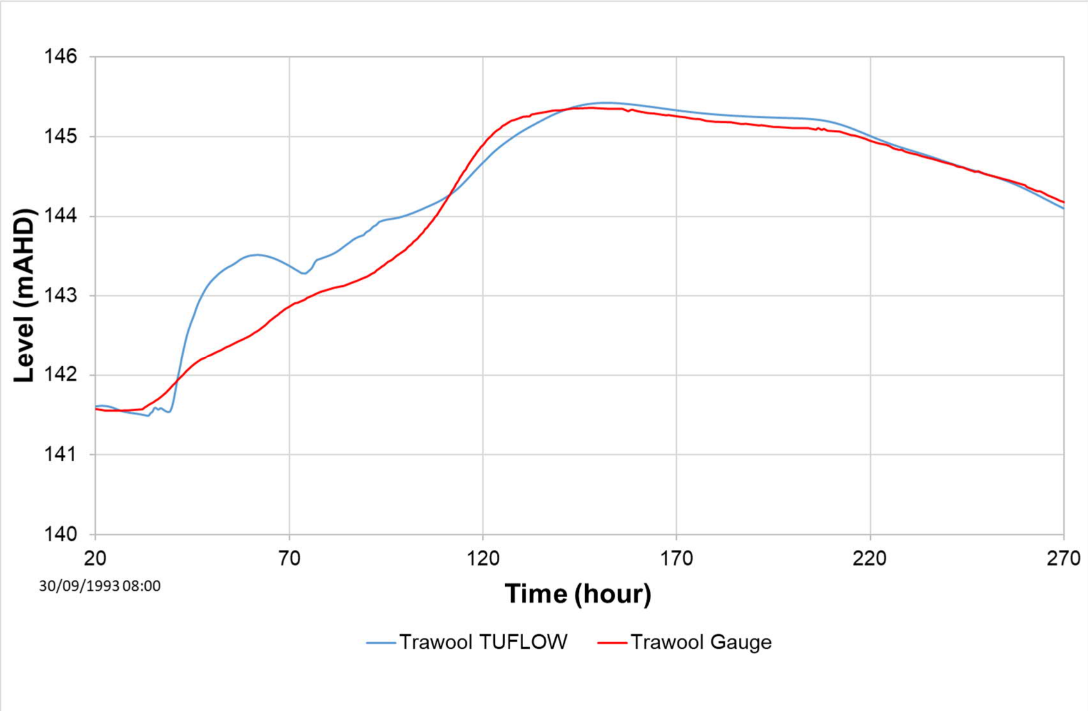


Figure 8-41 Height Validation Results 1993 Event - Trawool Gauge

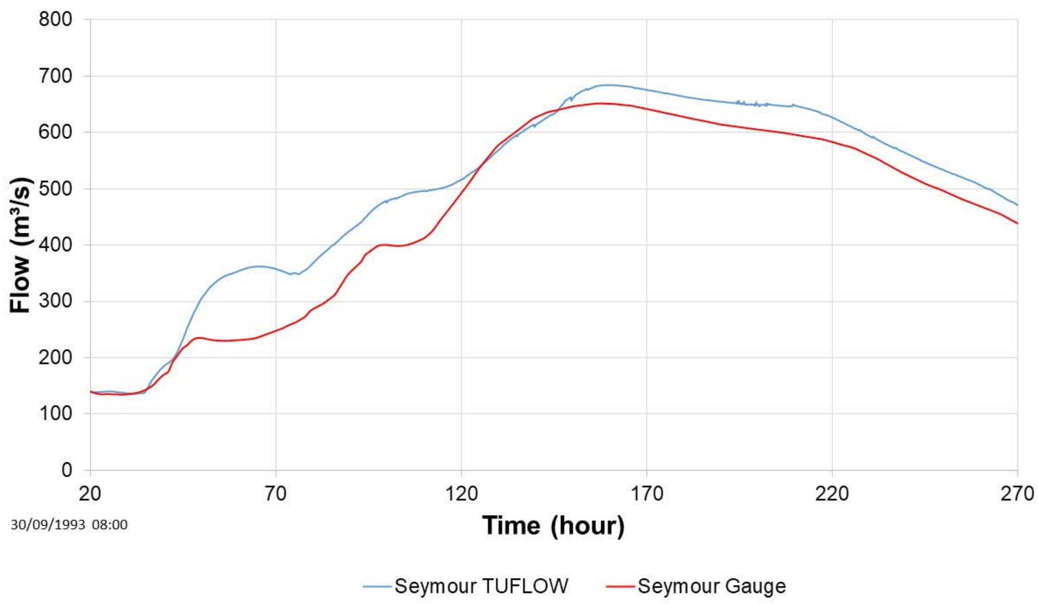


Figure 8-42 Flow Validation Results 1993 Event – Seymour Gauge

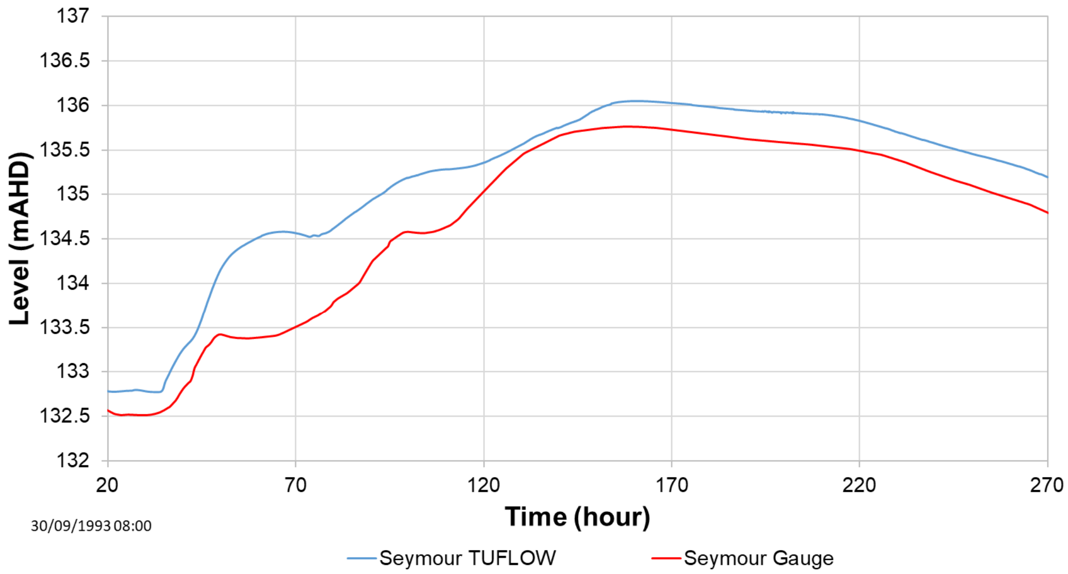


Figure 8-43 Height Validation Results 1993 Event – Seymour Gauge

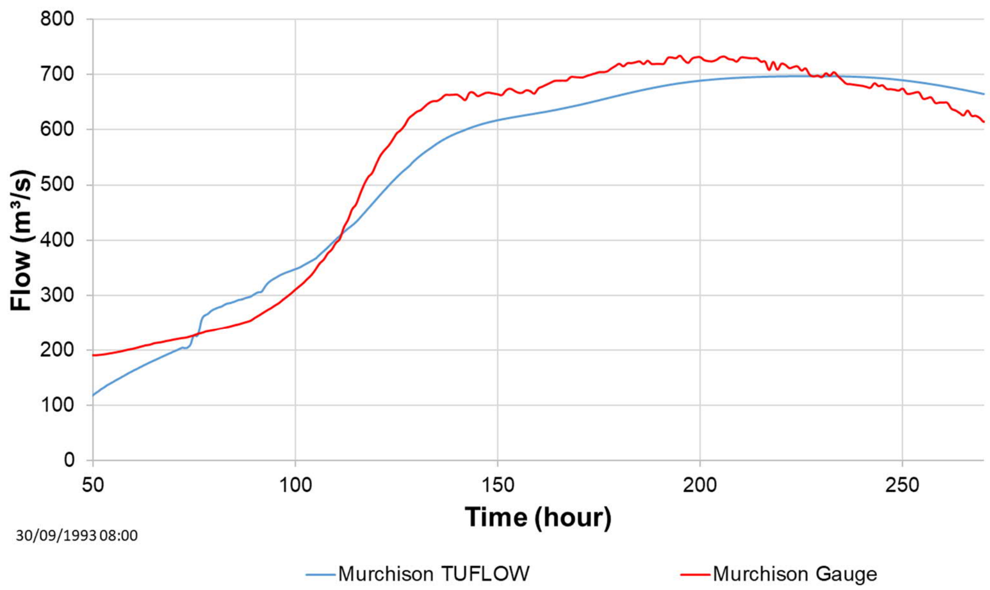


Figure 8-44 Flow Validation Results 1993 Event – Murchison Gauge

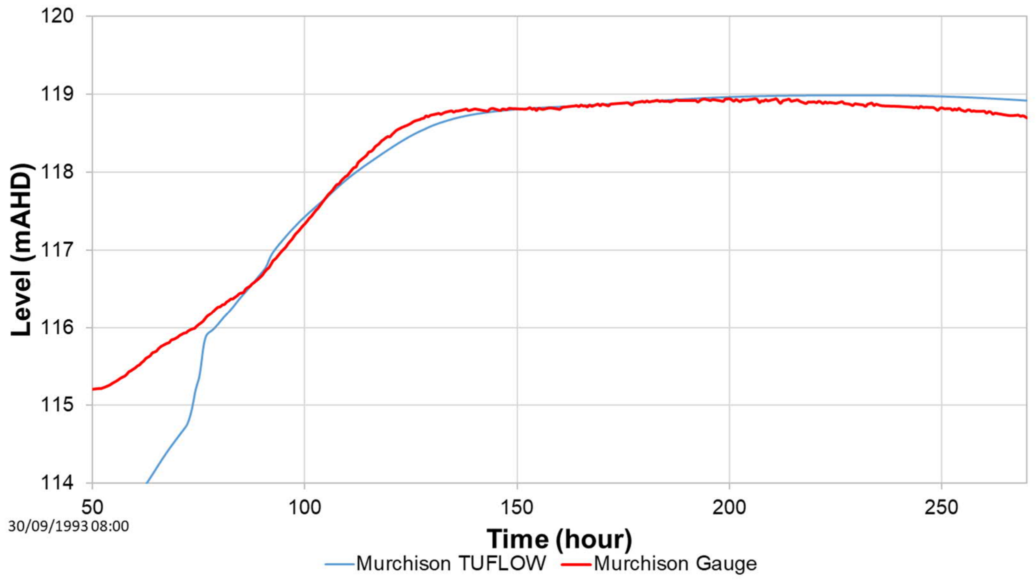


Figure 8-45 Height Validation Results 1993 Event – Murchison Gauge

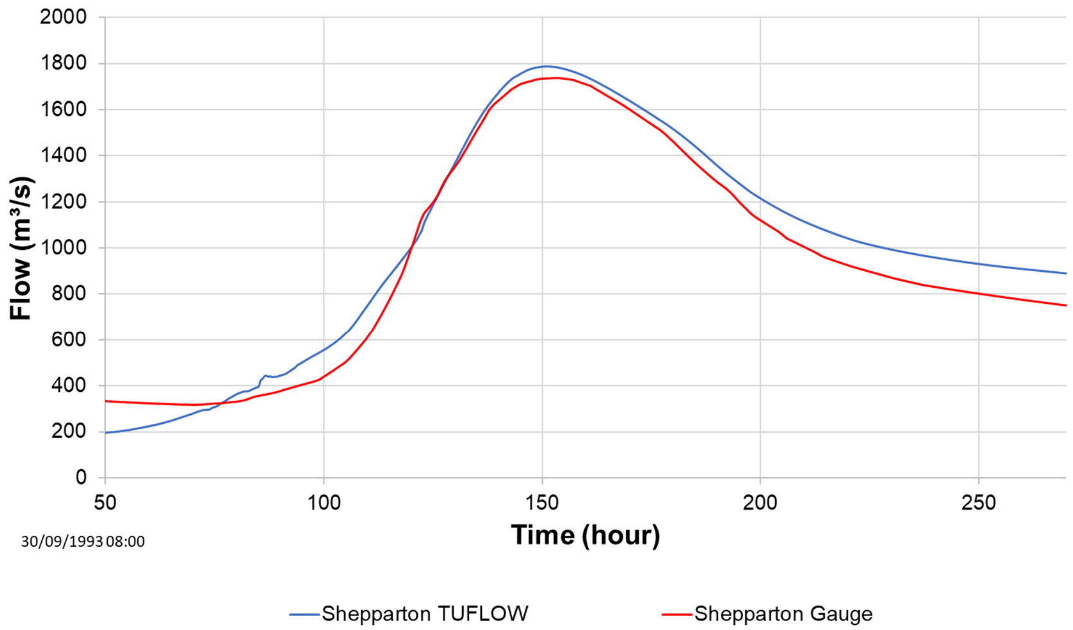


Figure 8-46 Flow Validation Results 1993 Event – Shepparton Gauge

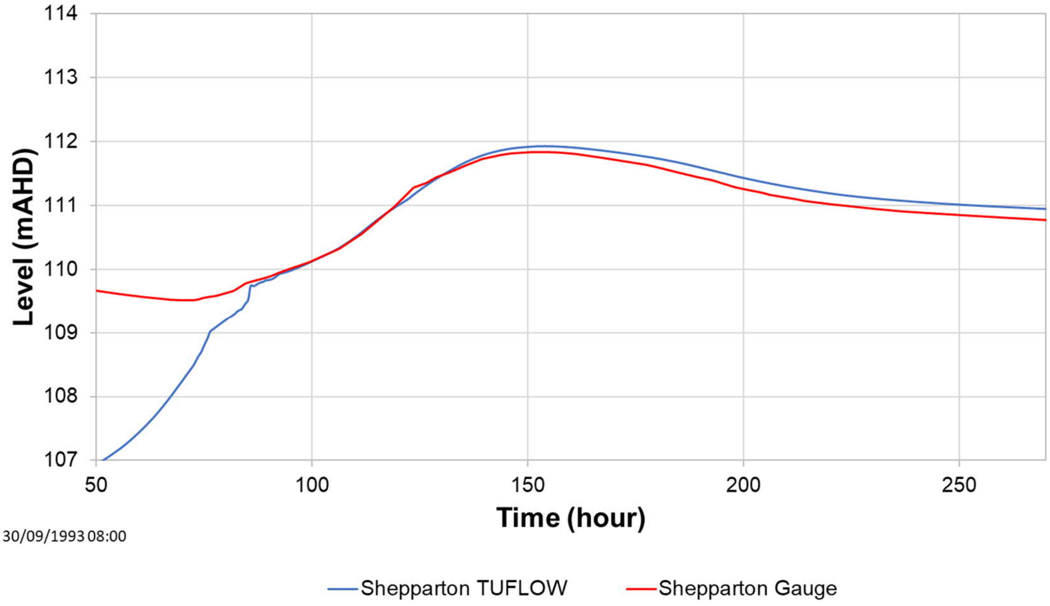


Figure 8-47 Height Validation Results 1993 Event – Shepparton Gauge

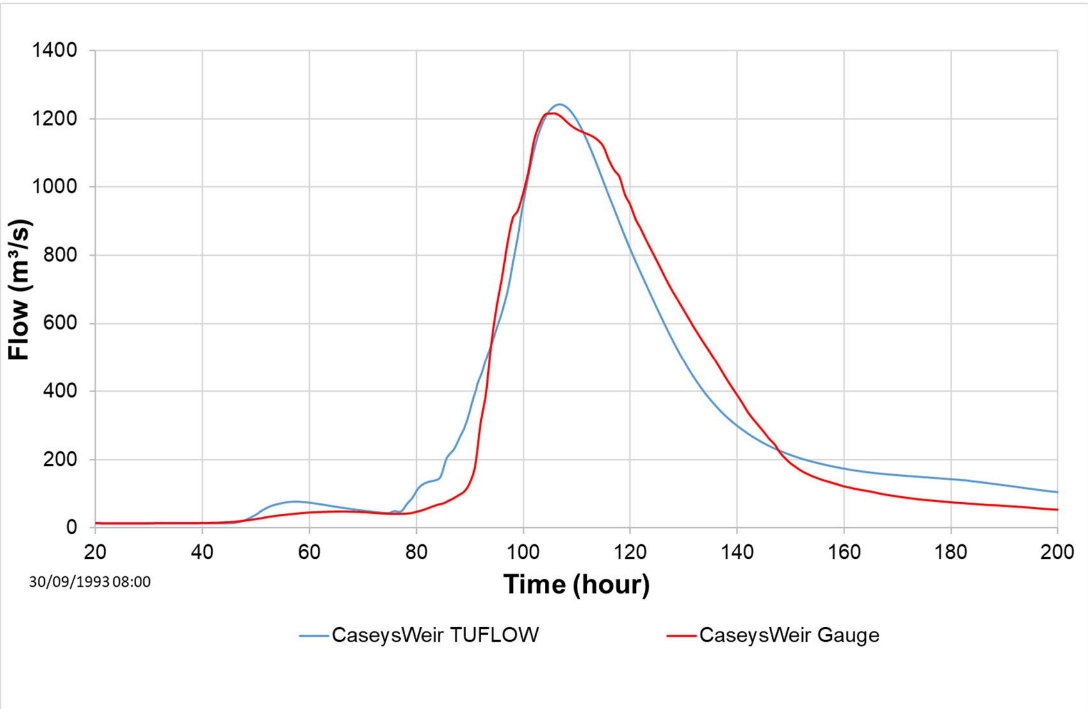


Figure 8-48 Flow Validation Results 1993 Event – Caseys Weir Gauge

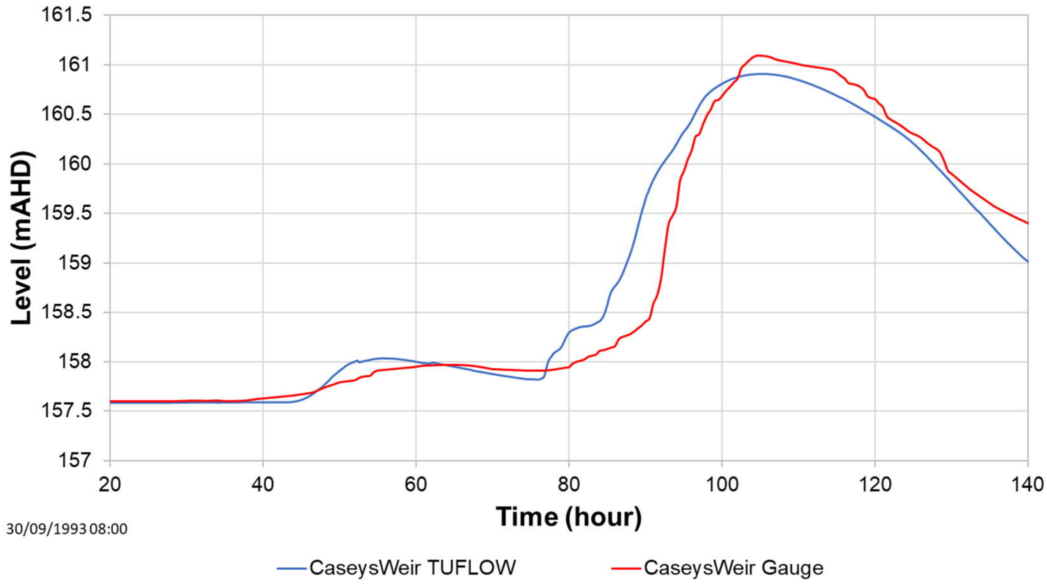


Figure 8-49 Height Validation Results 1993 Event – Caseys Weir Gauge

8.2.5.2 Flood Mark Validation

Whilst 1305 flood marks are available for the 1993 event, close inspection of the flood marks raised concern over the reliability of many of the recorded flood marks. Many locations were identified where significant variance was found within a local area. An example of this is demonstrated in Benalla where there is nearly 700 mm of vertical differences in the flood marks as shown in Figure 8-50.

During the study an assessment of the VFD flood marks was performed by the GBCMA (email from GBCMA dated 21/02/2020, which included GIS files and attached notes summarising the approach). The spatial and vertical accuracy was reviewed using GIS layers (e.g. aerial imagery, cadastre, LiDAR, etc) against the original surveyor's sketch drawings and field photography. The majority of the flood marks were spatially repositioned with some elevations altered.

Figure 8-52 presents the modelled flood depths along with a spatial comparison of the modelled and observed flood level at the flood marks. It should be noted that due to the number of flood level marks there are numerous instances of marks of one colour completely obscuring marks of another colour beneath. Because of this Figure 8-52 should be considered in tandem with Table 8-12 and Figure 8-51, which present the statistical summary of the comparison of observed and modelled 1993 flood levels.

Overall, a reasonable calibration was achieved, with 53% of the marks falling within the ± 200 mm range. In general, the model is lower than the observed flood makes, but there are some areas where the flood marks indicate the model is consistently high or low as follows:

- Upstream of Benalla there is cluster of high flood marks (see Figure 8-53). A number of adjustments were made to the model to try and improve these levels (refer to Section 8.3) but the routing in TUFLOW was different to RORB despite the modification trial. In the end the model was split at the Hume Highway so the errors did not translate to Benalla. In addition, there is a cluster of low flood marks to the east of Benalla.

- The model levels are low compared with the flood marks on the Goulburn River downstream of the confluence with the Yea River (see Figure 8-54). Adjustment to ‘n’ along this section was considered, but it was not possible to differentiate it from other areas where a good calibration was achieved in addition to the fact that this was a validation event. Being downstream of the Yea River confluence consideration was given to Yea River flows being too low. A review of the hydrology was undertaken and increasing the inflows could not be justified.
- Upstream of Seymour there is an area where the modelled results are low compared with the surveyed marks (see Figure 8-54). Upstream of this location there is good agreement. Given the model validation upstream and downstream it seems likely that the flood marks along this section of the river have not picked up the peak flood level or there is an issue with the survey datum for this cluster of flood marks.

Figure 8-50 Example inconsistent vertical levels of 1993 flood marks

Table 8-12 Statistical Summary of Comparison of Observed and Modelled Flood Levels - 1993 Flood Event

Data Source	Historic Flood Heights
No. of Points Interrogated	1087
Mean	-0.19
Median	-0.17
Standard Deviation	0.37
Lower Quartile	-0.31
Upper Quartile	-0.04

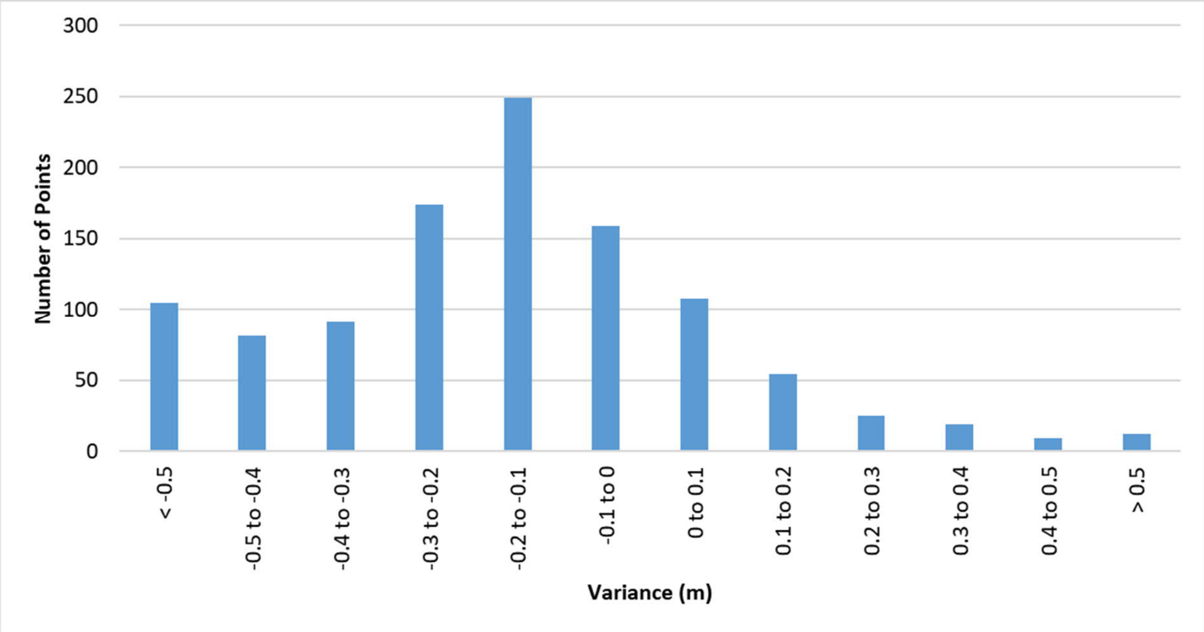


Figure 8-51 Statistical Summary of Comparison of Observed and Modelled Flood Levels - 1993 Flood Event

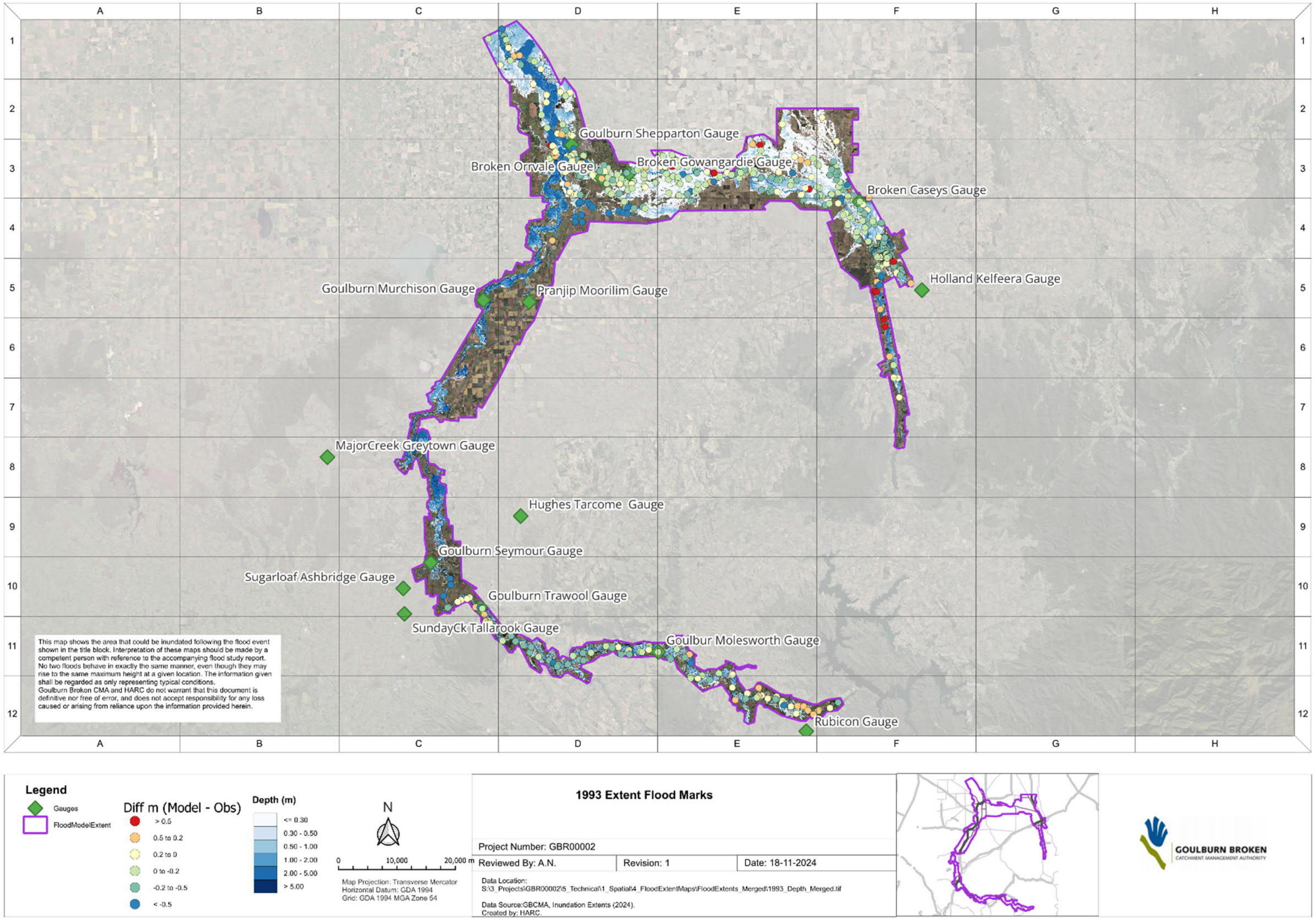


Figure 8-52 Hydraulic Model Calibration Results 1993 Event

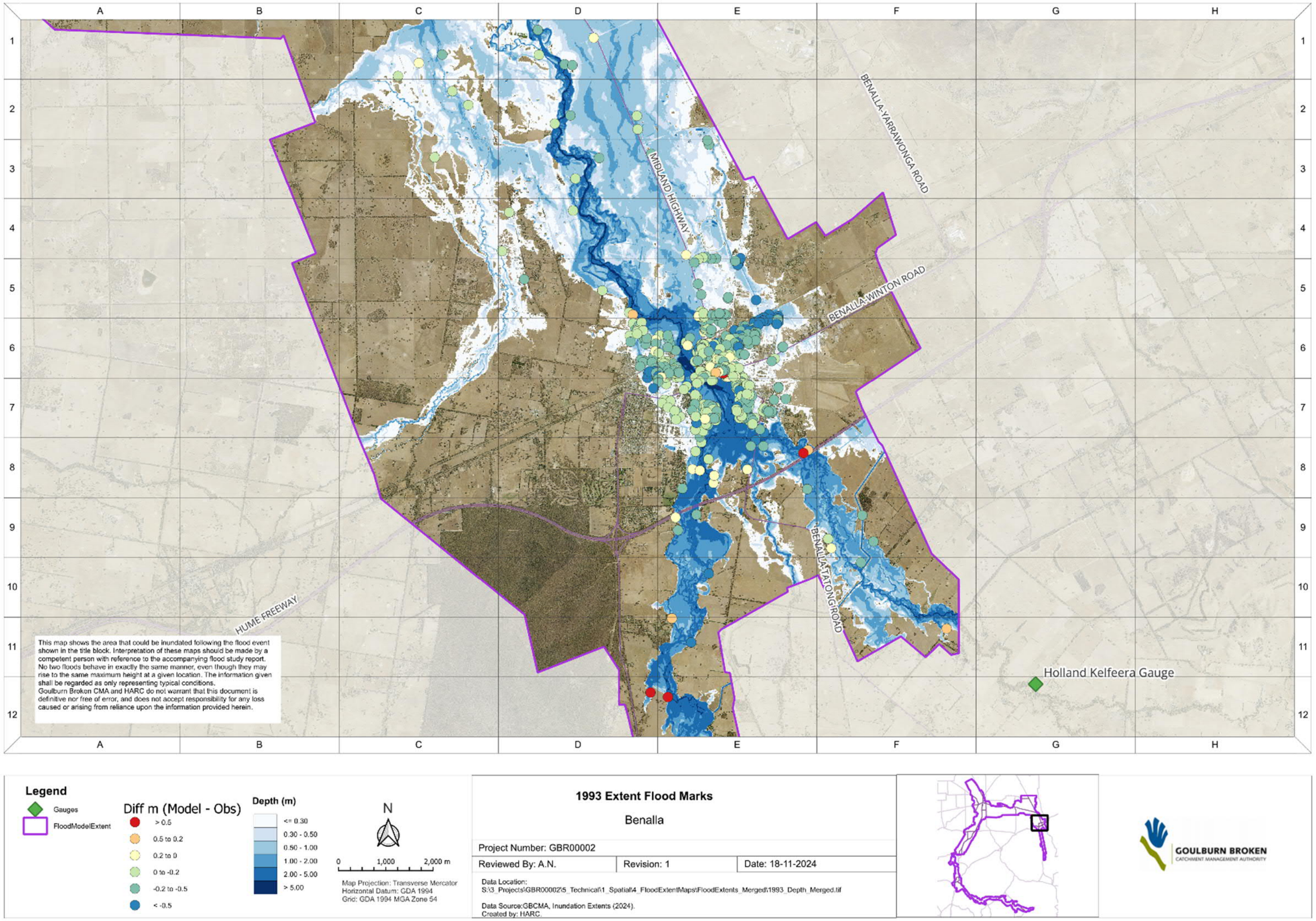


Figure 8-53 Hydraulic Model Validation Results 1993 Event (Benalla)

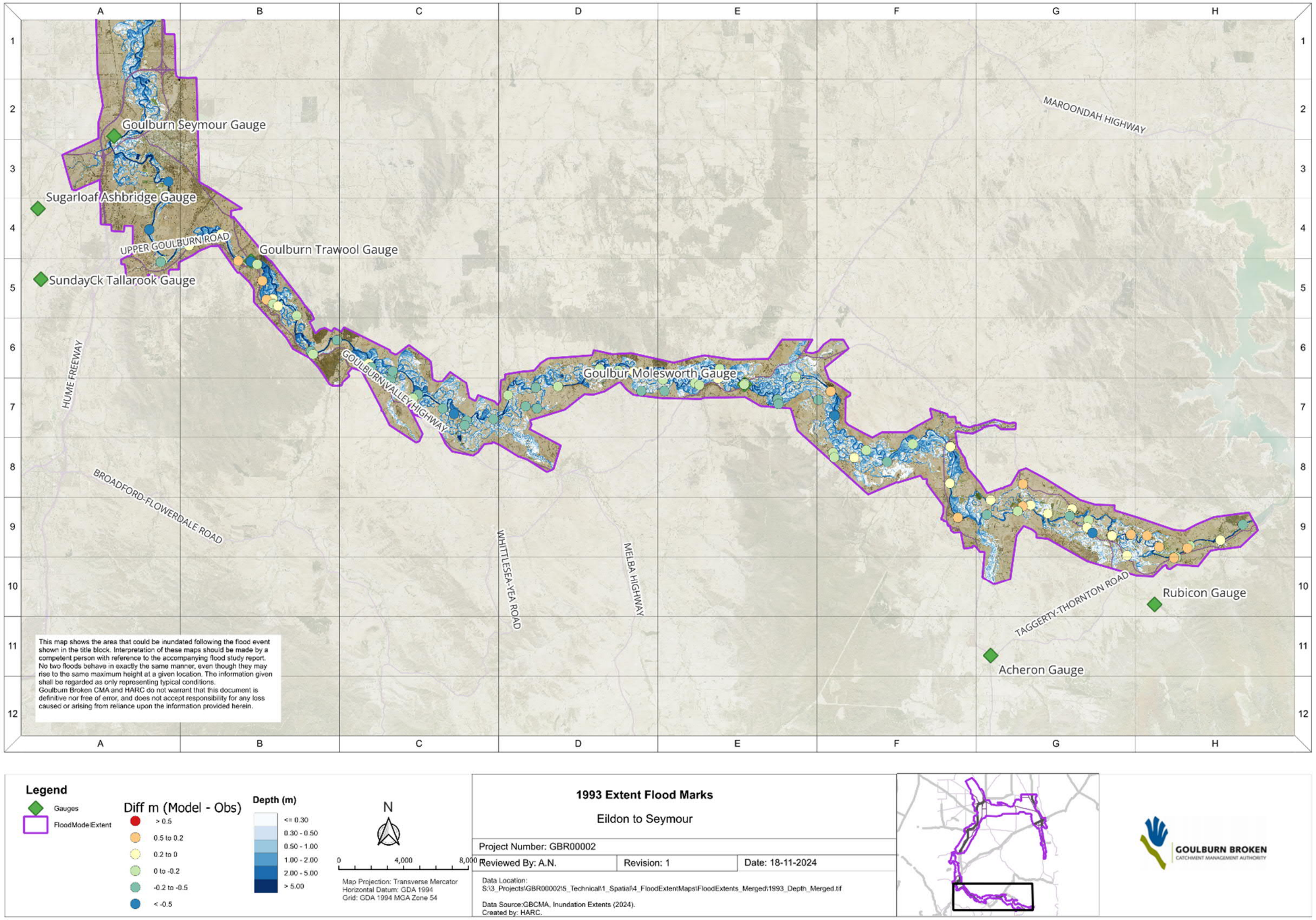


Figure 8-54 Hydraulic Model Validation Results 1993 Event (Eildon to Seymour)

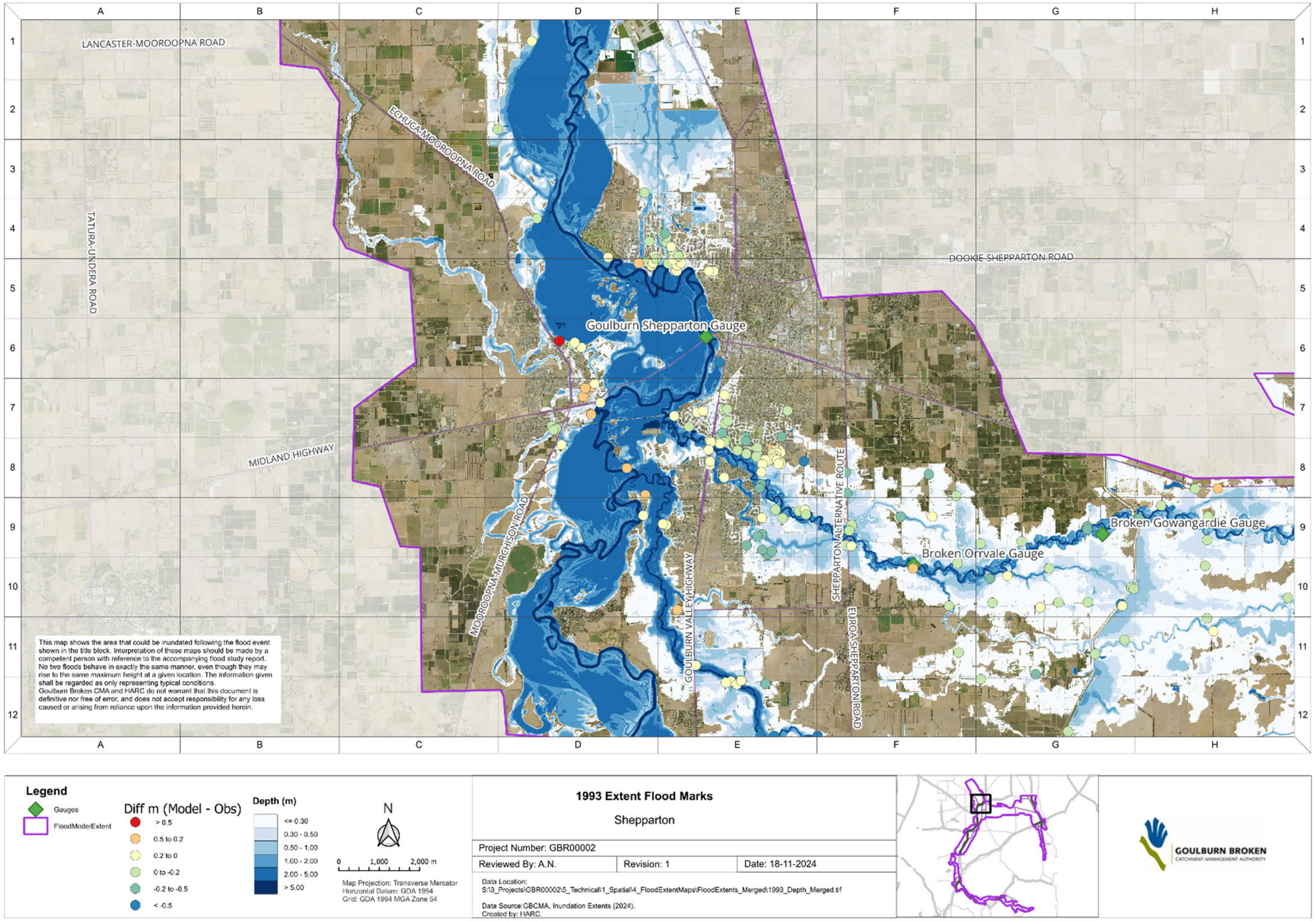


Figure 8-55 Hydraulic Model Validation Results 1993 Event (Shepparton)

8.3 Summary of Calibration / Validation Testing Regime

The calibration / validation process was an extensive one, with over 150 calibration / validation runs performed and many model parameters repeatedly altered in order to achieve an acceptable calibration. Table 8-13 summarises the parameters changed and the outcomes. It should be noted that the grid size of the calibration testing runs was increased to quickly test many of the changes. Periodically the model was run at the intended final resolution to understand the sensitivity to grid size.

Table 8-13 Calibration Parameter Alterations

Parameter Altered	How Altered	Outcome
Manning's n – Farm Zone	Changed between 0.04 – 0.06	Lower farm zone Manning's n value resulted in lower levels along the upper portions of the Goulburn and Broken Rivers but increased levels below Shepparton. Final value: 0.05
Manning's n – River channel	Changed between 0.02 – 0.035 River channel divided into straight or constructed channels (lower Manning's n) and 'natural' channel (higher Manning's n)	Changing the river Manning's n value had little effect on the Broken River. Lower channel Manning's n values resulted in lower levels along the Goulburn River, with the greatest effects found at the Goulburn Weir. Levels along both the Goulburn and Broken Rivers are lower, with the greatest effects found on the Goulburn River. Having higher river Manning's n values and lower straight channel values had the effect of lowering the water levels in the straight channels relative to the 'natural' channels. This was important for mitigating the high levels at the top of the Goulburn River. Natural channel final value: 0.025 Straight channel final value: 0.018
Manning's n – Vegetation	Scattered vegetation changed between 0.06 – 0.08 Moderate vegetation changed between 0.08 – 0.10 Dense vegetation changed between 0.10 – 0.12	Lowering the vegetation Manning's n value resulted in lower levels. However, it did not have as much effect as modifying the farm zone or channel values. Final values: Scattered: 0.07 Moderate: 0.08 Dense: 0.10
Topography – depth of channel reinforcing	The supplied Water Technology reinforcing came in several depths. The reinforcing linking the sections supplied by Water Technology was adjusted to match these depths.	Lowering the bed level of the channels lowers the water level. This has significantly more effect on sections of the river that are confined, such as the Goulburn River above Seymour. Final values: Deepest reinforcing used

Parameter Altered	How Altered	Outcome
Topography – shape of channel reinforcing	The cross section available in Seymour were used to shape the river bed through Seymour	In general, the shape of the river was made wider. This confirmed that the bathymetry had a reasonable impact on the flood levels. Final: Adopted adjusted bathymetry through Seymour.
Topography – scour	The upper portions of both the Goulburn and Broken Rivers were tested for the effects of 1-2 m of scour during the flood events.	There was no particular knowledge of scouring during the 2022 or 1993 event, but testing was done to see if this might explain the high model flood level in the upper reaches. Lowering the bed level on the upper reaches of the Goulburn and Broken Rivers did lower the water levels, but not by an appreciable amount and did not resolve the calibration issues. Final: no scour
Topography – Goulburn Weir depth	Level of the base of Goulburn Weir changed between ~124 m AHD (DEM supplied) - ~111 m AHD. The 111 m AHD was estimated from plans of the weir.	Lowering the levels in the Goulburn Weir allowed greater depths in the weir, which are more representative of reality. Additionally, lower levels allowed the weir structure to receive appropriate inflows and function as intended. Final: ~111 m AHD
Timing of Inflow Hydrographs	Pranjip, Castle and Seven Creek inflows delayed by 12-24 hours	Delaying the inflows from Pranjip, Castle and Seven Creeks, which all join the Goulburn upstream of Shepparton, in such a manner so as to allow them to peak around the same time as the flood peaked in Shepparton increased the modelled flood level in Shepparton. Final: Seven Creeks delayed by 24 hours.
Goulburn Weir – Operating Procedures	Opening time changed between 1 – 30 minutes Offtake canals offtake levels changed between 123.0 – 124.1 m AHD	Shorter weir opening times resulted in the Goulburn Weir rapidly opening and closing to keep the water level in the upstream dam at FSL. This created bursts of flow out of the weir that caused instabilities in the model. When the opening time was slowed, the flow out of the dam was smoother and model health improved. The offtake canals initially removed water from the dam if the water levels were above 123 m AHD, but this resulted in the water levels being drawn down considerably by the time the flood peak arrived. This artificially reduced the effect of the flood below the dam by lowering levels and altered the timing of the peak. Higher offtake levels improved the calibration of levels downstream of Goulburn Weir. Final values: Opening time: 30 minutes Offtake canals offtake levels: 124.1 m AHD
Goulburn Weir – 1d2d boundary	SX line changed in length and location	When the SX line was only in front of the weir, there was a rapid draw down to the weir. It was trialled to extend the SX line across the majority of the dam, this eliminated the drawdown entirely, leaving the water level of the dam constant. A midpoint between the two eliminated the

Parameter Altered	How Altered	Outcome
		drawdown but allowed some hydraulic gradient across the dam. Final: SX extended ~500 m upstream of the weir
Sub-grid sampling*	SGS enabled	SGS lowers levels across the model with levels along the Goulburn River more affected more than on the Broken. Final: SGS enabled
Viscosity Formulation*	Smagorinski changed to Wu	The Wu formulation increased levels across the model by a significant amount. This had a greater effect on the confined flow paths of the Goulburn than along the Broken River. Final: Wu viscosity formulation
Smagorinski coefficients	Smagorinksi coefficients: (variable, constant)	Extensive testing was done on the relationships between grid size, Manning's n and the Smagorinski coefficients. This testing was performed using the Trawool rating curve model. The initial rating curve found a good agreement with a grid size of 5 m and a channel manning's of 0.035. However, when the cell size was increased to 10 m, a manning's 'n' value of 0.027 was required to find an agreement. When the cell size was increased to 15 m (the norm for the calibration runs) a manning's value of less than 0.010 was required to reach agreement. As this is not a plausible manning's value for a natural channel, the Smagorinksi coefficients were adjusted instead. Smagorinski coefficients of (0.18, 0.05) with a channel manning's of 0.027 on a 15 m grid were found to produce a good agreement. Lowering the Smagorinski coefficient lowered the water level minimally across the model. The findings from this testing became redundant with the introduction of the Wu formulation in the current TUFLOW release. Changed between (0.5, 0.05) – (0.15, 0.05)
Wu coefficients*	Wu coefficients: (2D, 3D) Changed between (7, 0) – (3, 0)	Lower Wu coefficients than the default of (7,0) lowered levels across the model. Final value: (7,0)
Model split into sections	The model was split into three sections. On the Broken River downstream of Lake Nillacootie to upstream of Benalla and Benalla to Orvale gauge. On the Goulburn River downstream of Eildon to Loch Garry	For the validation event it was shown that the TUFLOW model was routing flow a lot quicker than RORB in the upper reaches of the Broken River. Various Manning's values were tried to slow the flow down but still the routing could not be matched. As a result, the model was split upstream of Benalla and the flow reset to prevent the errors transferring to Benalla.

Parameter Altered	How Altered	Outcome
		The model was split at Orrvale to test the impact on placing the recorded flow at Orrvale on Shepparton. This was done as there is not enough LIDAR to the north of Broken River to capture the full extent and breakouts. Final: Three hydraulic models adopted.

*Only available in TUFLOW 2020-01-AA or later

9. Conclusions

This report describes the development, calibration, verification and use of a RORB hydrologic model and a TUFLOW hydraulic model covering the entire Goulburn River and Broken River catchment to Loch Garry.

Some of the notable outcomes from the hydrology section of this study include:

- Development of a detailed RORB model covering the entire catchment upstream of Loch Garry, a total area of close to 16,000 km^2, and comprising 741 individual sub-areas and 15 sub-catchments (31 including areas upstream of Lake Eildon).
- Incorporation of a range of special storages and other controls in to the RORB model including Lake Eildon (and flood operations of its spillway gates), Lake Nillahcootie, Goulburn Weir, Casey's Weir, Lake Mokoan/Winton Wetland and cross-catchment flows from the Broken River to Broken Creek.
- Calibration of this model to recorded streamflow hydrographs at numerous locations within the catchment for three major historic flood events. Generally speaking, the quality of the calibration results are good given the complexity of the catchment.
- Derivation of hydrologic inputs including design rainfall depths, areal reduction factors and sets of complete rainfall space-time patterns for five distinct sub-catchments within the study area.
- Verification of the model to at-site gauged flood frequency estimates at five locations. Like the calibration, the verification results obtained are remarkable given the length of record and nature of the catchment.

On the whole, the results obtained from the hydrological modelling component of this study are robust, defensible and internally consistent and suitable to developing design flow estimates.

Some of the notable outcomes from the hydrauilc section of this study include:

- A TUFLOW 1D/2D model of the Goulburn Broken River system has been developed. The model was calibrated to the 2022 event and validated to the 1993 event.
- Overall, a good calibration to both gauges and flood marks was achieved for the 2022 event considering the size and complexity of the catchment, both hydrologically and hydraulically.
- For the validation event a reasonable calibration is achieved with the exceptions noted in the report. An exhaustive calibration / validation process was undertaken in an attempt to match flood level and marks which provided modest improvements.

On the whole, the results obtained from the hydraulic modelling component of this study are suitable to developing design flood extents.

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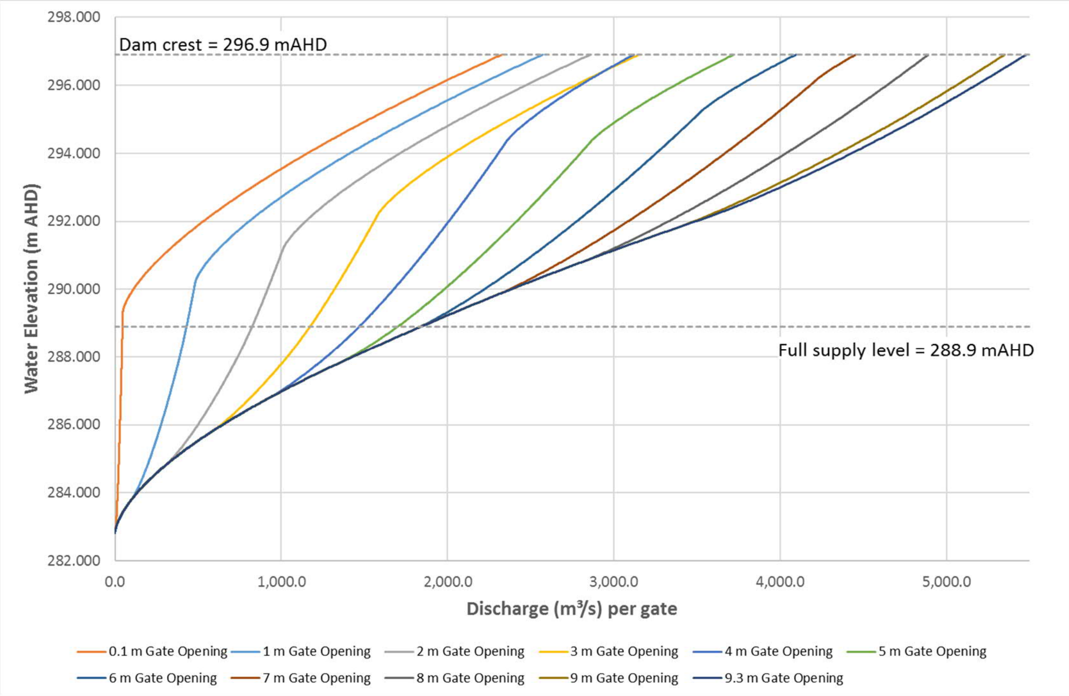
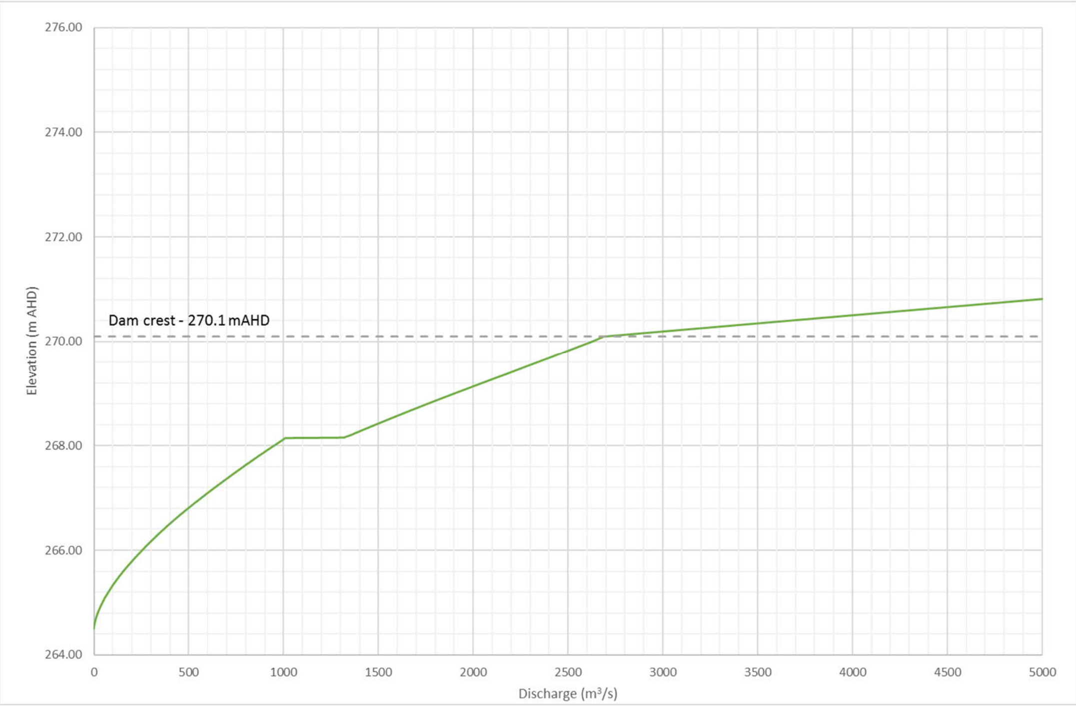
Appendix A Dam and weir data



■ Figure A-1 Lake Eildon stage-discharge curve



■ Figure A-2 Lake Nillahcootie stage-discharge curve

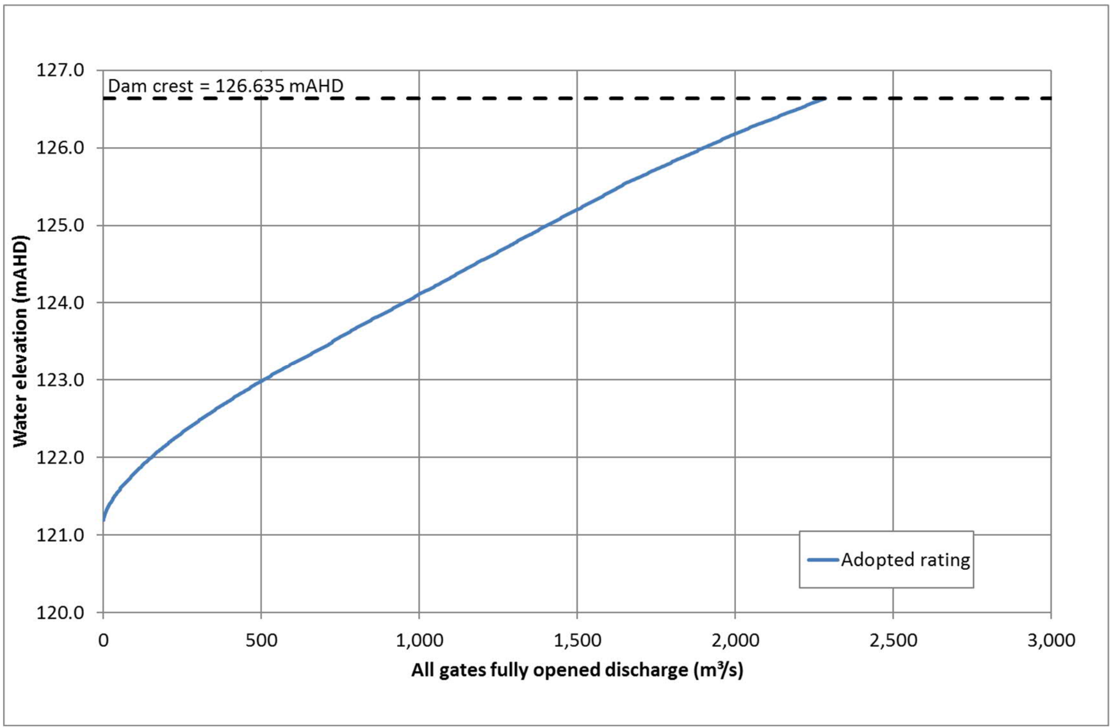


Figure A-3 Goulburn Weir stage-discharge curve

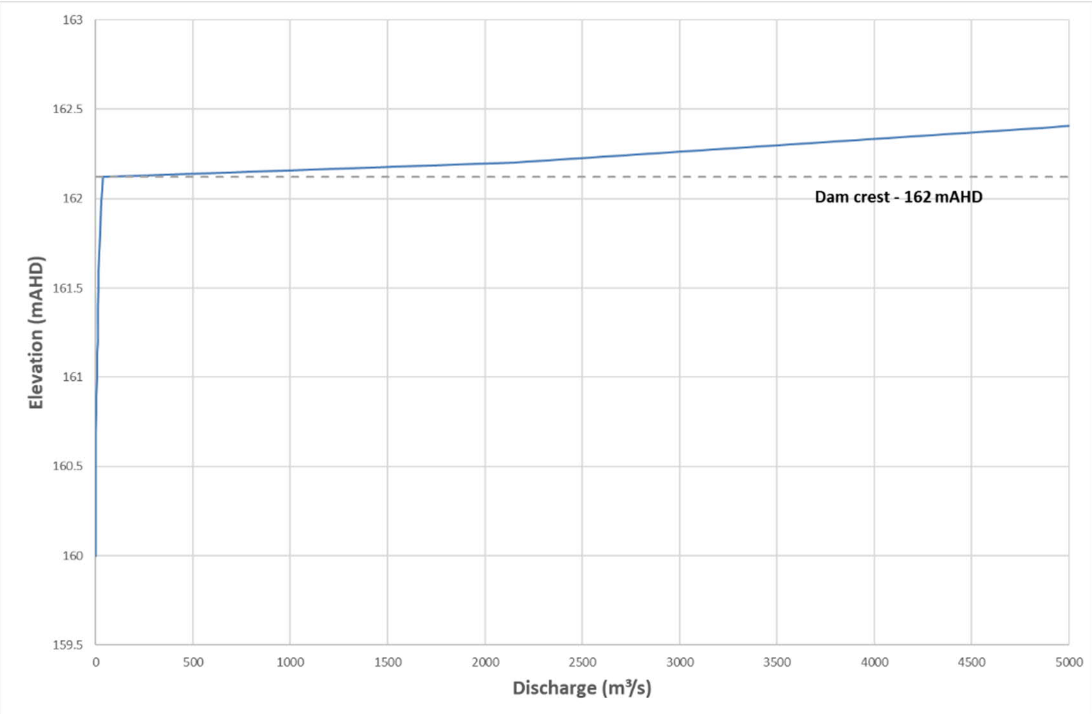
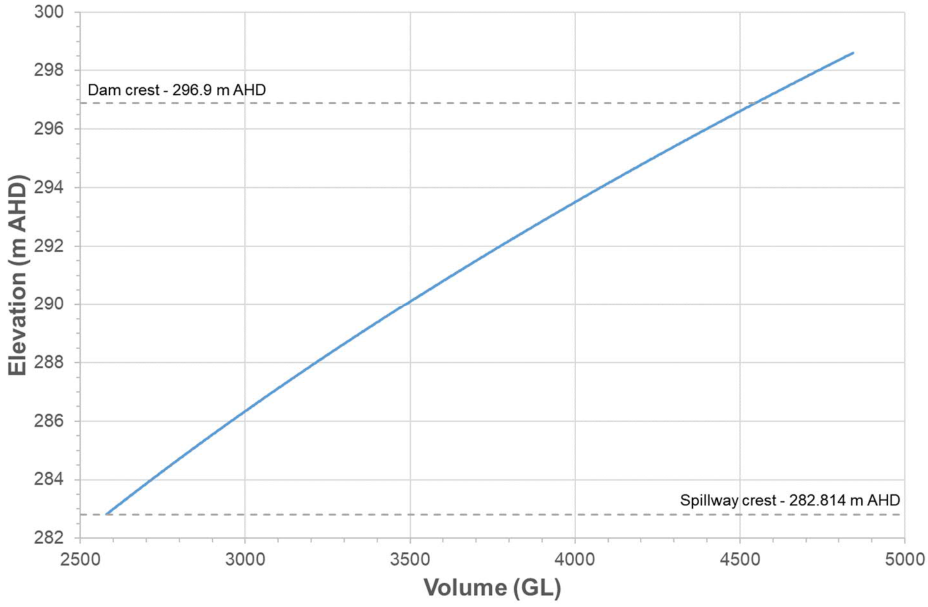
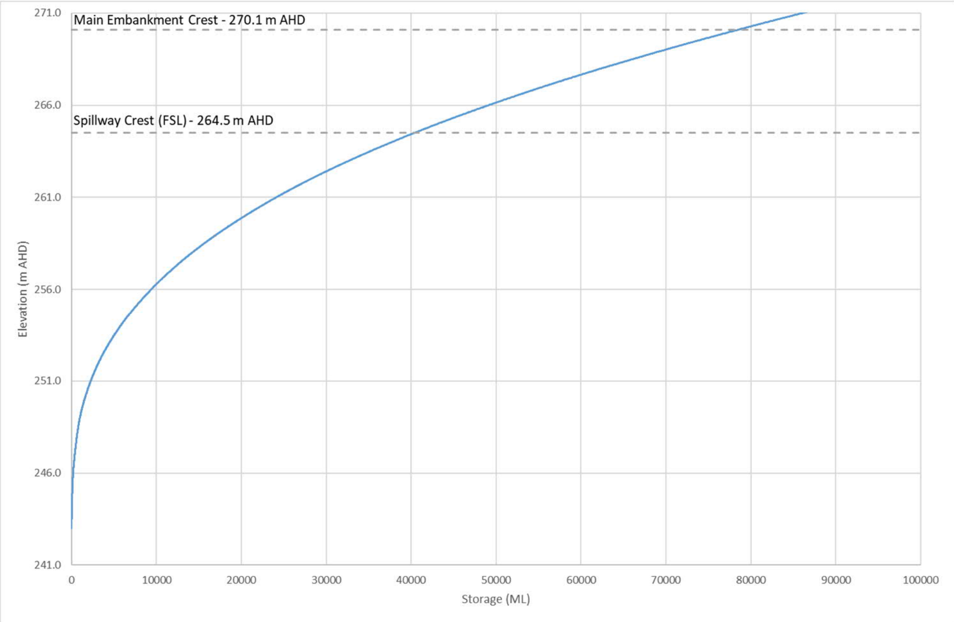


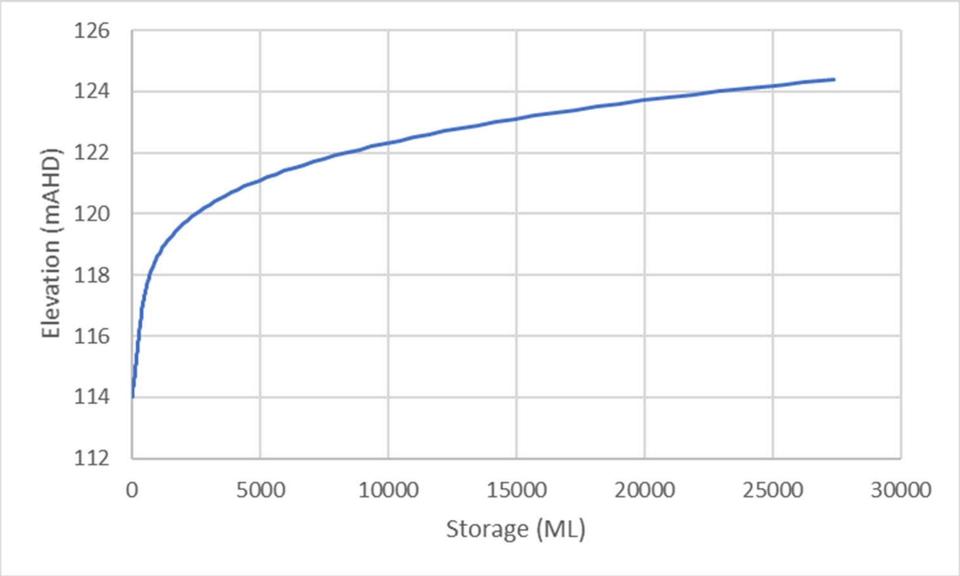
Figure A-4 Lake Mokoan stage-discharge curve



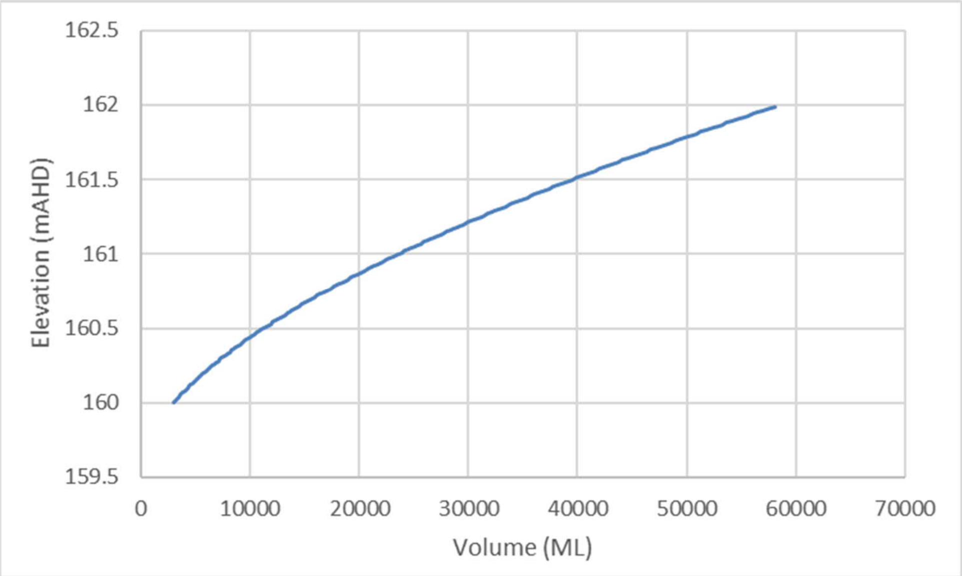
- Figure A-5 Eildon stage-storage curve



- Figure A-6 Nillahcootie stage-storage curve



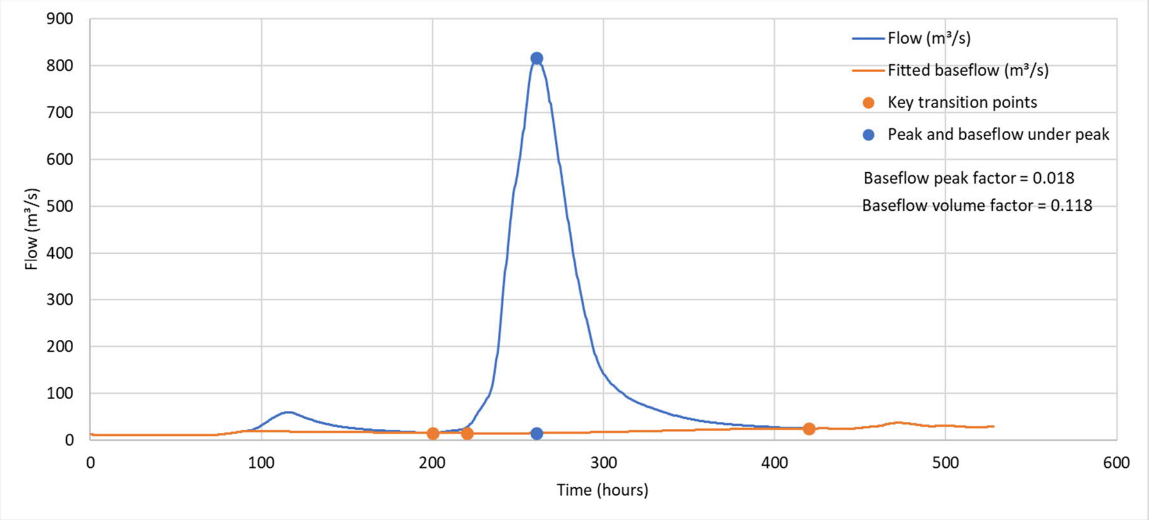
■ Figure A-7 Goulburn Weir stage-storage curve



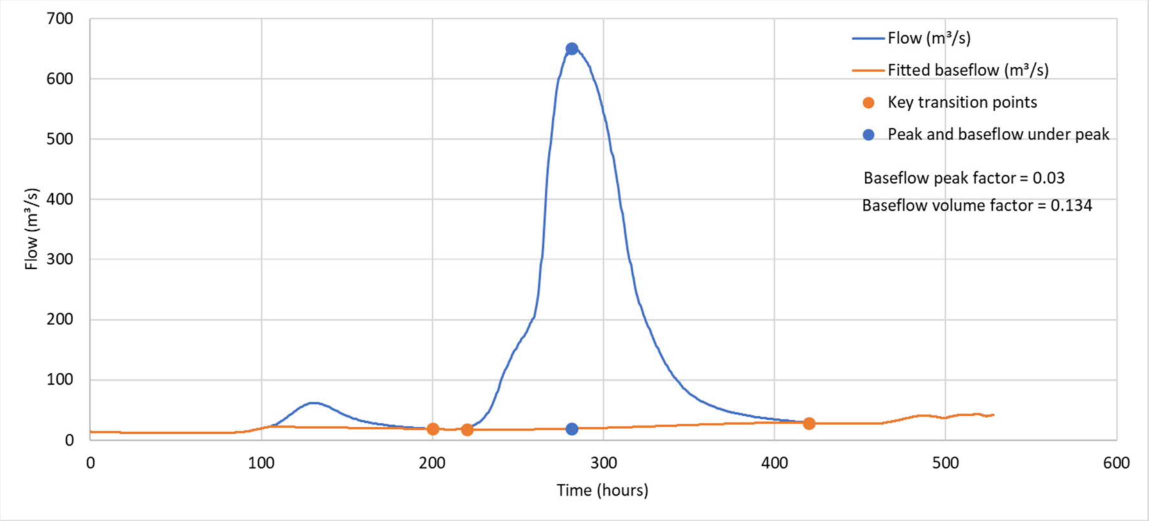
■ Figure A-8 Lake Mokoan stage-storage curve

Appendix B Baseflow separation for calibration

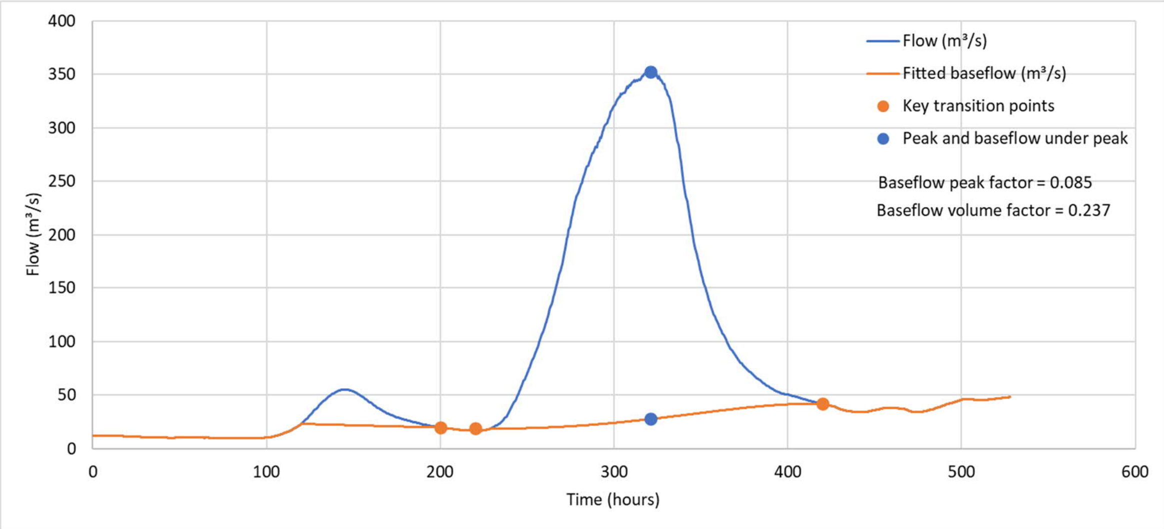
October 2022 event



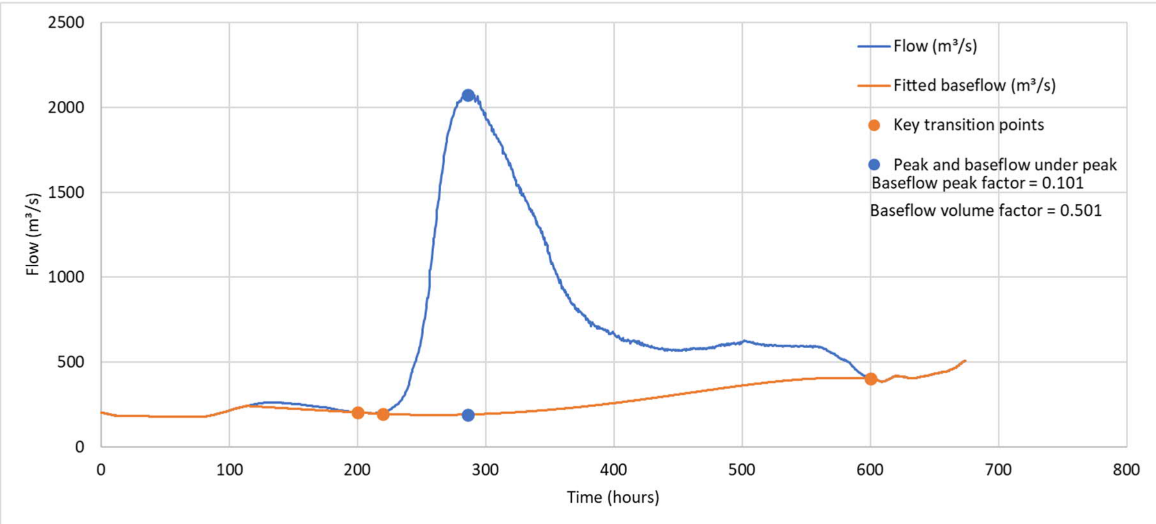
▪ Figure B-1 Broken River at Casey's Weir



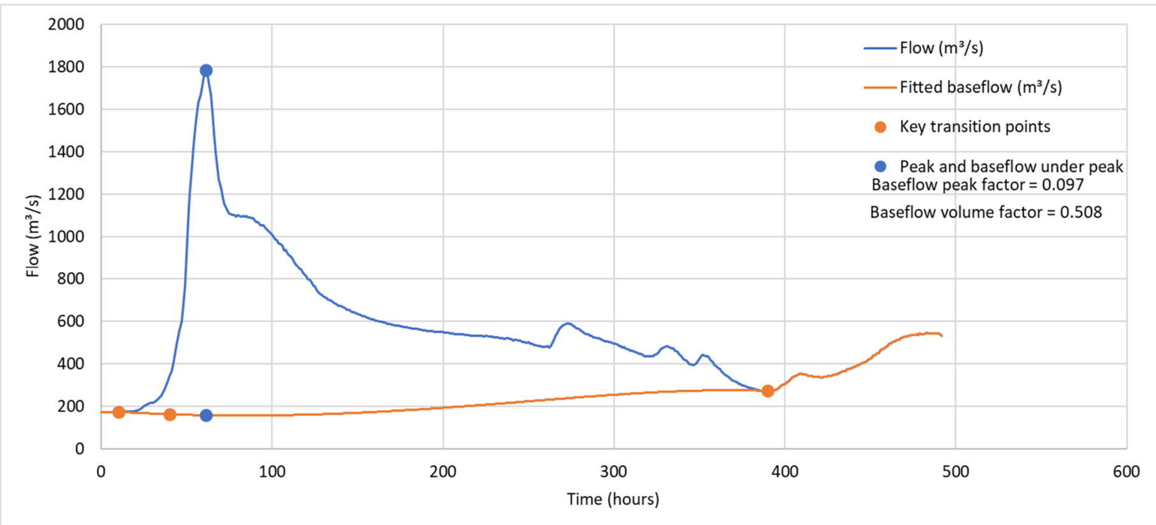
▪ Figure B-2 Broken River at Gowangardie



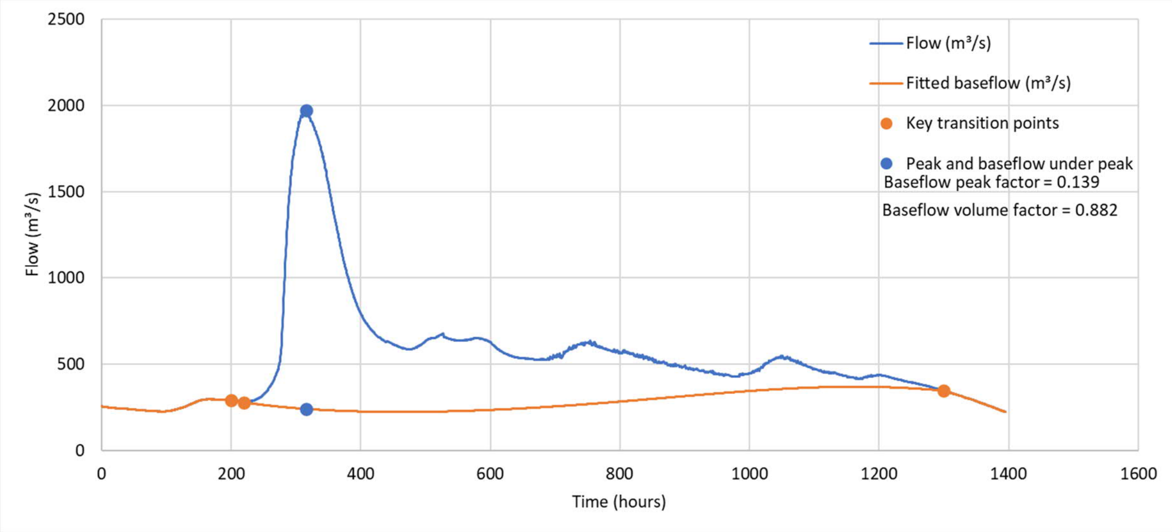
■ Figure B-3 Broken River at Orrvale



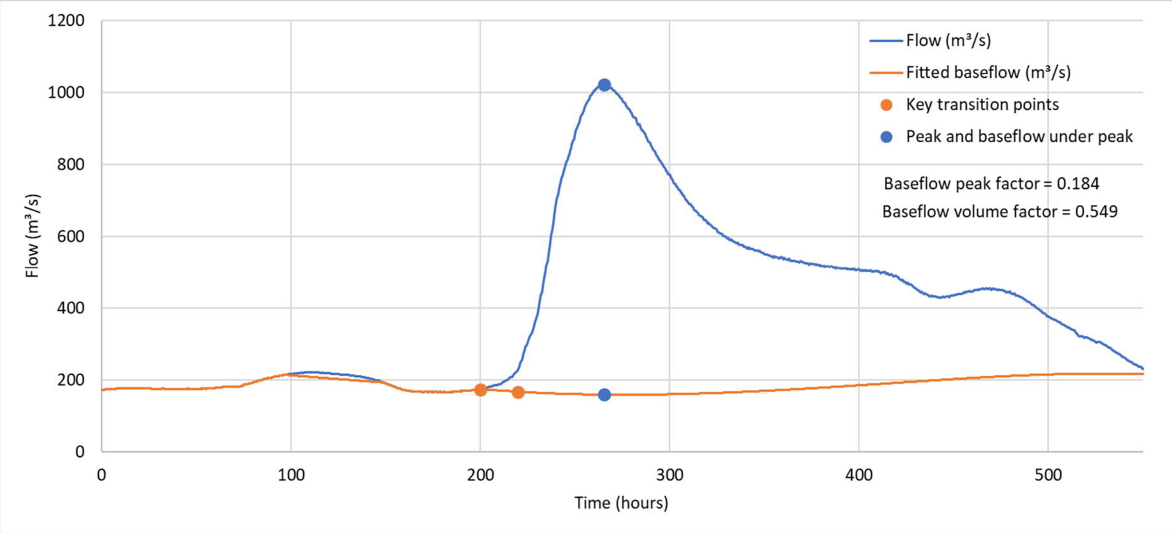
■ Figure B-4 Goulburn River at Murchison



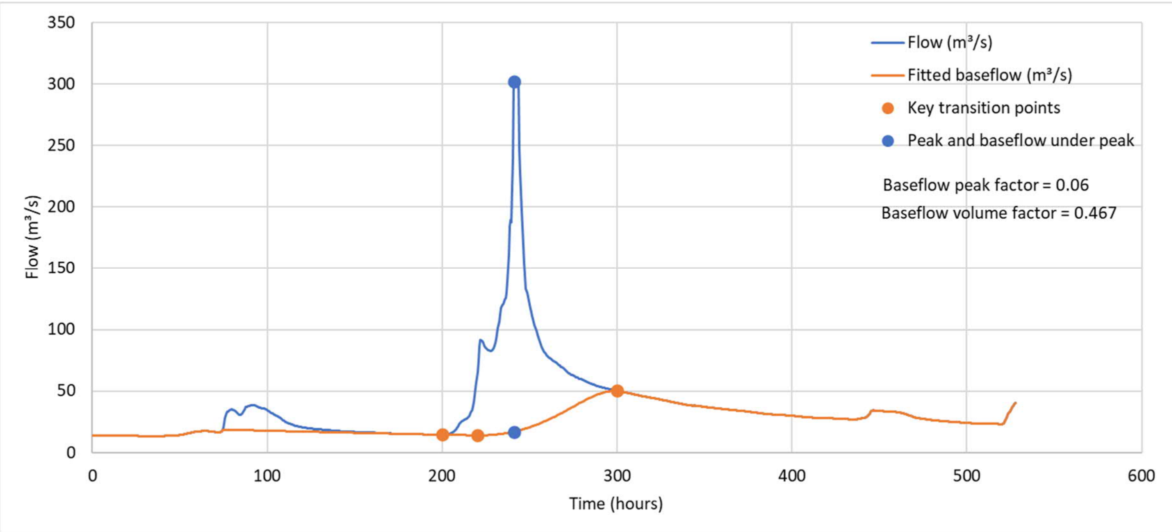
■ Figure B-5 Goulburn River at Seymour



■ Figure B-6 Goulburn River at Shepparton



■ Figure B-7 Goulburn River at Trawool



■ Figure B-8 Acheron River at Taggerty

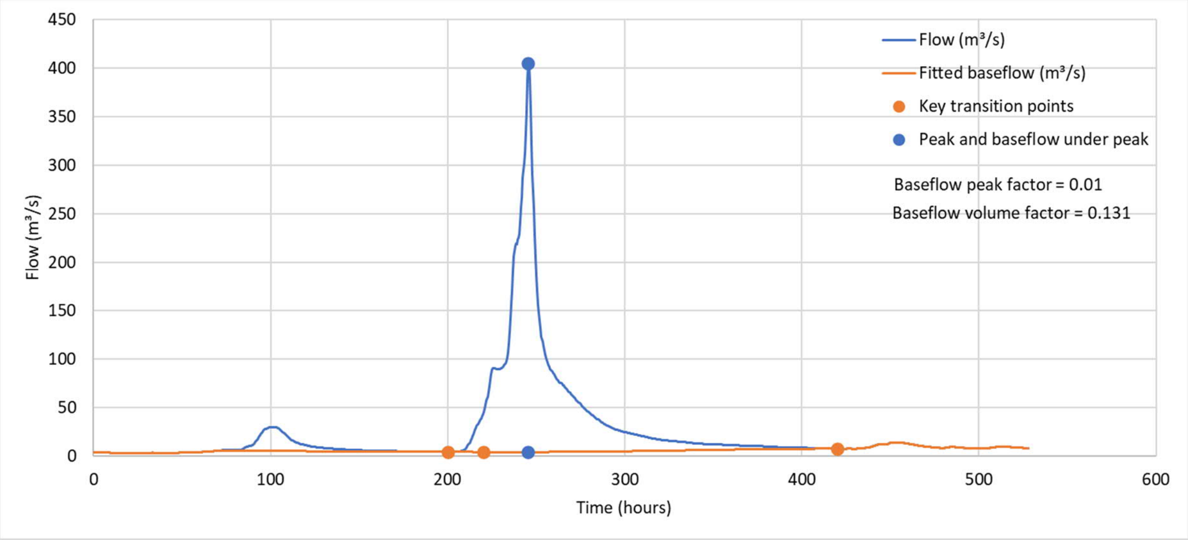


Figure B-9 Holland Creek at Kelfeera

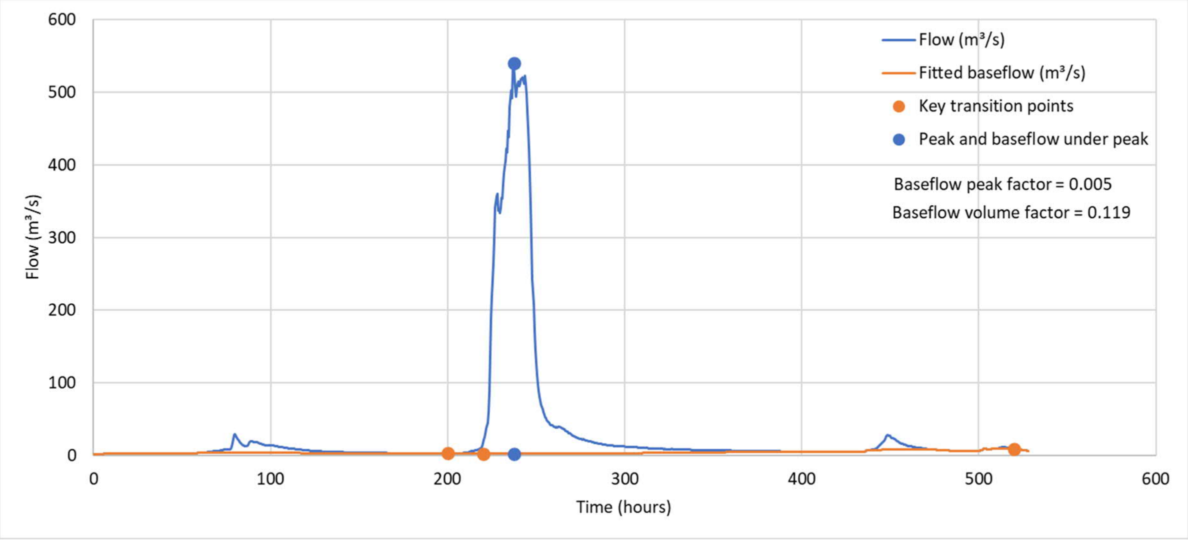


Figure B-10 Hughes Creek at Tarcombe

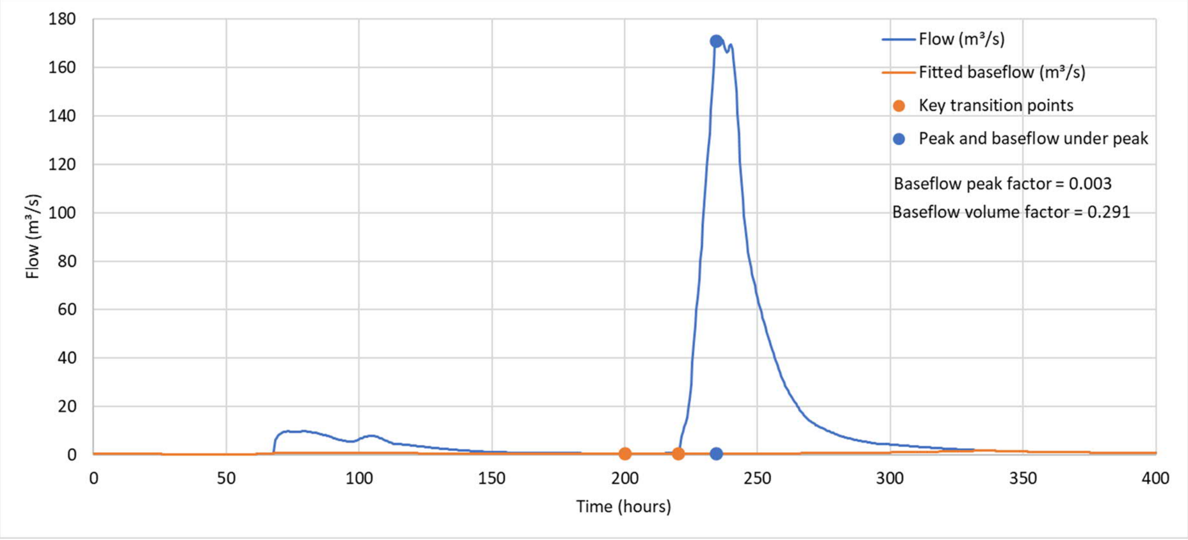
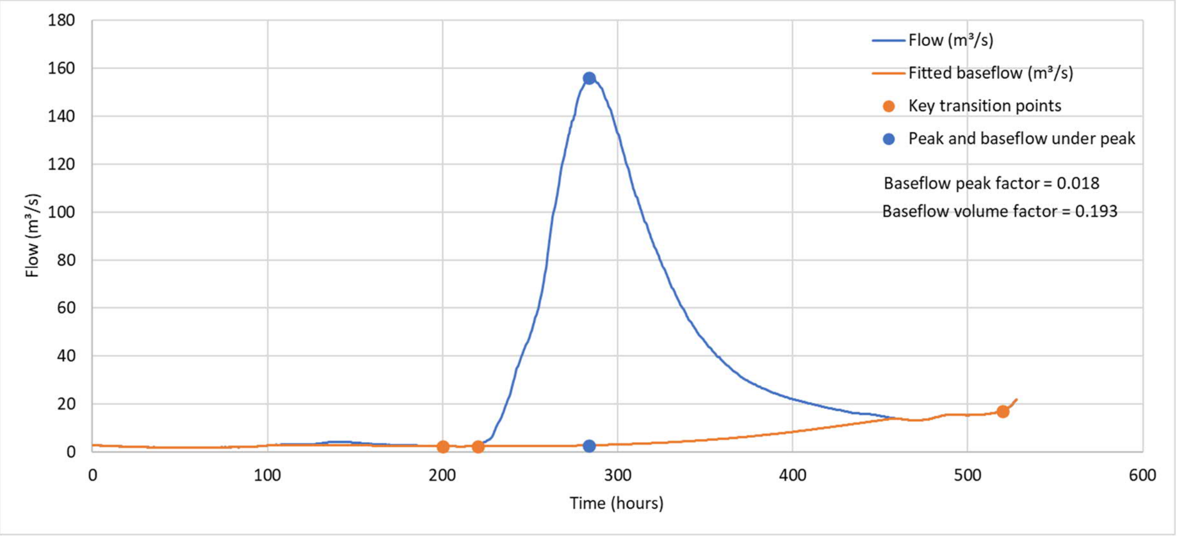
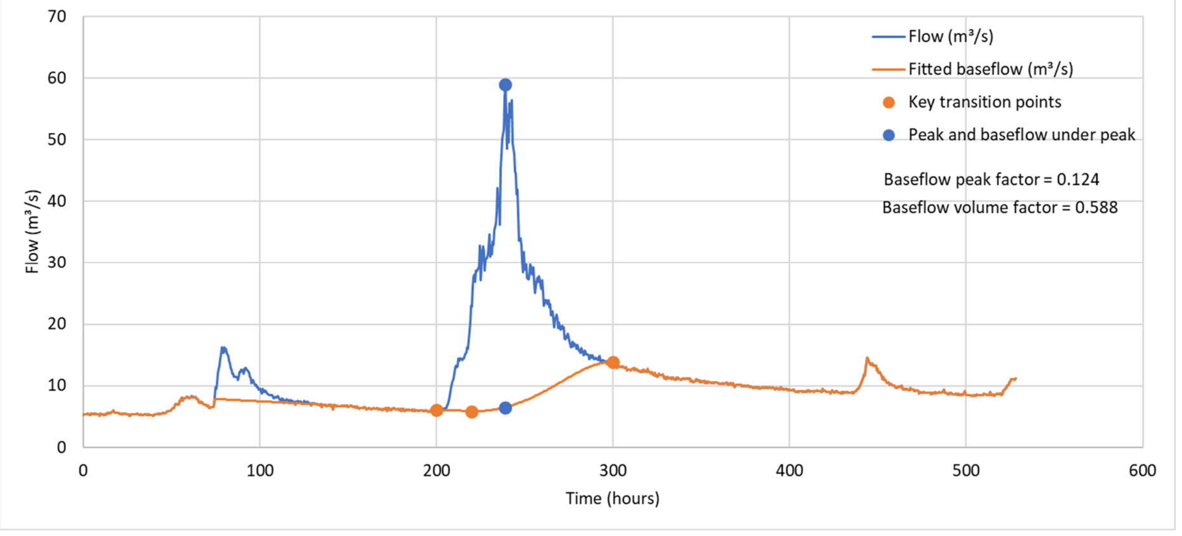


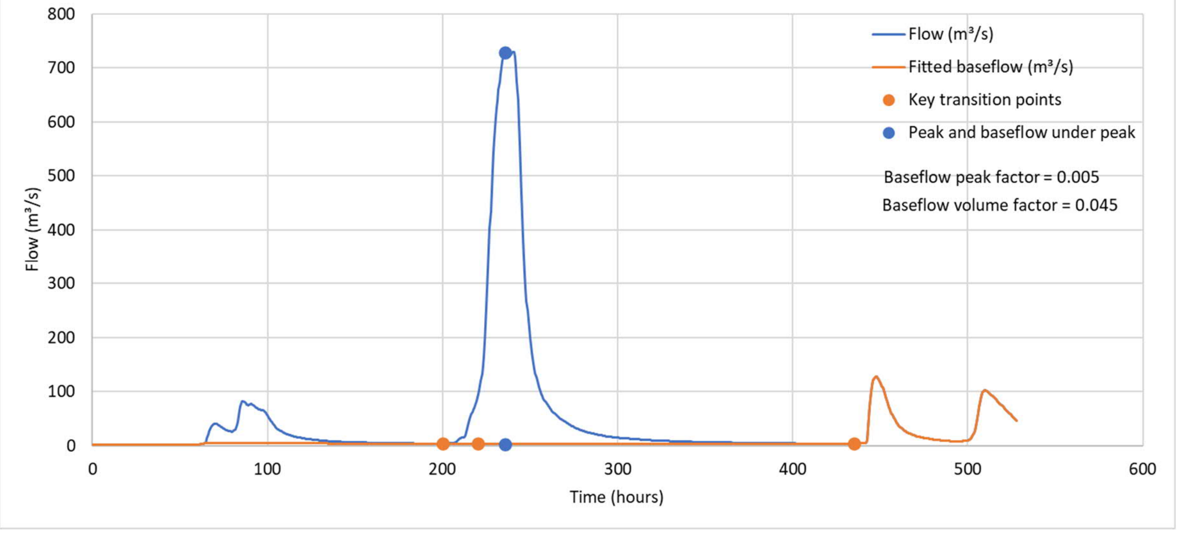
Figure B-11 Major Creek at Greytown



■ Figure B-12 Pranjip Creek at Moorilim



■ Figure B-13 Rubicon River at Rubicon



■ Figure B-14 Sugarloaf Creek at Ashbridge

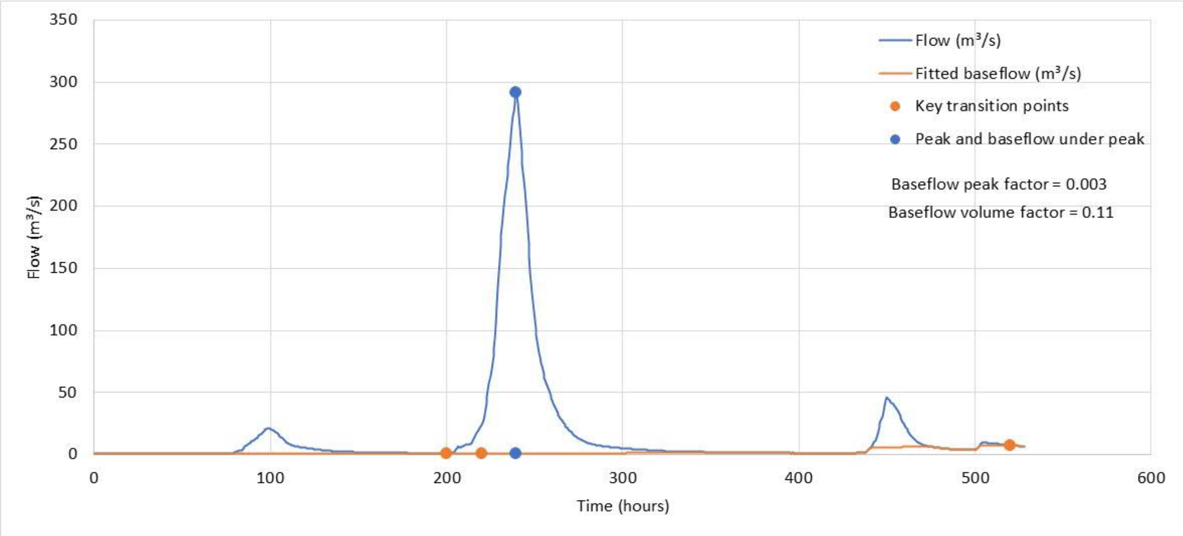


Figure B-15 Sunday Creek at Tallarook

October 1993 event

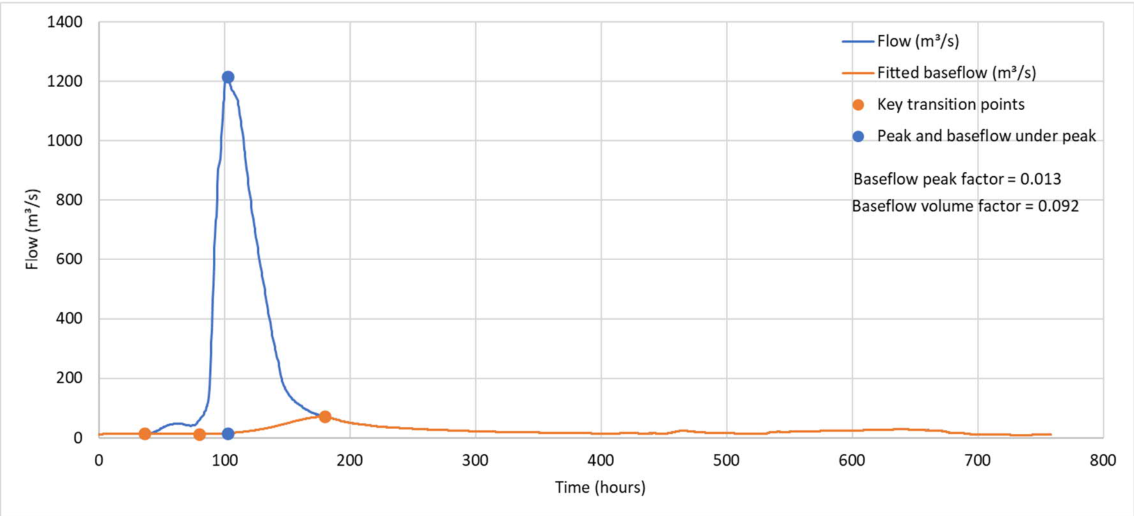


Figure B-16 Broken River at Casey's Weir

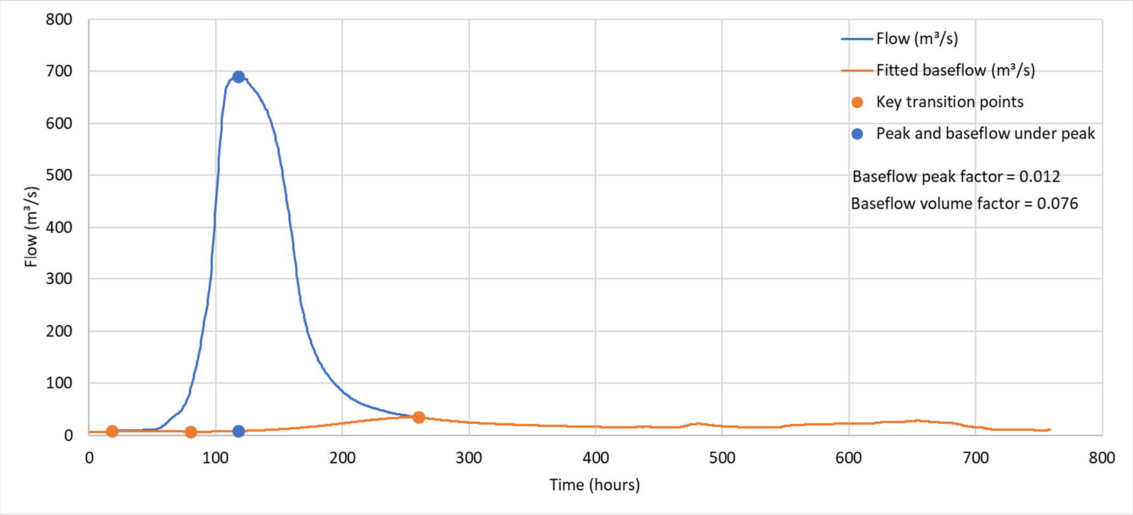


Figure B-17 Broken River at Gowangardie

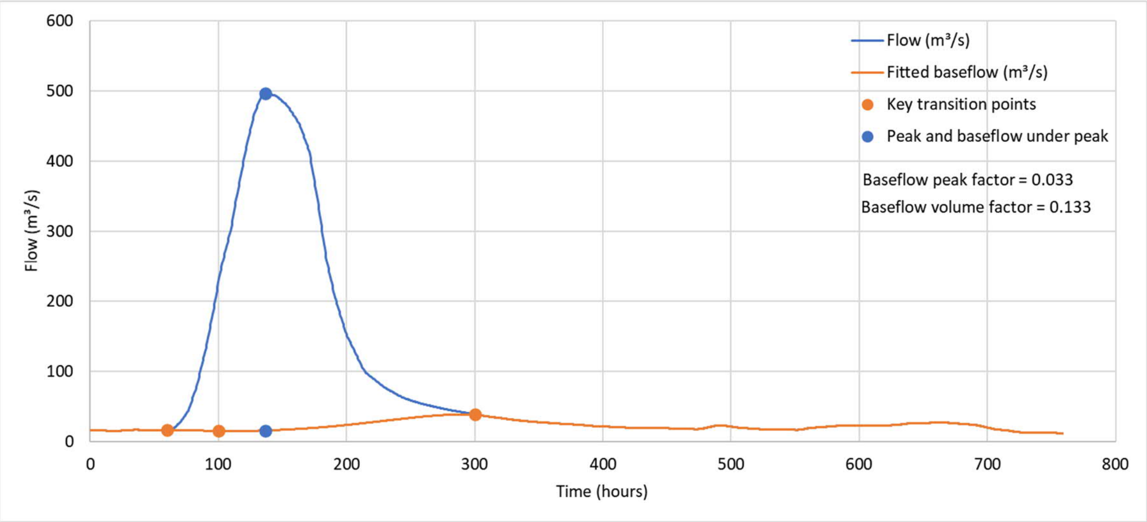


Figure B-18 Broken River at Orrvale

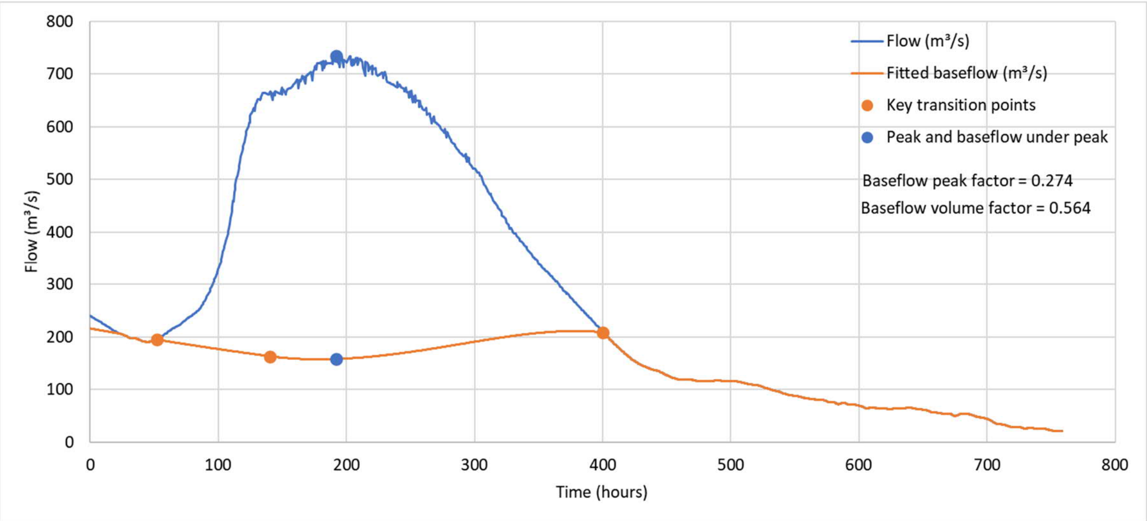
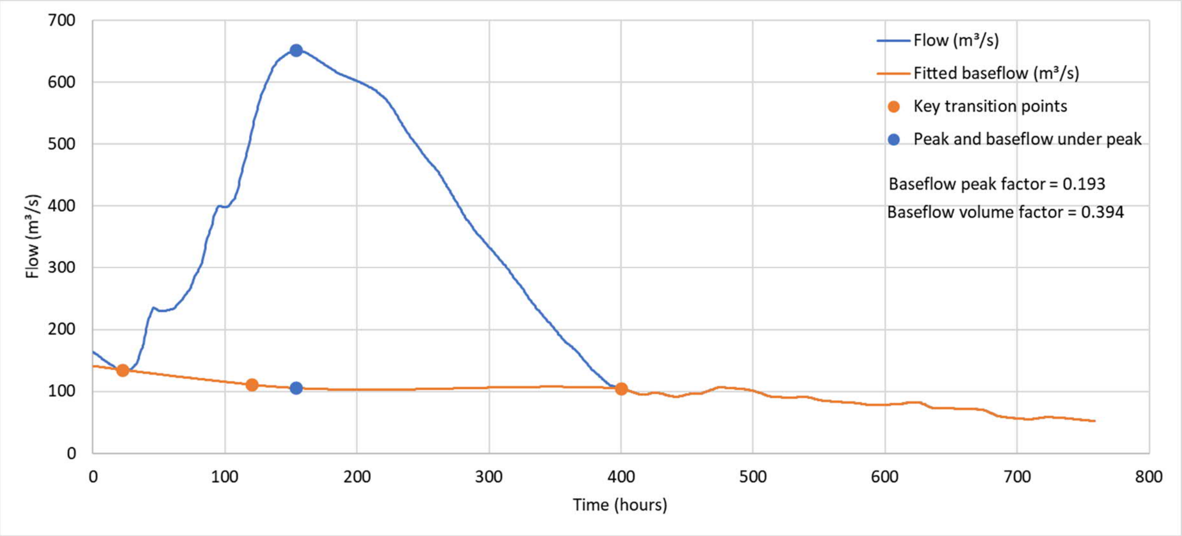
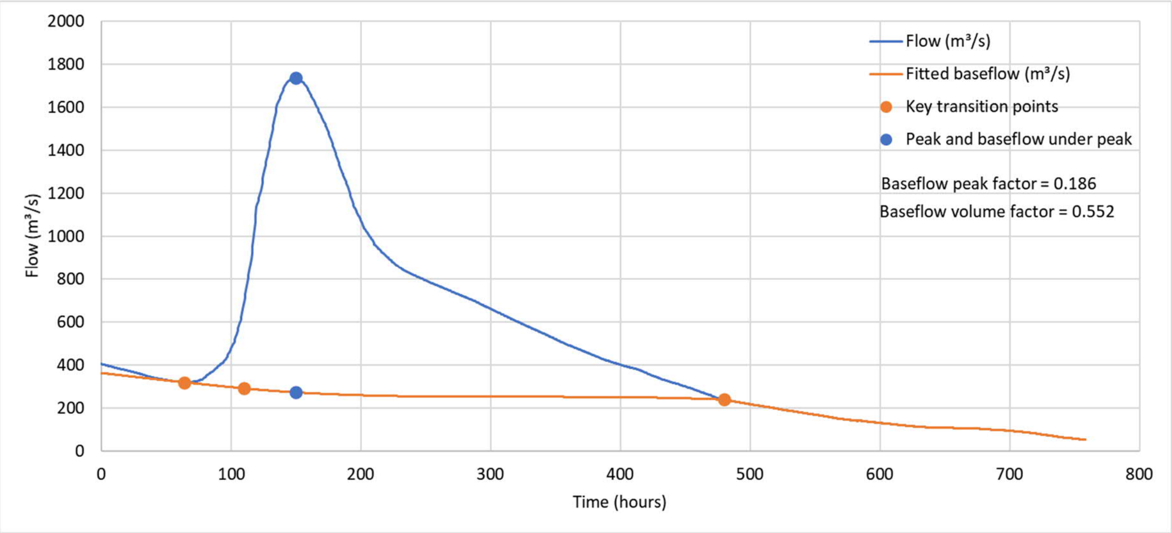


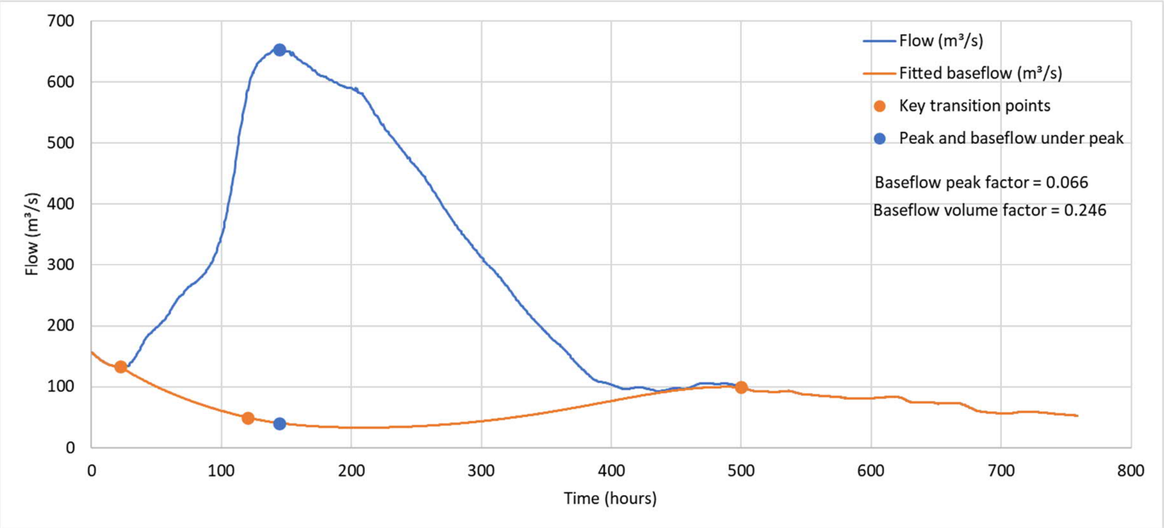
Figure B-19 Goulburn River at Murchison



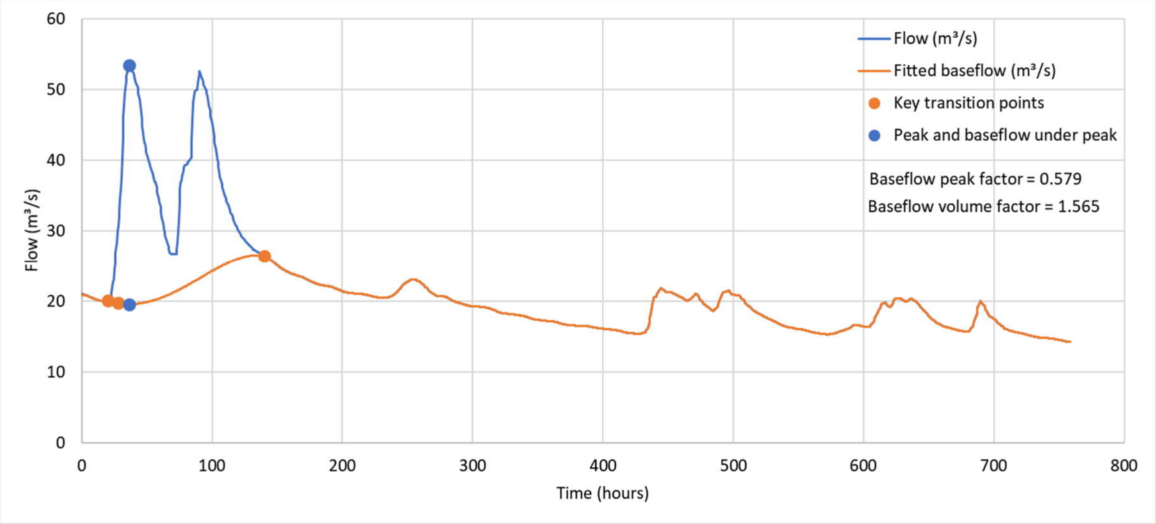
■ Figure B-20 Goulburn River at Seymour



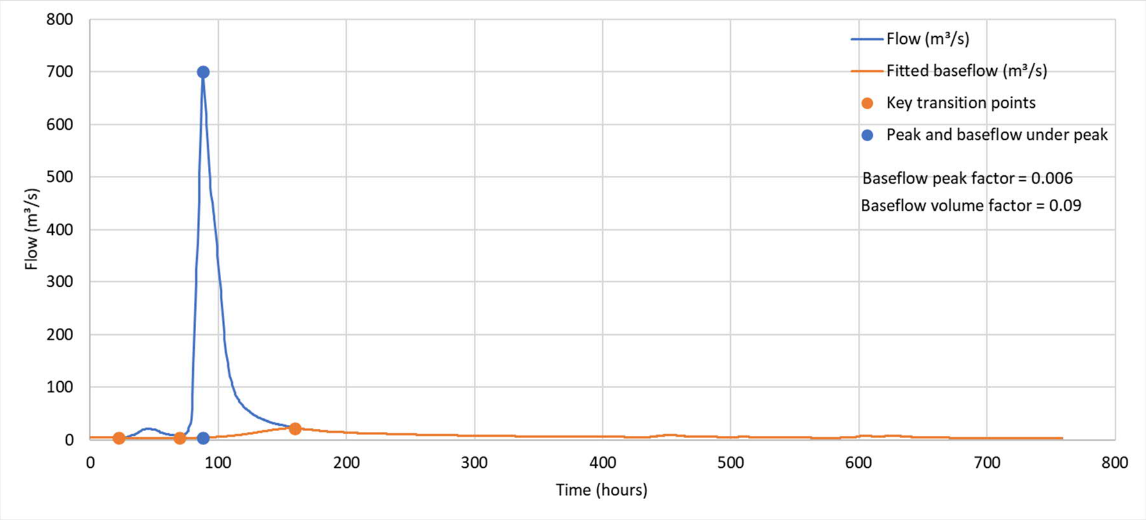
■ Figure B-21 Goulburn River at Shepparton



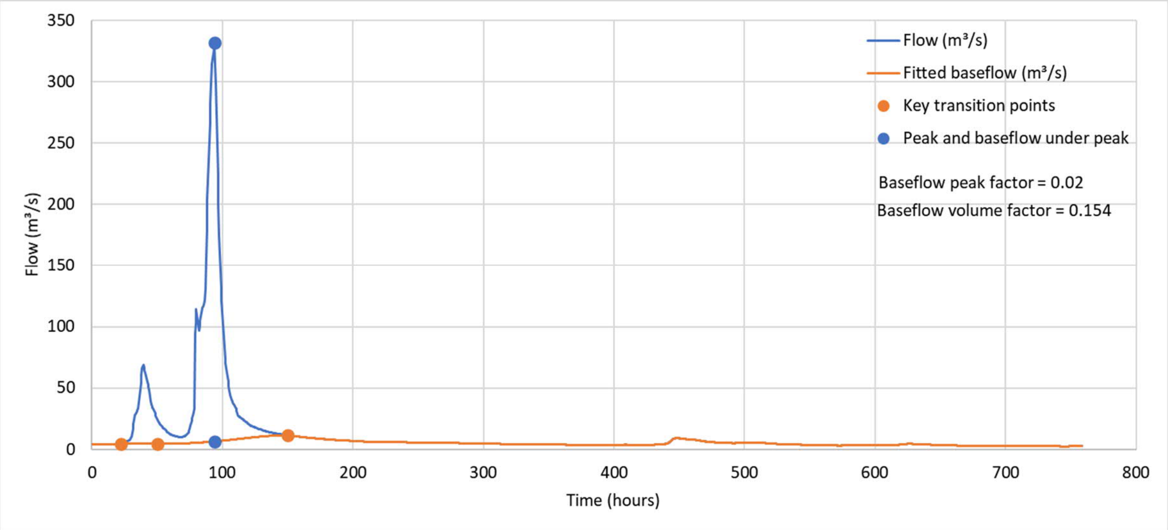
■ Figure B-22 Goulburn River at Trawool



■ Figure B-23 Acheron River at Taggerty



■ Figure B-24 Holland Creek at Kelfeera



■ Figure B-25 Hughes Creek at Tarcombe

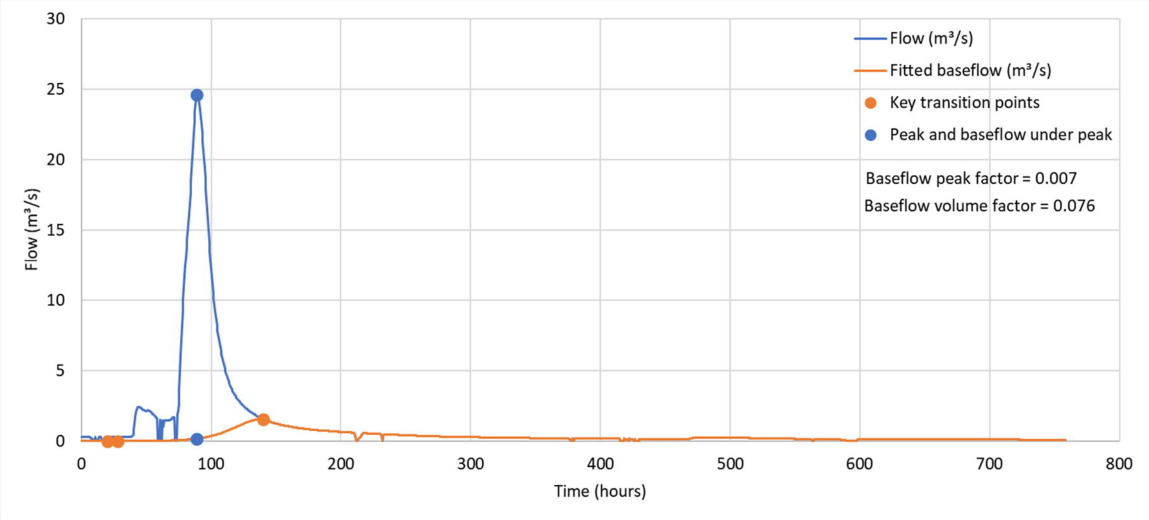


Figure B-26 Major Creek at Greytown

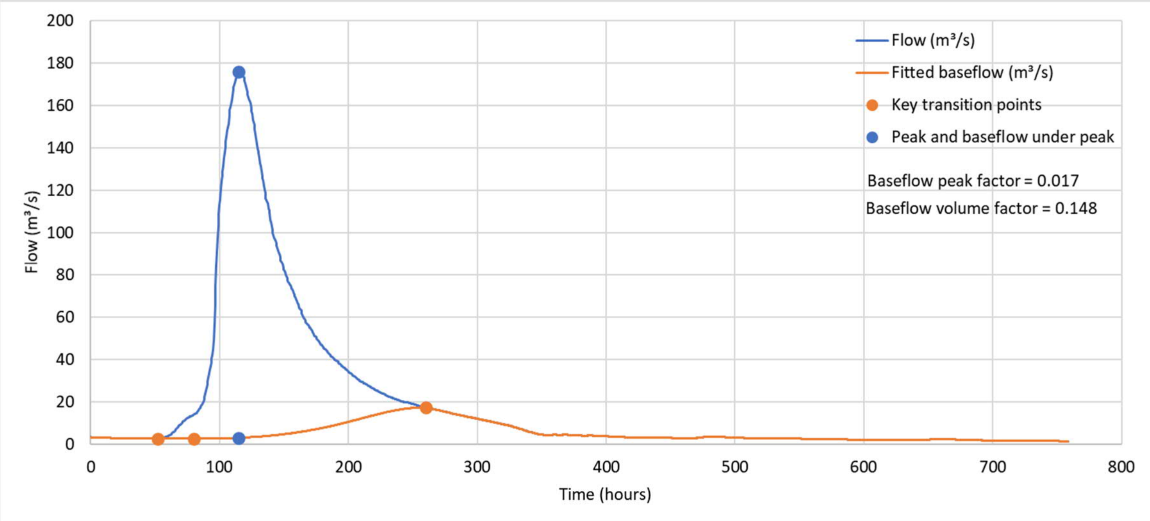


Figure B-27 Pranjip Creek at Moorilim

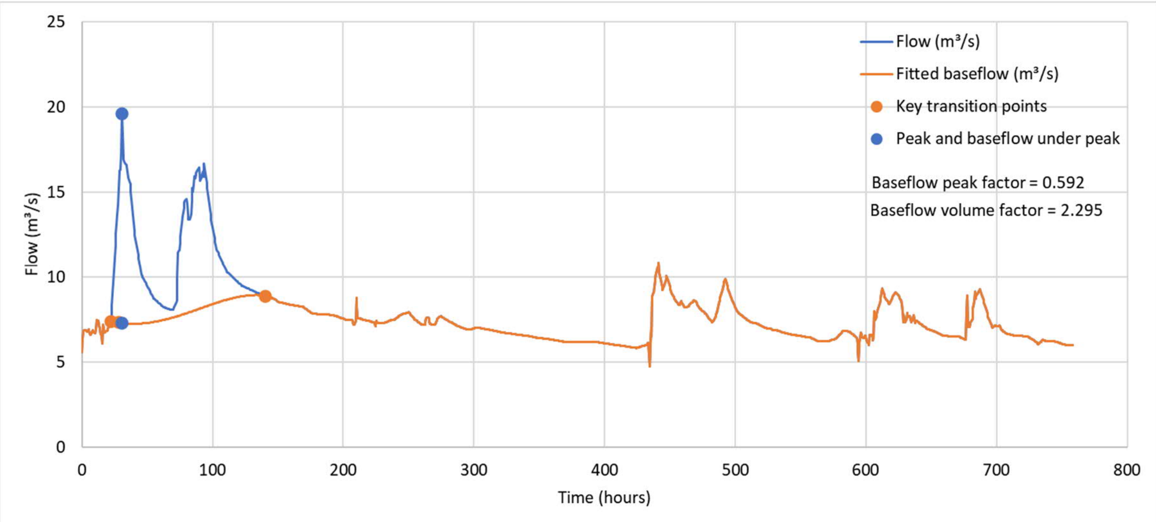
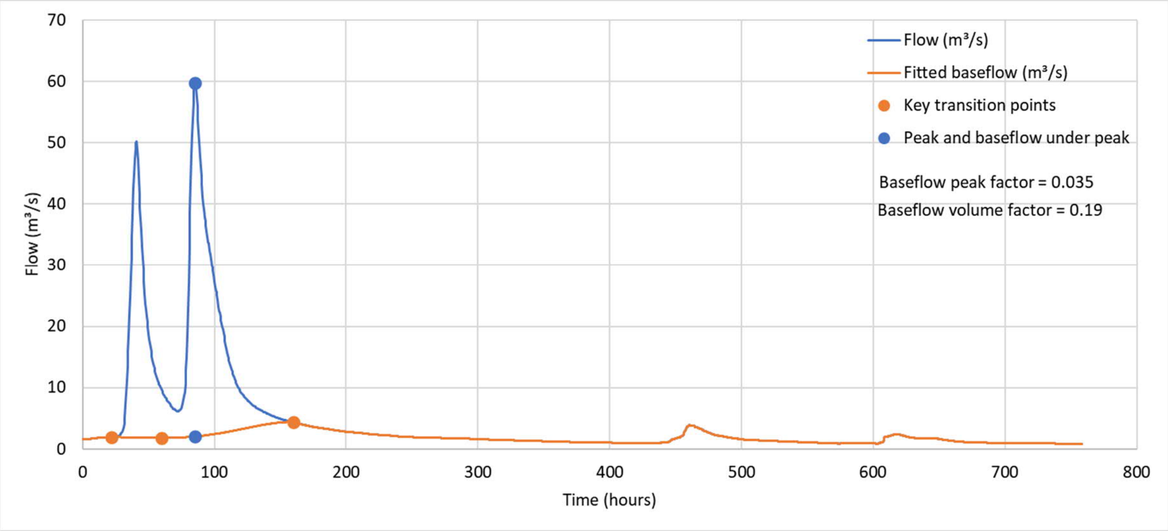
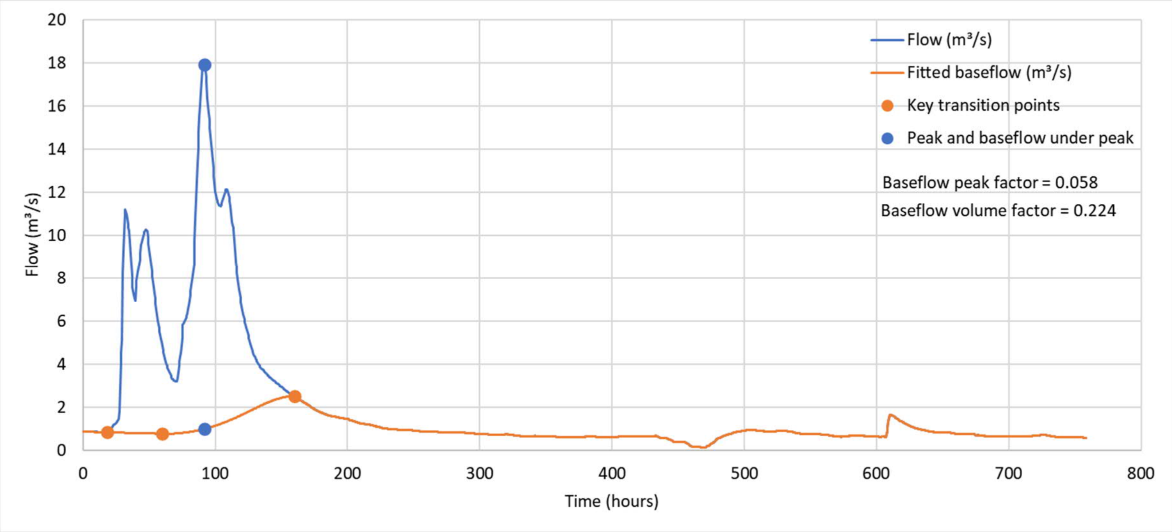


Figure B-28 Rubicon River at Rubicon

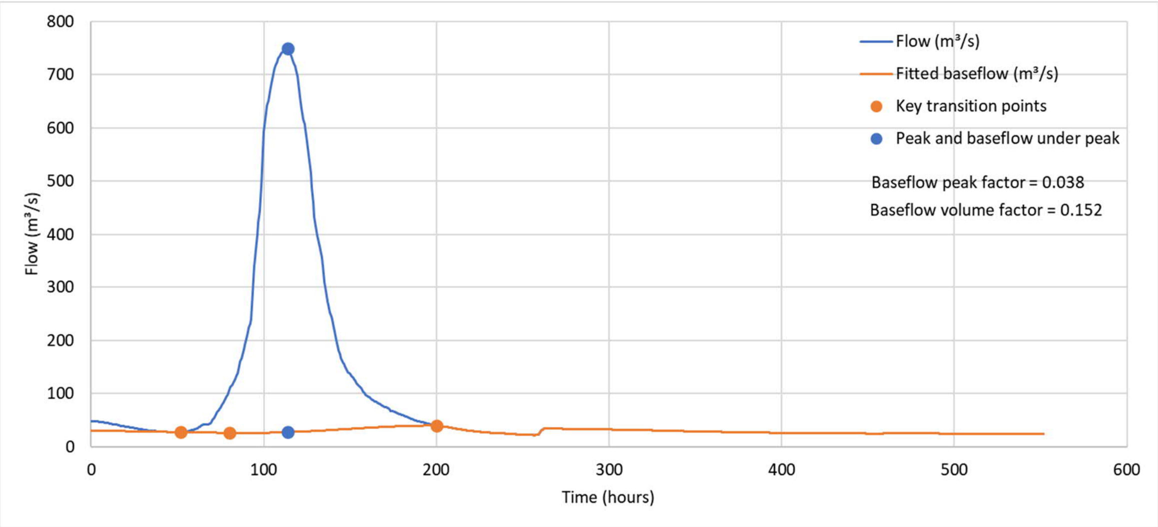


▪ Figure B-29 Sugarloaf Creek at Ashbridge

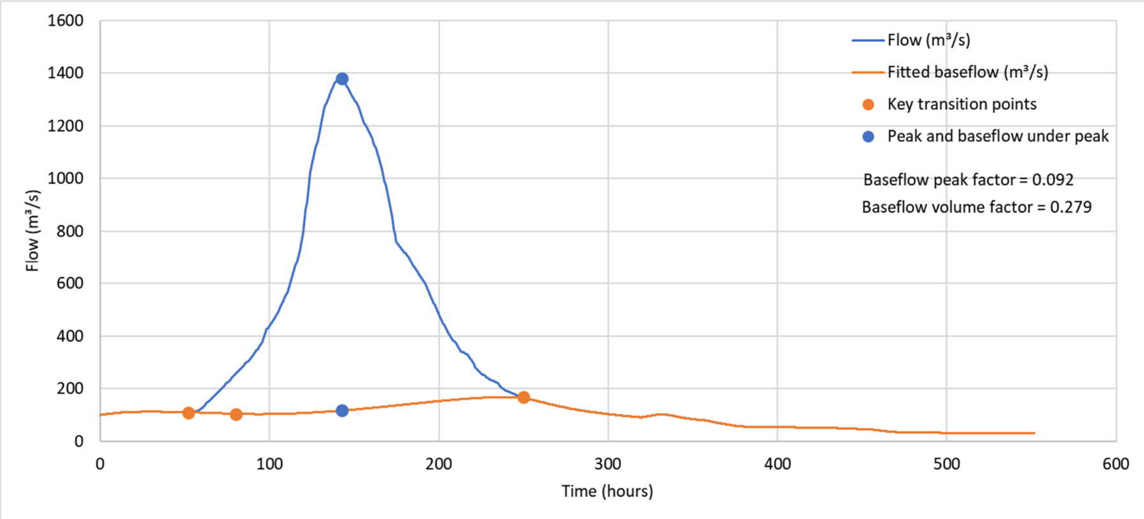


▪ Figure B-30 Sunday Creek at Tallarook

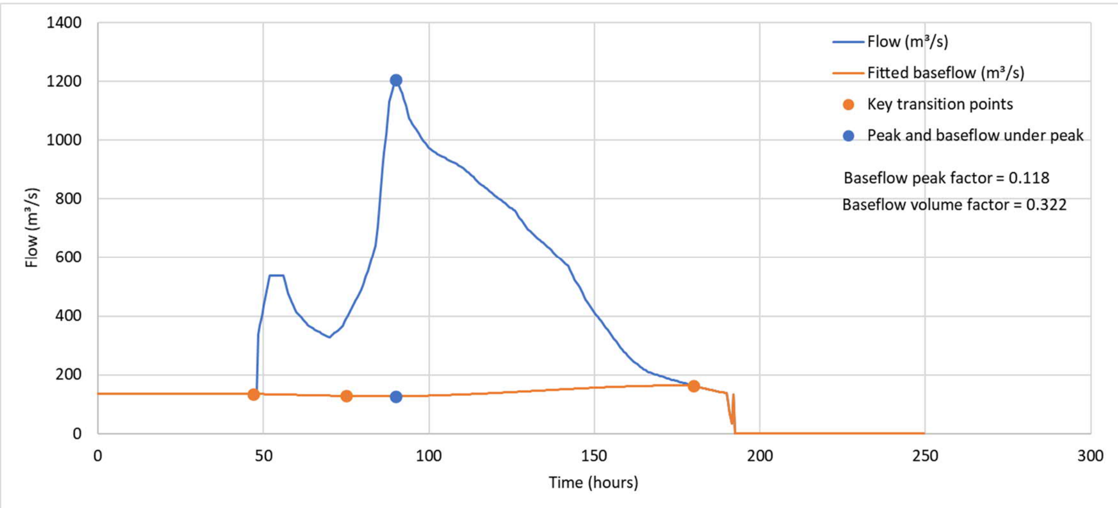
May 1974 event



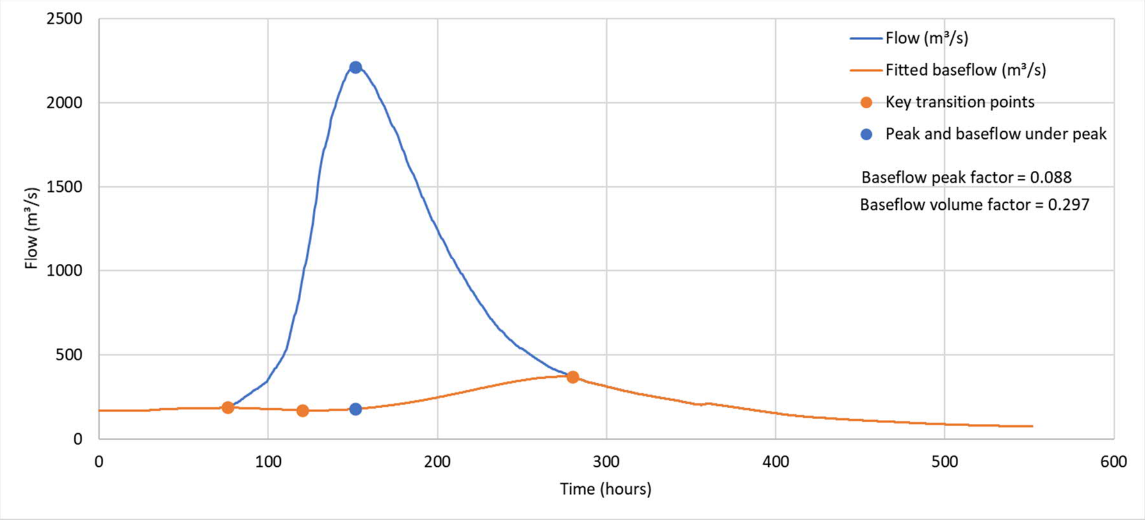
- Figure B-31 Broken River at Casey's Weir



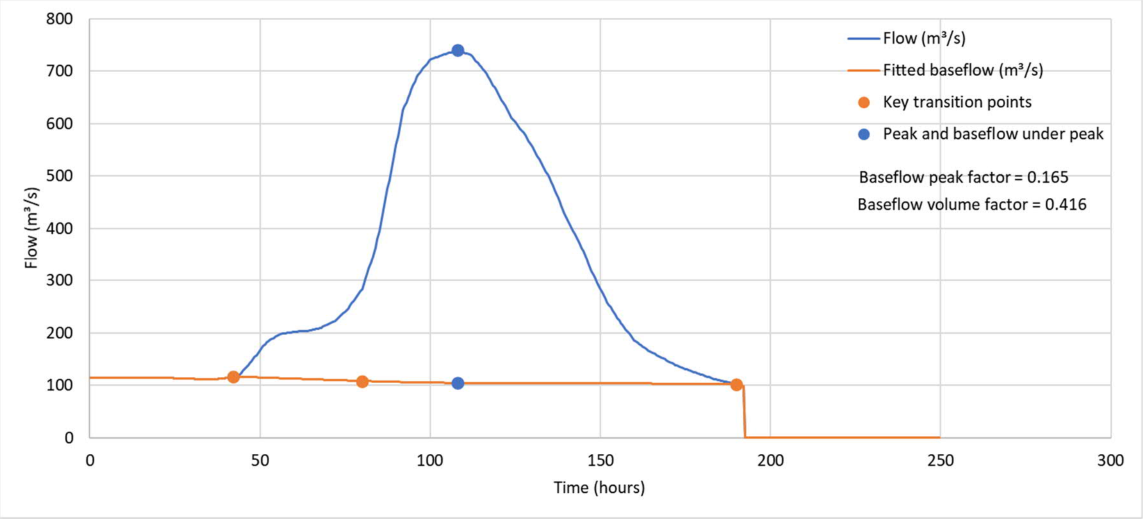
- Figure B-32 Goulburn River at Murchison



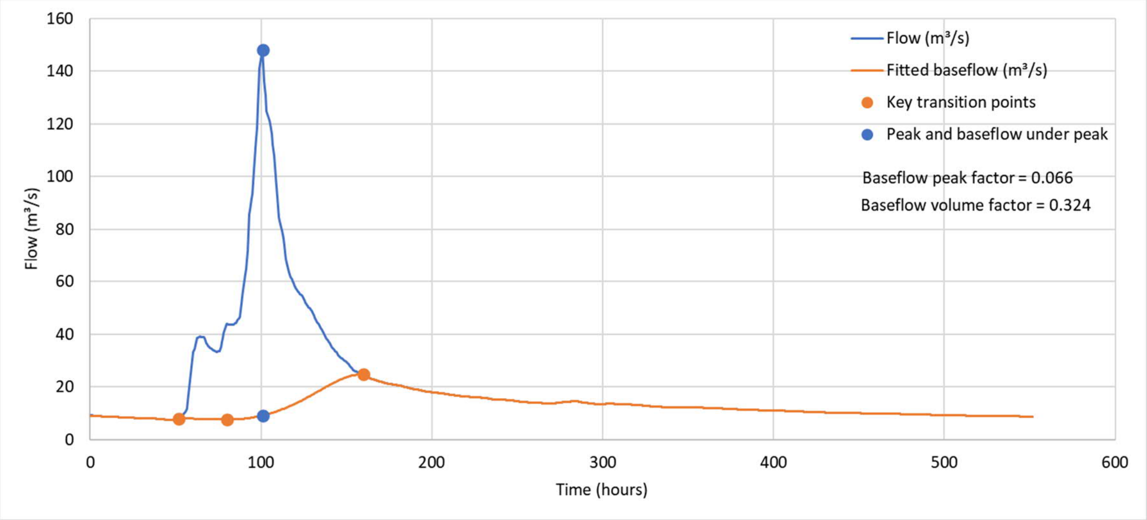
- Figure B-33 Goulburn River at Seymour



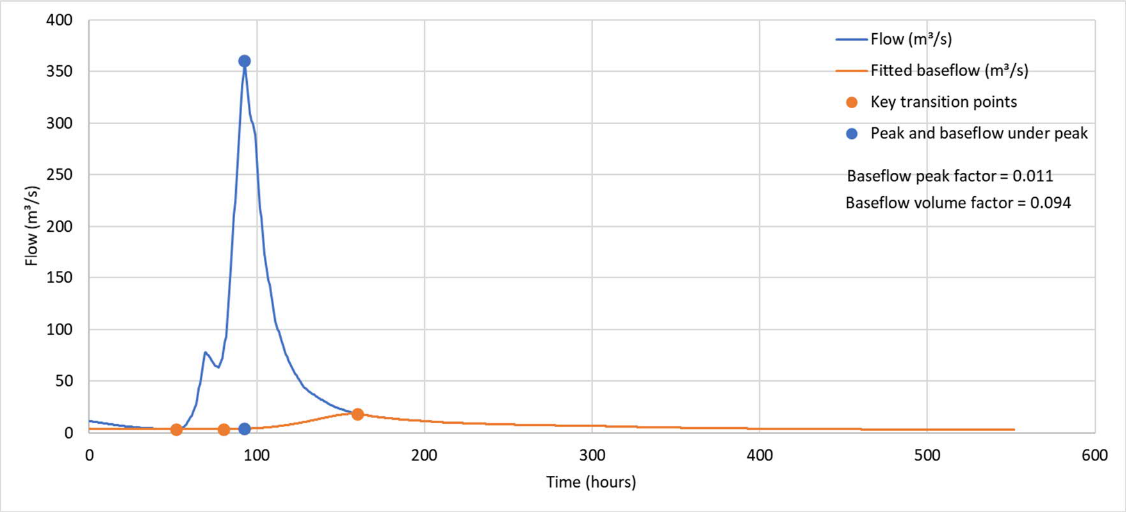
■ Figure B-34 Goulburn River at Shepparton



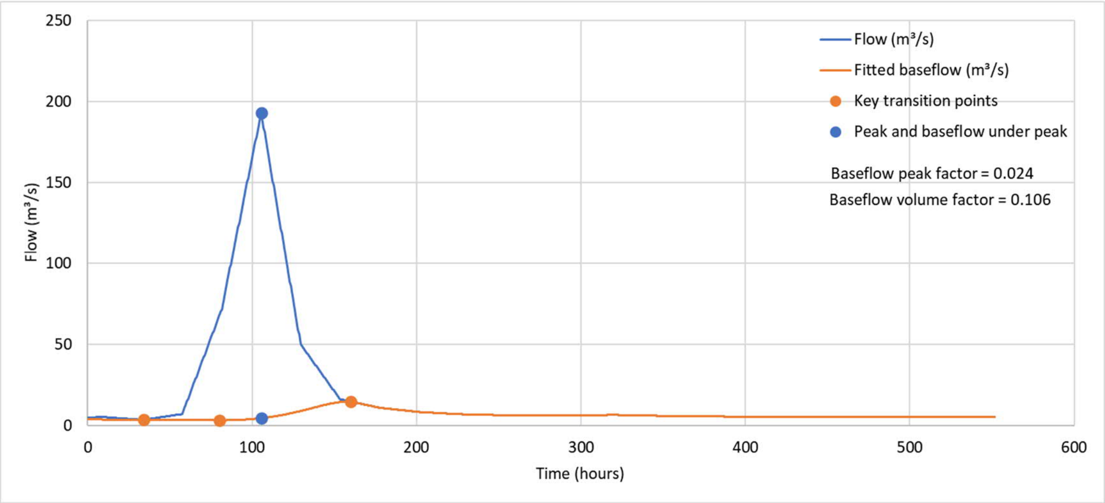
■ Figure B-35 Goulburn River at Trawool



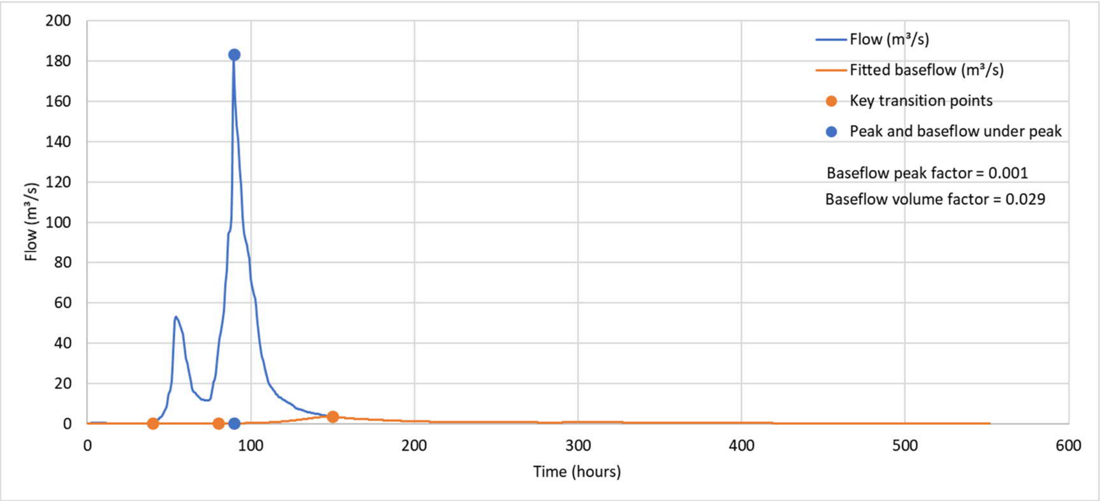
■ Figure B-36 Acheron River at Taggerty



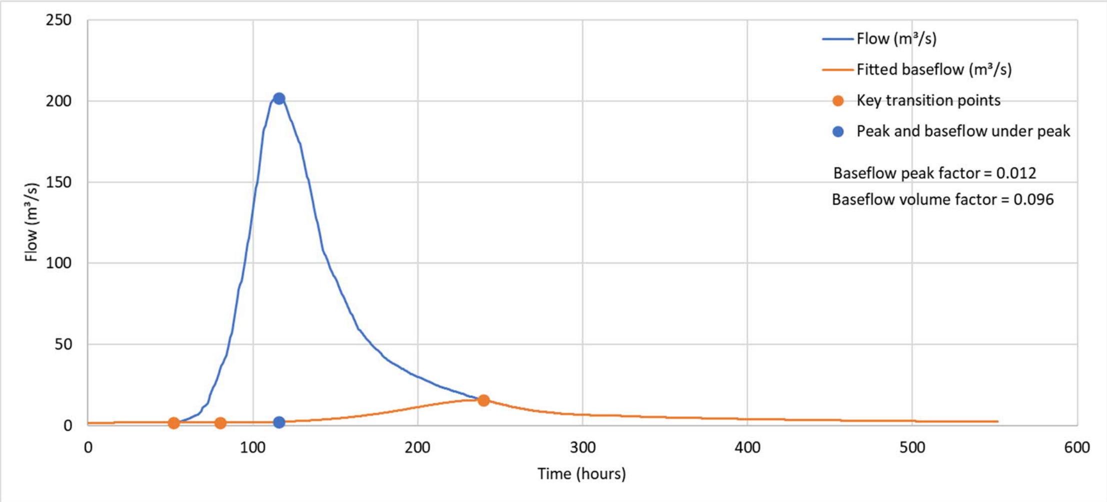
■ Figure B-37 Holland Creek at Kelfeera



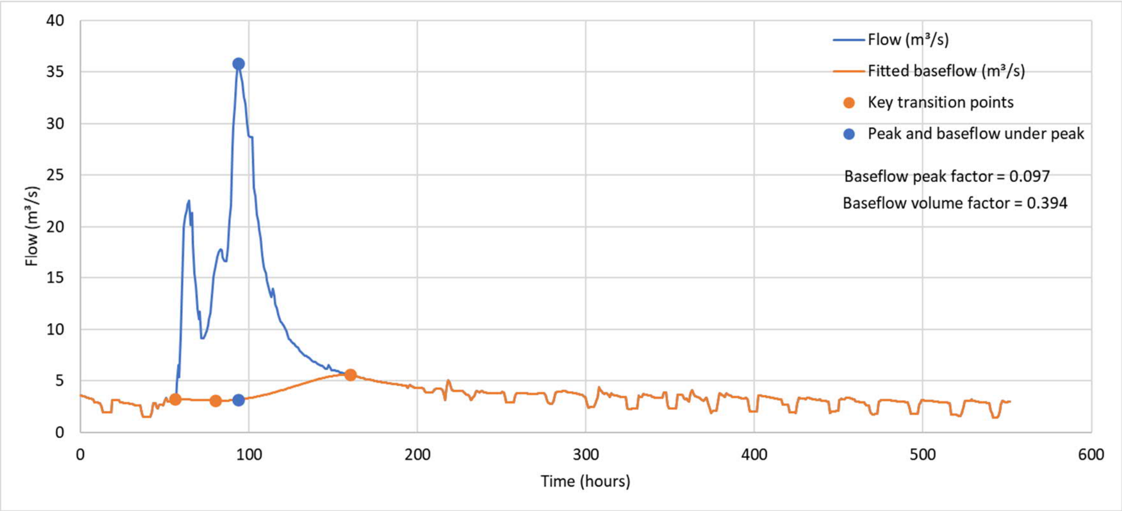
■ Figure B-38 Hughes Creek at Tarcombe



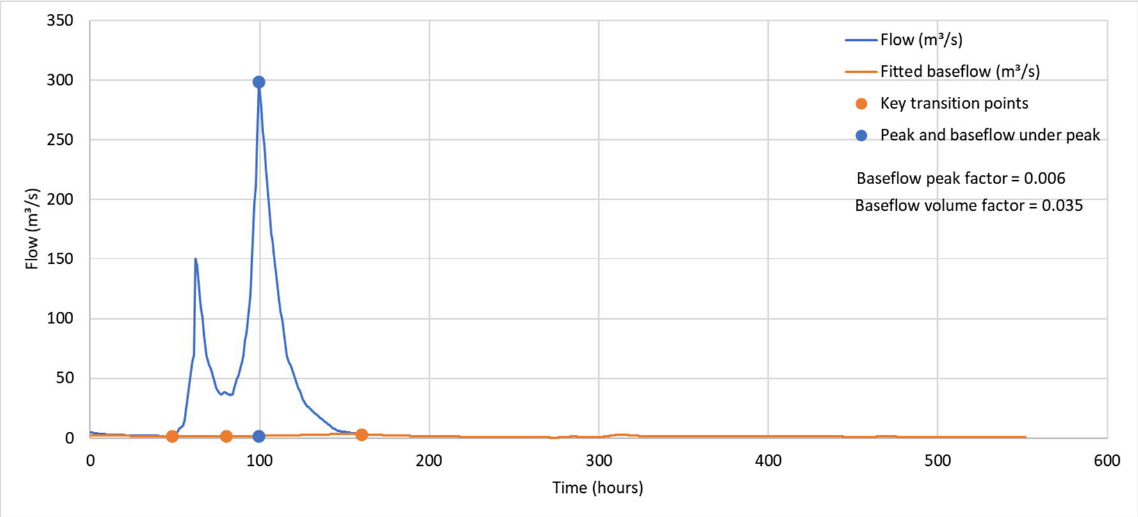
■ Figure B-39 Major Creek at Greytown



■ Figure B-40 Pranjip Creek at Moorilim



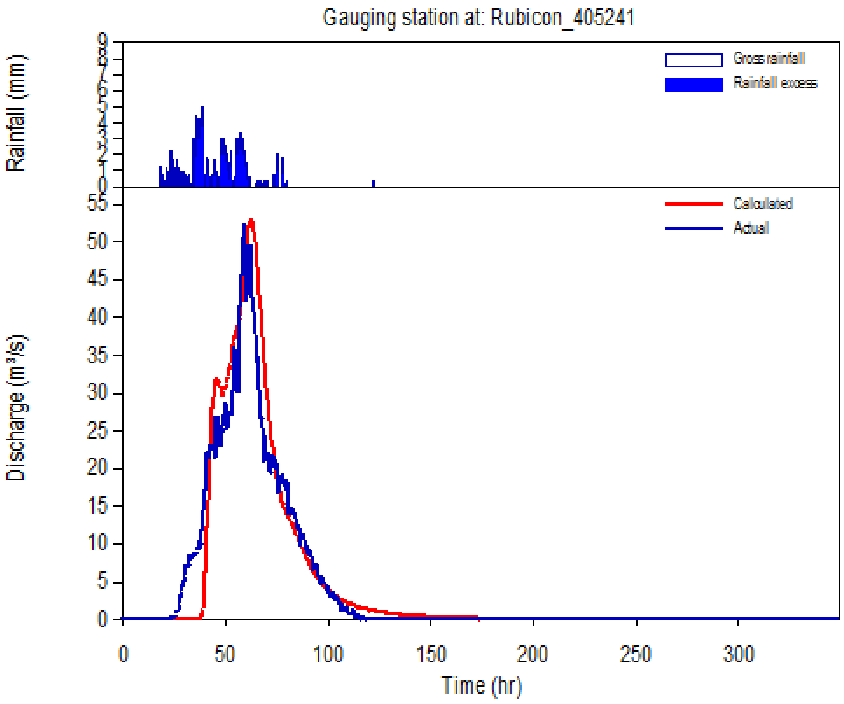
■ Figure B-41 Rubicon River at Rubicon

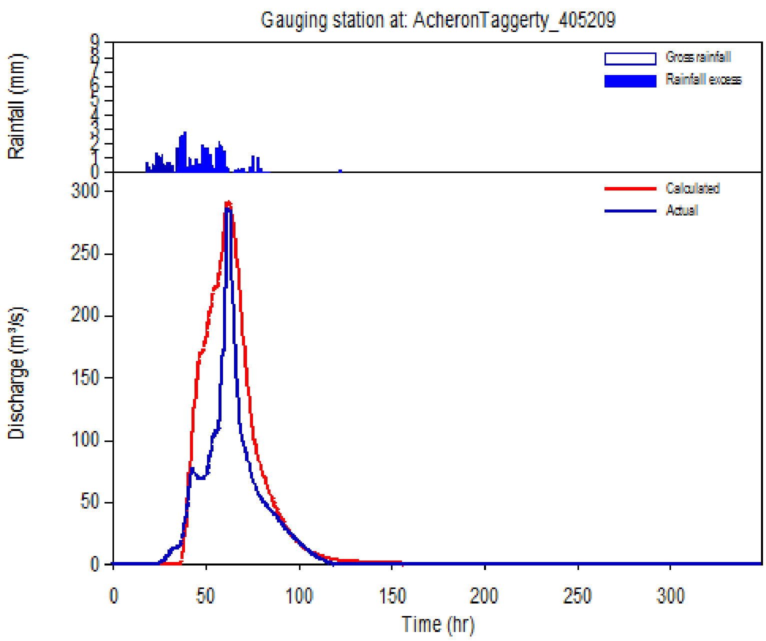


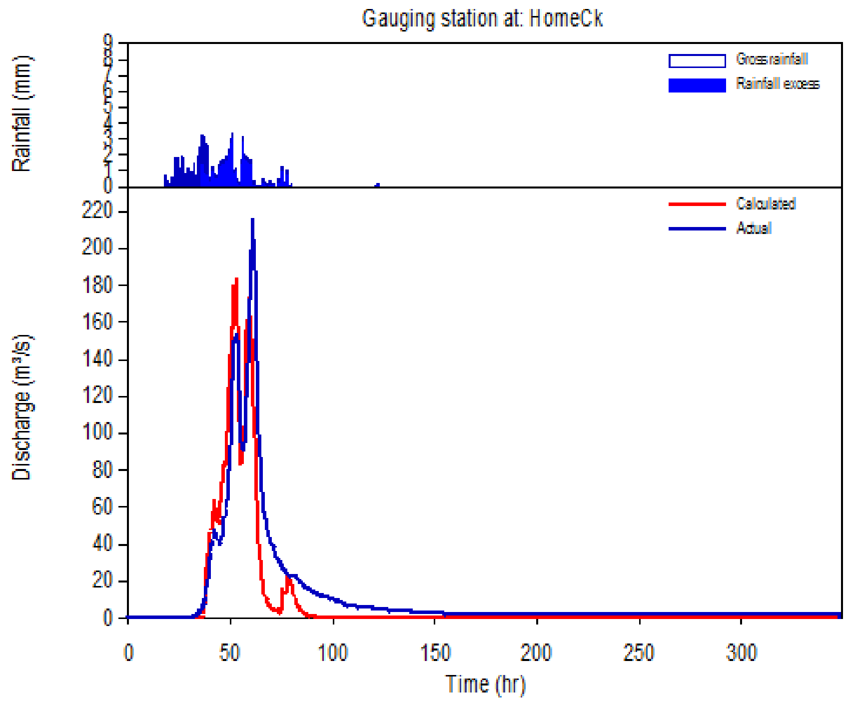
■ Figure B-42 Sunday Creek at Tallarook

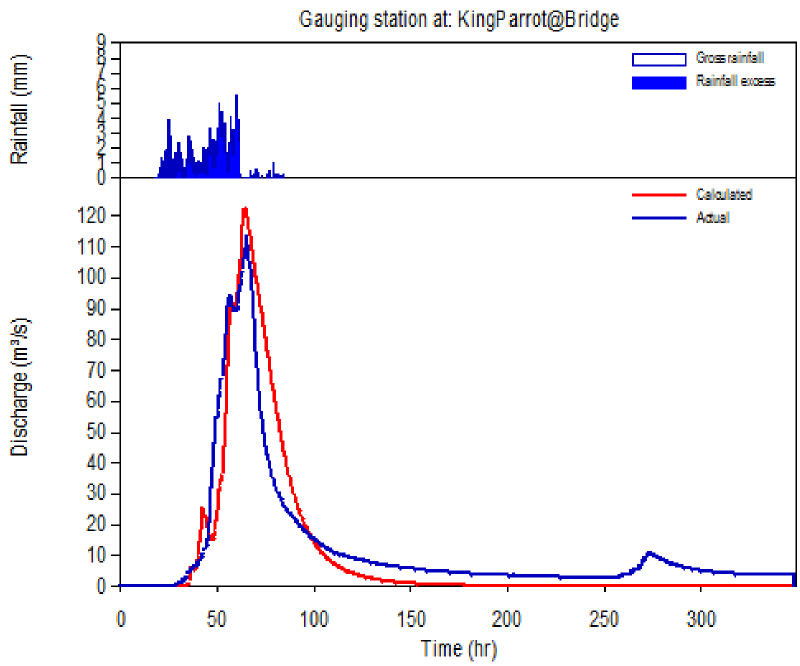
Appendix C Detailed calibration results

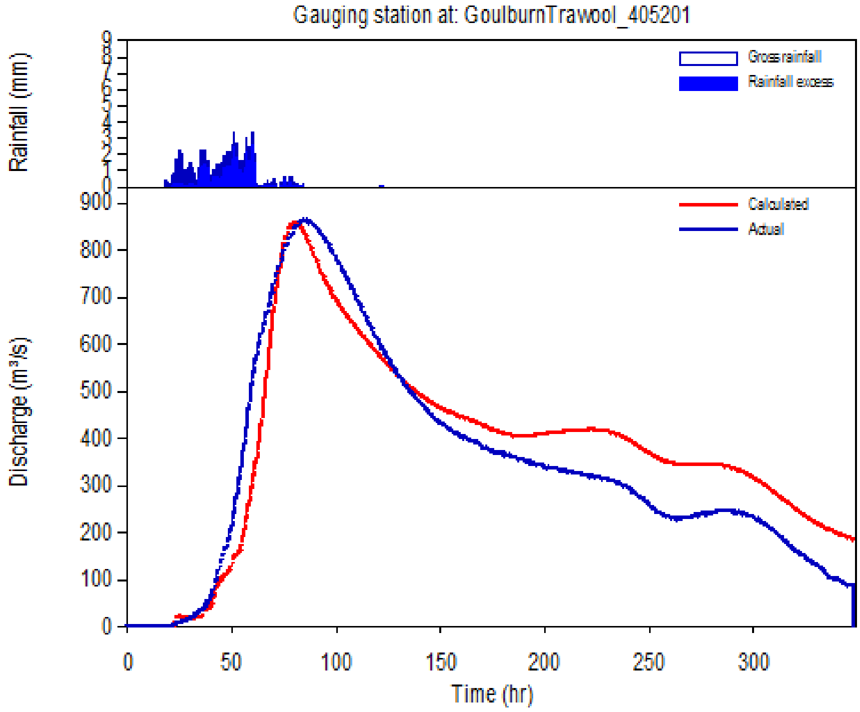
October 2022 event

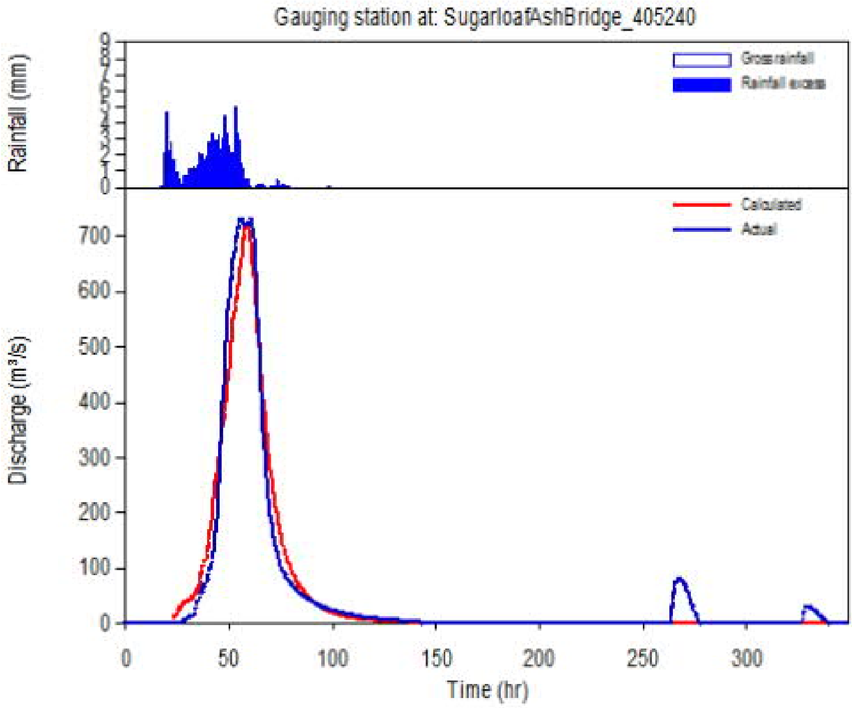


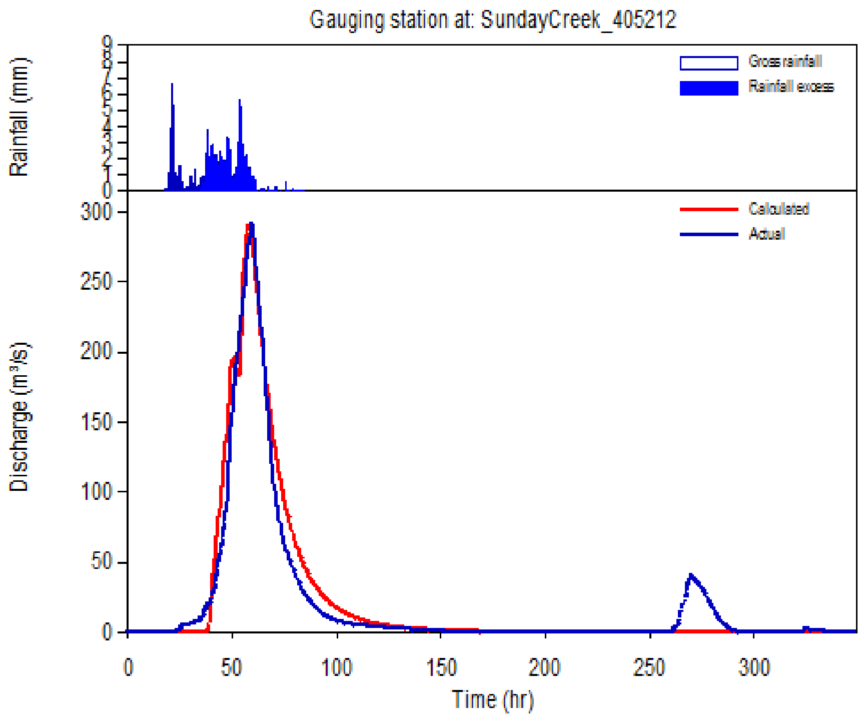


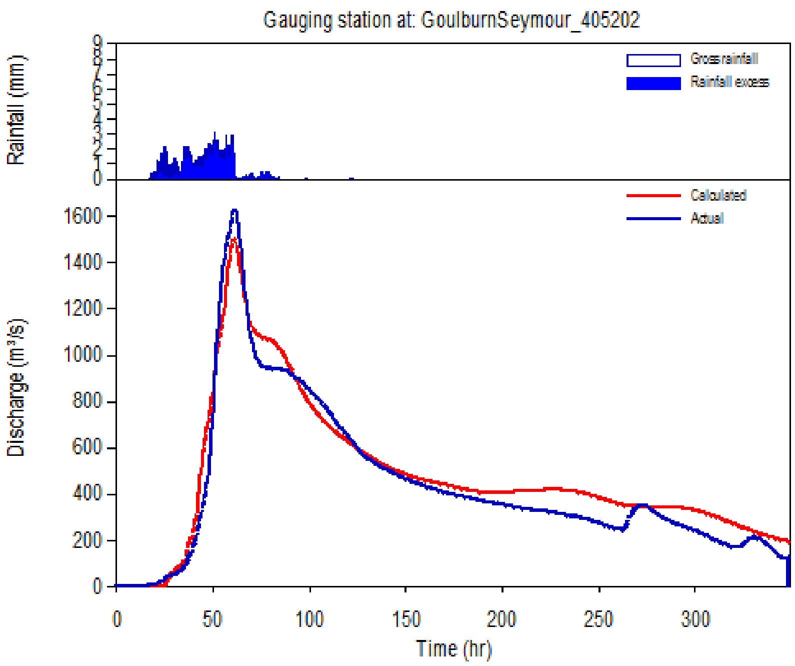

Gauging station at: HomeCk
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

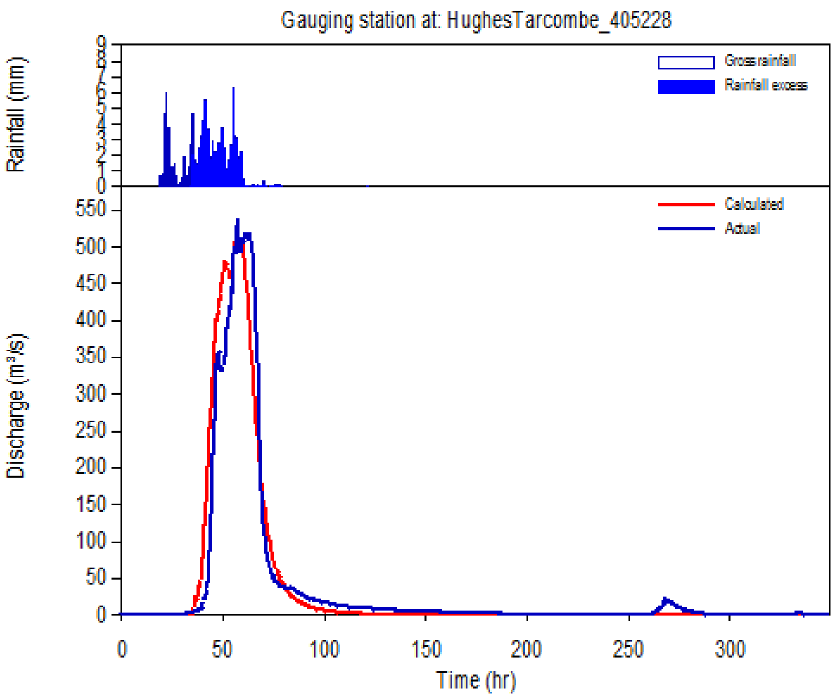

Gauging station at: KingParrot@Bridge
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

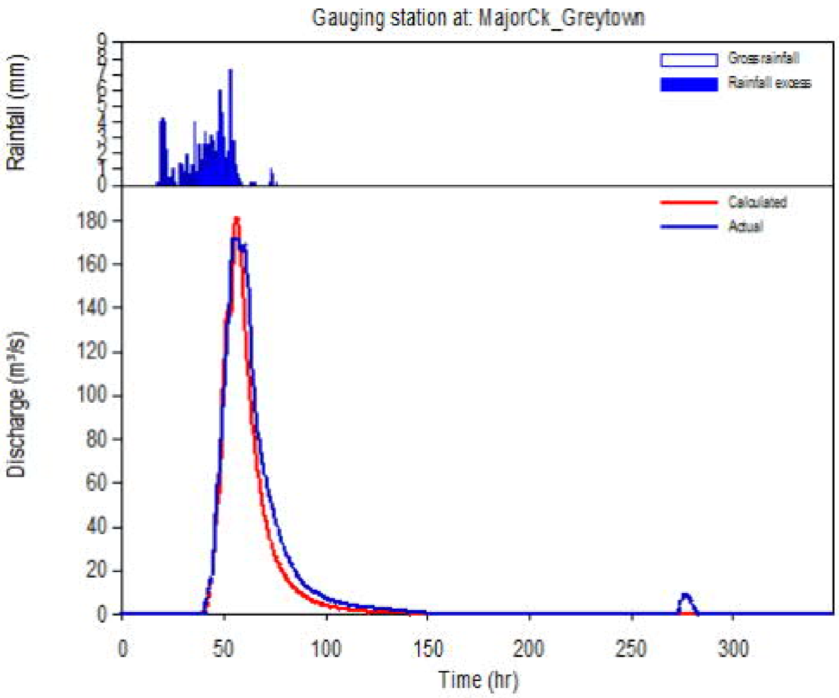

Gauging station at: GoulburnTrawool_405201
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

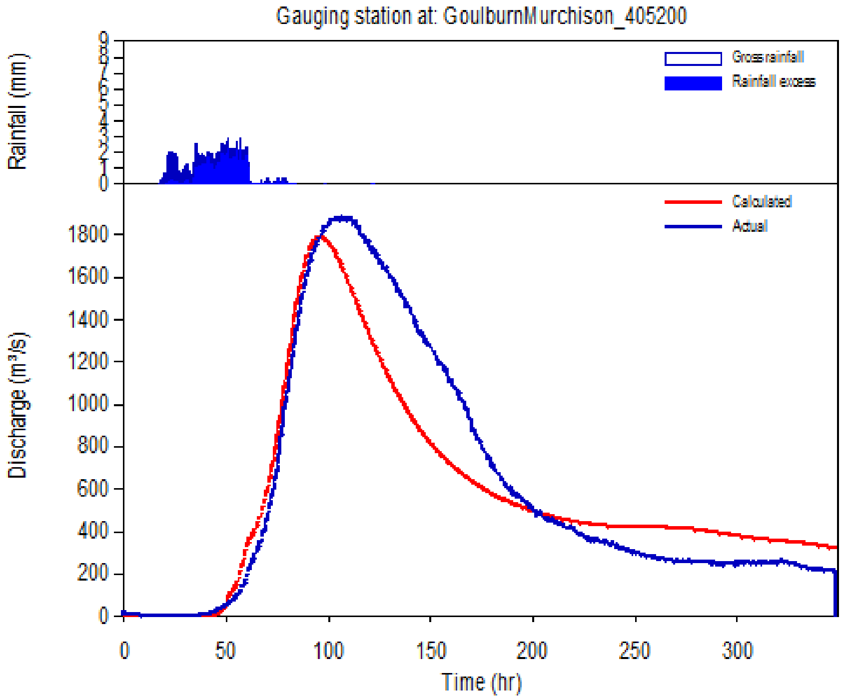

Gauging station at: SugarloafAshBridge_405240
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

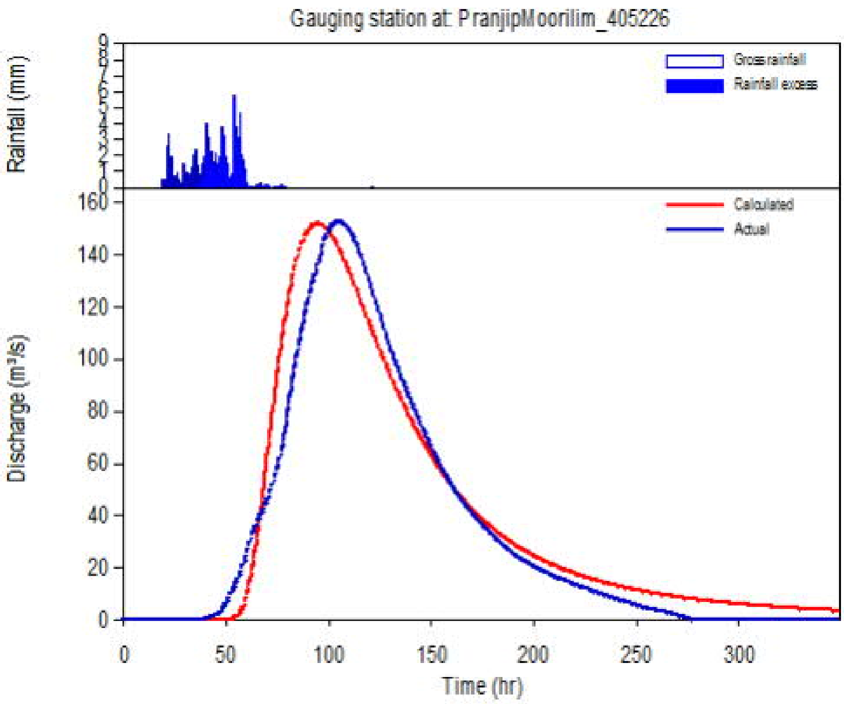

Gauging station at: SundayCreek_405212
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

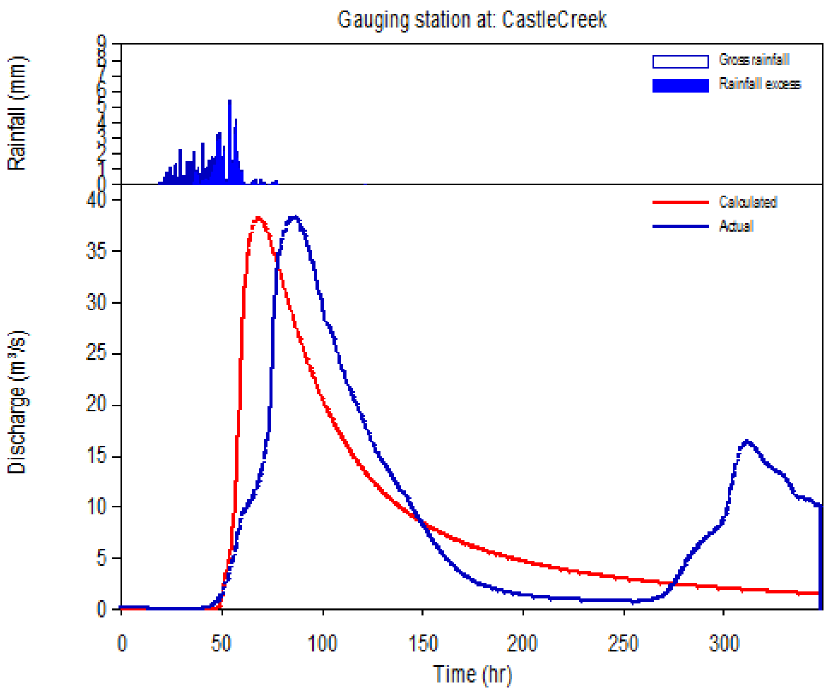

Gauging station at: GoulburnSeymour_405202
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

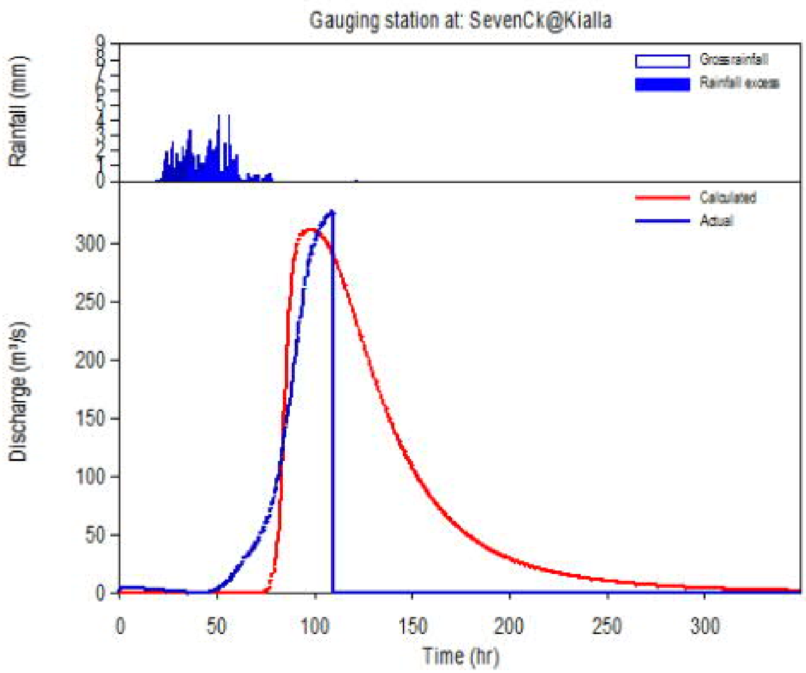

Gauging station at: HughesTarcombe_405228
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

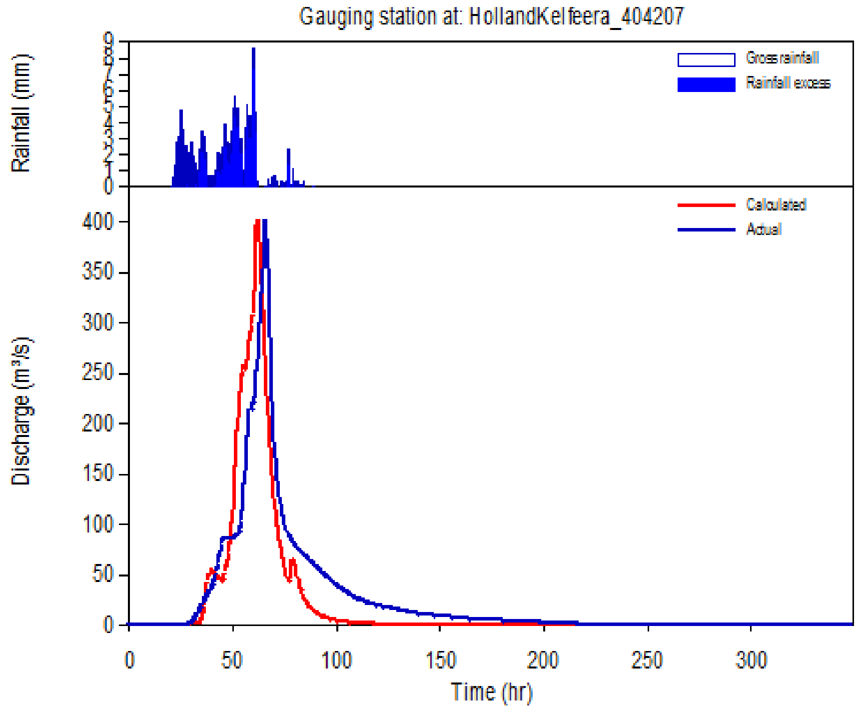

Gauging station at: MajorCk_Greytown
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

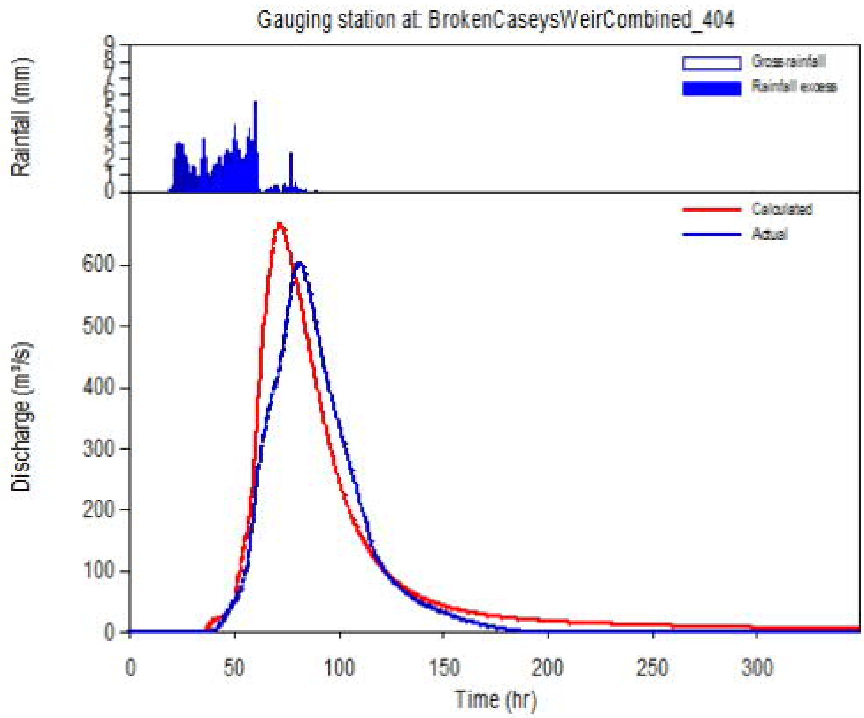

Gauging station at: GoulburnMurchison_405200
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

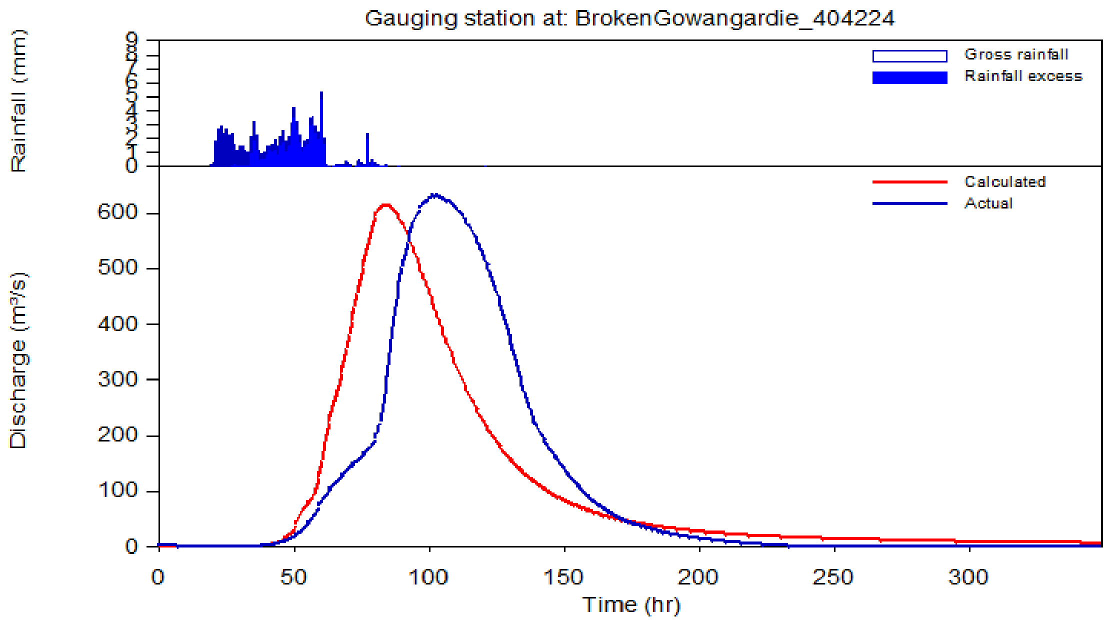

Gauging station at: PranjipMoorilim_405226
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

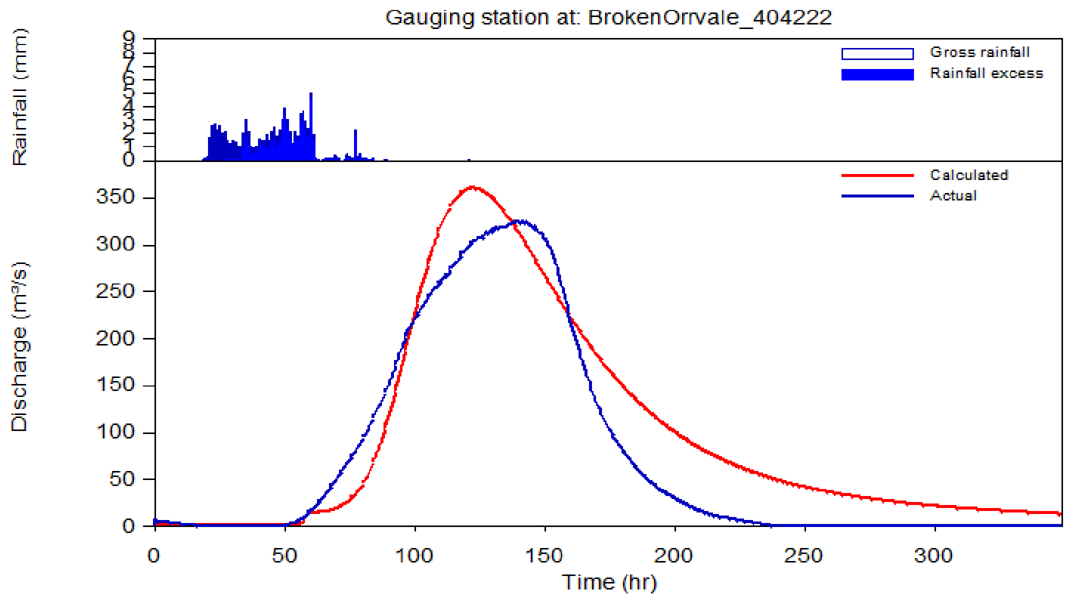

Gauging station at: CastleCreek
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

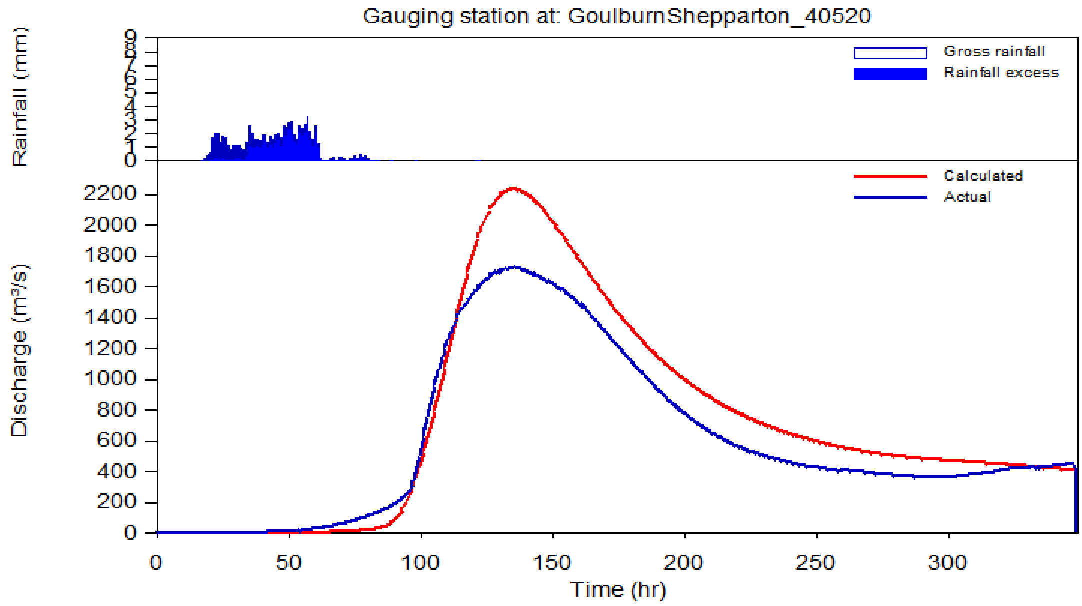

Gauging station at: SevenCk@Kialla
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)


Gauging station at: HollandKelfeera_404207
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

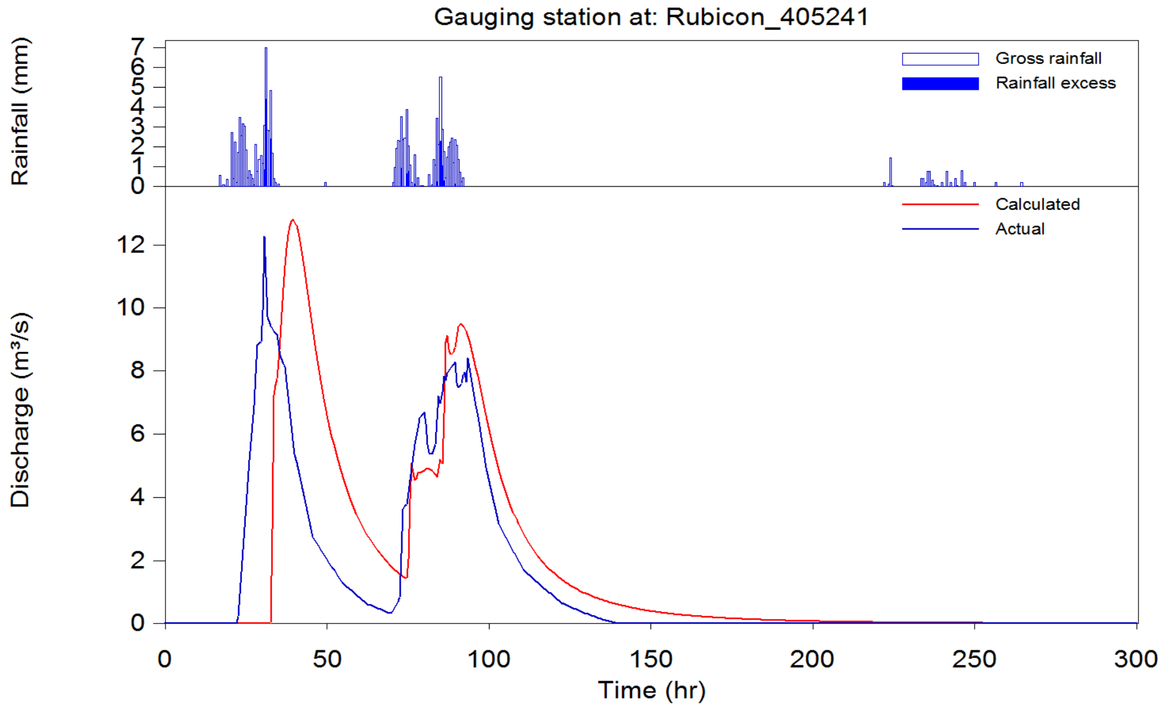

Gauging station at: BrokenCaseysWeirCombined_404
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

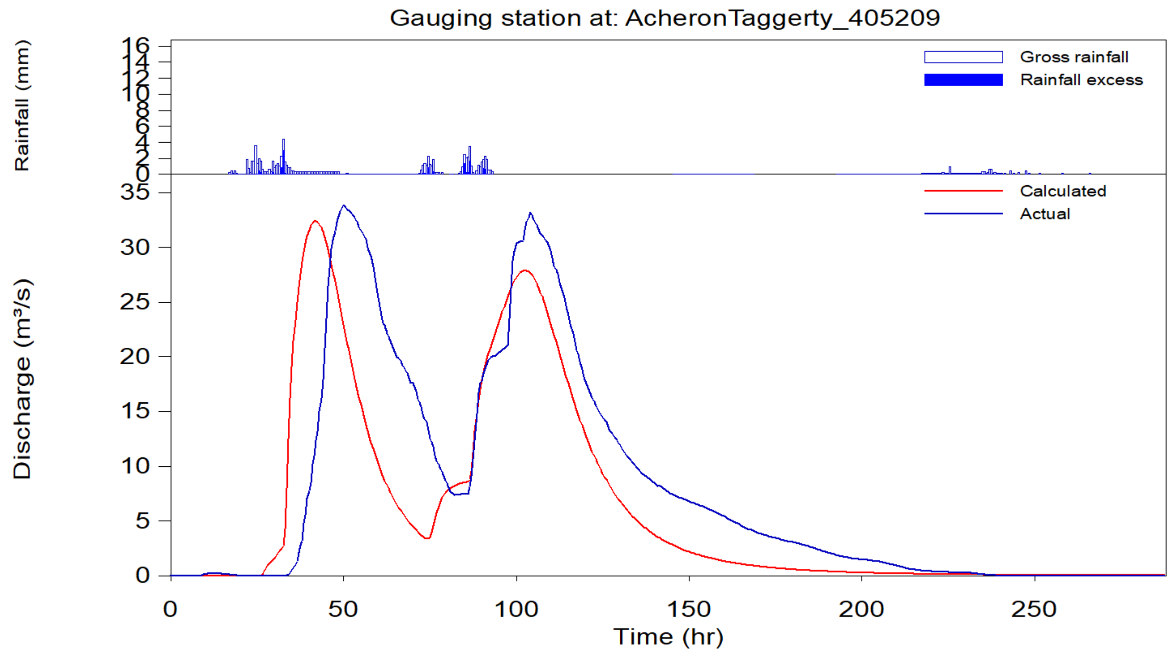

Gauging station at: BrokenGowangardie_404224
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

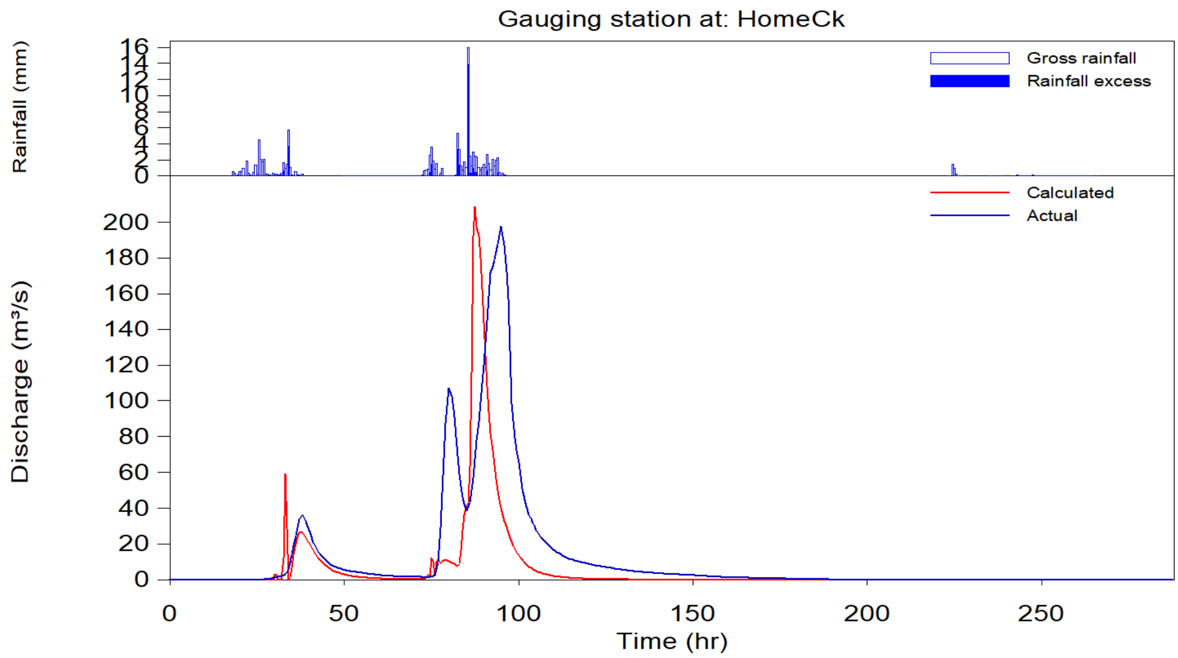

Gauging station at: BrokenOrrvale_404222
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

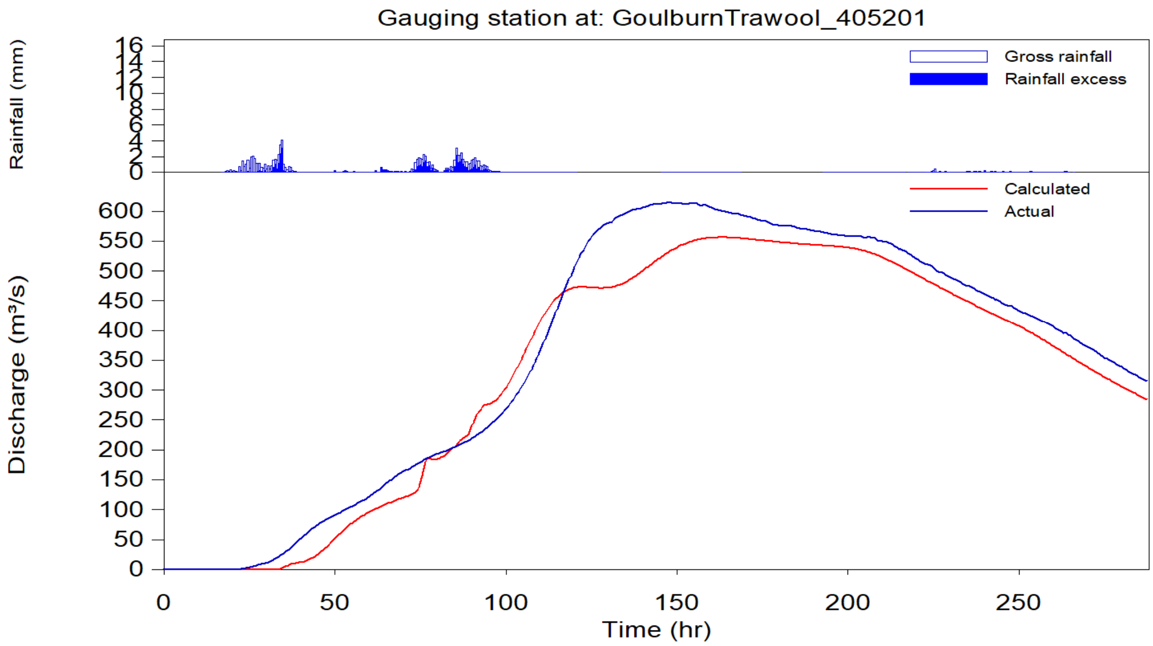


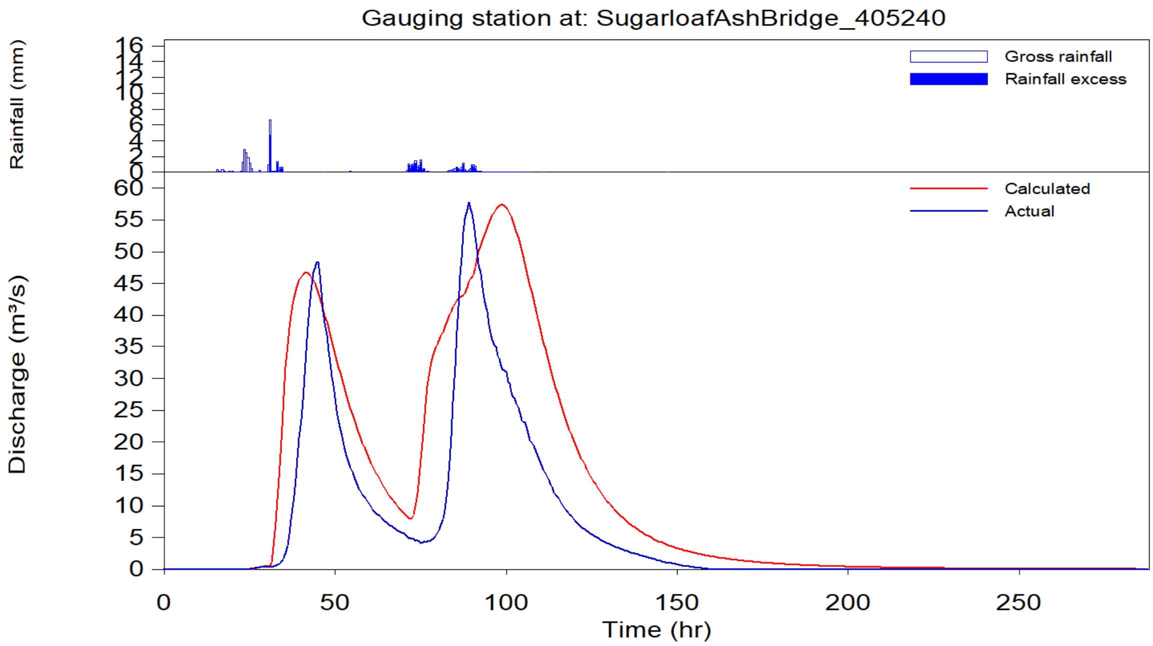
October 1993 event

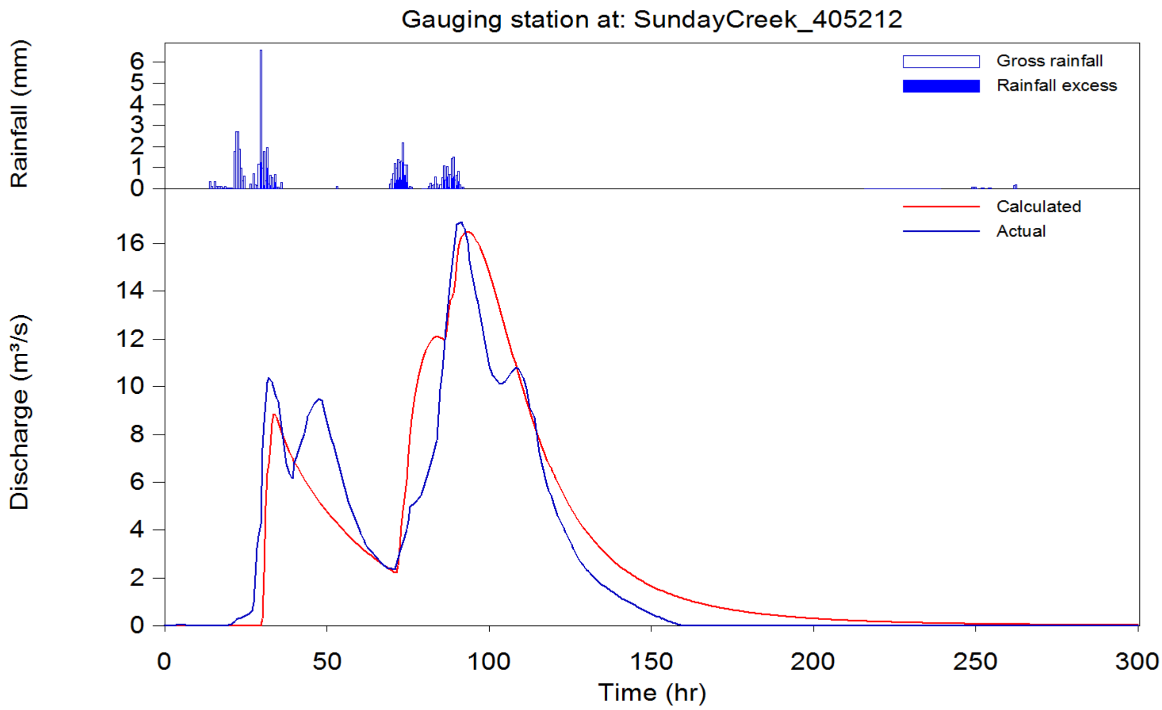


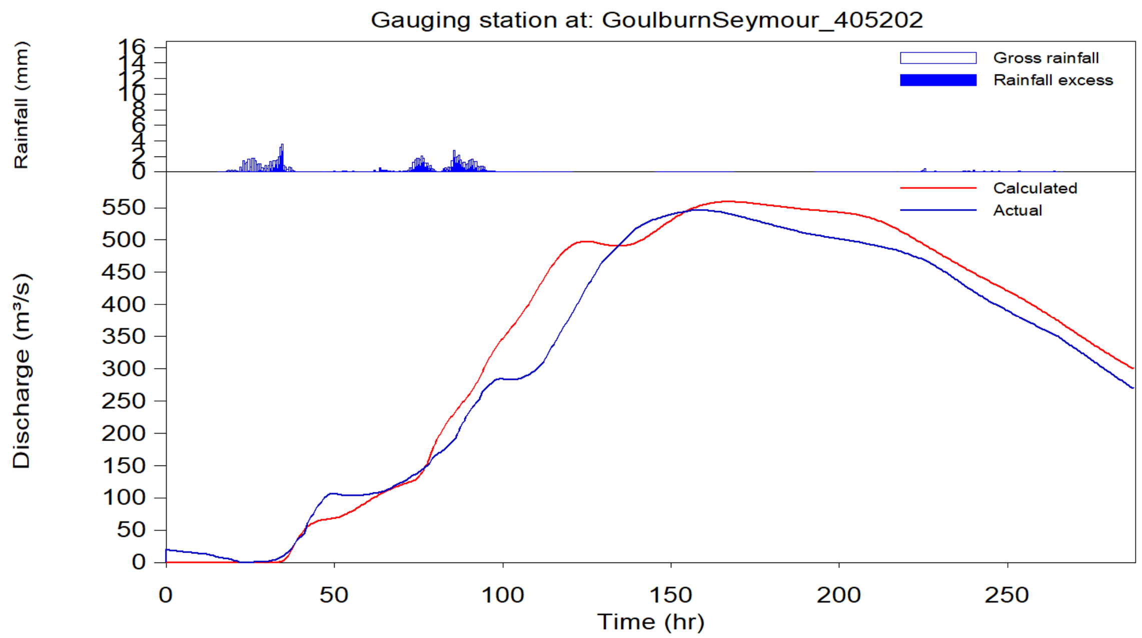

Gauging station at: AcheronTaggerty_405209
Rainfall (mm)
Discharge (m³/s)
Gross rainfall
Rainfall excess
Calculated
Actual
Time (hr)

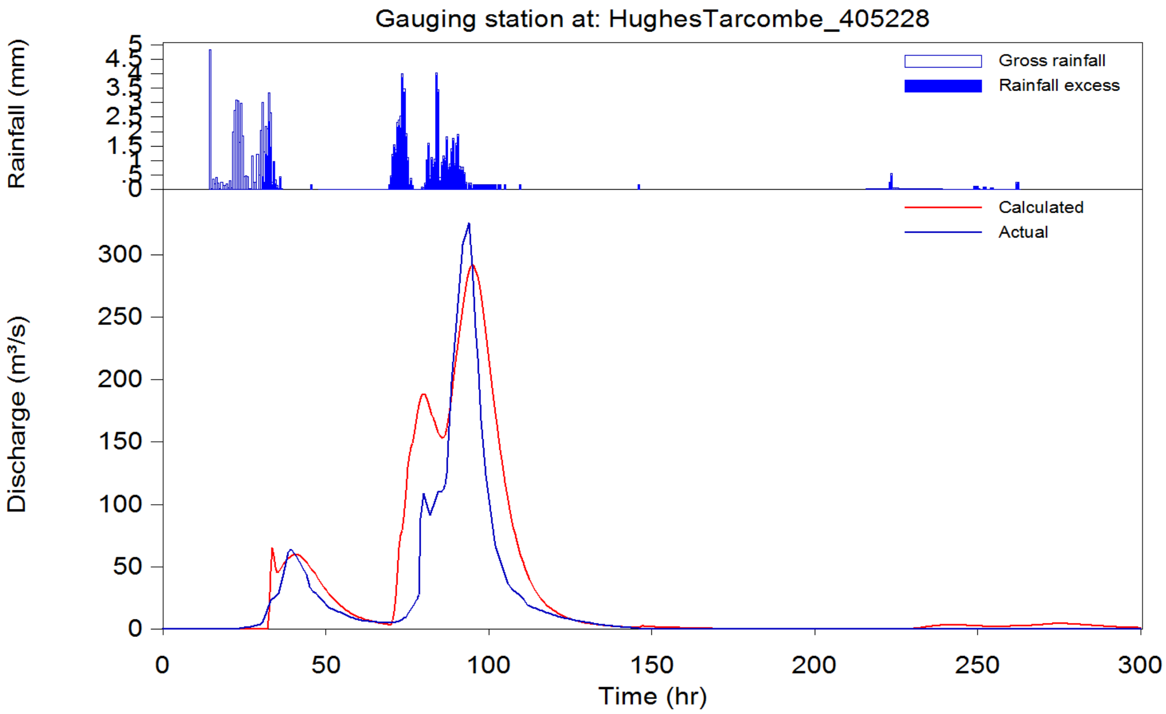

Gauging station at: HomeCk
Rainfall (mm)
Discharge (m³/s)
Gross rainfall
Rainfall excess
Calculated
Actual
Time (hr)

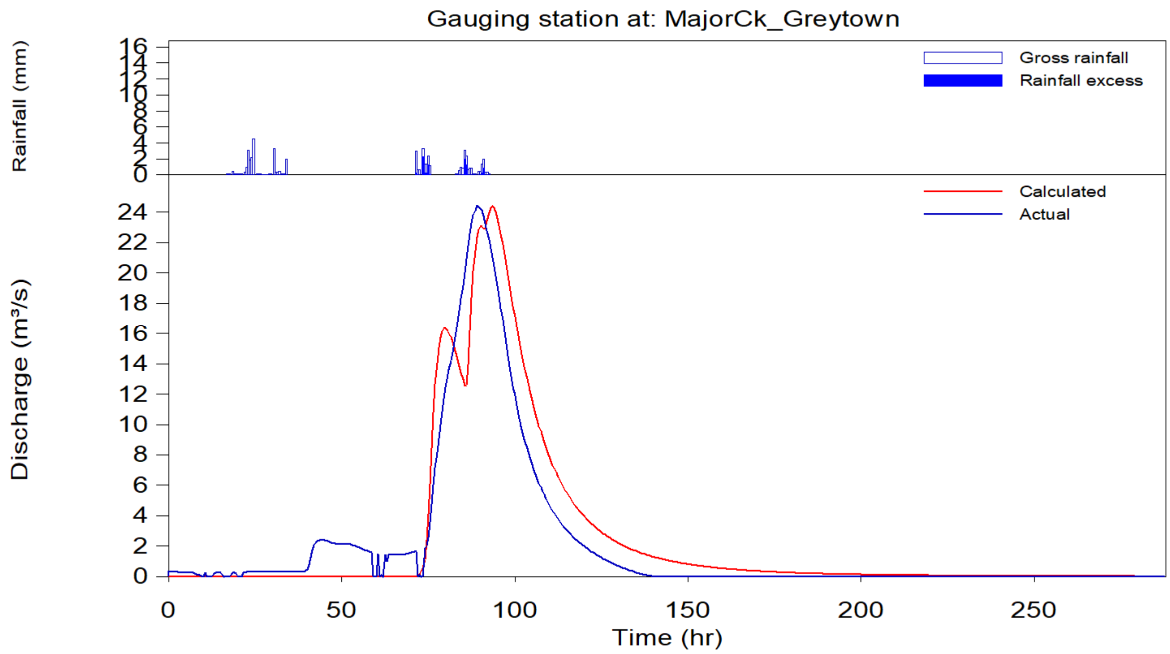

Gauging station at: GoulburnTrawool_405201
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

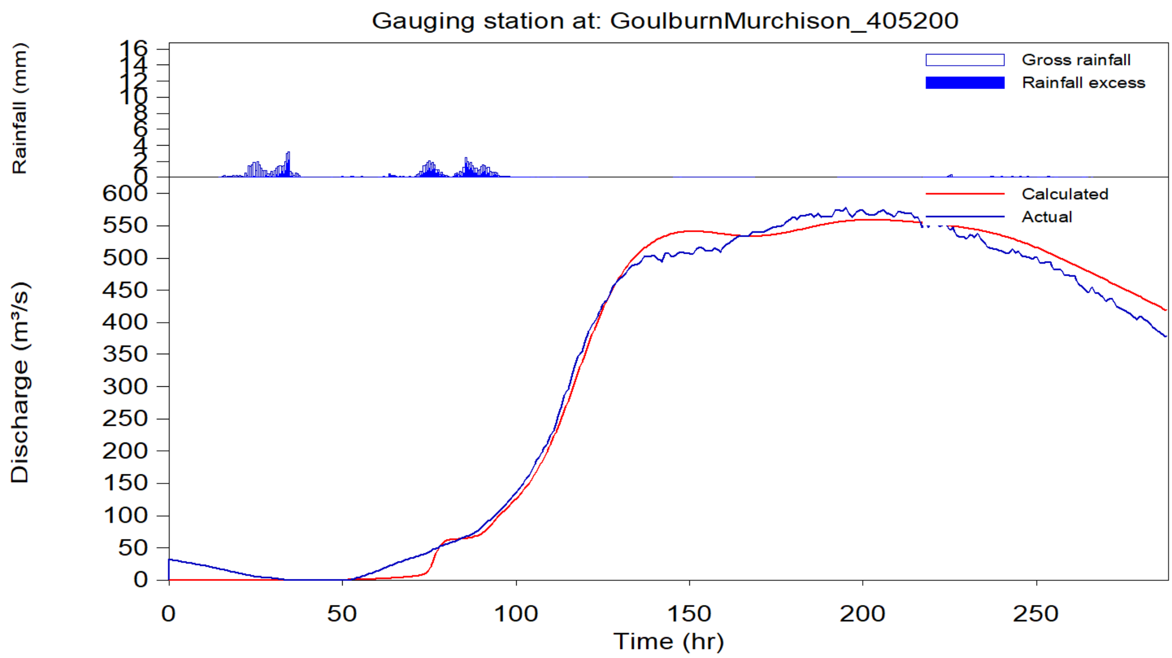

Gauging station at: SugarloafAshBridge_405240
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

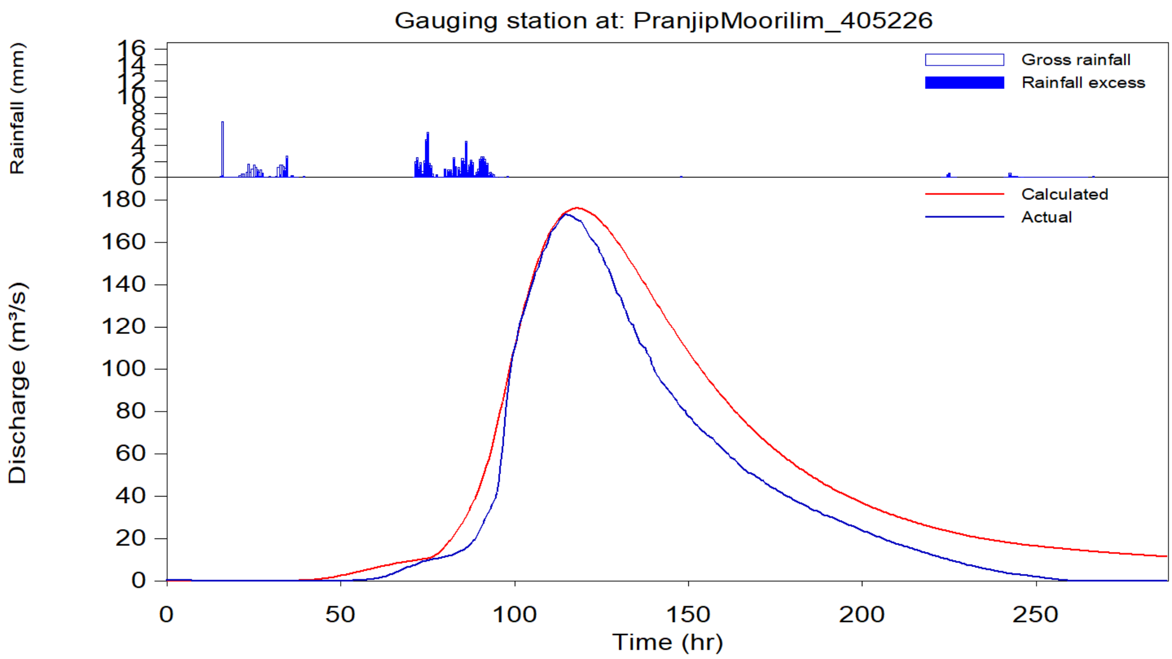

Gauging station at: SundayCreek_405212
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

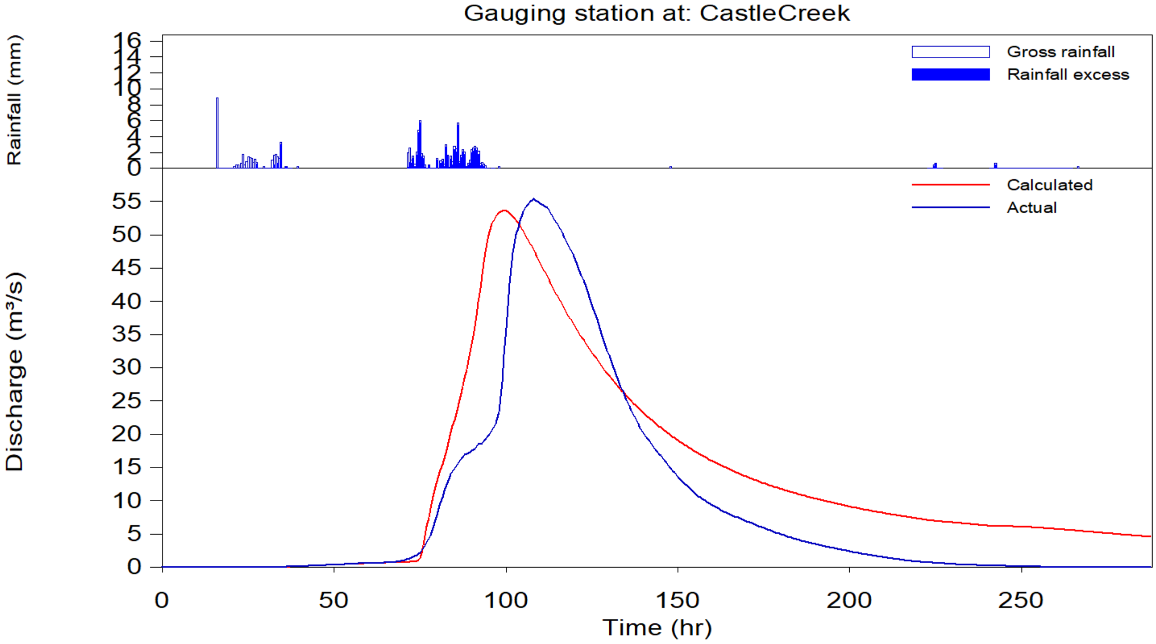

Gauging station at: GoulburnSeymour_405202
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

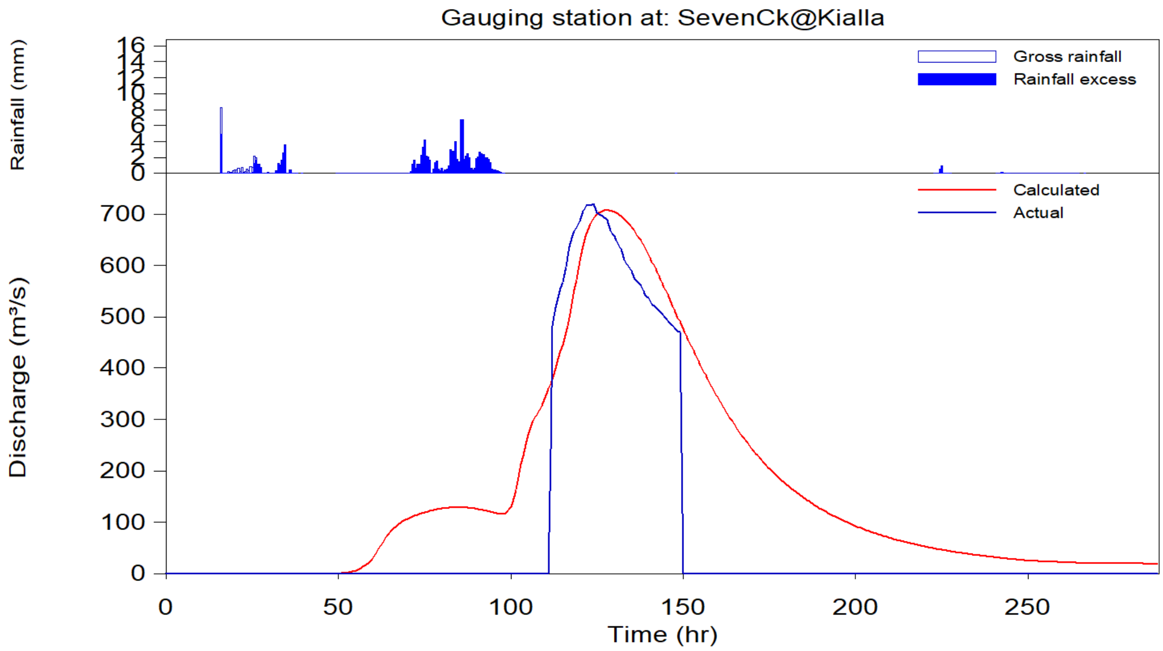

Gauging station at: HughesTarcombe_405228
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
0
50
100
150
200
250
300
Time (hr)

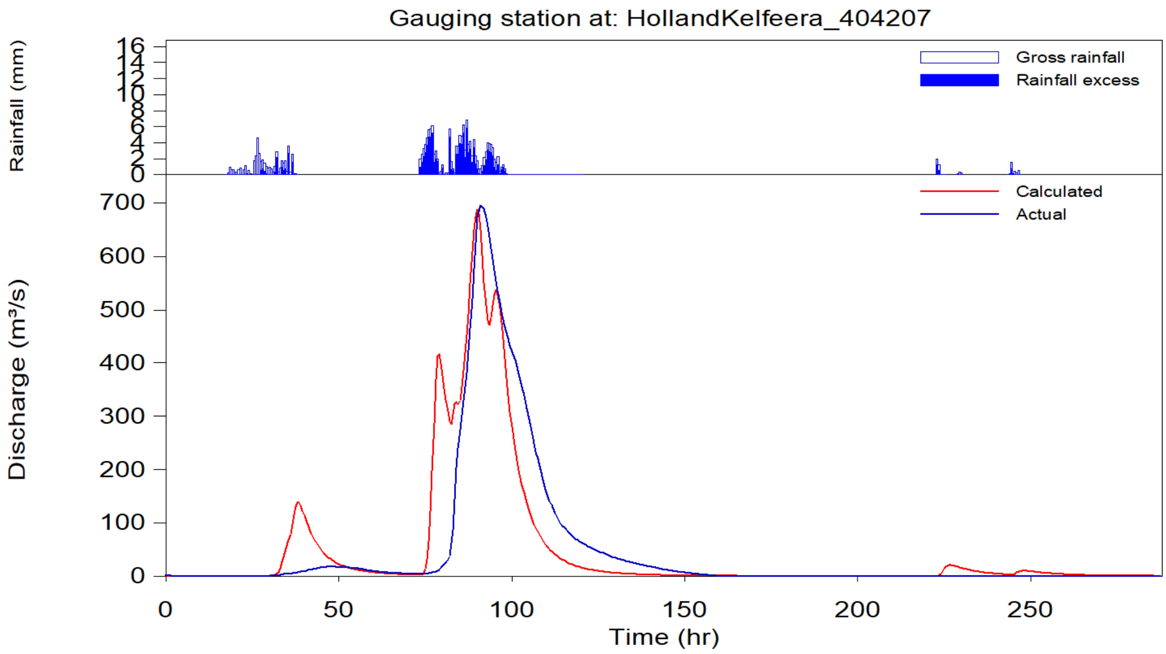

Gauging station at: MajorCk_Greytown
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
0
2
4
6
8
10
12
14
16
18
20
22
24
50
100
150
200
250
Time (hr)

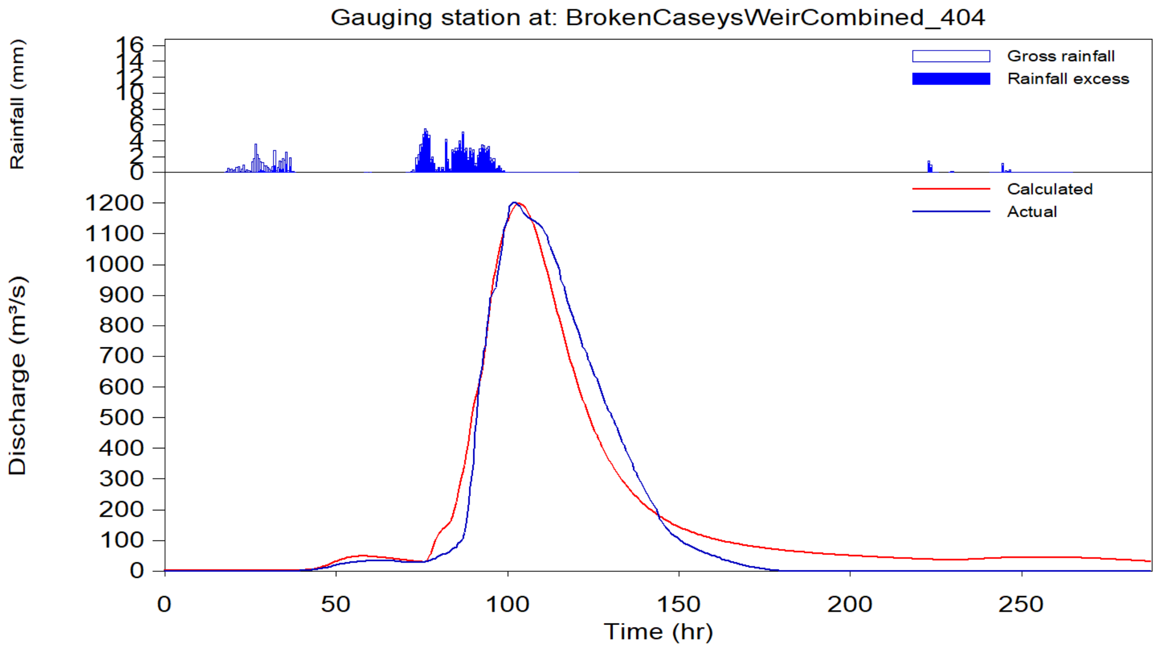

Gauging station at: GoulburnMurchison_405200
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

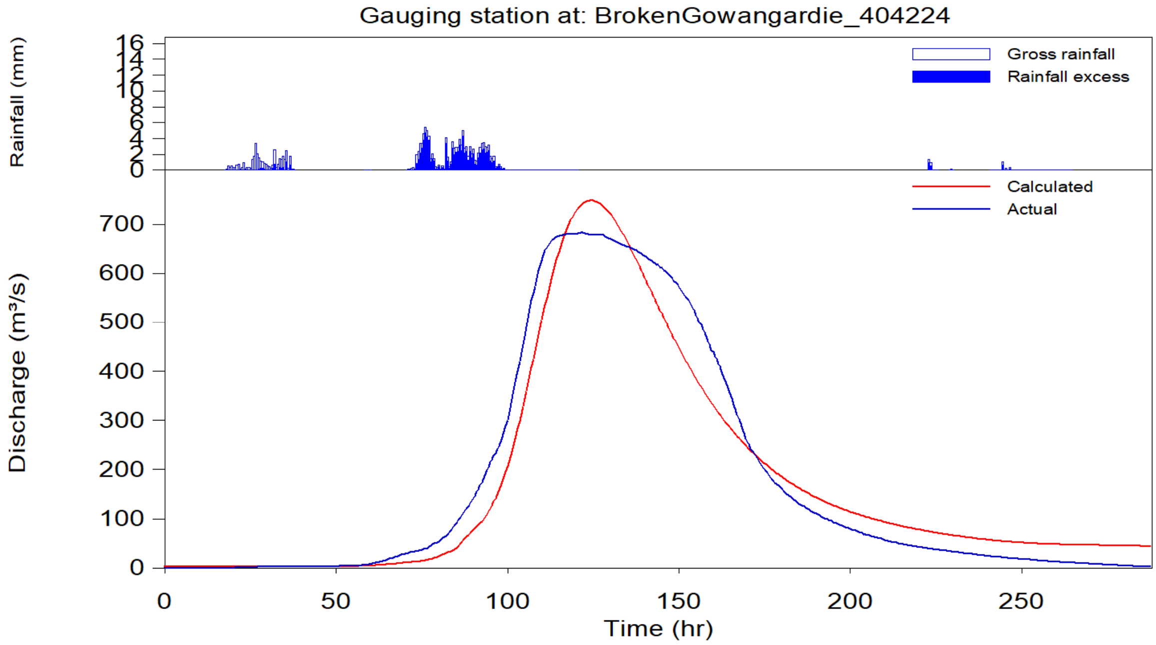

Gauging station at: PranjipMoorilim_405226
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

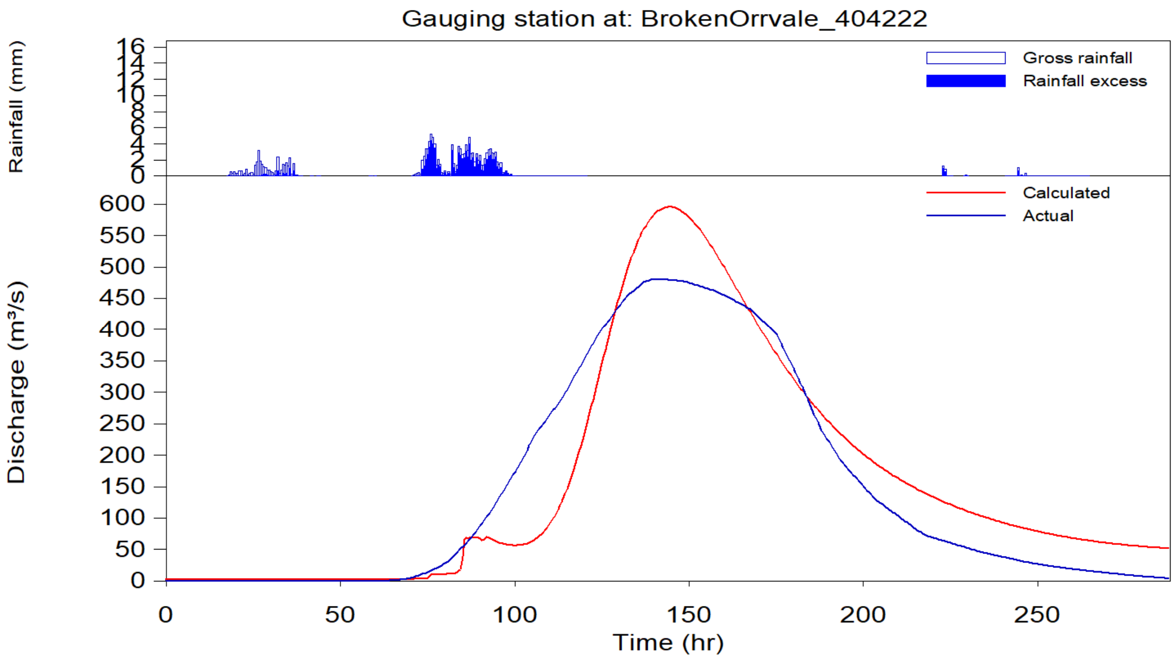

Gauging station at: CastleCreek
Rainfall (mm)
Discharge (m³/s)
Gross rainfall
Rainfall excess
Calculated
Actual
Time (hr)

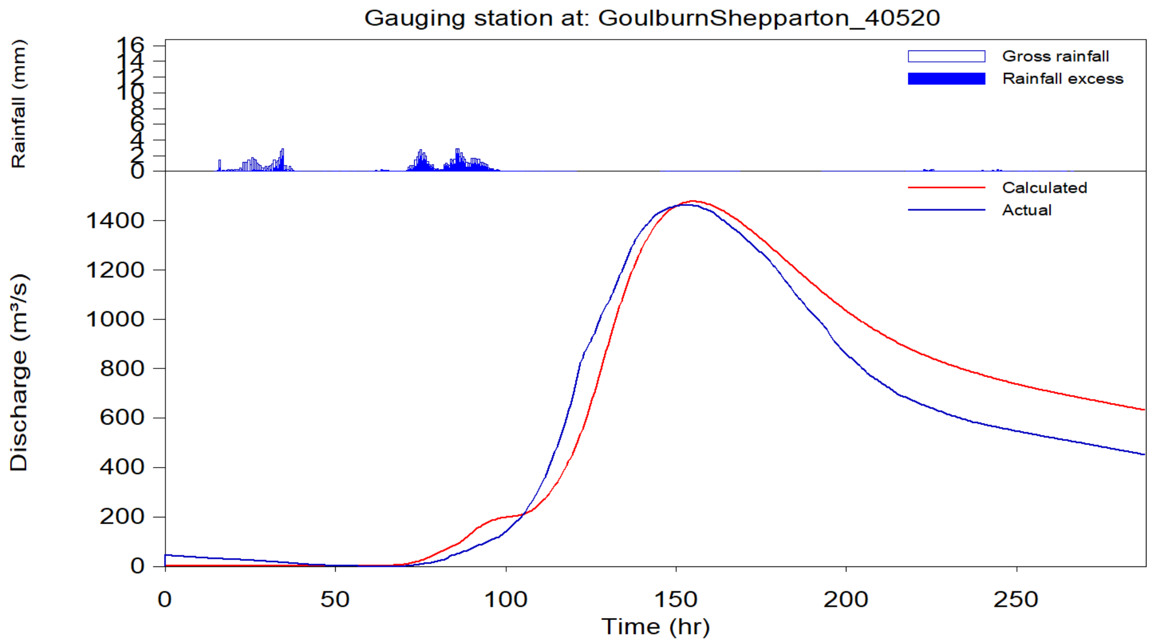

Gauging station at: SevenCk@Kialla
Rainfall (mm)
Discharge (m³/s)
Gross rainfall
Rainfall excess
Calculated
Actual
Time (hr)


Gauging station at: HollandKelfeera_404207
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

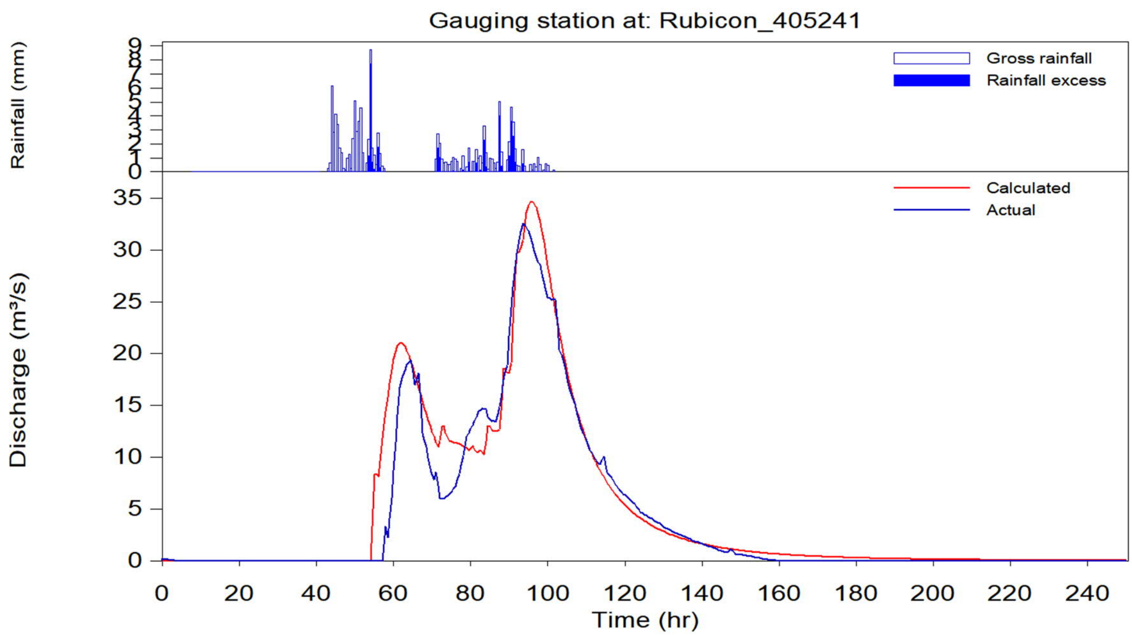

Gauging station at: BrokenCaseysWeirCombined_404
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

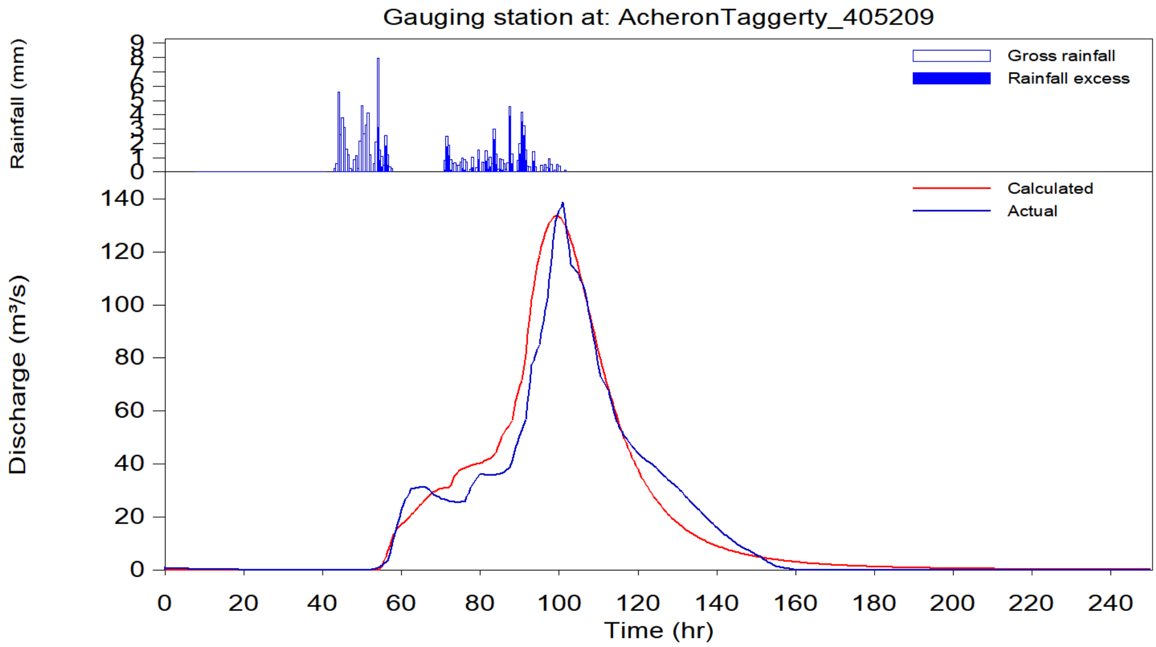

Gauging station at: BrokenGowangardie_404224
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

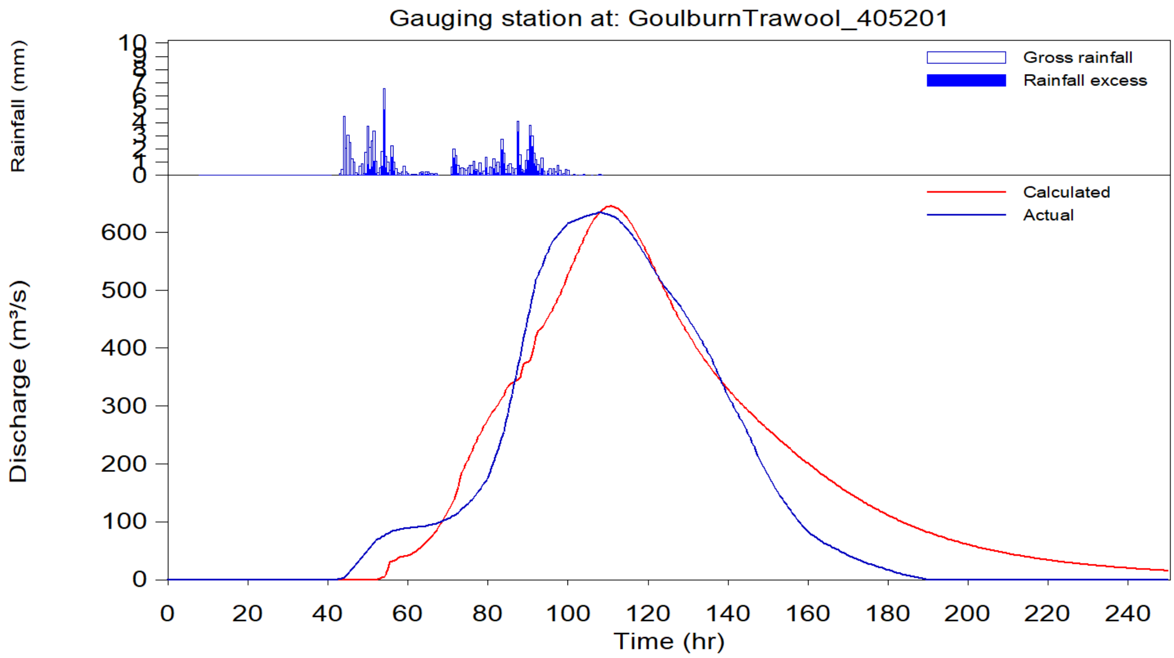

Gauging station at: BrokenOrrvale_404222
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

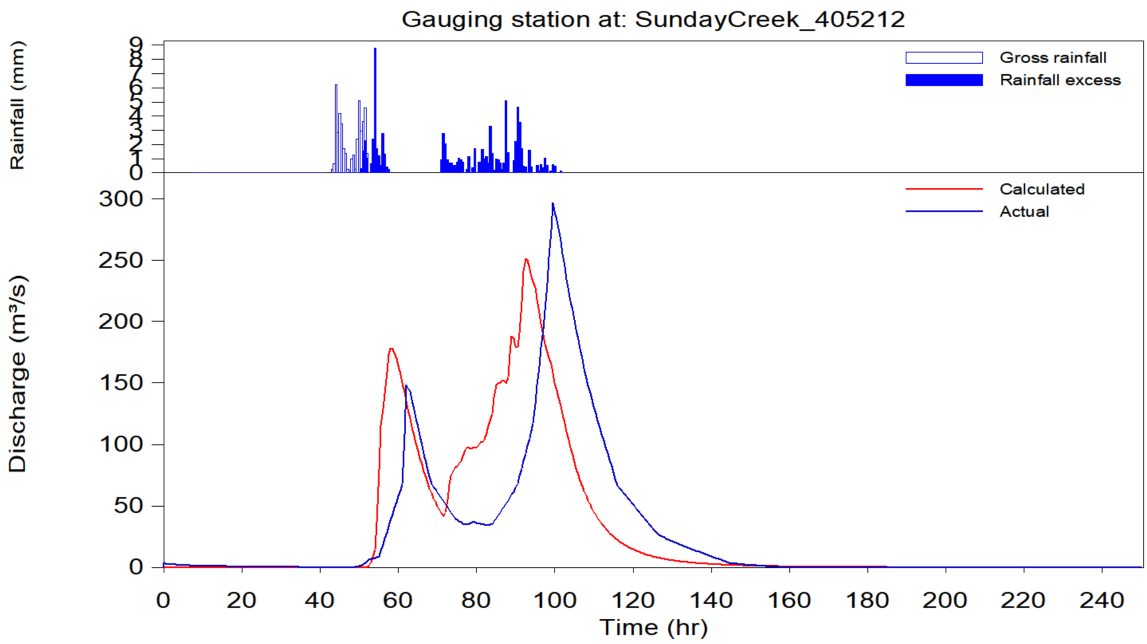


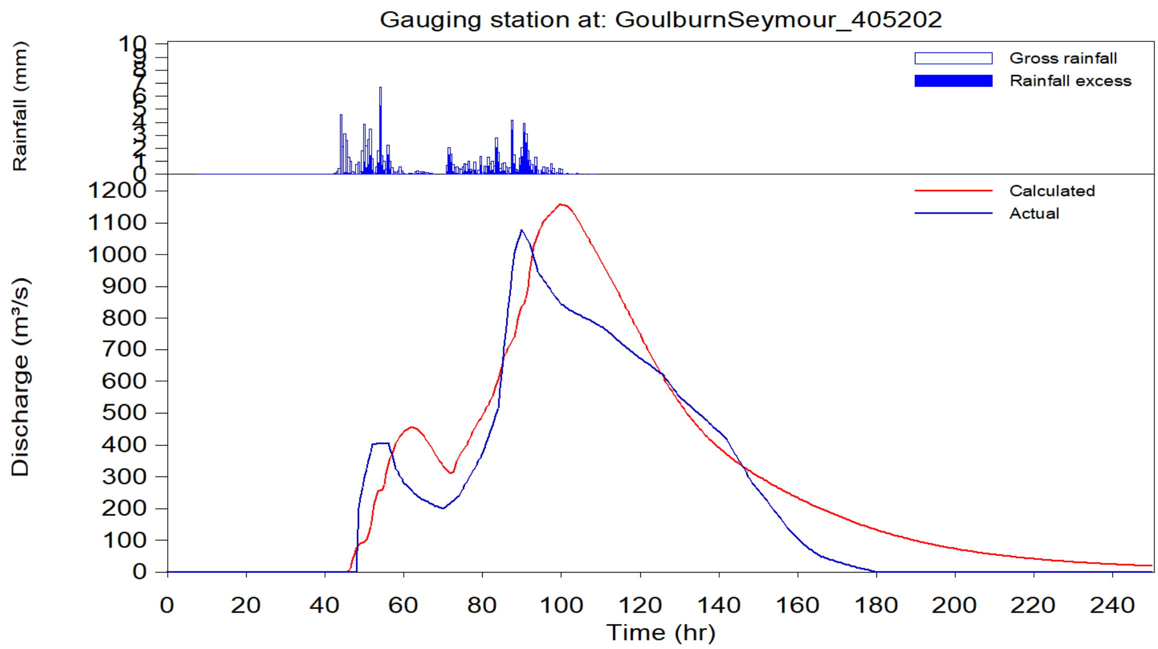
May 1974 event

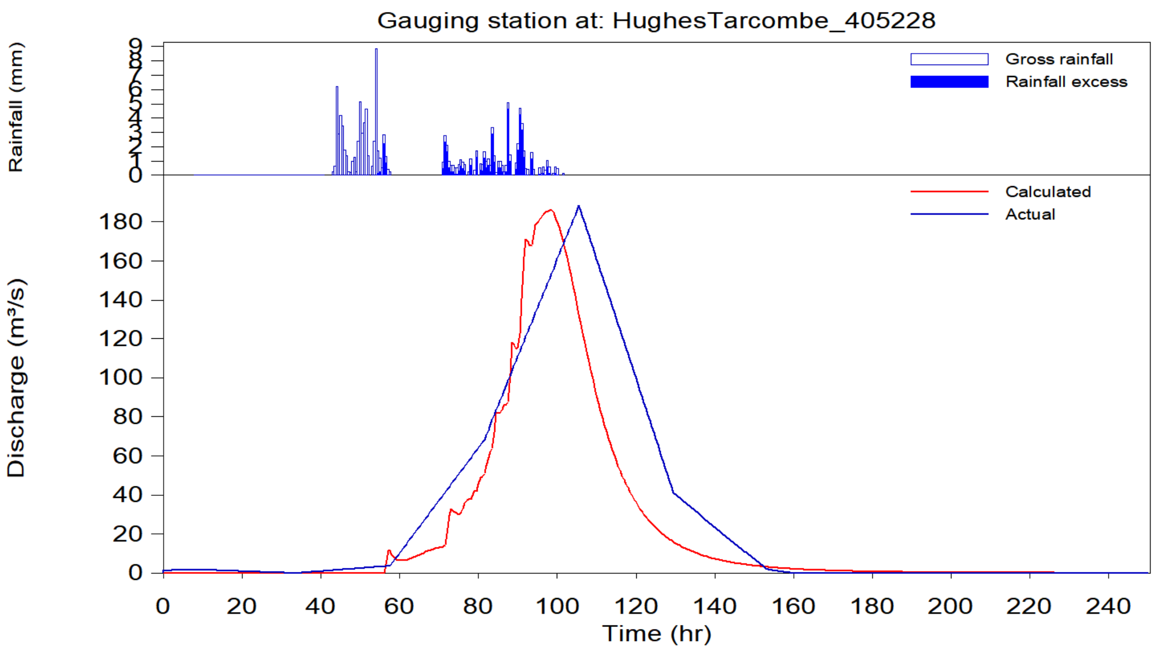


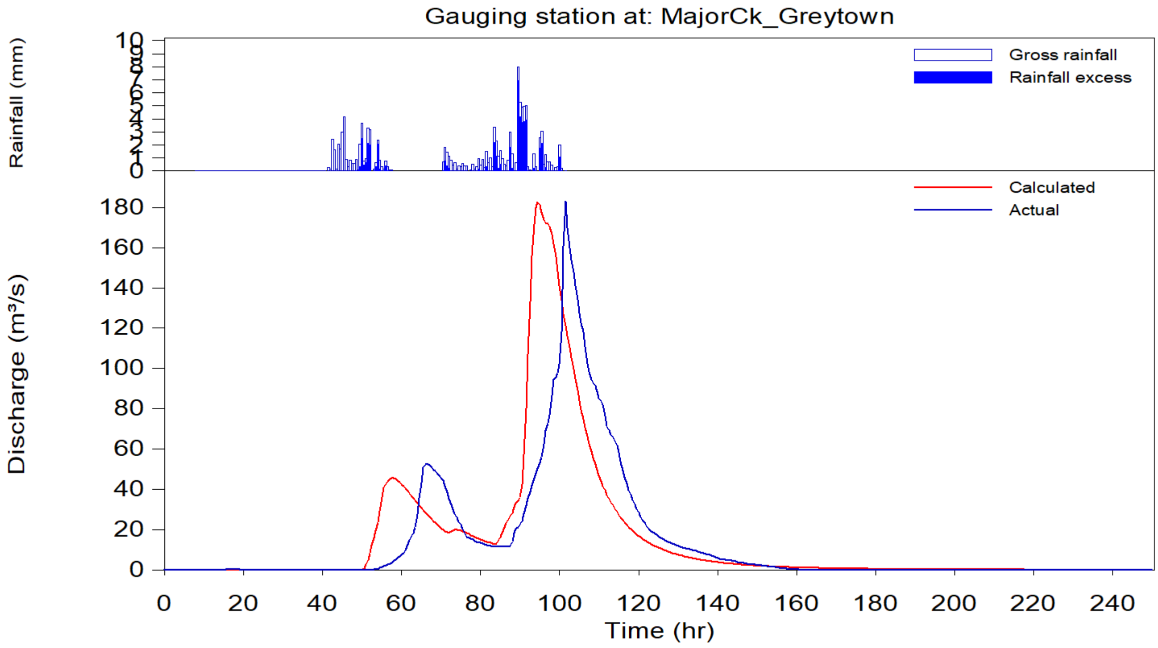

Gauging station at: AcheronTaggerty_405209
Rainfall (mm)
Discharge (m³/s)
Gross rainfall
Rainfall excess
Calculated
Actual
Time (hr)

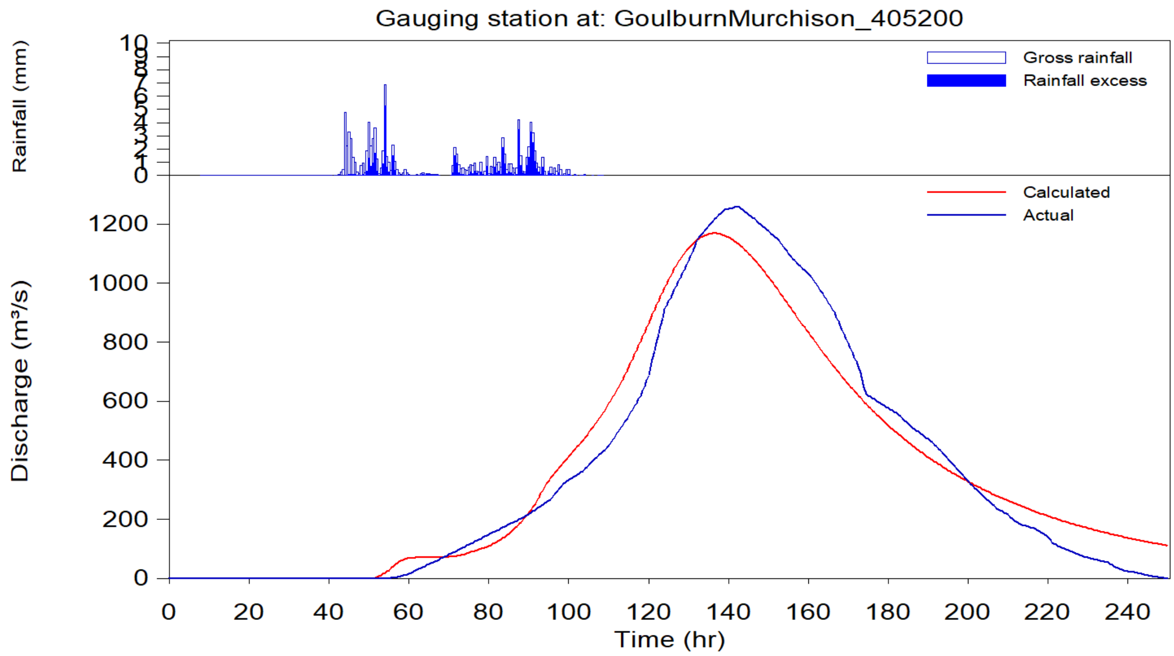

Gauging station at: GoulburnTrawool_405201
Rainfall (mm)
Discharge (m³/s)
Gross rainfall
Rainfall excess
Calculated
Actual
Time (hr)

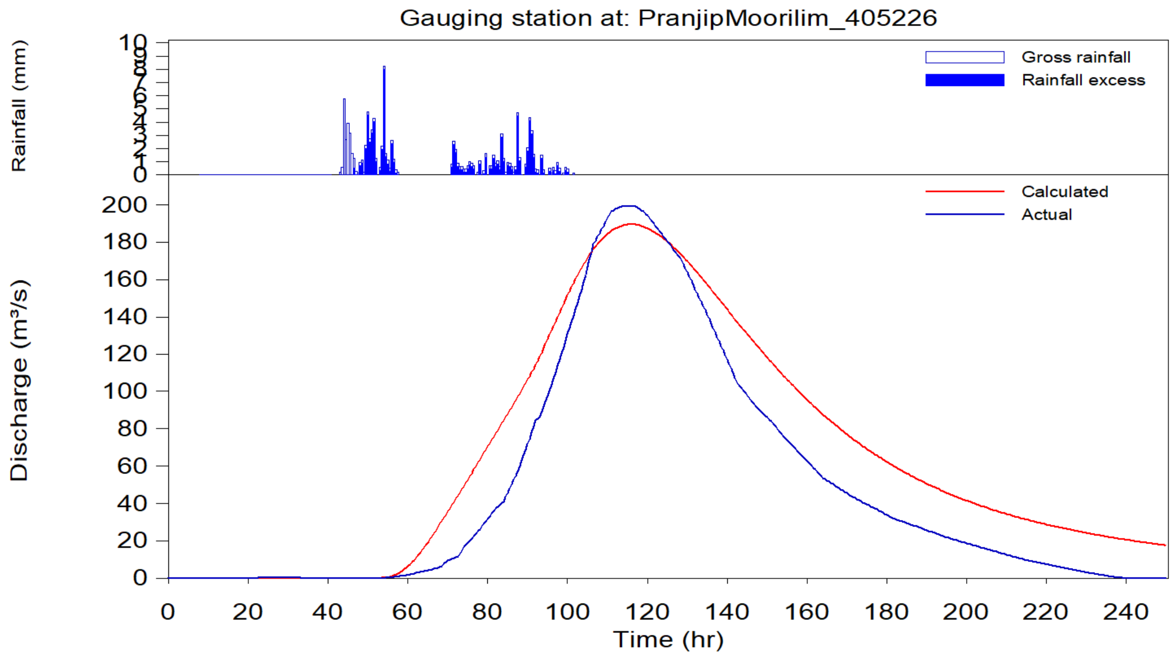

Gauging station at: SundayCreek_405212
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
0
50
100
150
200
250
300
0
20
40
60
80
100
120
140
160
180
200
220
240
Time (hr)

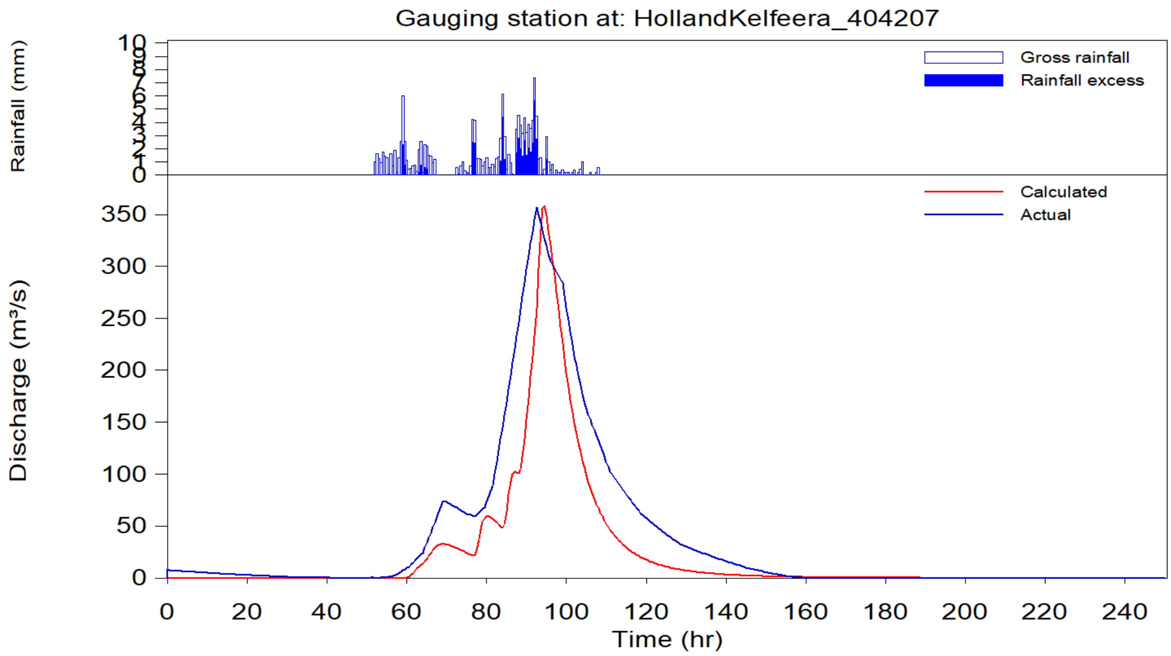

Gauging station at: GoulburnSeymour_405202
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
0
20
40
60
80
100
120
140
160
180
200
220
240
Time (hr)

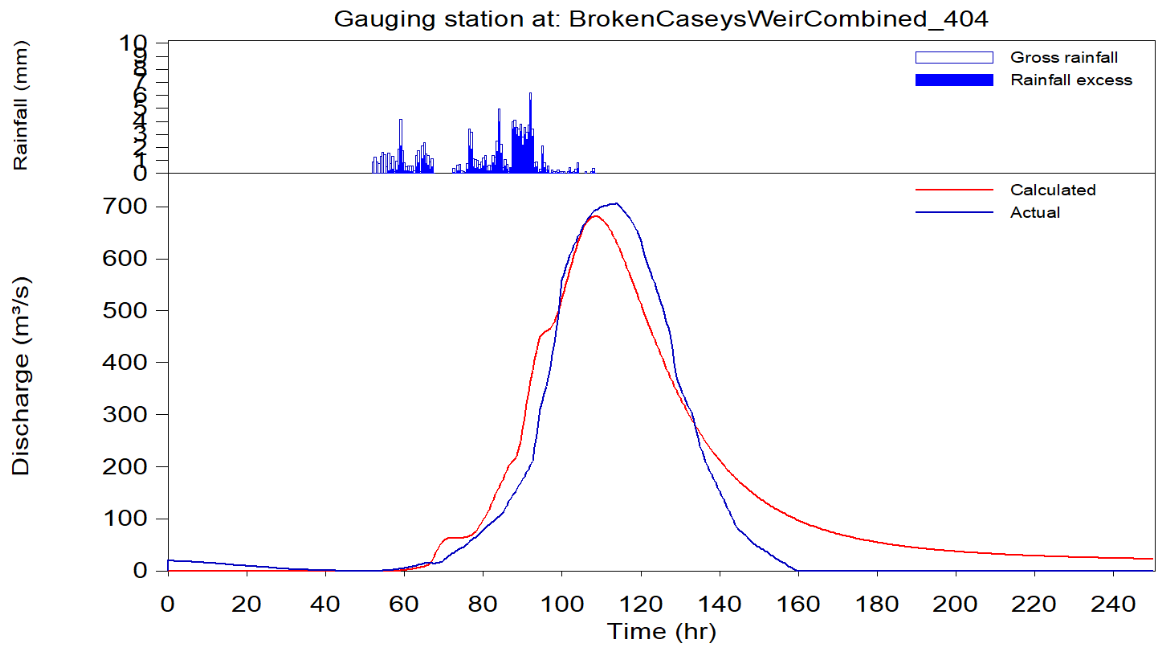

Gauging station at: HughesTarcombe_405228
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
0
20
40
60
80
100
120
140
160
180
0 20 40 60 80 100 120 140 160 180 200 220 240
Time (hr)

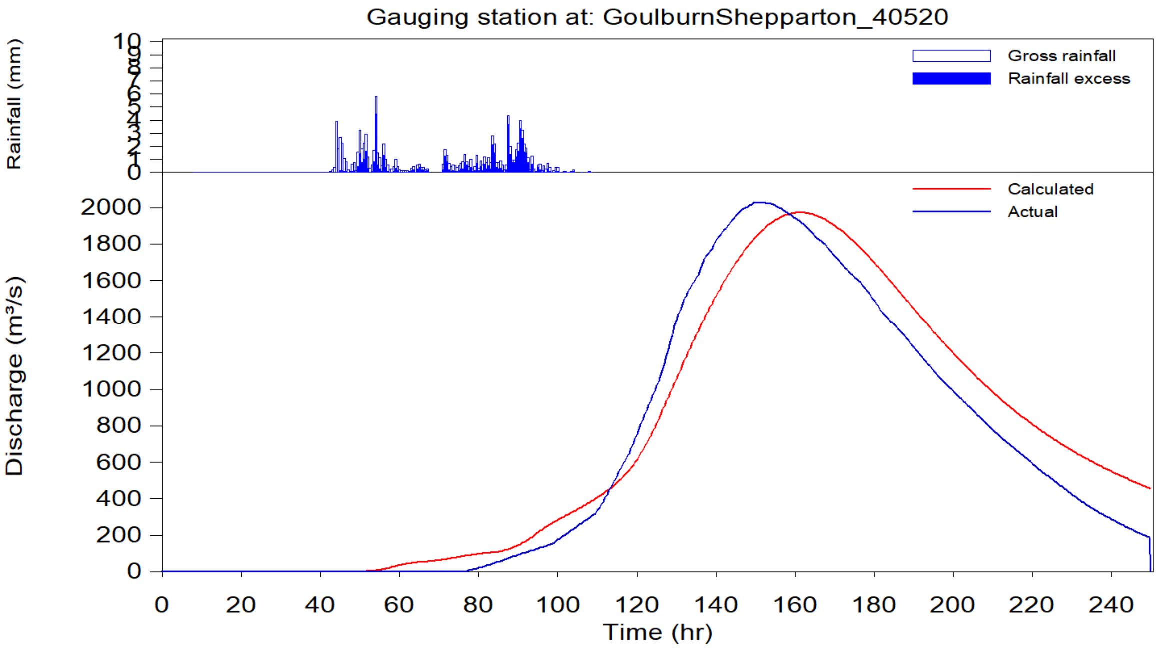

Gauging station at: MajorCk_Greytown
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
0
20
40
60
80
100
120
140
160
180
0 20 40 60 80 100 120 140 160 180 200 220 240
Time (hr)


Gauging station at: GoulburnMurchison_405200
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)

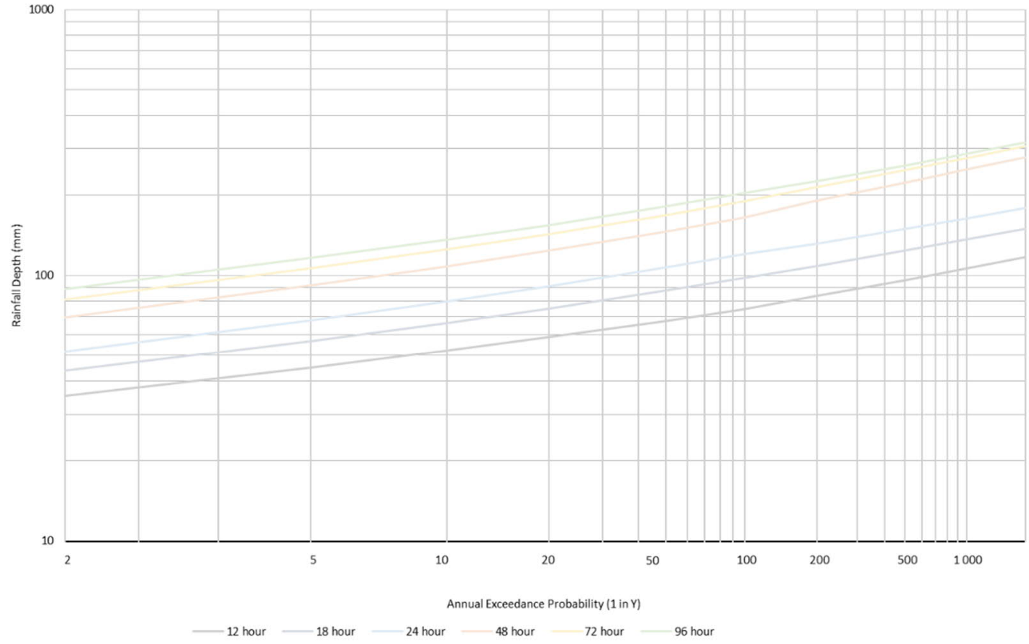

Gauging station at: PranjipMoorilim_405226
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)


Gauging station at: HollandKelfeera_404207
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
0
50
100
150
200
250
300
350
0
20
40
60
80
100
120
140
160
180
200
220
240
Time (hr)

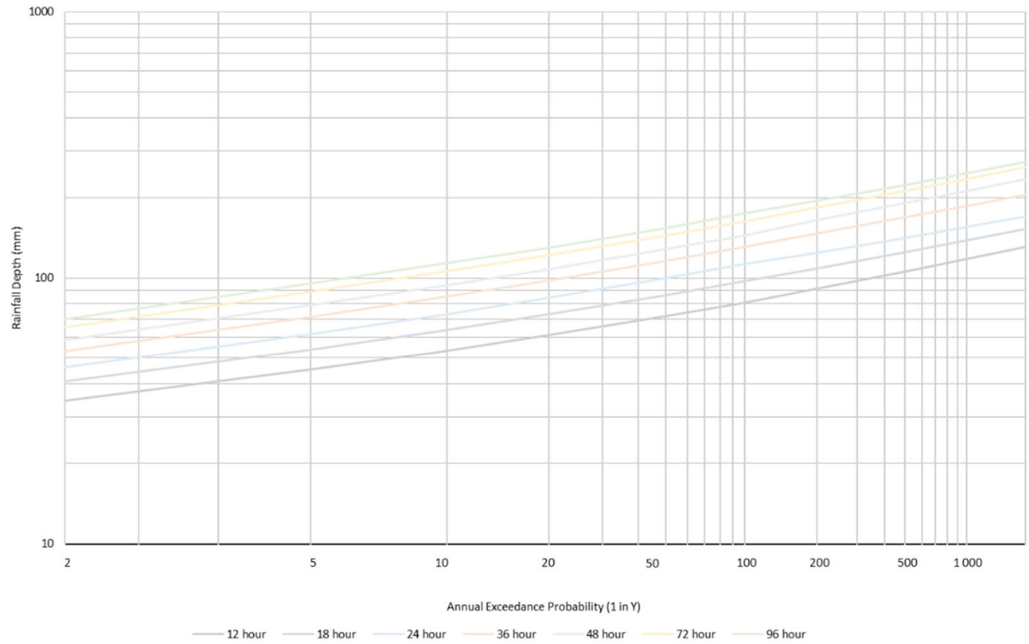

Gauging station at: BrokenCaseysWeirCombined_404
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
0
100
200
300
400
500
600
700
0
20
40
60
80
100
120
140
160
180
200
220
240
Time (hr)


Gauging station at: GoulburnShepparton_40520
Rainfall (mm)
Gross rainfall
Rainfall excess
Calculated
Actual
Discharge (m³/s)
Time (hr)
0
20
40
60
80
100
120
140
160
180
200
220
240
2000
1800
1600
1400
1200
1000
800
600
400
200

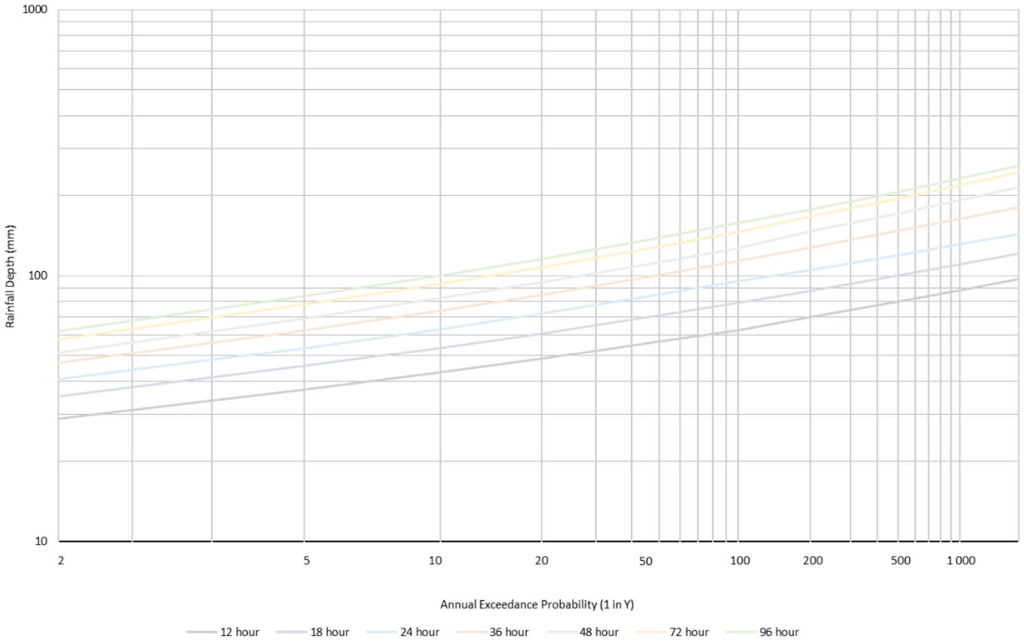
Appendix D Design rainfall depths



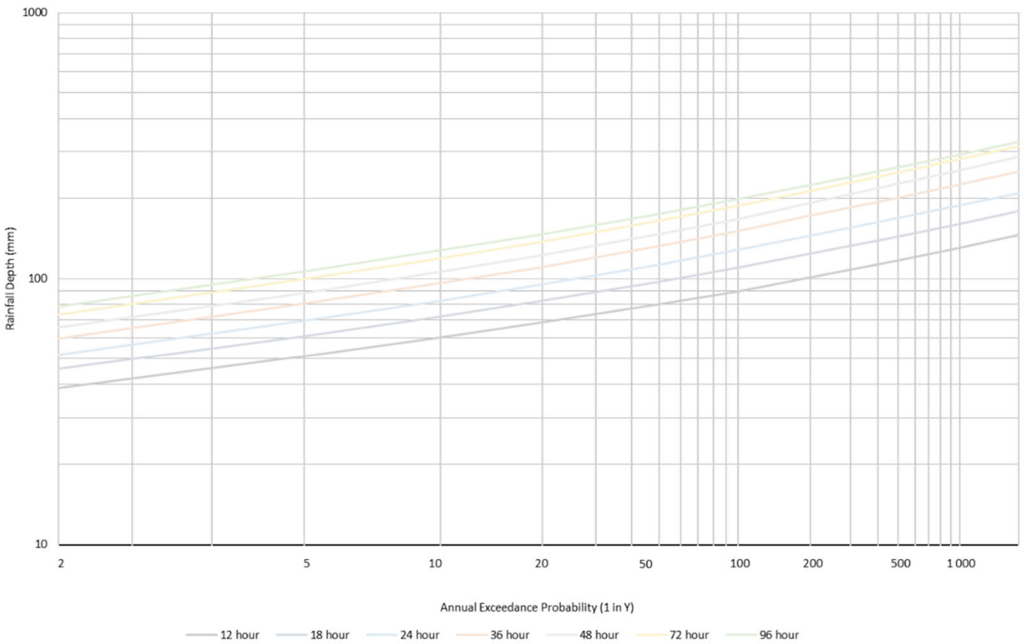
- Figure D-1 Design rainfall frequency curve – Goulburn River at Trawool



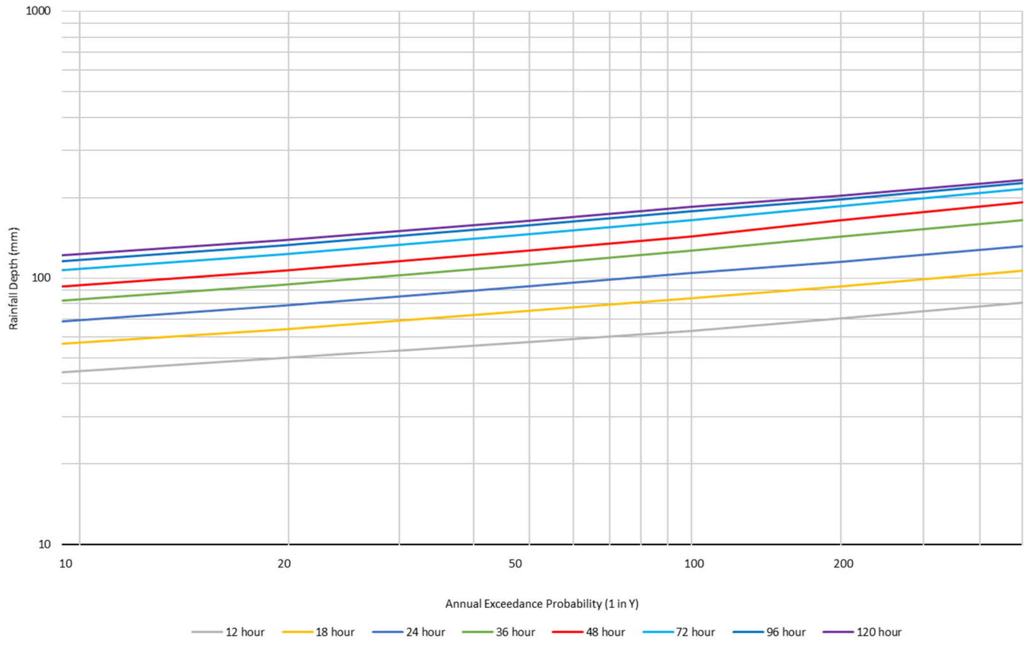
- Figure D-2 Design rainfall frequency curve – Goulburn River at Seymour



- Figure D-3 Design rainfall frequency curve – Goulburn River at Murchison

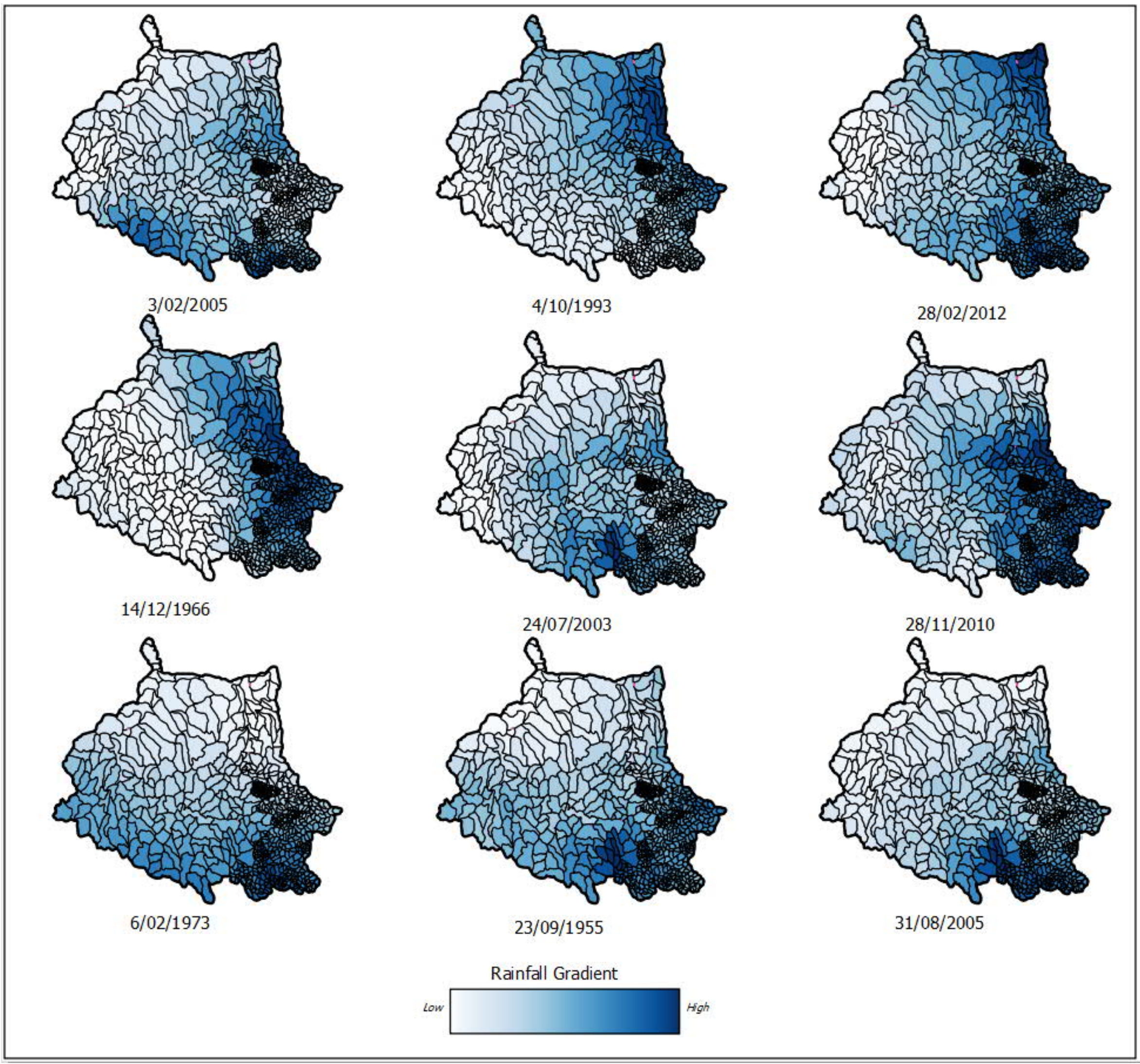


- Figure D-4 Design rainfall frequency curve – Broken River at Casey's Weir

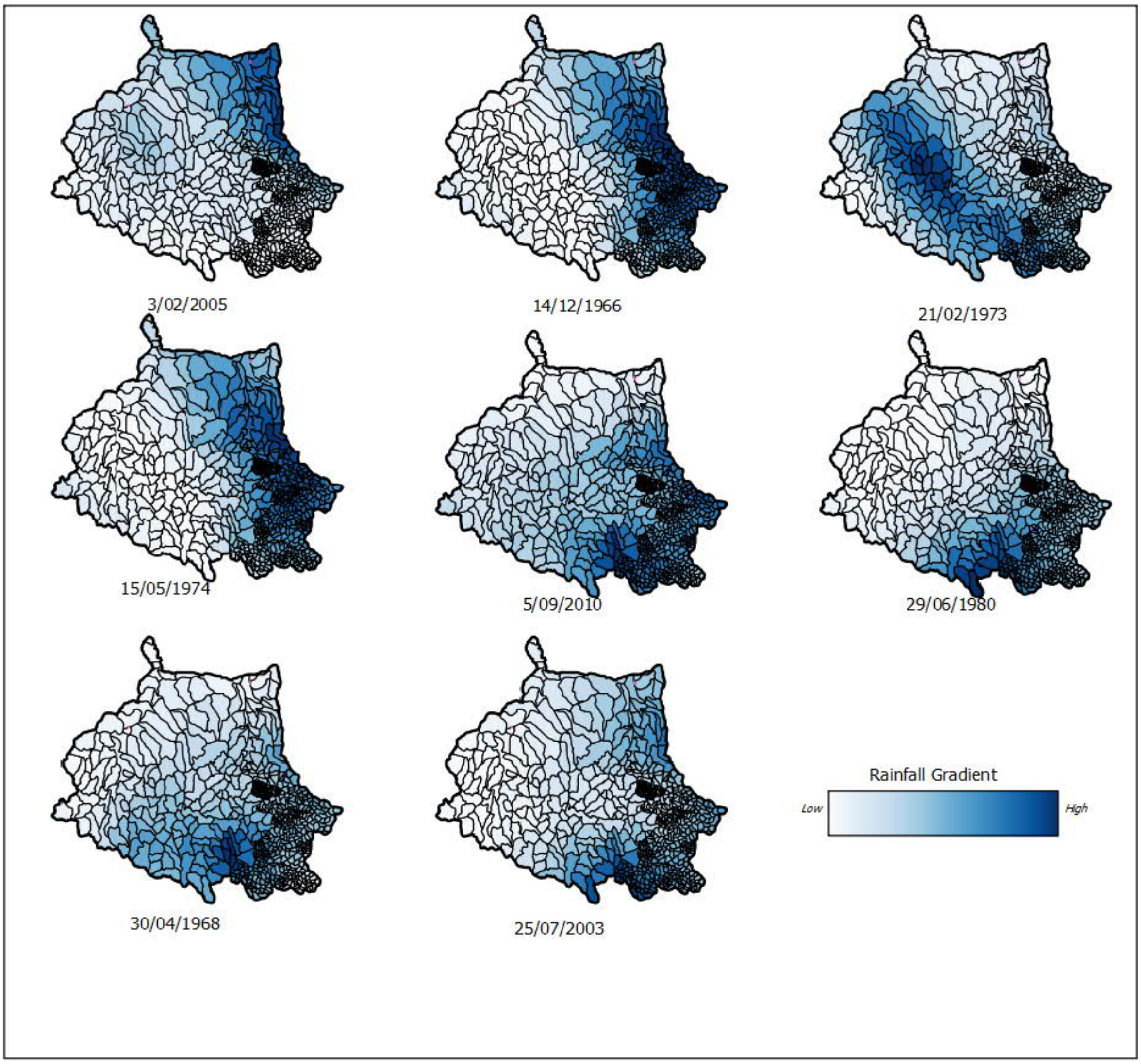


- Figure D-5 Design rainfall frequency curve – Goulburn River at Shepparton

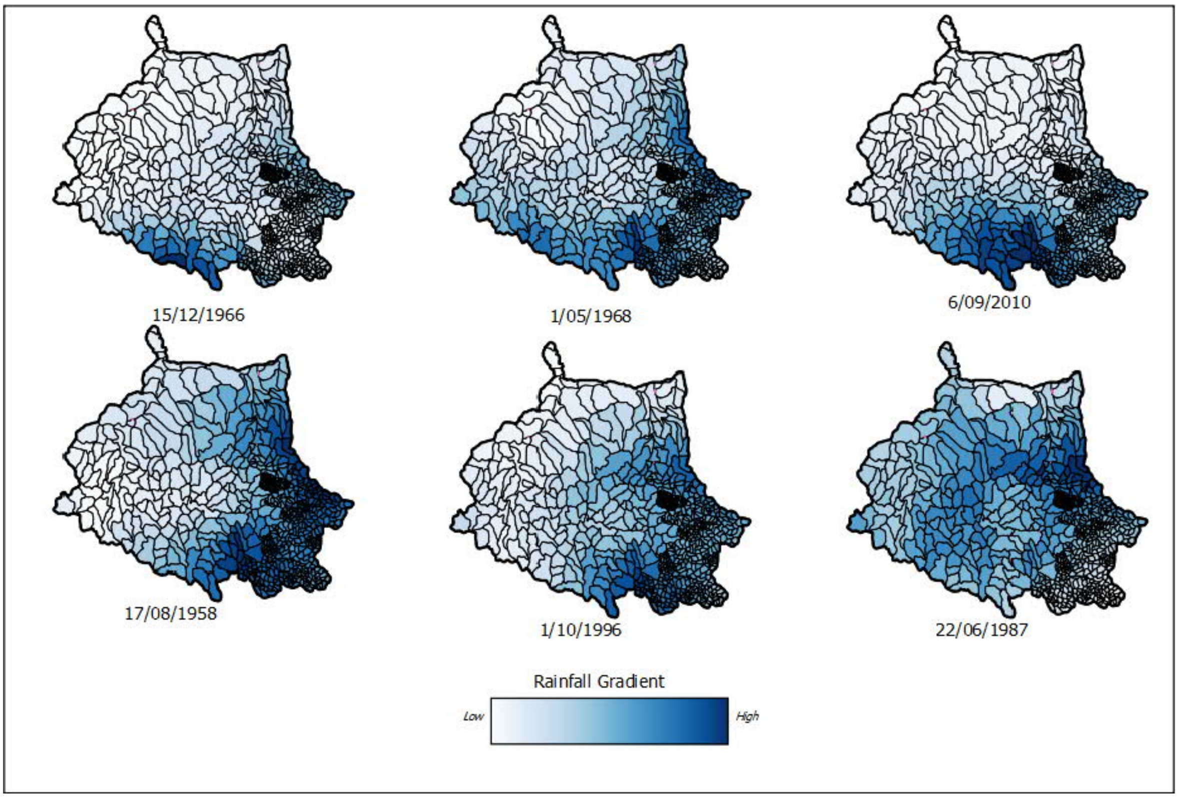
Appendix E Design rainfall space-time patterns



▪ Figure E-1 Shepparton 24 hour design rainfall space-time patterns – spatial pattern



- Figure E-2 Shepparton 48 hour design rainfall space-time patterns – spatial pattern



- Figure E-3 Shepparton 72 hour design rainfall space-time patterns – spatial pattern

Appendix F Design baseflow data

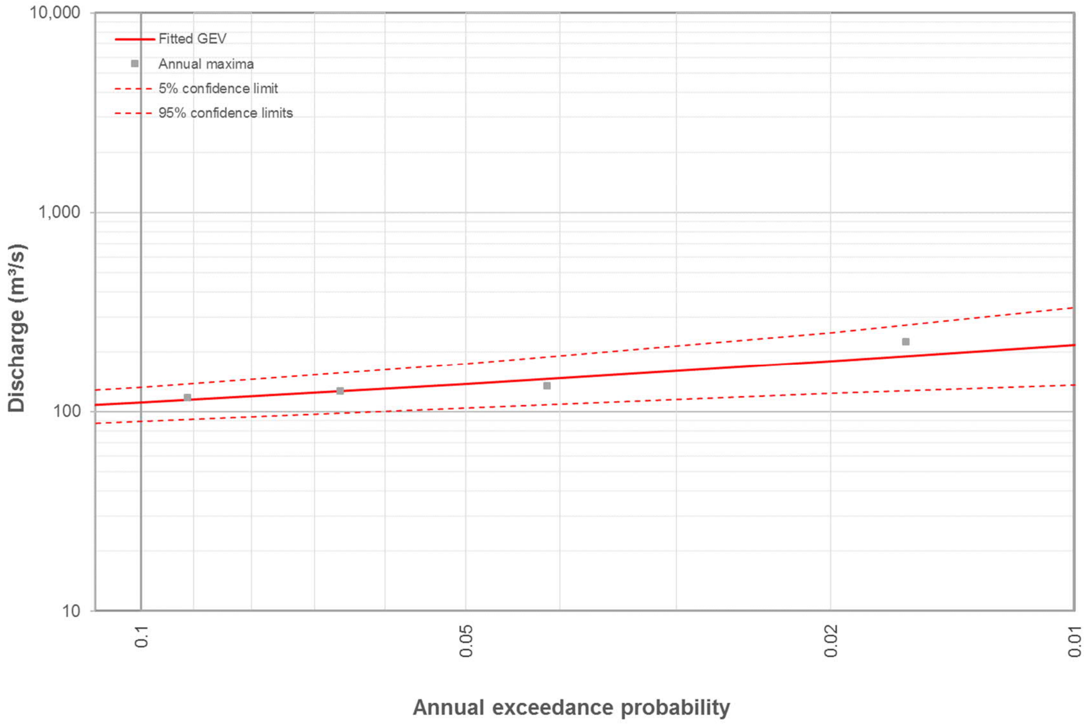


Figure F-1 Flood frequency analysis fitted to gauged annual maxima baseflow associated with gauged annual maxima flows for Goulburn River at Trawool

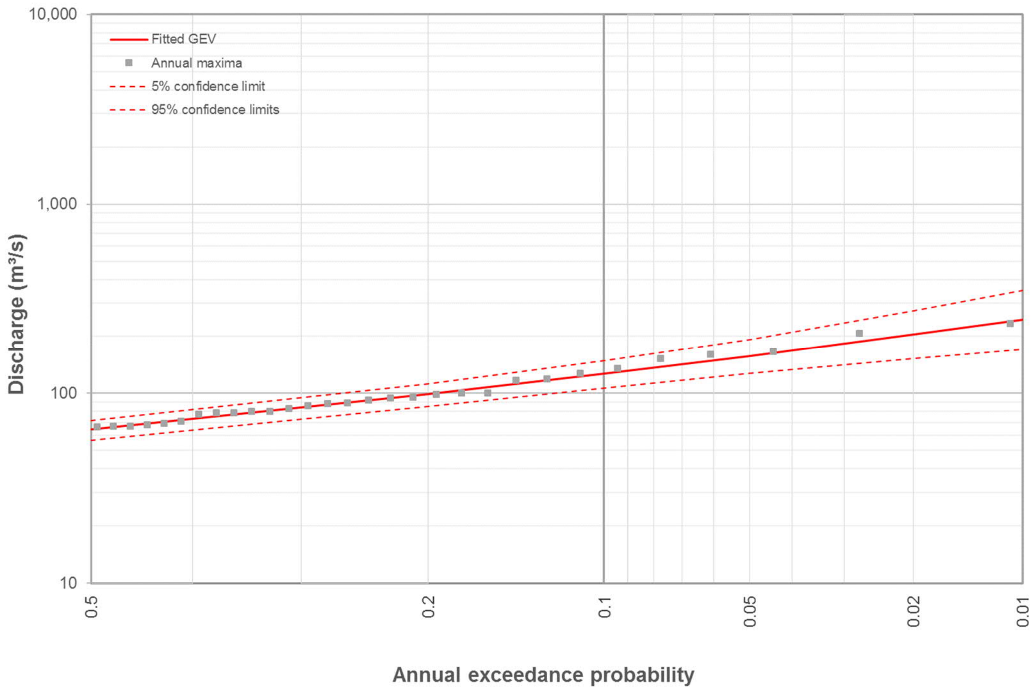
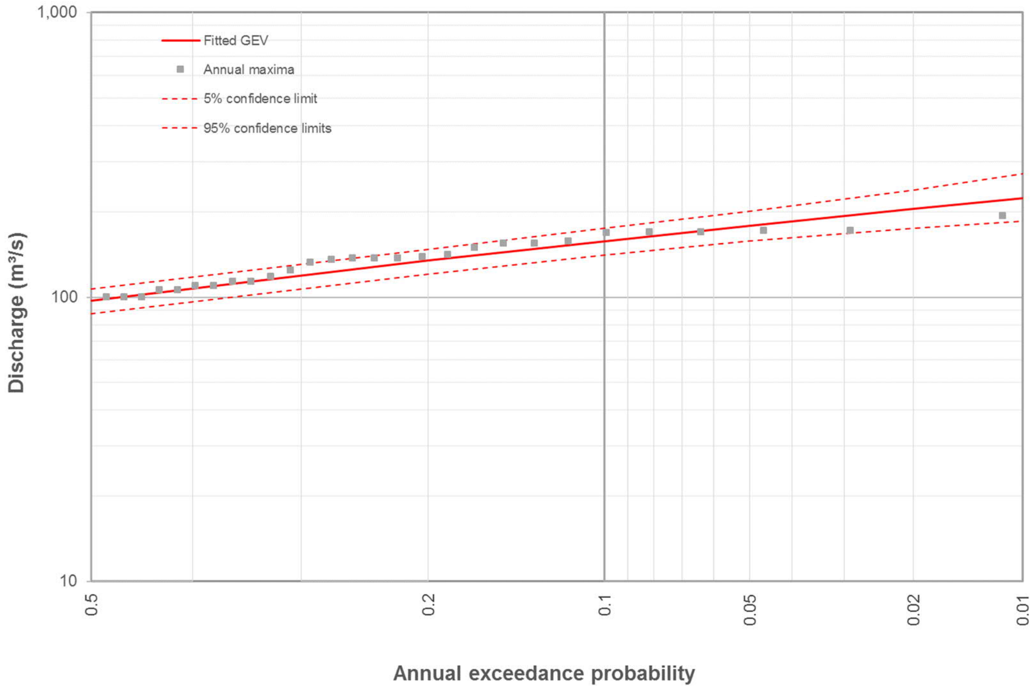
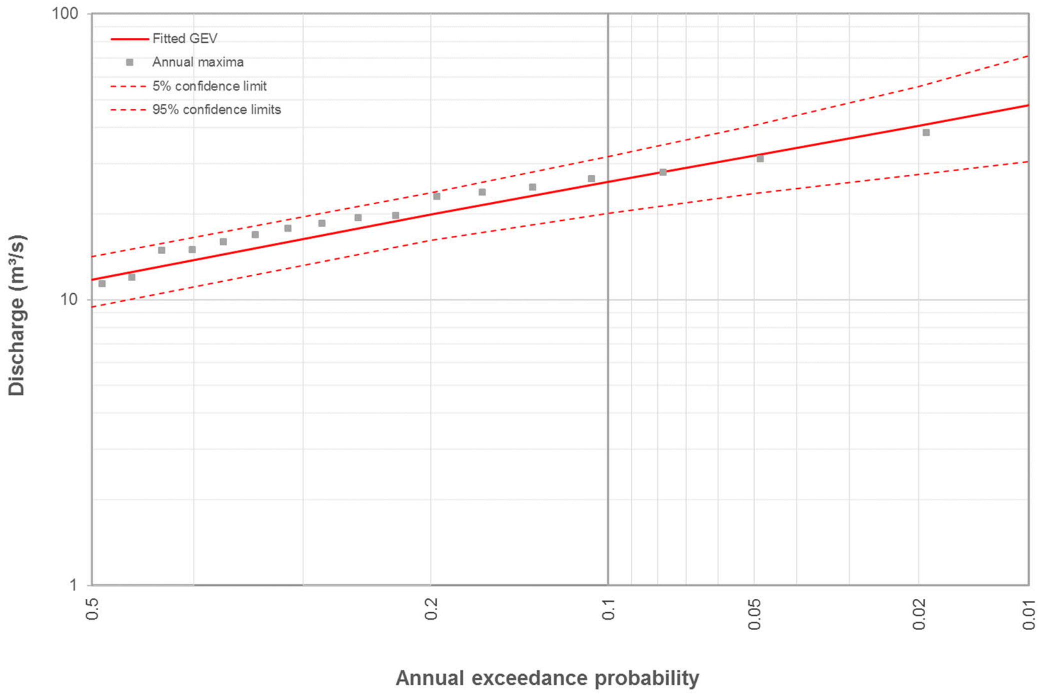


Figure F-2 Flood frequency analysis fitted to gauged annual maxima baseflow associated with gauged annual maxima flows for Goulburn River at Seymour



- Figure F-3 Flood frequency analysis fitted to gauged annual maxima baseflow associated with gauged annual maxima flows for Goulburn River at Murchison



- Figure F-4 Flood frequency analysis fitted to gauged annual maxima baseflow associated with gauged annual maxima flows for Broken River at Casey's Weir

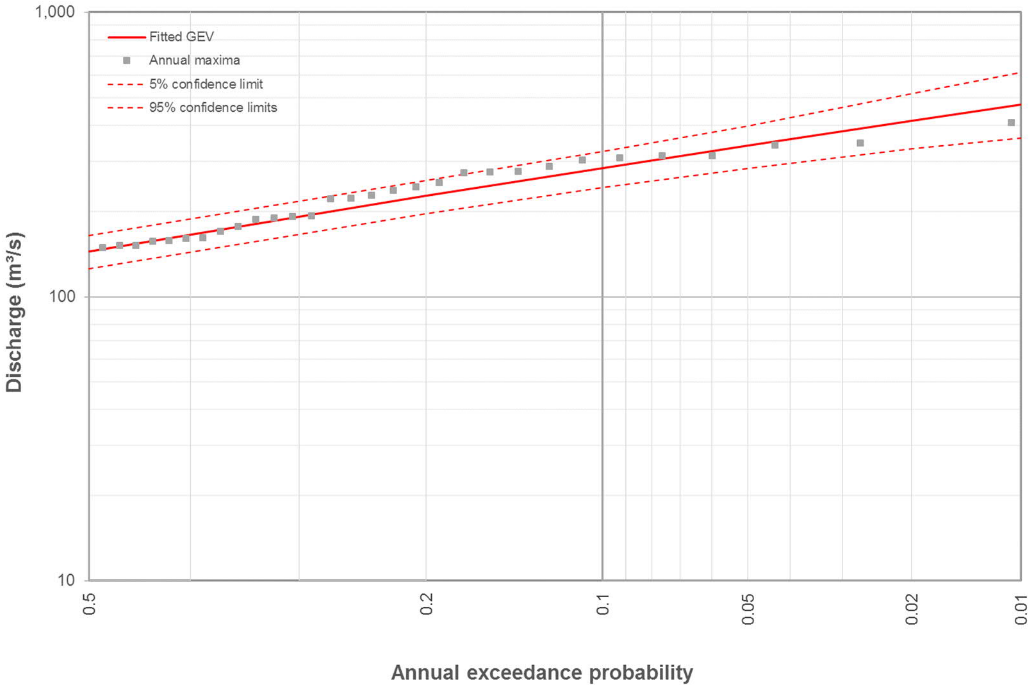


Figure F-5 Flood frequency analysis fitted to gauged annual maxima baseflow associated with gauged annual maxima flows for Goulburn River at Shepparton