Design of North Warrnambool Floodplain Management Plan Implementation Works

RM2208 v1.0 FINAL

Prepared for City of Warrnambool

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Cover image: *Plan view of Warrnambool (Google Earth Pro)*

EXECUTIVE SUMMARY

Cardno has been engaged by the Warrnambool City Council and the Glenelg Hopkins Catchment Management Authority to undertake the "Design of North Warrnambool Floodplain Management Plan Implementation Works". To develop the Floodplain Management Plan implementation works, the following tasks were undertaken:

- Hydrological assessment of the Merri River and Russell Creek Catchments
- Hydraulic Modelling of the Merri River and Russell Creek Catchments
- Detailed analysis of the 1946 Merri River flood event
- Delineation of preliminary Flood Planning Zones
- Flood Damage Assessment for the Merri River and Russell Creek Catchments

The project has identified the design flood levels and depths expected in the Merri River and Russell Creek floodplains. These can be found in figures 6.1 to 6.6 of this report. For the 100 year Average Recurrence Interval (ARI) flood event (an event with a 1% chance of occurring in any year). The number of properties and floors flooded in the catchment is shown below. Note that the properties with overground flooding include the properties with over floor flooding.

	5yr	10yr	20yr	50yr	100yr	200yr
Properties with over floor flooding	3	7	16	47	151	246
Properties with over ground flooding	295	360	423	598	847	1069

The Annual Average Damages (AAD) for the catchment is estimated at $500,000. The damages for each modelled flood event are shown below.

	Flood Event					
Category	**5yr**	**10yr**	**20yr**	**50yr**	**100yr**	**200yr**
Residential	$682,503	$1,086,125	$1,534,937	$2,775,854	$6,806,551	$11,837,078
Commercial				$1,352,201	$1,762,440	$2,053,254
Road and infrastructure damage	$59,352	$97,090	$135,271	$180,792	$254,431	$313,854
Total	**$741,854**	**$1,183,215**	**$1,670,208**	**$4,308,847**	**$8,823,422**	**$14,204,187**

Potential floodplain mitigation strategies are to assessed in Phase 2 of this project.

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GLOSSARY

Annual Exceedence Probability (AEP)	Refers to the probability or risk of a flood of a given size occurring or being exceeded in any given year. A 90% AEP flood has a high probability of occurring or being exceeded; it would occur quite often and would be relatively small. A 1% AEP flood has a low probability of occurrence or being exceeded; it would be fairly rare but it would be relatively large.
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level.
Average Recurrence Interval (ARI)	The average or expected value of the period between exceedances of a given discharge or event. A 100-year ARI event would occur, on average, once every 100 years.
Catchment	The area draining to a site. It always relates to a particular location and may include the catchments of tributary streams as well as the main stream.
Design flood	A significant event to be considered in the design process; various works within the floodplain may have different design events e.g. some roads may be designed to be overtopped in the 1 in 1 year or 100% AEP flood event.
Development	The erection of a building or the carrying out of work; or the use of land or of a building or work; or the subdivision of land.
Discharge	The rate of flow of water measured in terms of volume over time. It is to be distinguished from the speed or velocity of flow, which is a measure of how fast the water is moving rather than how much is moving.
Flood	Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or overland runoff before entering a watercourse and/or coastal inundation resulting from super elevated sea levels and/or waves overtopping coastline defences.
Floodplain	Area of land which is subject to inundation by floods up to the probable maximum flood event, i.e. flood prone land.
Geographical information systems (GIS)	A system of software and procedures designed to support the management, manipulation, analysis and display of spatially referenced data.

Hydraulics	The term given to the study of water flow in a river, channel or pipe, in particular, the evaluation of flow parameters such as stage and velocity.
Hydrograph	A graph that shows how the discharge changes with time at any particular location.
Hydrology	The term given to the study of the rainfall and runoff process as it relates to the derivation of hydrographs for given floods.
Mathematical/computer models	The mathematical representation of the physical processes involved in runoff and stream flow. These models are often run on computers due to the complexity of the mathematical relationships. In this report, the models referred to are mainly involved with rainfall, runoff, pipe and overland stream flow.
Probability	A statistical measure of the expected frequency or occurrence of flooding. For a fuller explanation see Annual Exceedence Probability.
Risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. For this study, it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
Runoff	The amount of rainfall that actually ends up as stream or pipe flow, also known as rainfall excess.
Topography	A surface which defines the ground level of a chosen area.

1 INTRODUCTION

Cardno has been engaged by the Warrnambool City Council and the Glenelg Hopkins Catchment Management Authority to undertake the “Design of North Warrnambool Floodplain Management Plan Implementation Works”. In doing so Cardno has undertaken a review of the North Warrnambool Floodplain Management Plan and prepared designs and specifications of works to implement the Plan.

1.1 Scope of Works

This report outlines the work undertaken as part of this Design of the Warrnambool Floodplain Management Plan. The scope of work contained in this report includes:

- A comprehensive review of the relevant policies and guidelines and ensure that our approach is guided by them. Research and document existing available information of relevance to the study as part of a Gap Analysis in consultation with the Project Manager. This step will identify key data required to develop robust hydrological and hydraulic models
- Collate existing aerial and ground surveys, and undertake additional ground/river survey as required, to provide a detailed database to facilitate the preparation of a calibrated hydraulic model and flood inundation maps that meet the study requirements.
- Specify, set-up, calibrate and validate a suitable hydrologic model for application to the study area. The model will be subject to appropriate sensitivity analysis, design flood hydrographs will be produced, and an analysis and discussion of potential impacts of climate change as agreed with the Project Manager (e.g. sea level rise and change in rainfall regimes) will be provided.
- Specify, set-up, calibrate and validate a suitable hydraulic model for application to the study area. The model will be subject to appropriate sensitivity analysis, flood levels and extents will be determined, and floodway areas will be delineated. Cardno will liaise closely with the Project Manager throughout model development, calibration and validation.
- Deliver all flood related information collected and developed through the study as fully attributed VFD compliant datasets in ArcGIS Version 9.* format ready for upload to the Master Datasets. This information includes separate, clear, high-quality flood-inundation maps at modelled flood intervals, including information on properties at risk, and the preparation of flooding overlay maps (i.e. Urban Floodway Zones (UFZ), Floodway Overlays (FO), and Land Subject to Inundation Overlays (LSIO)).
- Undertake a flood risk assessment for the study area (ToR 6). This will involve developing a flood damage assessment model to determine flood damage potential at varying flood intervals, socio-economic benefits and costs, and environmental impacts.
- Examination of possible flood mitigation measures. Appropriate land use planning and building controls will be discussed, as well as the potential need for and benefits of a flood warning system.
- Review the 2006 Floodplain Management Plan.
- Design flood mitigation works. This will involve testing the current proposed works and analysing the risk. If required, Cardno will modify or design new mitigation measures similar to those currently proposed.

- Draft an urban drainage strategy that enables Council to implement it for any future development activity within the City's North East corridor development.
- Conduct hydraulic modelling with flood mitigation works in place using fully developed conditions. Determine appropriate levels of mitigation and gain approval from all stakeholders including floodplain management authorities and the general public.
- Prepare 'stand-alone' draft and final reports, maps and models describing each completed task in sufficient detail to allow further work to build on this study.

2 CATCHMENT AND STORMWATER SYSTEM DATA

2.1 Summary of Data Sources

The following data was acquired for use in the study:

- Aerial LIDAR survey (supplied by WCC, August 2009)
- Warrnambool 50cm Contours (supplied by WCC, August 2009)
- Flow Data for the Merri River at Woodford (www.vicwaterdata.net, August 2009)
- Merri River RORB model (supplied by Water Technology, August 2009)
- Various flood study reports in the area including;
    - The South Warrnambool Flood Study (Water Technology)
    - The North Warrnambool Flood Study for Merri River and Russell Creek (GHD)
    - Dennington Flood Study (Water Technology)
    - Review of Flood Studies (RJ Keller & Associates)

2.2 Site Inspection

A site reconnaissance was undertaken in order to become familiar with local topography and physical features of the site. The field inspection was carried out during September 2009. The location of significant floodplain features was noted. These included:

- Bridges
- Culverts
- Roadways

2.3 Survey Data and Digital Terrain Model

LIDAR (Aerial Laser Survey) data was supplied by WCC, enabling the development of a fine scale Digital Terrain Model (DTM) to define the existing overland drainage network.

2.3.1 Ground Survey

Cardno have undertaken ground data analysis and subcontracted Surfcoast Survey & Drafting Services P/L to conduct additional ground and river survey to facilitate the preparation of a calibrated hydraulic model and flood inundation that meet the study requirement.

2.3.1.1 Permanent Survey Marks

The Digital Elevation Model (DEM), produced from LIDAR, was compared to known permanent survey marks (PSM) in the study area. This assesses the accuracy of the LIDAR terrain model and allows for the measurement and removal of any small systematic error with reference to the true ground level. There are over 200 PSM's in the study area and from this dataset, 21 PSMs were identified with elevations given at the land surface. Their locations are shown in Figure 2.1. The location of the PSM elevation was obtained from the datasheets used to record each PSM found at the Land Channel website (www.land.vic.gov.au) and preference was given to survey marks recorded recorded within the last ten years.

The Department of Sustainability and Environment were also contacted to confirm the accuracy of the provided dataset. They advised that the LIDAR dataset had met their QA requirements for a vertical accuracy of +/- 0.1m at one standard deviation. The spatial resolution of the DTM sampled for the model is approximately 1m*1m. The data set has been thoroughly checked against known benchmark points which checks both the horizontal and vertical accuracy.

From Table 2.1, ground levels from the PSM were close to the elevation from the DEM with a mean difference of 0.020m and a standard deviation of 0.082m. The difference was within a vertical accuracy of +/- 0.1m at one sigma and was sufficient to define storage capacity, roads, and levee banks for this flood study.

Table 2.1 – PSM Survey Ground Level vs DEM Elevation

ID	Number	Name	MGA94 Easting	MGA94 Northing	Survey Height	Model Topography Data	Difference in Height
1	372907340	WANGOOM PM 734	628012.6	5753857	13.160	13.129	-0.031
2	372904680	WANGOOM PM 468	631220	5751850	11.463	11.653	0.190
3	372906350	WANGOOM PM 635	631388.3	5751543	11.224	11.326	0.102
4	372900870	WANGOOM PM 87	628111.9	5753831	15.137	15.045	-0.092
5	372906500	WANGOOM PM 650	628143.1	5754044	9.483	9.596	0.113
6	372903660	366	630836.1	5753003	27.616	27.592	-0.024
7	372905990	WANGOOM PM 599	631089.9	5753323	30.321	30.380	0.059
8	372900420	WANGOOM PM 42	630560	5752480	9.154	9.074	-0.080
9	372901370	WANGOOM PM 137	626920	5754450	9.841	9.889	0.048
10	372901380	WANGOOM PM 138	626899.8	5754435	9.807	9.860	0.053
11	372901730	WANGOOM PM 173	629652	5752724	5.947	6.030	0.083
12	372901770	WANGOOM PM 177	628823.1	5752843	11.990	11.904	-0.086
13	372901780	WANGOOM PM 178	628814.1	5752862	11.811	11.773	-0.038
14	372901920	WANGOOM PM 192	632230	5753360	33.621	33.588	-0.033
15	372901930	WANGOOM PM 193	632250	5753350	33.645	33.740	0.095
16	372903960	WANGOOM PM 396	630835.7	5752896	24.481	24.382	-0.099
17	372904370	WANGOOM PM 437	631260	5751550	11.171	11.243	0.072
18	395000360	YANGERY PM 36	628578.5	5754608	37.655	37.743	0.088
19	372905971	XWA 597/1	631203.3	5753514	28.400	28.390	-0.010
20	372906830	WANGOOM PM 683	631258	5752968	35.200	35.290	0.090
21	395000420	YANGERY PM 42	629319.2	5754509	36.200	36.116	-0.084
					Mean:		0.020
					Standard Deviation:		0.082

3 SURFACE HYDROLOGY

3.1 Available Data

There are two major waterways that lie within the North Warrnambool Floodplain Area, the Merri River and Russell Creek. While Russell Creek is ungauged, the Merri River has a number of gauged records in the Warrnambool area as listed below:

- Merri River at Woodford (236205) Victorian Water Data Warehouse instantaneous flow data from 1965-2008. Lies to the North of Warrnambool
- Merri River at Woodford (236205) State Rivers and Water Supply Commission Red Book ('The Red Book') incomplete instantaneous flow data from 1966-1981
- Merri River at Woodford (236205) The Red Book maximum mean daily flow data from 1949-1981
- Merri River at Warrnambool (236217) record from 1977-1985. Located at the downstream end of the River, west of Warrnambool
- Merri River at Dennington (236218) record from 1975-1985. Located at the Princes Hwy Bridge, west of Warrnambool

The records for the Merri River gauge at Woodford contain the longest and most appropriate flow information for use in this study.

3.2 Previous Investigations

The hydrology of the Merri River at Warrnambool has been investigated previously in a number of studies. Estimates of the magnitude of flood flows are found in the following reports:

- Flood Frequency Analysis of the Merri River (Thiess, 1999)
- North Warrnambool Flood Study for Merri River and Russell Creek (GHD, 2003).
- South Warrnambool Flood Study (Water Technology, 2007)
- Dennington Flood Study (Water Technology, 2007)

Flood analysis has been undertaken at Russell Creek in the following reports:

- North Warrnambool Flood Study for Merri River and Russell Creek (GHD, 2003)
- Russell Creek Flood Modelling (Cardno Lawson Treloar, 2007)

Cardno has undertaken a review of these studies and the findings are outlined below.

3.2.1 Thiess Flood Frequency Analysis

Thiess undertook a Flood Frequency Analysis (FFA) utilising recorded data from 1966-1998. The analysis of the flow record excluded years with peak flows less than 1,000ML/day. The results are shown in Table 3.1.

Table 3.1 – Thiess FFA for Merri River

Design Flood ARI (years)	LP3 Distribution Peak Design Flow		Confidence Limits (ML/d)	
	(ML/Day)	(m^3/s)	5%	95%
2	6,500	75	5,100	8,300
5	12,000	139	9,100	16,300
10	16,100	186	12,000	23,500
20	21,900	253	15,600	34,700
50	26,500	307	18,300	44,500
100	31,300	362	21,000	55,400

The rationale behind the decision to remove flows less than 1000 ML/d in the analysis is unknown.

3.2.2 GHD Analysis

GHD conducted an FFA using a synthetically extended time series. They applied a daily water balance model, Australian Water Balance Model (AWBM) to create a modelled time series of flow. The AWBM utilised rainfall records at 3 locations in and around Warrnambool as well as flow data from the Merri River gauge at Woodford. This allowed for the creation of a flow data series dating back to 1900, giving 100 years of flow data, including an estimation of the 1946 flood event, which is considered an extreme event in Warrnambool. The results of this are shown in Table 3.2. As Woodford is upstream of the study area, GHD scaled the design flows at Woodford by catchment size to infer peak flows on the Merri upstream and downstream of the Russell Creek confluence. Their adopted design flows are also shown in Table 3.2.

Table 3.2 – GHD FFA for Merri River

Design Flood ARI (years)	Design Flow at Woodford		Upstream of Russell Creek Confluence		Downstream of Russell Creek confluence	
	(ML/Day)	(m^3/s)	(ML/Day)	(m^3/s)	(ML/Day)	(m^3/s)
2	-	-	-	-	-	-
5	10,800	125	11,100	128	12,400	144
10	14,800	171	15,202	176	17,252	200
20	18,900	219	19,500	226	21,600	250
50	24,700	286	25,400	294	29,300	339
100	29,200	338	30,000	347	35,400	410

The analysis undertaken by GHD was reviewed by Erwin Weinmann and RJ Keller and Associates. They found that it was likely that the methodology used for the flood frequency analysis would under-predict the large flood events at the Woodford gauge.

3.2.3 Water Technology Analysis

Water Technology undertook a FFA utilising flow data sets obtained from the Victorian Water Resources Data (VWDW) warehouse. Three data sets were used, including historical flows from 1948-1974 flow data as well as average daily and instantaneous flow data from 1974-2003. The historical data series was measured once daily and hence is unlikely to include the actual daily maximum flow required in an FFA. To remedy this issue, Water Technology established a relationship between the maximum instantaneous flow for each month and the maximum computed daily average flow for each month. This relationship was found to be $y=1.3117x+1.8934$, with an $R^2=0.9803$. They assumed that the historical data series was equivalent to the calculated average daily flow and applied the relationship to the historical data series, generating daily maximum flows for an extended time series.

Using this extended time series a FFA was undertaken, the results of which are shown in Table 3.3. Flows less than 1000 ML/d were excluded from the analysis; the primary reason for this was to reduce the skew of the fitted statistical data. The 1946 flood event was not included in their analysis due to it being a statistical outlier as defined by AR&R. Given the length of known record the 1946 event was large enough to be classified as a clear high statistical outlier by AR&R.

The flows at Woodford were considered to be representative of flows in the Merri River at Warrnambool. Water Technology acknowledges that at the study site there is additional catchment area downstream of the Woodford gauge, however stipulate that RORB modelling and inference from catchment geometry would suggest that flood peaks are not likely to vary significantly.

Table 3.3 – Water Technology FFA for Merri River

Design Flood ARI (years)	LP3 Distribution Peak Design Flow		Confidence Limits (ML/d)	
	(ML/Day)	(m^3/s)	5%	95%
2	6,290	73	5,061	7,817
5	12,455	144	10,008	15,501
10	17,235	199	13,485	22,034
20	22,270	258	16,583	29,900
50	29,360	339	20,015	43,056
100	35,070	405	22,136	55,552

These results are not significantly different from the adopted flows from the North Warrnambool Flood Study. Hence, for consistency, they adopted the design flows for the Merri River downstream of the Russell Creek confluence as defined in the GHD North Warrnambool Flood Study. The Weinmann review considered the FFA developed by Water Technology to be more robust than the GHD report.

3.3 Recommended Approach

The study brief and the Weinmann and Keller Review (2006) indicated that there was scope for improvement in the methodology used in the generation of the design flows for the determination of flood levels and extents in the North Warrnambool region. Weinmann and Keller (2006) indicate that the approach used by GHD is likely to under-predict the high flows that are important in flood analysis, in particular, the 1946 event. As the 1946 event was used by GHD to calibrate their hydraulic model, this under-estimate of the flow event is likely to lead to systemic errors when defining flood levels under design conditions.

Based on the completed studies and available data the approach proposed to determine the design flow rates for this assessment is to utilise the available streamflow data and develop an independent FFA. This FFA will then be compared directly to the estimated flood peaks obtained from the various studies undertaken on the Warrnambool region to assess the range of difference between the estimated peak events. In addition, each set of estimates from the previous studies will be attempted to be replicated to validate the peaks determined from each study.

The available data shows that the instantaneous maximum flow data at the Merri River at Woodford is available from 1966 to 2008 and this was used for the FFA. The data set was then extended back to 1949 using the recorded mean daily flows from the Red Book which includes this information from 1949 to 1981. Where possible, instantaneous maximum daily flow data will be used from the Red Book where this information shows the likely peak event for the year although the data recorded in the Red Book is sporadic over the 1949 to 1965 period.

The adopted methodology follows an assumption that the FFA will produce a more reliable peak flow estimate for the 2 to 100 year ARIs in this catchment than other available methodologies such as hydrological modelling. The FFA is utilised to generate the peak flood estimates for the full range of ARIs and the RORB hydrological model was then used to generate these event hydrographs using calibrated model parameters.

In considering historical events for use in the FFA, we note the following estimates of the 1946 flood event event:

- 469 m^3/s by GHD (GHD, 2003)
- 860 m^3/s by WaterTech (WaterTech, 2007)
- 950 m^3/s by SRWSC (WaterTech, 2007)
- 1,050 m^3/s by the Country Roads Board (later VicRoads) (WaterTech, 2007).

Due to the large range of estimates, and the fact that the ARI is unknown, this event should not be used in the derivation of the FFA. It is also clearly defined as a high outlier using the AR&R FFA methodology if the magnitude is within the 860 – 1,050 m^3/s range. However, it can be used to check that the upper end of the FFA fitter curve is representing the higher events acceptably. We note that rainfall analysis by Weinmann indicates that the ARI of the storm event that caused the 1946 flood is considered to be in the order of the 1000 year event.

3.4 Flood Frequency Analysis

Cardno has undertaken a FFA for the Woodford flow gauge using the latest available information from the VWDW. This consists of the instantaneous flow data series from 1966 to 2008. The rating table at Woodford is rated up to 8.6 mAHD, which corresponds to a flow rate of 422 m^3/s. This rating table should give accurate results for events up to this rating. Whilst examining this data a number of discrepancies were found between this flow series and that used by Water Technology.

In assessing the applicability of the data, the historical flow series from the Red Book and Blue Book were plotted against the current VWDW data and the Water Technology FFA data (from Figure 6.2 of their report) in Figure 3.1. The figure shows that there are several years where the Water Technology maximum flow rate is significantly different to the Red Book instantaneous maximum flows and maximum mean daily flow. Of particular note are the years 1950, 1954, 1972, 1976 and 1980 where the Water Technology data is much higher than the Red Book data, and 1953, 1971 and 1975 where the Water Technology data is lower. Figure 3.1 also shows that the current VWDW data is very similar to the Red Book data for the 16 years where they overlap, indicating that they are based on the same or similar measurements. The VWDW data is considered to be the most current and accurate data set for the catchment.

Figure 3.1 also shows a synthesised flow series for the years 1949-1964 using a regression relationship between the instantaneous maximum daily flow from the VWDW versus the maximum mean daily flow from the Red Book. The regression relationship, shown in Figure 3.2, was drawn from data covering the years 1965-1981 and is highly correlated. The analysis indicates that the instantaneous daily maximum flow can be inferred from the average daily flow by applying a scaling factor of 1.26. For the years of 1951, 1953 and 1964 the instantaneous maximum flow was used as the peak flows were recorded in the Red Book (this data was used in preference to the adjusted maximum mean daily flows).

An FFA was undertaken using the combined synthesised flow series (for the years 1949-1964) and the instantaneous max daily flow from VWDW (for the years 1951, 1953, 1964, 1965-2009). The combined flow series is shown in Figure 3.3.

The FFA was undertaken using the process outlined in AR&R Vol 4, Section 2.5 for fitting an annual FFA using the Log Pearson Type III distribution. An in-house Cardno program was used to process the FFA using the Bulletin 17B method and produce flood frequency curves. The AR&R method was used for removing low and high outliers, with further outliers removed at the users discretion (as recommended in AR&R).

Three methodologies of outlier removal were assessed, with the aim of reducing the skew to between -0.4 and 0.4, including:

- Classifying outliers according to the AR&R methodology in Book 4, Section 2.5, resulting in the removal of no years.
- Removing all years with maximum flows less than 300 ML/day resulting in the removal of the lowest 8 years of data (aiming to reduce the statistical skew of the data set).
- Removing all years with maximum flows less than 1000 ML/day resulting in the removal of the lowest 12 years of data (aiming to reduce the statistical skew of the data set).

The results of the analysis are shown in Table 3.4 and the associated confidence limits are shown in Table 3.5. The flood frequency plots are shown in Figures 3.6 - 3.8 respectively.

Table 3.4 – Results of FFA for Merri River

Design Flood ARI (years)	Outliers removed according to ARR methodology		Maximum flows <300ML/D removed		Maximum flows <1000ML/D removed	
	(ML/Day)	(m^3/s)	(ML/Day)	(m^3/s)	(ML/Day)	(m^3/s)
2	4,131	48	4,770	55	4,902	57
5	12,731	147	11,587	134	10,692	124
10	19,661	228	16,888	195	15,757	182
20	26,249	304	22,113	256	21,487	249
50	33,983	393	28,724	332	30,140	349
100	38,984	451	33,419	387	37,531	434

Table 3.5 – Confidence limits from FFA for Merri River

Design Flood ARI (years)	Outliers removed according to ARR methodology (ML/D)		Maximum flows <300ML/D removed (ML/D)		Maximum flows <1000ML/D removed (ML/D)	
	5%	95%	5%	95%	5%	95%
2	2,886	6,001	3,670	6,249	3,993	6,029
5	8,604	20,148	8,700	16,215	8,559	13,877
10	12,914	32,673	12,362	24,674	12,276	21,400
20	16,860	45,219	15,837	33,444	16,294	30,422
50	21,355	60,566	20,094	45,023	22,110	44,816
100	24,199	70,791	23,044	53,523	26,907	57,695

3.5 Discussion of Flood Frequency Analysis Results

Table 3.4 and Table 3.5 indicate that the removal of low flows strongly impacts the FFA results. Removing the low flows improves the confidence limits, however the magnitude of the design floods alter significantly. The threshold limits of 300 ML/day and 1,000 ML/day are arbitrary values based on the removal of low flows to provide a better fit to the Log-Pearson Type III distribution (designated by a skew value of between -0.4 and 0.4). The flood frequency plots for the three methodologies are shown in Figures 3.4 to 3.6.

Removing maximum flows less than 1000 ML/day from the dataset results in the best statistical fit to the LPIII distribution, giving a 100-yr design flood with a magnitude of 434 m^3/s. This value is greater than both the previous analyses described above, but is considered a reasonable approximation of the expected design flows and is consistent with the previous approaches. The review undertaken by Weinmann and Keller and Associates (2006) recommended the Water Technology 100 year ARI peak flow of 405 m^3/s over the GHD estimates and this corresponds well to the current estimate of 434 m^3/s found from this FFA.

A point of contention in all FFAs for the Warrnambool region was the 1946 major historical flood event. Figure 3.7 shows the current FFA estimate of the 1946 event magnitude, the 95% confidence intervals and the 4 other known estimates of the 1946 event. The ARI of the 1946 has been previously estimated from rainfall assessment by GHD to have approximately a 500 year ARI, however the Weinmann and Keller (2006) review suggest that a more plausible estimate of the 1946 ARI should be closer to the 1000 year ARI, and this has been adopted for the plot in Figure 3.7. Figure 3.7 shows that the current FFA estimate confidence limits enclose all of the estimates for the 1946 event, except the GHD estimate, which has been stated by Weinmann and Keller (2006) as being an underestimate of the event. The FFA approximates the 1000 year event at 800 m^3/s which is similar to the Water Technology estimate of 860 m^3/s.

Figure 3.7 also shows that the fitted flood frequency distribution diverges from the known existing data for the 100 year ARI event. It was thought that the estimated flood peak for the 100 year ARI was under estimated and hence, the upper section of the FFA was adjusted. The justification for fitting the upper section of the distribution was based on the understanding that the 1946 event was approximately at the 800 to 900 m^3/s flow rate. The FFA manages to fit the known (and estimated) data well and manages to predict the 1946 event and for this reason the FFA predicted peak flows has been adopted as shown in Table 3.4.

It should be noted that in this project, the 1946 flood event has not been used as a calibration event for the hydraulic model, due to the lack of gauge information, the uncertainty surrounding the flow estimate for the event and landform changes since 1946. The hydraulic model has been calibrated to other storm events (1978 and 2001) to provide greater certainty to the hydraulic model calibration. The 1946 event has been assessed using the calibrated hydraulic model by trialling various flow rates and comparing these against recorded flood levels. It was found that a flow rate of 850 m^3/s is a reasonable estimate of the 1946 flood event, after additional detailed rainfall analysis. This is discussed in section 5 below.

Other statistical distributions were explored but were found to under predict the 1946 estimate and because of this, these methods were not extensively investigated as part of the project. In addition, AR&R suggests that the Log Pearson Type III distribution is the most robust method for flood frequency analysis, especially for the higher ARI events.

3.6 RORB Modelling

3.6.1 Model Setup

The RORB hydrological model version 6.0 (Laurenson, Mein and Nathan, 2007) was used for this study. RORB calculates flood hydrographs from storm rainfall hyetographs and can be used for modelling natural, part urban and fully urban catchments. RORB is an industry standard model that has been used widely in previous studies around Victoria.

The model created for the Merri River Catchment by Water Technology has been utilised in this study and was developed as per section 6.3 of the South Warrnambool Flood Study report. The model includes the Russell Creek Catchment. The 1,018 km^2 Merri River catchment was divided into 46 sub-catchments. The catchments were delineated using CatchmentSIM, a 3D-GIS topographic parameterisation and hydrologic analysis tool (Water Technology, 2007). The catchment was considered rural and hence there was 0% fraction impervious and all watercourses were classified as natural reaches. The RORB catchment file is shown in Appendix A.

The calibration undertaken by Water Technology involved using recommended k_c and m values of 58 and 0.8 respectively. These parameters have been maintained for this assessment.

In order to justify the k_c, the k_c that is recommended from utilising the regional equation for Victoria for catchments with a mean annual rainfall >800 mm within RORB was also 58 (for the full catchment area of 1,018 km^2). However, the calibration was undertaken on the peak flows that are calibrated for the Merri River at Woodford gauge and the Merri River has a catchment area of approximately 900 km^2 to this point, when this value is used in the Victorian regional k_c equation a k_c of 55 was obtained. The regional k_c estimate validates the use of the k_c value of 58 within the model.

3.6.2 Model Validation

Cardno undertook a model validation run using the parameters from Water Technology's South Warrnambool Flood Study, as shown in Table 3.6. The 'Intensity Frequency Duration' (IFD) coefficients listed in Table 3.7 were used for the generation of design storm events and are identical to those used in the South Warrnambool Flood Study.

Table 3.6 – Merri River RORB Parameters (after Water Technology, 2007)

Design Flood ARI (years)	Catchment Parameters		Rainfall Loss Parameters	
	k_c	m	IL	CL
100	58	0.8	20	3.9

Table 3.7 – Merri River IFD coefficients (after Water Technology, 2007)

IFD Coefficient	Value
${}^{2}I_{1}$	16.0
${}^{2}I_{12}$	3.4
${}^{2}I_{72}$	0.875
${}^{50}I_{1}$	33.0
${}^{50}I_{12}$	5.8
${}^{50}I_{72}$	1.6
G	0.55
F2	4.33
F50	14.65

In order to replicate the Water Technology results, the areal reduction factor used in the RORB model needed to be set to unity (no reduction). Cardno believes that this is an unreasonable assumption, given that the general practice standard is to apply an areal reduction factor for any catchment with an area greater than 4 km^2 (AR&R Book 2). IFD values are applicable only at a point and "*it is not realistic to assume that the same intensity can be maintained over the entire area, thus some reduction has to be made*" (AR&R, Book 2). Not applying an areal reduction factor on a catchment of 1,018 km^2 significantly increases the volume of runoff in the modelled storm, increases the peak flows and changes the timing of the storm.

In light of this, Cardno undertook the RORB modelling using the Siriwardena and Weinmann method to derive the areal reduction factor for each design storm event. The RORB flows at Woodford were matched to the flows estimated from the FFA by modifying the continuing loss parameter. The associated parameters and results are shown in Table 3.8. The initial loss was set to a constant for all storm events as this loss is linked to the physical characteristics of the catchment and is not expected to vary considerably due to the various design flood ARIs. The continuing loss increases with the design ARI as part of the calibration to the design FFA and is consistent with the previous studies on Warrnambool.

The relatively low continuing loss rates for the more frequent ARIs are expected as no baseflow contribution was added to the flow hydrographs from direct runoff. As the baseflow was not added, the lower losses for the more frequent ARIs (2 to 10 year ARI) would account for this baseflow component not being directly accounted for. It should be noted that the initial loss and continuing loss are rough approximations for a complex system and are the only methods for calibration the RORB models.

Table 3.8 – Adopted Merri River RORB Parameters and Model Results

Design Flood ARI (years)	Catchment Parameters		Rainfall Loss Parameters		Flow in the Merri River at Woodford (m^3/s)		Flow @ Warrnambool North (m^3/s)	RORB Critical Storm Duration
	k_c	M	IL	CL	FFA	RORB	RORB	(hrs)
2	58	0.8	20	1.26	57	57	54	30
5	58	0.8	20	1.35	124	124	119	72
10	58	0.8	20	1.45	182	182	175	72
20	58	0.8	20	1.79	249	249	239	72
50	58	0.8	20	2.07	349	349	337	72
100	58	0.8	20	2.13	434	434	423	72
200	58	0.8	20	2.13	541	541	530	36

It can be seen from Table 3.8 that the design storm that generates the peak flow in RORB is the 72 hour event for all ARI events other than the 2-year and 200-year. The results are different to those reported by Water Technology that described the 30-hour event as critical at Warrnambool and this is due to the lack of the areal reduction factor in that modelling.

The flow rates from RORB at North Warrnambool are taken upstream of the Russell Creek confluence and are slightly reduced from those at Woodford. This behaviour corresponds to our expectations of the hydrological performance of the river system.

3.6.3 Design Flood Hydrographs

The adopted design flows for the hydraulic model are shown in Table 3.9. Plots of the flood hydrographs are shown in Appendix B.

Table 3.9 – Adopted Design Flows, Merri River

Design Flood ARI	Design Flow @ Warrnambool North (m^3/s)
2	54
5	119
10	175
20	239
50	337
100	423
200	530

3.7 Russell Creek Hydrology

Cardno has previously undertaken flood analysis of the Russell Creek catchment in 2007. A RORB model was developed to assess the expected design flows for the Russell Creek catchment. The total catchment area of Russell Creek is 32.7 km^2. A total of 17 sub-catchments were used to define the drainage properties of the catchment and are shown in Figure 3.. The RORB catchment file is shown in Appendix A.

Impervious area percentages were based on the following assumptions:

- Rural land = 0.05 (Assumed that all flows will be restricted to existing conditions as per Clause 56);
- Existing residential areas = 0.52;
- Proposed residential areas and racecourse = 0.45.

Consideration was made for developed urban areas with the Russell Creek catchment with the natural channels converted to lined urban channels as appropriate within the RORB vector to allow for the increase in runoff rate. A summary of the sub-catchment characteristics is provided in Table 3.10.

RORB allows for the modification of a number of hydrological parameters for calibration purposes including:

- Coefficient of runoff;
- Initial rainfall loss;
- Variation of the stream lag parameter ‘k_c’ (affecting the routing time of flow through a sub-catchment);
- The non-linearity factor ‘m’.

As the Russell Creek Catchment is ungauged, two assessments have been undertaken using RORB to assess the possible design flows, primarily by the modification of the adopted kc and design loss parameters. The first method used the probabilistic rational method and the other used a regional analysis technique.

Table 3.10 – Russell Creek Sub-catchment Parameters

Sub-catchment	Area (km^2)	Impervious Fraction
A	4.68	0.05
B	3.68	0.05
C	2.19	0.05
D	3.09	0.05
E	2.33	0.05
F	3.65	0.18 km^2 @ 0.52; 3.47 km^2 @ 0.05
G	3.29	0.05
H	2.15	0.05
I	0.99	0.44.km^2 @ 0.45; 0.55 km^2 @ 0.52
J	1.25	0.81 km^2 @ 0.45; 0.44 km^2 @ 0.52
K	0.86	0.52
L	1.4	0.49 km^2 @ 0.45; 0.91 km^2 @ 0.52
M	0.61	0.52
N	0.87	0.52
O	1.08	0.52
P	0.3	0.52
Q	0.25	0.52
Total area	32.67	

3.7.1 RORB assessment - Rational Method

The rational method (AR&R, 1998) was used to estimate peak catchment flows and verify the peak modelled 100 year ARI flow. The time of concentration (t_c) was determined to be 172 mins for the entire catchment using the Adams method. The overall catchment fraction impervious is 0.16 and was determined based on the information in Table 3.10

The kc used in the RORB model was found through an iterative approach. The RORB model was run using various values for kc until flows at the outlet matched those predicted by the rational method. The RORB parameters used in the modelling are shown in Table 3.11. Initial and continuous loss rates were as assumed by GHD in 2003. The Siriwardena and Weinmann method was used to generate areal reduction factors. Storms from 1 hour to 36 hours were modelled. The IFD coefficients listed in Table 3.11 were used for the generation of design storm events (AR&R Vol 2, 1987).

Table 3.11 – Russell Creek RORB Parameters

Design Flood ARI (years)	Catchment Parameters		Rainfall Loss Parameters	
	k_c	m	IL	CL
5	4.5	0.8	26	3
10	4.5	0.8	26	3
20	4.5	0.8	26	3
50	4.5	0.8	26	3
100	4.5	0.8	26	3.5

Table 3.12 – Russell Creek IFD Coefficients

IFD Coefficient	Value
$^{2}I_{1}$	15.74
$^{2}I_{12}$	3.39
$^{2}I_{72}$	0.90
$^{50}I_{1}$	30.00
$^{50}I_{12}$	5.75
$^{50}I_{72}$	1.60
G	0.57
F2	4.32
F50	14.61

Table 3.13 – Russell Creek RORB Comparison to Rational Method

		RORB		Rational Method		
Location	ARI	Peak discharge (m3/s)	Storm Duration	Peak discharge (m3/s)	Time of Conc. (min)	Percentage Difference in discharge
Russell Creek	100yr	56.9	6hr	56.4	172	1%

Table 3.14 shows the results from the RORB modelling at four key locations.

Table 3.14 – RORB Estimated Russell Creek Flows, Rational Parameters

Design Flow ARI (years)	Flow at Aberline Rd (m^3/s)	Flow at Wangoom Road (m^3/s)	Flow at Mortlake Road (m^3/s)	Flow at Merri River Confluence (m^3/s)
5	6.2	3.2	10.9	12.2
10	10.5	4.9	17.6	19.1
20	14.0	6.5	23.5	25.3
50	25.5	11.4	41.3	43.6
100	34.6	15.6	54.4	56.9

These flows are slightly lower than the GHD study (estimated 100 yr flows of 62 m3/s at Merri River), but are within a similar range. It is noted that the rational method estimate has a large error range (in the order of +/- 50%).

3.7.2 RORB Assessment - Regional Method

RORB parameters are able to be inferred from neighbouring catchments, under the assumption that neighbouring catchments are hydraulically similar. In order to be consistent with the analysis undertaken for the Merri River, this assessment has assumed that the k_c value of 58 determined above for the Merri is an appropriate value. We note that some uncertainty exists around this value.

The k_c value from the Merri River catchment was adapted in accordance with the methodology specified in the RORB manual as follows:

$$\frac{k_c}{d_{av}}(Merri) = \frac{k_c}{d_{av}}(Russell)$$

This equation yielded an estimated k_c of 6.45 for Russell Creek. Design losses (Table 3.8) were also carried over from the Merri River RORB model. This k_c value of 6.45 is similar to the k_c used by GHD for Russell Creek of 4.7 (GHD, 2003). The flows at each key location are shown in Table 3.15.

Table 3.15 – RORB Esitmated Russell Creek Flows, Regional Parameters

Design Flow ARI (years)	Flow at Aberline Rd (m^3/s)	Flow at Wangoom Road (m^3/s)	Flow at Mortlake Road (m^3/s)	Flow at Merri River Confluence (m^3/s)
5	15.9	7.1	24.7	26.0
10	20.4	8.9	32.6	34.5
20	26.3	11.2	42.3	45.8
50	36.5	15.1	56.3	61.8
100	45.0	18.6	69.5	76.9
200	54.7	22.8	84.1	93.9

The flow rates in Table 3.15 are approximately double those indicated by the Rational Method analysis for the 5-year ARI event and 25% higher for the 100-year event. These are broadly within the error range of the Rational Method and likely provide a reasonable upper estimate on the flow rate in Russell Creek. The flow rates are also higher than those assumed by GHD.

3.7.3 Regional Flow Estimation

In order to provide a comparison to other flow estimates in the Glenelg Hopkins CMA, 100-year flow estimates were examined for 24 catchments within the GHCMA. This information was obtained from a spreadsheet provided by GHCMA summarising the FFAs for catchments under the GHCMA's responsibility. These are shown in Figure 3.9. Out of these 24 catchments, only 3 catchments had an area which was similar to Russell Creek. These three catchments are all larger than Russell Creek's, ranging from 42 to 69 km^2. Figure 3.10 shows a plot of the smallest three catchments size against the 100-year ARI flow estimate. There appears to be no real trend in the regional catchments equating catchment size and flow rate for the smaller catchment areas. The plot also shows that either estimate (rational or regional) could be considered reasonable for the Russell Creek catchment.

In comments made on this report, Keller and Associates stated that *"as Russell Creek is a partly urbanised catchment, the flood estimate for this catchment should plot significantly above any regional relationship derived from generally rural catchments"* (Weinmann, 2010). In this case, the estimates of the 100 year ARI peak flow for Russell Creek are above the two trendlines which are shown in Figure 3.9.

3.7.4 Russell Creek – k_c Value Sensitivity

In order to test the sensitivity of the peak flows in Russell Creek the k_c parameter within RORB was varied by +/- 20%. By varying the k_c value within RORB, the time it takes for the peak flow to reach the outlet of the catchment is modified. A larger k_c tends to slows the flow within the catchment resulting in a hydrograph that is longer but with a lower peak flood level. The opposite is experienced if the k_c is reduced (attenuation decreased, higher peak flow with a shorter hydrograph). This sensitivity analysis attempts to quantify the uncertainty in the peak flow rates from the hydrologic model.

The Russell Creek catchment was run with varying k_c values to determine the sensitivity of the model to this parameter. Values of +/- 20% of the selected k_c value of 6.45 were used. Table 3.16 shows the peak flows obtained from these parameters.

Table 3.16 – Flows for Varying k_c Values

Design Flow ARI (years)	Flow at Aberline Rd (m^3/s)			Flow at Wangoom Road (m^3/s)			Flow at Mortlake Road (m^3/s)			Flow at Merri River Confluence (m^3/s)		
	k_c = 5.16	k_c = 6.45	k_c = 7.74	k_c = 5.16	k_c = 6.45	k_c = 7.74	k_c = 5.16	k_c = 6.45	k_c = 7.74	k_c = 5.16	k_c = 6.45	k_c = 7.74
5	17.8	15.9	14.2	8.1	7.1	6.2	28.8	24.7	21.9	29.9	26.0	23.3
10	23.6	20.4	18.7	9.8	9.0	8.1	36.3	32.6	29.2	39.2	34.5	29.9
20	30.3	26.3	24.0	12.6	11.2	10.3	45.2	42.3	38.7	50.9	45.8	39.8
50	41.8	36.5	31.8	16.8	15.1	14.0	61.8	56.3	52.2	67.2	61.8	54.3
100	51.2	45.0	39.9	20.8	18.6	17.1	76.0	69.5	65.3	82.4	76.9	68.4
200	62.6	54.7	26.3	25.4	22.8	21.1	92.1	84.1	79.9	99.2	93.9	84.4

Table 3.17 and Table 3.18 show the differences obtained against the base k_c scenario. By varying the k_c values, differences in the peak flow of up to 17% are experienced, however in terms of absolute flow this results in a change of only 4 m^3/s. Figure 3.11 plots the variance between the three k_c values. From the results obtained, it is noted that a 20% reduction in k_c results in a 6% (or 5 m^3/s) increase in peak flow at the Merri River in the 100 year ARI event. A 20% increase in k_c correlates to 10% decrease in peak flow at the Merri river in the 100 year ARI event. This falls within the expected accuracy range of hydrological models and also identifies that the model is not particularly sensitive to this parameter.

Table 3.17 – Difference in Flow from base case, k_c 5.45 (positive indicates increase)

Design Flow ARI (years)	At Aberline Rd		At Wangoom Road		At Mortlake Road		At Merri River Confluence	
	Flow Difference	% Difference	Flow Difference	% Difference	Flow Difference	% Difference	Flow Difference	% Difference
5	1.97	11	1.01	12	4.09	14	3.85	13
10	3.14	13	0.84	9	3.72	10	4.74	12
20	3.99	13	1.44	11	2.91	6	5.12	10
50	5.31	13	1.66	10	5.49	9	5.43	8
100	6.25	12	2.22	11	6.53	9	5.45	7
200	7.94	13	2.55	10	8.03	9	5.3	5

Table 3.18 – Difference in Flow from base case, k_c = 7.74 (positive indicates increase)

Design Flow ARI (years)	At Aberline Rd		At Wangoom Road		At Mortlake Road		At Merri River Confluence	
	Flow Difference	% Difference	Flow Difference	% Difference	Flow Difference	% Difference	Flow Difference	% Difference
5	-1.65	-12	-0.91	-15	-2.85	-13	-2.74	-12
10	-1.74	-9	-0.88	-11	-3.39	-12	-4.61	-15
20	-2.33	-10	-0.89	-9	-3.63	-9	-5.99	-15
50	-4.65	-15	-1.1	-8	-4.03	-8	-7.5	-14
100	-5.05	-13	-1.48	-9	-4.17	-6	-8.54	-12
200	-5.31	-11	-1.71	-8	-4.15	-5	-9.47	-11

3.7.5 Adopted Russell Creek Design Flows

It was decided, in conjunction with the CMA that the regional parameters would be used for the design runs. The flows as a result of the regional parameters are summarised in Table 3.19 at four locations along Russell Creek. The RORB model was used to determine the hydrographs (for each catchment of the RORB model as shown in figure 3.8) with the sub-catchments A-F combined at the upstream end of the model, as well as catchments H-G. All other catchments had individual inputs into the hydraulic model. Plots of the modelled flood hydrographs are found in Appendix B. In order to replicate the Russell Creek hydrograph and flows more accurately these inflows are input into the hydraulic model at discrete locations adjacent to the centroid of each sub-catchment area (for sub-catchments within the 2D model). This allows for a more accurate representation of the flood hydrograph.

Table 3.19 – Adopted Russell Creek design flows

Design Flow ARI (years)	Flow at Aberline Rd (m^3/s)	Flow at Wangoom Road (m^3/s)	Flow at Mortlake Road (m^3/s)	Flow at Merri River Confluence (m^3/s)
5	15.9	7.1	24.7	26.0
10	20.4	8.9	32.6	34.5
20	26.3	11.2	42.3	45.8
50	36.5	15.1	56.3	61.8
100	45.0	18.6	69.5	76.9
200	54.7	22.8	84.1	93.9

3.8 Consideration of Joint Flood Assessment

Given the relative scales of the Russell Creek and Merri River catchments, it is unlikely that Russell Creek will have a significant impact on the flooding in the Merri River. This is due to the Merri River flood flows reaching Warrnambool approximately 1 day after the storm event begins (see Appendix B – Figure B.1 for a comparison). Peak Russell Creek flows reach the Merri River confluence well before this time, even for the 36 hour event.

The model has been run assuming the start of each flood event is concurrent, regardless of the relative duration. The Merri River peak flood event occurs as a result of the 72 hour storm for all ARI's and at various locations on Russell Creek, the peak flow occurs in floods ranging from the 15 minute to the 36 hour durations. We have assumed that these events occur simultaneously and that the start time of each flood hydrograph is concurrent. The highest flood level, as determined by the hydraulic model from any combination of events (from a single ARI), is then given as the flood level for the property. This approach was agreed with the GHCMA.

Sensitivity analysis was undertaken on the 100 year ARI 36 hour duration event by running three models:

- Design Case – hydrographs starting concurrently regardless of peak.
- Case 1 – hydrographs peaking at identical times.
- Case 2 – Russell Ck hydrograph starts at Merri River peak.

The results of this analysis are presented in Section 7.3.

4 HYDRAULIC MODELLING

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

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

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

4.1 Hydraulic Model Development

The hydraulic models consist of two main hydraulic components:

- The channel network (1D); and
- 2D grid of the surface topography.

The establishment of these two components of the model is described in the following sections.

4.1.1 Channel System (1D)

Survey was undertaken on specific locations along Merri River and Russell Creek to obtain cross sections for use in the 1D channel network. In addition to the cross sections, the dimensions of the major bridges and culverts were captured and are included in the 1D network. The location of the cross sections and structures are shown in Figure 4-1.

A 1D channel network was developed for both Merri Creek and Russell Creek and culverts and bridges were included in the model as discrete elements.

4.1.2 Topography (2D)

The topography was defined using a Digital Terrain Model (DTM) of the region. Two topographic layers were established, a reduced grid for the calibration and a larger grid of the region for the final model runs. The reduced grid focussed on Merri River and excluded the upper reaches of Russell Creek. The grid was reduced to minimise run times to aid the calibration process and was utilised because the calibration points were all within the Merri River floodplain. The full model was established to include the upper reaches of Russell Creek to ensure the full flood area was modelled.

The dimensions of the grids are summarised in Table 4.1. The 2D model extent is shown in Figures 4-2 and 4-3 for the calibration and full model respectively. The grid size was set at 5 m as this was deemed sufficient to capture the topography and detail of the model while allowing run times to be reasonable.

The topography along the river in the 2D grid was flattened to the approximate top of bank level as the 1D network of cross sections represented the river storage. This removes the double counting of volume storage within the system and improves stability of the 1D to 2D interaction.

Table 4.1 – Topography grid size

Parameter	Calibration model	Full model
Cell size	5m x 5m	5m x 5m
Grid Cells (x direction)	858 columns	1276 columns
Grid Cells (y direction)	800 rows	842 rows

4.1.3 Hydraulic Roughness

The hydraulic roughness for the overland flow model was described using a two-dimensional roughness map of Manning's "n" values. This was developed by digitising different land-use zones from the digital aerial images within a GIS environment (MapInfo). The roughness values were set to the values as shown in Table 4.2. The final roughness grid is shown in Figure 4-4.

The roughness parameters are consistent with the values specified by Chow (1973), the manning's 'n' for the roads, residential and commercial are consistent with previous modelling experience and practices.

As discussed in the GHD review of the North Warrnambool floodplain (2009), the Manning's 'n' values in previous studies varies considerably. For the Merri River main channel previous estimates have included 0.02, 0.03 and 0.045 by GHD in their 2001 calibration within the review, WaterTech for the Dennington Flood study and Maunsell respectively. The selected Manning's 'n' value of 0.035 for the Merri River lies within the range of these estimates and corresponds to the values specified within Chow (1973) for a natural stream which has a clean, straight, full stage, no rifts or deep pool but with some stones and weeds.

The Manning's 'n' value of 0.07 for the flood plain corresponds to medium to dense brush which is considered reasonable for the Merri floodplain, GHD consider this value to be appropriate in the review of the North Warrnambool Flood Study (2009). The roughness parameter of 0.08 was adopted for the upper reaches of the Russell Creek floodplain as there is likely to be less flow path delineation within these sections of Russell Creek and images of the creek and surrounds shows medium to dense brush.

Table 4.2 – Calibrated Roughness Parameters, Mannings 'n'

Parameter	Roughness Manning's 'n'
Roads	0.018
Main river channel	0.035
River floodplain	0.07
Farmland / dense floodplain	0.08
Residential	0.2
Commercial	0.5

4.1.4 Boundary Conditions

Boundary conditions were established at both the upstream boundaries for Merri River and Russell Creek and at the downstream end of Merri River.

The upstream Merri River boundary was setup as a single 2D boundary with an inflow hydrograph representing the hydrology of the known calibration events. Where the event hydrograph was unknown this boundary was set as a steady state flow boundary, that is, it had a continuous flow entering the model until a steady state solution was reached. The topography was modified around this 2D boundary to aid the flows entering the model.

The downstream model boundary was setup as a flow-height (Q-h) boundary. The Q-h boundary was derived using the full Merri River cross section near the downstream end of the model and the Manning's Equation while assuming a uniform flow. This boundary was modified as part of the calibration to ensure it was representing the system correctly.

4.2 Calibration

The calibration was undertaken using two known events where both flow data existed and spot levels were recorded. The purpose of the calibration was to finalise the downstream Q-h boundary condition and to finalise the roughness grid.

These events occurred in 1978 and 2001 and had peak flows of 253 m^3/s and 243 m^3/s respectively. The peak flood depth estimates were provided by the Glenelg Hopkins Catchment Management Authority (GHCMA) from the Victorian Flood Database for the 1978 event and were taken from the Dennington Flood Study for the 2001 event.

In addition to these runs, the 1946 event was run to determine the approximate flow rate that matched where possible the recorded observations. These simulations were run using a low estimate of 850 m^3/s entering the model and a high estimate of 1000 m^3/s entering the model. The aim of this modelling was to provide confidence in the hydrological assessment undertaken in Section 3.

4.2.1 1978 Event

The 1978 event had a peak of 253 m^3/s and had three useable recorded flood heights on the Merri River for calibration. These points were at Wollaston Road to the east of Wollaston Bridge, At Queens Road south of the Merri River main channel and at Bromfield Street. The maximum observed levels are shown in Table 4.3 along with the calibrated flood level. For this run the 1978 flood hydrograph was used as the inflow to the Merri River and a constant flow of 15 m^3/s was passed through Russell Creek. Road-works that had occurred in the mid 1980's near St James Park along Wollaston Road required the model topography to be amended in this area to replicate the 1978 land surface.

The calibration process involved adjusting the roughness grid and the downstream boundary conditions to match the observed flood peaks at the known locations. The downstream boundary condition was estimated as a stage-discharge relationship based on the known channel geometry. The raw Manning's results were slightly modified as a result of the 1978 calibration event. The stage-discharge table was not modified after the 1978 calibration and was used for the other calibration events. The calibrated roughness parameters are shown in Table 4.2.

The calibration results show that the flood peaks are closely matched with the flood level adjacent to Wollaston Bridge. At Queens Road the modelled flood levels were also very close to the observed levels at 0.01 m below the observed levels. The most downstream location at Bromfield Street adjacent to the weir was 0.20 m above the observed level. It was thought that this surveyed level at Bromfield Street (4.51 m) was not entirely consistent with the Queen Street level of 4.79 m as the two flood levels have a difference of 0.29 m but are within 400 m of each other. It is unlikely that the Bromfield Street Weir would influence the flood levels as the weir is only 1.8 m AHD. As no information as to how the flood level had been obtained there was no method of determining which level was more accurate and the validation was accepted with the 0.20 m difference in modelled and observed flood levels at adjacent to Bromfield Weir.

Overall, the downstream boundary conditions coupled with the friction grid produced a reasonable calibration to the observed levels for the 1978 event.

Table 4.3 – 1978 Calibration Results

Location	Observed level	Model Level	Difference
U/S Wollaston Road (east of Wollaston Bridge)	5.13 m	5.12 m	- 0.01 m
Queens Road (south of Merri River)	4.79 m	4.78 m	- 0.01 m
Bromfield Street (south of Merri River)	4.51 m	4.71 m	+ 0.20 m

4.2.2 2001 Event

The calibration for the 2001 was undertaken using data obtained for the Dennington Flood Study (WaterTech, 2007). The event had a peak of 243 m^3/s and had observed levels adjacent to Woodend Rd and at upstream and downstream of Cassidy's Bridge. This event was included in the calibration as it was the most recent significant flood event and this event was also used in the Dennington Flood Study. This flood utilised the same friction grid and downstream boundary conditions as the 1978 event and form a validation of the roughness grid and boundary conditions.

The calibration was undertaken using a steady state model as the flood hydrograph was unknown. A steady state run was considered a reasonable method of assessing the performance of the model as the downstream boundary condition controls the flood flows through the model and this allows the flood event to be well represented. Although a steady state run is conservative in its application, it is unlikely that the results would vary significantly due to flood storage volume because the flood peak was up above 220 m^3/s for over 12 hours. This is reasonably approximated by a steady state run. The calibration results are shown in Table 4.4 at Woodend Road and at Cassidy's Bridge. The results show that the model is matching the observed levels well with a 0.08 m difference at Woodend Road. The results at the upstream side of Cassidy's Bridge were 0.22 m below the observed level but only 0.08 m below the observed level on the downstream side. The head loss across bridge was 0.11 m which was less than the 0.25 m of head loss from the observed levels. This could be caused by a number of factors including debris collecting at the bridge during the flood event or scour at the bridge. This is hard to represent consistently across a range of models and the predicted head loss over the bridge was deemed acceptable given that downstream of Cassidy's Bridge was outside of the model area.

Overall the calibration was slightly above the observed levels but within acceptable levels. The main unknown in the calibration was the magnitude of the flows through Russell Creek. In this calibration the Russell Creek flow were approximated at the 20 year ARI event (42.3 m^3/s), derived from the hydrologic study. Sensitivity runs have shown that the Russell Creek assumed flows can impact on the levels downstream of the Russell Creek confluence, this specifically includes all of the observed 2001 event flood levels. Given more information regarding the behaviour of Russell Creek during this event, a more robust calibration could be achieved. However, this calibration, coupled with the calibration for the 1978 event, suggests that the roughness grid and downstream boundary conditions are appropriate for the model simulations.

Table 4.4 – 2001 Calibration Results

Location	Observed Level	Model Level	Difference
Woodend Road	4.38 m	4.42 m	0.04 m
U/S Cassidy's Bridge	3.65 m	3.58 m	- 0.07 m
D/S Cassidy's Bridge	3.40 m	3.41 m	0.01 m

5 1946 EVENT ANALYSIS

As discussed in Section 3.5, the estimates of the flow associated with the 1946 event vary significantly. It has been suggested by Weinmann and Keller (2006) that this event was approximately a 1000 year event. In the Dennington Flood Study WaterTech (2007) utilised a flow of 860 m^3/s to represent the 1946 event. In order to clarify and attempt to improve the understanding on the uncertainty around the 1946 event three approaches have been considered:

- Rainfall analysis on a range of rainfall gauges to attempt to determine the ARI of the 1946 event. This can be compared to the CRC Forge estimates.
- The 1000 year ARI design rainfall will be run through the RORB model for the Merri River using a range of loss conditions to determine if the estimated 1946 event flow is supported by the RORB peak flows.
- Estimate the peak design flows using the recorded 1946 flood levels by running the hydraulic model. For this study two flow estimates were modelled at 850 m^3/s and 1000 m^3/s. These values cover the range of flow estimates and correspond to the approximate 1000 year event.

The outcome of these investigations will improve the understanding of the estimated flow rate and ARI for the 1946 event. Each assessment method will be outlined in the sections below.

5.1.1 Rainfall Analysis

In order to assess the rainfall for the March 1946 event over the Merri River catchment the 6 nearest gauges which had significant records and included the 1946 event were used. The 6 gauged used in this assessment are summarised in Table 5.1.

Table 5.1 – Rainfall gauges used in the analysis

No.	Name	Start Date	End Date	Longitude	Latitude
090016	CARAMUT (BARWIDGEE)	01/1932	12/1977	-38.0	142.5
090039	ELLERSLIE POST OFFICE	01/1905	04/1991	-38.2	142.7
090045	HAWKESDALE (POST OFFICE)	01/1884	Open	-38.1	142.3
090051	KOROIT	03/1889	Open	-38.3	142.4
090062	PENSHURST (THE GUMS)	01/1906	Open	-37.9	142.4
090084	WOOLSTHORPE	01/1884	Open	-38.2	142.5

The 6 gauges all had extensive records and as a result were not infilled. Where it was indicated that the data was accumulated over subsequent days the total rainfall was distributed uniformly across the days of accumulation. This was undertaken to ensure that no abnormal rainfall totals were included in the assessment.

Initially the rainfall totals for the 15th to the 18th March 1946 were extracted for each location and these were used to determine the peak 24, 48 and 72 hour total rainfall.

In order to estimate the ARI for each of these events, two methods were employed:

- Utilising the CRC Forge estimated peak rainfalls.
- Utilising a rainfall frequency analysis for each gauge to approximate the ARI.

CRC Forge was utilised to develop an estimate of the rainfall totals for the 24, 48 and 72 hour duration events for the Merri River catchment. The catchment area was set at 900 km^2 and the areal rainfall estimates were utilised. The rainfall totals are summarised in Table 5.2. The frequency analysis for each gauge is shown in Appendix C. The location of the gauges and the spatial distribution of rainfall for the 1946 event is shown in Figure 5.1.

Table 5.2 – CRC Forge peak rainfall estimates for the Merri River catchment

ARI (years)	24 hr event (mm)	48 hr event (mm)	72 hr event (mm)
50	81.2	103.6	114.6
100	91.8	118.3	128.3
200	103.0	133.4	143.9
500	119.1	155.3	166.5
1000	132.4	173.4	185.2
2000	146.6	193.0	205.7

The total rainfall recorded on during the 1946 event is shown in Table 5.3, Table 5.4 and Table 5.5 for the 24, 48 and 72 hours durations respectively. The ARI for each of these events was estimated from the two methods: the CRC Forge and from the rainfall frequency assessment.

Table 5.3 – Rainfall totals for the 24 hour duration 1946 rainfall event

Gauge Name	No.	Rainfall Total (mm)	Estimated ARI (CRC Forge)	Estimated ARI (Freq. analysis)
CARAMUT (BARWIDGEE)	90016	74.9	< 50	29
ELLERSLIE POST OFFICE	90039	81.3	50	28
HAWKESDALE (POST OFFICE)	90045	142.5	1711	901
KOROIT	90051	201.9	> 2000	1266
WOOLSTHORPE	90084	128.0	835	770
PENSHURST (THE GUMS)	90062	124.2	692	197

Table 5.4 – Rainfall totals for the 48 hour duration 1946 rainfall event

Gauge Name	No.	Rainfall Total (mm)	Estimated ARI (CRC Forge)	Estimated ARI (Freq. analysis)
CARAMUT (BARWIDGEE)	90016	102.3	< 50	63
ELLERSLIE POST OFFICE	90039	95.0	< 50	21
HAWKESDALE (POST OFFICE)	90045	185.9	1638	1698
KOROIT	90051	247.1	> 2000	1114
WOOLSTHORPE	90084	160.3	638	420
PENSHURST (THE GUMS)	90062	147.1	388	313

Table 5.5 – Rainfall totals for the 72 hour duration 1946 rainfall event

Gauge Name	No.	Rainfall Total (mm)	Estimated ARI (CRC Forge)	Estimated ARI (Freq. analysis)
CARAMUT (BARWIDGEE)	90016	118.0	62	90
ELLERSLIE POST OFFICE	90039	115.1	52	35
HAWKESDALE (POST OFFICE)	90045	218.9	> 2000	1587
KOROIT	90051	266.2	> 2000	997
WOOLSTHORPE	90084	187.5	1112	544
PENSHURST (THE GUMS)	90062	160.3	418	377

As shown in Figure 5.1 the Woolsthorpe gauge is the most central gauge to the bulk of the Merri catchment. The two gauges to the east of the catchment, Hawkesdale and Koroit, show that the event was recorded as more severe with estimates for the rainfall event being between 900 and over 2000 year ARI. To the north of the catchment at Penhurst, the event was not as severe but was still estimated at approximately a 400 year ARI event. To the west, Caramut and Elleslie gauges, the event can be seen to have been less severe with an ARI estimated at less than 100 year ARI. To examine the average rainfall across the entire catchment, the rainfall data was analysed over the Merri River catchment in approximately 25 km^2 parcels. The average rainfall across the catchment for the 24, 48 and 72 hour periods and the estimate of the ARI based on the CRC-FORGE and Woolsthorpe gauge frequency analysis are shown in Table 5.6.

Table 5.6 – Average Catchment Rainfall, 1946 Event

Storm Duration (hours)	Average Rainfall (mm)	Estimated ARI (CRC FORGE)	Estimated ARI (Woolsthorpe)
24	130	1000	1000
48	161	650	400
72	181	900	500

The assessment of the rainfall totals for the 1946 event at North Warrnambool indicates that the rainfall event was between a 500 and 2000 year event over the Merri River catchment. This is broadly in line with the designated flood ARI estimated at a 1000 year ARI. The calculated average catchment rainfall when compared to the CRC-FORGE estimates of rainfall also indicates the magnitude of the storm event is approximately equivalent to the 1000 year ARI event.

It should be noted that daily rainfall totals were used for this assessment and if 6 minute or hourly pluviograph rainfall totals were available there may have been more intense 24, 48 or 72 hours periods for each of these rainfall gauges. The rainfall totals are slightly conservative as a result.

5.1.2 RORB Model Estimate

The North Warrnambool RORB model was utilised to check some of the assumptions of the rainfall depths and resulting peak flow rates for the 1946 event. In order to check that the assumptions made about the ARI of the flood event were around the correct ARI, the CRC Forge 1000 year ARI total rainfall estimate for the event for the 48 hour duration was used as an input into the RORB model. The peak flow rates could then be determined at key locations in the model and compared flood frequency assessment for the Woodford flow gauge.

The total rainfall for the CRC Forge 48 hour 100 year ARI event was 173.4 mm. As for the hydrologic modelling, a k_c of 58 and an 'm' value of 0.8 were used. A range of loss rates were explored to determine the impact of the initial and continuing loss on the peak of the flood hydrograph. The GSAM Coastal temporal pattern was utilised to distribute the rainfall event total over the 48 hour period. The results of this assessment are summarised in Table 5.7.

Table 5.7 – Peak flows from RORB using 1000 year ARI CRC Forge 48 hour rainfall

Location	Initial Loss / Continuing Loss						
	0 mm / 0 mm/h	10 mm / 1 mm/h	10 mm / 1.5 mm/h	10 mm / 2 mm/h	20 mm / 1 mm/h	20 mm / 1.5 mm/h	20 mm / 2 mm/h
Woodford	1097.0	812.3	705.0	599.0	768.7	656.9	562.2
U/S Russell Creek	1136.1	844.8	725.9	608.2	808.6	692.9	579.0
Dennington	1227.6	923.7	790.8	658.8	884.7	751.9	633.2
Model outlet	1251.3	937.1	802.1	668.4	892.0	764.5	639.3

The model was run with no losses initially to gauge the upper limit of the peak and the peak flow rate of 1,136 m^3/s was obtained at the upstream edge of the Merri Creek hydraulic model. It is unrealistic to assume that there would be no losses during a rainfall event in the catchment and to explore the resulting peak flow rates initial losses of 10 mm and 20 mm were utilised with continuing losses ranging from 1 mm/hr to 2 mm/hr. These loss rates are consistent with the loss rates utilised for the design events.

The peak flows at the upstream boundary of the North Warrnambool model ranged from 579 m^3/s up to 845 m^3/s. However, the rainfall patterns leading up to the 1946 event suggests that the catchment may have already been quite damp with a regional event occurring on the 10th, 11th and 12th Match 1946. This suggests that the 10 mm initial loss rate may be more appropriate (than the 20 mm initial loss). This set of runs has been highlighted in Table 5.7. The peak flows at the upstream boundary of the North Warrnambool model range from 608 to 850 m^3/s. The upper estimate of the peak flow rate coincides with the flow rate found in the hydraulic model (see section 5.1.3) to best represent the recorded flood levels in the North Warrnambool area.

5.1.3 Hydraulic Model Estimate

For this study two flow estimates were modelled at 850 m^3/s and 1000 m^3/s. These values cover the range of flow estimates and correspond to the approximate 1000 year event. The 850 m^3/s is a similar flow rate to the Dennington Flood Study (2007) and the 1000 m^3/s examines flow rates predicted by SRWSC (~950 m^3/s) and the Country Roads Board (~1050 m^3/s). The model was run as a steady state run and a nominal flow (30 m^3/s) was passed down Russell Creek, as the peak flow and through Russell Creek is unknown. The flows through Russell Creek were expected to have little bearing on the levels at the downstream section of Russell Creek as they would be controlled predominantly by Merri River backflows. A steady state model was used due to the large magnitude and duration of the 1946 flood event. It is understood that the 1946 event occurred over a number of days and the assumption was that the flood storage would have been filled in the lead up to the peak event.

The 1946 peak flow estimates have been utilised from previous studies and from the FFA. A peak rainfall analysis method was not employed in this assessment as this method had already been attempted by GHD in their North Warrnambool Flood Study (2003) which resulted in an estimate of the 1946 event of approximately 470 m^3/s downstream of the Merri River and Russell Creek confluence. The consensus of the estimates of the peak flow rate during the 1946 event on the Merri River was almost double this number. In addition, the Port Fairy Regional Flood Study (2008) showed a hydrologic estimate of the 1946 peak flow for the Moyne River at Toolong. These estimates were based on recorded rainfall in the region and were run through an existing RORB model with a range of continuing losses (1 mm/hr up to 3 mm/hr). The resulting peak flow rates ranged from 412 m^3/s up to 689 m^3/s, This is a very large range and provides only rough guidance on the peak flow rate. This is mentioned to highlight that fact that the uncertainty associated with the hydrologic approach is not likely to aid the estimation of the 1946 peak flow rate.

The method employed by Cardno was to utilise a range of flow rates from past estimated of peak flows and then use the hydraulic model to attempt to match the known flood levels. This process then intuitively determines the peak flow rates for the 1946 event and can then be compared to the estimates for the 1000-year ARI from the FFA.

The results for the 1946 modelled event compare well to the observed data with all results being within +/- 0.31 m under the 850 m^3/s scenario. The modelled levels and observed levels are summarised in Table 5.8 and the flood depths are shown in Figure 5.2. The observed levels at Queens Road on the Merri River floodplain seem to contradict the observed level at the corner of Ardley St/Nairne Close as the level is 0.29 m higher at the Ardley St/Nairne Close location (albeit on the Russell Creek tributary) compared to the observed level at Queens Road. It would be expected that the hydraulic gradeline would reduce from upstream to downstream and as the Ardley St/Nairne Close observed level would be controlled by the backwater from the Merri River. Logically this level should be lower than the observed level at Queens Road. It appears likely that the Queens Road level is incorrect, given that the model replicates the flood heights at Nairne St and Cassidys Bridge.

The modelled results (with levels of 8.02 m and 7.91 m respectively for the Queens Rd and Ardley St/Nairne Close locations) show a consistent pattern to the expected levels, although they do differ from the observed levels with the Queen Road level being 0.31 m higher and Ardley St/Nairne Close level being 0.09 m lower than the observed levels. This suggests that the 1946 flow rate would be around the 850 m^3/s but as the modelled flood levels are above observed in some areas and below in others it is not possible to get a perfect calibration. Further increases in flow rate will cause the modelled Queens Road flood levels to increase, reducing the calibration accuracy at this location, and any further decreases in the flow rate will result in a the levels at Ardley St/Nairne Close to reduce, which will decrease the calibration accuracy at this location.

It should be noted that the observed levels are likely to have significant error as it is unknown how these levels were measured and water level marks can be influenced by debris and wave action. The 1000 m^3/s scenario produced consistently high results as compared to the observed levels and suggests that this flow rate is higher than the 1946 event.

Overall, it is evident that the 850 m^3/s produces a reasonable estimate of the 1946 event, especially at the downstream end of the model at Cassidy's Bridge with the levels being only 0.01 m higher than the observed levels. The head loss across Cassidy's Bridge was identical in the model as compared to the observed levels, which indicates that the bridge friction parameter is correct and the structure is representing the physical bridge accurately. Overall, the hydraulic model has represented this large 1946 event well.

Table 5.8 – 1946 Model Results

Location	Observed level	850 m^3/s Run		1000 m^3/s Run	
		Model Level	Difference	Model Level	Difference
Queens Road	7.71 m	8.02 m	+ 0.31 m	8.45 m	+ 0.74 m
Cnr Ardley St & Nairne Close	8.00 m	7.91 m	- 0.09 m	8.33 m	+ 0.33 m
U/S Cassidy's Bridge	6.58 m	6.59 m	+ 0.01 m	6.81 m	+ 0.23 m
D/S Cassidy's Bridge	6.25 m	6.26 m	+ 0.01 m	6.39 m	+ 0.14 m
100m D/S Cassidy's Bridge	6.18 m	6.19 m	+ 0.01 m	6.28 m	+ 0.10 m

5.1.4 Summary

This assessment has attempted to provide clarification of the uncertainty around the 1946 event as to the exact ARI of this event and the associated peak flow rate. Three methods were employed in this study to attempt to quantify this event and the consensus was that the storm was approximately a 1000 year ARI rainfall event, which corresponded to a 1000 year ARI flow event, which had a peak flow rate of around 850 m^3/s. These estimates are reinforced by the regional CRC Forge rainfall totals and from the hydrologic modelling using the CRC Forge rainfall estimates.

The rainfall ARI was shown to vary spatially with more intense rainfall recorded on the eastern side of the Warrnambool catchment and lower rainfall totals recorded to the west. The ARI ranged from over a 2000 year event to below a 500 year event. This adds difficulty in determining the exact ARI for the flood event, however it does provide some guidance. Subsequent assessment using the CRC Forge extreme rainfall totals indicated that the 1000 year ARI 48 hour rainfall total produced peak flow rates that were consistent with the hydraulic model estimate.

6 RESULTS

The calibrated SOBEK model for Russell Creek and Merri River was used to analyse the extent, location and depths for the durations of 5, 10, 20, 50, 100 and 200 year ARIs. For each ARI a range of event durations were examined to find the maximum flood extents and depths.

For Merri River the peak flows were found in the 72 hour event, with the exception of the 200 year ARI where the shorter 48 hour event was critical. For Russell Creek multiple durations were modelled and included all durations where the peak flow was observed at any of the distributed inflow points as defined by the RORB catchments. This ensures that the peak flood extents and depths are captured by the modelling. The durations modelled are summarised in Table 6.1. We have assumed concurrent starting times of flood hydrographs in the Merri River and Russell Creek.

Table 6.1 – Modelled recurrence intervals and durations

Recurrence Interval	Durations in Merri River	Durations in Russell Ck
5 year ARI	72hr	15min, 24hr, 36hr
10 year ARI	72hr	15min, 12hr, 36hr
20 year ARI	72hr	15min, 9hr, 36hr, 48hr
50 year ARI	72hr	15min, 9hr, 36hr, 48hr
100 year ARI	72hr	15min, 12hr, 36hr, 48hr
200 year ARI	48hr	15min, 3hr, 9hr, 36hr

The flood extents and depths for the peak 5, 10, 20, 50, 100 and 200 year ARI events are shown in Figures 6.1 to 6.6 respectively. A range of maximum water surface elevations have been extracted and summarised on Figures 5.1 to 5.6 and in Table 5.2. Figure 6.7 shows the increasing inundated area for the flood scenarios. There is a drainage connection between St James Park, just downstream of Wollaston Road and the Merri River that was not explicitly included in the model. It is expected that flows will flood St James Park through this connection even if Wollaston Road is not overtopped. As such, where Wollaston Road is not overtopped in the design event, we have adopted a level in St James Park equivalent to the flood level at the downstream side of the Wollaston Road Bridge.

Figure 6.1 shows the peak 5 year ARI flood depths with water surface elevation data shown at a range of locations. The Merri River 5 year ARI flood is predominantly contained within the main river channel. Russell Creek has some overland flooding. At the upstream end of Russell Creek there is an accumulation of flood water upstream of Wangoom Road due to the effect of the culverts. The flows are constrained within Russell Creek downstream of this area. The flow then breaks out of the main channel near Garden Street and Brierly Street. Downstream of Queens Street the flood breaks out of the main Russell Creek and flows over the floodplain until the flood levels are constricted at Daltons Road before joining with the Merri River.

Figure 6.2 shows the peak 10 year ARI flood depths with water elevation data. The 10 year event breaks out of the Merri River main channel in this event and causes some floodplain inundation. This inundation is mainly restricted to adjacent to and downstream of the

Bromfield weir. Within Russell Creek similar areas area flooded during the 10 year ARI event as for the 5 year ARI event. Some additional flooding was observed along Brenton St which caused some inundation at the corner of Brenton St and Hayley Drive.

The 20 year ARI flood results are shown in Figure 6.3. The Merri River flood depths increased and flow breaks out upstream of Wollaston Bridge to the east. The area inundated within the Merri River floodplain has increased as and the water surface elevations indicate increases within the floodplain of about 0.5 m. On Russell Creek the inundated area upstream of Wangoom Road has increased under the 20 year ARI. Flows are still contained within the channel down to Brierly Street and Garden Street. The main areas of inundation as shown under the 5 and 10 year ARI events has not significantly increased under the 20 year ARI, however the flood depths have increased.

The 50 year ARI results are shown in Figure 6.4. The flood inundation on Merri River has increased with the area to the east of Wollaston Bridge becoming inundated to the north and south of Wollaston Road. Flood depths though the main Merri River flood plain increased by approximately 0.5 m when compared to the 20 year ARI event. The Russell Creek inundation upstream of Wangoom Road did not increase in the 50 year ARI and flows are still contained within the Russell Creek channel until Brierly Street and Garden Street. The inundation along Brunton Street has increased with overland flooding reaching Mortlake Road. Mortlake Road is not overtopped in this event. The flooding extent downstream of Mortlake Road did not significantly increase, however the depth increased by approximately 0.10 m.

The 100 year ARI event flood extent is shown in Figure 6.5. The Merri River flood extent did not increase significantly over the 50 year ARI scenario however the flood depths increased by approximately 0.5 m. For Russell Creek the main increase in inundated area occurred when Mortlake Road was overtopped at Allans Street and Donovans Road. Additional overland flooding is observed adjacent to Queen Street and Bromfield Street. Overall, the overland flood depths increased by approximately 0.10 m to 0.20 m.

The 200 year ARI event flood extent is shown in Figure 6.6. This was the most severe event modelled and has the largest flood extent. The Merri River flood extent is not significantly increased over the 100 year ARI event, however the flood depths are increased by another 0.5 m above the 100 year ARI event. Upstream of Wangoom Road the inundation increased to the point where a breakout occurred where water was flowing overland west towards the Merri River. Downstream of Wangoom Road the flood is mainly contained within the Russell Creek main channel again until the Brierly Street and Garden Street area where the flows breakout of the main channel. The inundation upstream of Mortlake Road is similar to the 100 year ARI scenario although the depths were increased. Downstream of Mortlake Road the overland flooding increased with flows along Donovans Road reaching the downstream inundation of Russell Creek. Overall the flood depth increases by 0.10 m to 0.20 m over the 100 year ARI.

Table 6.2 – Maximum water surface elevation for each event at key locations

Site location	Maximum surface water elevation (m AHD)					
	5 year ARI	10 year ARI	20 year ARI	50 year ARI	100 year ARI	200 year ARI
Merri River						
500m U/S Wollaston Bridge	3.54	4.38	5.14	5.92	6.50	7.12
U/S Wollaston Bridge	3.40	4.14	4.82	5.49	6.04	6.68
D/S Wollaston Bridge	3.34	4.04	4.68	5.27	5.74	6.34
Southern End of Queens Street	3.30	3.98	4.58	5.10	5.53	6.07
Northern End of Bromfield Street	3.01	3.73	4.36	4.92	5.37	5.93
Confluence of Merri R and Russell Ck	2.70	3.22	3.9	4.62	5.15	5.77
Eastern End of Tarhook Road	2.53	2.89	3.53	4.30	4.88	5.55
U/S Cassidy's Bridge	2.43	2.7	3.23	3.83	4.32	4.87
Russell Creek						
U/S Wares Rd	18.42	18.58	18.74	19.00	19.25	19.53
U/S Footpath Bridge near Moonah St	11.24	11.29	11.34	11.42	11.50	11.58
U/S Mortlake Road	8.55	8.69	8.87	9.26	9.46	9.63
U/S Queens Rd	6.63	6.68	6.73	6.80	6.86	6.93
U/S Bromfield St	5.09	5.13	5.18	5.28	5.38	5.75
U/S Daltons Rd	4.46	4.68	4.83	4.96	5.17	5.75

6.1 Flood Planning Controls

The current planning framework for the floodplain is encapsulated in the *Warrnambool Planning Scheme*. The Scheme, prepared in accordance with Victorian State Planning Policy Framework (VPP), documents all planning controls in the study area. The scheme consists of a written document as well as maps, plans and related documents. It contains (as outlined in the accompanying *User Guide*):

- The objectives of planning in Victoria.
- Purposes of the planning scheme.
- A User Guide.
- The State Planning Policy Framework.
- The Local Planning Policy Framework.
- Zone and overlay requirements.
- Particular provisions.
- General provisions.
- Definitions.
- Incorporated documents.

The *State Planning Policy Framework (SPPF)* covers strategic issues of State importance. It lists policies under six headings: settlement, environment, housing, economic development, infrastructure, and particular uses and development. Every planning scheme in Victoria contains this policy framework, which is identical in all schemes.

The *Local Planning Policy Framework (LPPF)* contains a municipal strategic statement and local planning policies. The framework identifies long term directions for land use and development in the Warnambool region; presents a vision for its community and other stakeholders; and provides the rationale for the zone and overlay requirements and particular provisions in the scheme.

With regard to flooding, clause 22.02-1 of the scheme, entitled 'Urban Floodway Local Policy' provides for the control of flooded areas as follows:

- All land thought liable to be at risk of flooding will be zoned either Urban Floodway, or covered by a Land Subject to Inundation Overlay (LSIO), generally in accordance with the controls introduced by Amendment L17B to the Warrnambool Planning Scheme on 20 June 1997, or subsequent amendments.
- Pursuant to Clause 37.03-3, the subdivision of land partly in the Urban Floodway Zone (UFZ), and partly in any other zone will be controlled by the provisions of the other zone. Each new lot created must be suitable for the purposes of a dwelling to the satisfaction of the responsible authority.
- In areas subject to the LSIO, it is policy that as first preference no fill will be allowed. Fill under a designated building footprint, outside a building footprint or for safe and proper access to and from the site, will be discouraged. Written justification to the satisfaction of the responsible authority must be provided by the applicant for any such fill, including why other construction techniques cannot be used.

6.2 Flood Related Planning Zones and Overlays

The planning scheme allows for a number of flood related overlays to identify land liable to flooding and flood characteristics. In general, the nature of the flood risk and available flood information will determine to what extent the provisions are applied in the planning scheme. The flood zone and overlay provisions allow for control of the land use and development through the use of a planning process to ensure that development is in-line with the level of flood risk.

There are four flood zones and overlays available for use:

- Urban Floodway Zone (UFZ)
- Floodway Overlay (FO)
- Land Subject to Inundation Overlay (LSIO)
- Special Building Overlay (SBO).

Each of these zones and overlays are defined more clearly in the following sections.

6.2.1 Urban Floodway Zone (UFZ)

The Urban Floodway land use zoning is intended to protect land in urban areas that has a primary function of floodwater conveyance. It applies to urban areas where the potential flood risk is high due to the presence of existing development or to pressures from new or more intensive development. The UFZ restricts, to a very limited number, the use of land to those that are consistent with the primary function of flood conveyance.

In Urban Floodway Zone areas, the following uses are allowed without a permit:

- Apiculture - Must meet the requirements of the Apiary Code of Practice, May 1997.
- Extensive animal husbandry
- Informal outdoor recreation
- Mineral exploration
- Mining
- Natural systems
- Search for stone - Must not be costeaning or bulk sampling.
- Telecommunications facility.

The following uses are allowed with a planning permit:

- Agriculture (other than Apiculture and Extensive animal husbandry)
- Leisure and recreation (other than Informal outdoor recreation, Indoor recreation facility, and Motor racing track)
- Mineral, stone or soil extraction (other than Mineral exploration, Mining, and Search for stone)
- Road
- Utility installation (other than Telecommunications facility).

The following land uses are prohibited in an Urban Floodway Zone:

- Indoor recreation facility
- Motor racing track
- Any other use not listed above.

All planning permits and subdivisional applications are also subject to the same controls as required for an application on land covered by the Floodway Overlay described below.

6.2.2 Floodway Overlay (FO)

The purpose of the Floodway Overlay, as described in the planning scheme, is as follows:

- To implement the State Planning Policy Framework and the Local Planning Policy Framework, including the Municipal Strategic Statement and local planning policies.
- To identify waterways, major floodpaths, drainage depressions and high hazard areas, which have the greatest risk and frequency of being affected by flooding.
- To ensure that any development maintains the free passage and temporary storage of floodwater, minimises flood damage and is compatible with flood hazard, local drainage conditions and the minimisation of soil erosion, sedimentation and silting.
- To reflect any declarations under Division 4 of Part 10 of the Water Act, 1989 if a declaration has been made.
- To protect water quality and waterways as natural resources in accordance with the provisions of relevant State Environment Protection Policies, and particularly in accordance with Clauses 33 and 35 of the State Environment Protection Policy (Waters of Victoria).

A planning permit is required to construct a building or to construct or carry out works, including fences and roadworks on land covered by the floodway overlay, with some limited exemptions for public infrastructure works.

Subdivision of land covered by a FO/RFO can only be accomplished with a planning permit and under the following conditions:

- The subdivision does not create any new lots, which are entirely within this overlay. This does not apply if the subdivision creates a lot, which by agreement between the owner and the relevant floodplain management authority, is to be transferred to an authority for a public purpose.
- The subdivision is the re-subdivision of existing lots and the number of lots is not increased, unless a local floodplain development plan incorporated into this scheme specifically provides otherwise.

All planning applications where a local floodplain development plan has not been incorporated into the scheme require a flood risk study to be undertaken with regard to the following points:

- The State Planning Policy Framework and the Local Planning Policy Framework.
- The existing use and development of the land.
- Whether the proposed use or development could be located on flood-free land or land with a lesser flood hazard outside this overlay.
- The susceptibility of the development to flooding and flood damage.
- The potential flood risk to life, health and safety associated with the development.
- Flood risk factors to consider include:
  - The frequency, duration, extent, depth and velocity of flooding of the site and accessway
  - The flood warning time available
  - The danger to the occupants of the development, other floodplain residents and emergency personnel if the site or accessway is flooded.
  - The effect of the development on redirecting or obstructing floodwater, stormwater or drainage water and the effect of the development on reducing flood storage and increasing flood levels and flow velocities.
  - The effects of the development on environmental values such as natural habitat, stream stability, erosion, water quality and sites of scientific significance.

Possible methods for development of the FO are outlined in the “Advisory Notes for Delineating Floodways” (NRE, 1998). These methods include:

- Flood frequency
- Flood hazard
- Flood depth

For the flood frequency the advisory notes (Appendix A1) suggest that areas which have a high consequence of flooding, has flood depths that are moderate or high and flood frequently should generally be regarded as floodway. For the Russell Creek and Merri Rivers the frequent flooding has been defined as the 10 year ARI.

The flood hazard is defined by combining the flood depth and flow speed to form a hazard category for a given design event. The advisory notes suggest using Figure 6.8 for delineating the floodway based on flood hazard. An alternate definition of flood hazard (or safety risk) is provided by Melbourne Water based on both the velocity-depth product and the total flood depth. Melbourne Water defines 5 classes of safety risk as shown in Table 6.3. A draft floodway overlay has been developed where areas with a safety risk of greater than 2 (in the 100-year ARI event) are included.

Table 6.3 – Melbourne Water Safety Risk Definition

Safety Risk Category		Definition		
		V*D		Depth
High	5	> 0.84 m^2/s	or	> 0.84 m
Moderate to High	4	0.6 - 0.84 m^2/s		0.6 - 0.84 m
Moderate	3	0.4 - 0.6 m^2/s		0.4 - 0.6 m
Low to Moderate	2	0.2 - 0.4 m^2/s		0.2 - 0.4 m
Low	1	< 0.2 m^2/s		< 0.2 m

The last method was the flood depth method. For this the areas of the 100 year ARI flood inundation where the flood depths were **greater than 0.5 m** were included in the FO definition. Figures 6.9 - 6.11 show the potential floodway overlays.

6.2.3 Land Subject to Inundation Overlay (LSIO)

The LSIO aims to include land which is likely to be inundated by overland flow during the 100 year ARI flood. The LSIO is cover under Clause 44.04 of the VPPF for Warrnambool.

The purpose of the Land Subject to Inundation Overlay as described in the planning scheme is as follows:

- To implement the State Planning Policy Framework and the Local Planning Policy Framework, including the Municipal Strategic Statement and local planning policies.
- To identify land in a flood storage or flood fringe area affected by the 100 year ARI flood or any other area determined by the floodplain management authority.
- To ensure that development maintains the free passage and temporary storage of floodwaters, minimises flood damage, is compatible with the flood hazard and local drainage conditions and will not cause any significant rise in flood level or flow velocity.
- To reflect any declaration under Division 4 of Part 10 of the Water Act, 1989 where a declaration has been made.
- To protect water quality in accordance with the provisions of relevant State Environment Protection Policies, particularly in accordance with Clauses 33 and 35 of the State Environment Protection Policy (Waters of Victoria).
- To ensure that development maintains or improves river and wetland health, waterway protection and flood plain health.

A planning permit is required to construct a building or to construct or carry out works, including fences and roadworks on land covered by the LSIO, with some exemptions for public infrastructure works. Any subdivision of land requires a planning permit and the number of lots can be increased.

Applications for planning permits in areas covered by the LSIO have the following decision guidelines with respect to flooding:

- The State Planning Policy Framework and the Local Planning Policy Framework.
- Any local floodplain development plan.
- Any comments from the relevant floodplain management authority
- The existing use and development of the land.
- Whether the proposed use or development could be located on flood-free land or land with a lesser flood hazard outside this overlay.

- The susceptibility of the development to flooding and flood damage.
- The potential flood risk to life, health and safety associated with the development.
- Flood risk factors to consider include:
  - The frequency, duration, extent, depth and velocity of flooding of the site and accessway
  - The flood warning time available
  - The danger to the occupants of the development, other floodplain residents and emergency personnel if the site or accessway is flooded.
  - The effect of the development on redirecting or obstructing floodwater, stormwater or drainage water and the effect of the development on reducing flood storage and increasing flood levels and flow velocities.
  - The effects of the development on environmental values such as natural habitat, stream stability, erosion, water quality and sites of scientific significance.

As the LSIO defines flood areas which carry lower risk due to the frequency of inundation and impacts of flooding it is typically defined as the extent of less significant events. The LSIO covers areas that are not included within the FO or UFZ but are still exposed to flood risk. For the Warrnambool region it was considered appropriate to use the 100 year ARI event as the extent for the LSIO.

6.2.4 Special Building Overlay (SBO)

The SBO applies to areas that are subject to stormwater flooding in urban areas. That is to say areas which are inundated due to the inability of the stormwater infrastructure to convey the flood flows. This overlay is considered as many stormwater systems were implemented prior to current design standards and there has been substantial development since the infrastructure was completed.

Although we have included some consideration of stormwater drainage in this study, especially in the Russell Creek catchment, we do not believe that a SBO will be required in the Merri River and Russell Creek areas. Detailed modelling of the stormwater system of the Warrnambool area would be required to accurately develop the SBO.

6.3 Recommended Planning Controls

As the Warrnambool region is already well developed, we do not believe that there is a need to implement an Urban Floodway Zone in the catchment. Similarly, a SBO is unlikely to be required as the predominant flooding is from main channel flows rather than from stormwater flooding. Stormwater flooding was not specifically assessed as part of this project.

The recommended flood controls to be put in place are a FO and LSIO. The method of deriving the FO included using the 10 year ARI extent, the hazard class exceeding 2 for the 100 year ARI and where the depths were greater than 0.5 m during the 100 year ARI event. The three possible extents for the FO varied with each method protecting different areas. The main difference was that the FO included area around the Garden Street and Brierly Street intersection with Russell Creek under the 10 year ARI flood extents as compared to the other methods of deriving the FO. Similarly, the methods utilising the 100 year ARI using the depth greater than 0.5 m and the hazard class greater than 2 included more of the floodplain region of the Merri River, as well as additional area around Russell Creek.

The results of the three methodologies have been provided to allow the City of Warrnambool and Glenelg Hopkins CMA to determine a final Floodway Overlay shape.

The LSIO would include all areas inside the 100-year flood extent that are not covered by the final FO shape.

7 SENSITIVITY ANALYSIS

The sensitivity analysis aims to examine the impact on the hydraulic model changes to the roughness parameters and to the hydrograph shape and peak level. The sensitivity was undertaken on the 1% AEP design hydrograph for the roughness parameters and for all events for the hydrograph shape.

7.1 Sensitivity to Roughness Parameters

To examine the sensitivity to the Manning's roughness parameter, high and low roughness cases were developed, with values varying based on the land use. The scenarios chosen are summarised in Table 7.1. The main changes to the roughness parameters that should influence the model are the increase and decrease of the roughness in the main river channel, river floodplain and farmland roughness parameters. The roughness grids for the Low and High scenarios are shown in Figures 7.1 and 7.2.

Table 7.1 – Adopted Roughness Parameters, Mannings 'n'

Parameter	Roughness Manning's 'n'		
	Low	Calibrated	High
Roads	0.016	0.018	0.020
Main River channel	0.03	0.035	0.04
River floodplain	0.045	0.07	0.10
Farmland	0.045	0.08	0.10
Residential	0.15	0.2	0.3
Commercial	0.2	0.5	0.5

The changes in flood depth have been examined using the calibrated 100 year ARI event results as the base case. Figures 7.3 and 7.4 show the change in flood depth for the low and high roughness grid scenarios respectively as compared to the base case. The difference plot shows the increase of decrease of the parameter from the 100 year ARI base case.

The sensitivity analysis shows the impact of a range of roughness parameters. For the Merri River flood plain the difference was approximately +/- 0.10 m. This indicates that the roughness does not significantly impact on the peak water surface elevations. Within the Russell Creek section of the model the water surface elevations vary at the upstream section of the model by +/- 0.28 m, however at the downstream end of Russell Creek (from Moonah Street) the depths are similar and with the range of +/- 0.05 m. Overall the sensitivity analysis shows that model is not significantly sensitive to the roughness parameter.

Table 7.2 – Maximum water surface elevation for varying roughness at key locations (100-year ARI Event)

Site location	Maximum surface water elevation (m AHD)		
	Low roughness	Calibrated roughness (Base)	High roughness
Merri River	6.34	6.57	6.75
500m U/S Wollaston Bridge	5.93	6.12	6.31
U/S Wollaston Bridge	5.63	5.84	6.04
D/S Wollaston Bridge	5.41	5.63	5.85
Southern End of Queens Street	5.29	5.49	5.69
Northern End of Bromfield Street	5.17	5.31	5.48
Confluence of Merri R and Russell Ck	5.01	5.09	5.21
Eastern End of Tarhook Road	4.56	4.59	4.70
U/S Cassidy's Bridge	6.34	6.57	6.75
Russell Creek			
U/S Wares Rd	18.80	19.08	19.25
U/S Footpath Bridge near Moonah St	11.36	11.46	11.50
U/S Hopkins Hwy	9.37	9.38	9.38
U/S Queens Rd	6.79	6.82	6.86
U/S Bromfield St	5.20	5.32	5.49
U/S Daltons Rd	5.18	5.32	5.49

7.2 Hydrograph Shape and Peak

The timing of flood hydrographs can influence the extent and depth of inundation. To assess the sensitivity of the floodplain to a change in the hydrograph shape, and to a lesser extent the increase in peak flow, the results from the design RORB runs at Woodford were input to the model. These hydrographs have a slightly higher peak flow and essentially deliver the flood volume to the area at a faster rate. Peak levels are shown in Table 6.3. The flood depths are shown for the 5, 10, 20 and 50 year ARI events in Figures 7.5 to 7.8 respectively.

The difference in water surface elevations (comparing Table 6.2 and Table 7.3) are shown in Table 7.4. The differences show that the Woodford hydrographs were between 0.05 m and 0.16 m lower when using the transposed flood hydrographs (as compared to the Woodford hydrograph). Overall, the use of the transposed hydrograph from Woodford did not have a significant impact on the peak flood levels with most peak flood levels being within 0.16 m of the transposed flood peaks. This gives additional confidence in the flood levels as it removes some of the uncertainty from the hydrology.

Table 7.3 – Maximum water surface elevations for Merri River using the Woodford hydrographs for trial events at key locations

Site location	Maximum surface water elevation (m AHD)			
	5 year ARI	10 year ARI	20 year ARI	50 year ARI
Merri River				
500m U/S Wollaston Bridge	3.63	4.51	5.21	5.98
U/S Wollaston Bridge	3.48	4.27	4.88	5.54
D/S Wollaston Bridge	3.42	4.17	4.73	5.31
Southern End of Queens Street	3.38	4.10	4.63	5.14
Northern End of Bromfield Street	3.12	3.87	4.41	4.96
Confluence of Merri R and Russell Ck	2.79	3.38	3.99	4.67
Eastern End of Tarhook Road	2.59	3.02	3.63	4.35
U/S Cassidy's Bridge	2.48	2.81	3.32	3.88

Table 7.4 – Water surface elevation differences between model runs using the Woodford hydrographs compared to using the transposed hydrographs

Site location	Maximum surface water elevation (m AHD)			
	5 year ARI	10 year ARI	20 year ARI	50 year ARI
Merri River				
500m U/S Wollaston Bridge	-0.09	-0.13	-0.07	-0.06
U/S Wollaston Bridge	-0.08	-0.13	-0.06	-0.05
D/S Wollaston Bridge	-0.08	-0.13	-0.05	-0.04
Southern End of Queens Street	-0.08	-0.12	-0.05	-0.04
Northern End of Bromfield Street	-0.11	-0.14	-0.05	-0.04
Confluence of Merri R and Russell Ck	-0.09	-0.16	-0.09	-0.05
Eastern End of Tarhook Road	-0.06	-0.13	-0.10	-0.05
U/S Cassidy's Bridge	-0.05	-0.11	-0.09	-0.05

7.3 Hydrograph timing

In order to assess the influence of the hydrograph timing on the resulting flood depths a sensitivity analysis was undertaken on the 100 year ARI, 36 hour duration event by running three models:

- Base Case – hydrographs starting concurrently regardless of peak.
- Case 1 – hydrographs peaking at identical times.
- Case 2 – Russell Ck hydrograph starts at Merri River peak.

The differences are shown in Figures 7.9 and 7.10. When the peaks are matched the changes in the peak flood depth are evident throughout the Merri River floodplain. The floodplain depth increases by approximately 20-30 cm in the Merri River floodplain. At the downstream end of Russell Creek the flood depth increased by between 30-40 cm, however this was restricted to the area downstream of Queen Street. A small area downstream of the Hopkins Highway (adjacent to Donovans Road) shows some increase in flooding depth (between 2-10 cm) but limited increase in flood extent. The flood extent was only marginally increased and is represented by the magenta section of the flood difference plot.

When the Russell Creek hydrograph started at the time of the Merri River peak flow the change in flood depths was less than when the flood peaks were coincident. The difference in flood depths is shown in Figure 6.10. The Merri River floodplain increased by between 10-20 cm and the Russell Creek flood depths downstream of Queen Street increased by 20-30 cm. This was approximately 10 cm less than the scenario with the coincident flood peaks. As for Case 1, a small area downstream of the Hopkins Highway (adjacent to Donovans Road) shows some increase in flooding depth (between 2-10 cm) but limited increase in flood extent. The flood extent did not change considerably from the original 100 year ARI base scenario.

Overall, the timing of the Russell Creek and Merri River hydrograph peaks makes a maximum of between 30-40 cm increase in flood depths in the main part of the flood plain. It does not significantly increase the area inundated by the flooding. The flood depths remain unchanged for the bulk of the Russell Creek catchment with only the area downstream of Queen Street affected by the sensitivity analysis.

This sensitivity analysis shows the maximum impact that the timing of the event can have on the flood depths and aims to clarify the potential uncertainties associated with the timing of the hydrographs. It should be noted that the flood events are likely to start at a similar time due to rainfall events passing over the catchment and in most cases the peak for the Russell Creek would have passed before the Merri River reaches its peak flood depth.

We note that the sensitivity analysis indicates that a freeboard of 600 mm above the design 100-year ARI flood levels is likely to ensure that floor levels would not be inundated given the worst case flood scenario where all uncertainties are

8 ECONOMIC DAMAGES

The economic impact of flooding can be defined by what is commonly referred to as 'flood damages'. These flood damages can be defined as being direct, indirect or intangible as defined in Figure 8.1.

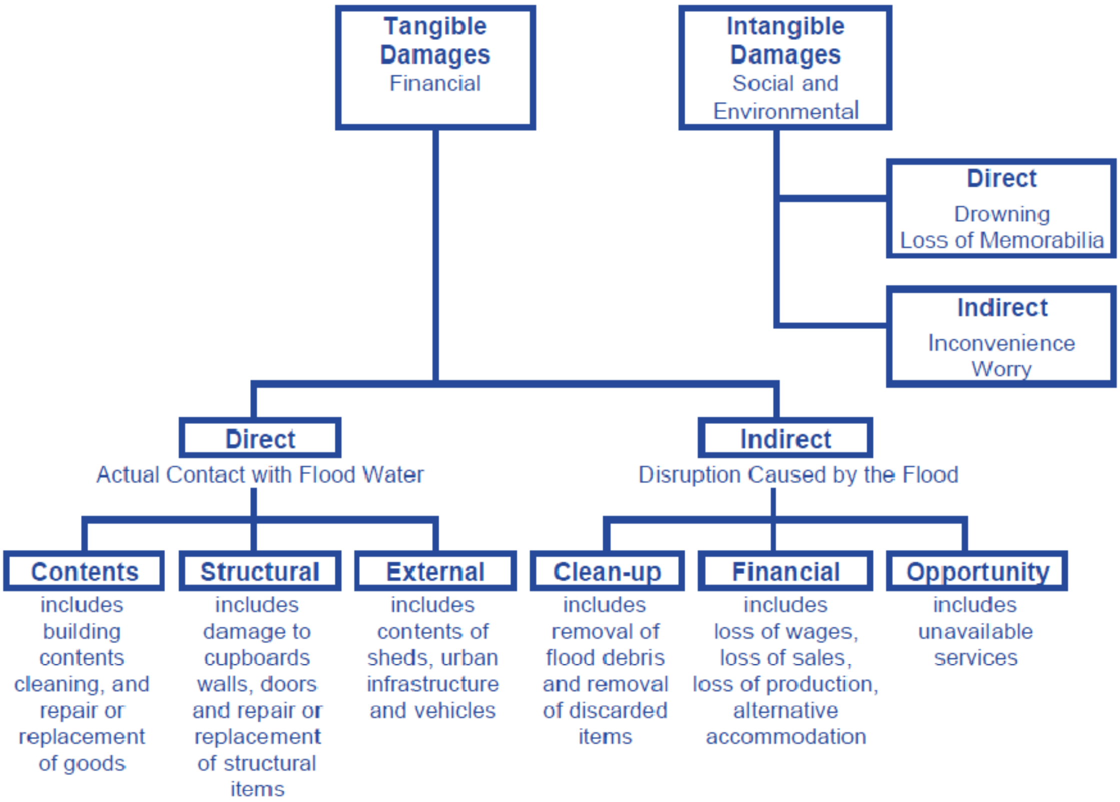

Figure 8.1 Types of flood damage (Floodplain Development Manual (NSW Gov, 2005))

The direct damage costs are just one part of the flood damage overall cost. The flood damages are broken down into two distinct groups, tangible and intangible damages. The damage assessment in this report is restricted to the tangible damages and makes no estimate of the costs associated with the 'intangible' costs, such as social distress and loss of memorabilia.

The 'tangible' damages are further divided into direct and indirect damages. The indirect damages are damages caused by the disruptions of the flooding (such as clean up costs and accommodation costs), whereas the direct damages are caused by contact with the flood waters directly (such as damage to carpets and household contents).

For Warrnambool it has been assumed that the residents will have no warning time and hence no allowance has been made for the residents protecting or removing their valuables. This assumption has been made as it gives a more conservative estimate of flood damages as the maximum 'potential' damage is assessed.

Flood damages can be assessed by a number of methods including the use of computer programs such as FLDAMAGE, ANUFLOOD or via more generic methods such using spreadsheets. For the purposes of this project, generic spreadsheets have been used based on experience by Cardno in this area. The use of both the Floodplain Management Manual (NSW Gov, 2005) and The Rapid Appraisal Method for floodplain Management (NRE, 2000) were utilised in this flood damage assessment.

8.1 Damage Analysis

A flood damage assessment has been undertaken for the existing catchment and floodplain as part of the current study. The assessment is based on damage curves that relate to the depth of flooding on a property to the likely damage to a property.

Ideally, the damage curves would be calibrated to the specific catchment for which the study was undertaken, however, damage data in most catchments is not available and as a result damage curves from other catchments are utilised. The Department of Environment, Climate Change and Water NSW (DECCW) has carried out research and prepared a methodology (draft) to develop damage curves based on state-wide historical data. This methodology is only for residential properties and does not cover industrial or commercial properties.

The DECCW methodology is only a recommendation and there are currently no strict guidelines regarding the use of damage curves in Victoria. The Rapid Appraisal Method (RAMS) suggests specific damage values for residential, commercial and industrial buildings, however, these values are not specific to Victoria and the flood damage curves developed by DECCW are based on a more robust methodology.

The following sections provide an overview of the methodology applied for the determination of damages within the Merri River and Russell Creek floodplain.

8.1.1 Residential Damage Curves

The *Floodplain Management Guideline No. 4 Residential Flood Damage Calculation* prepared by DIPNR (now DECCW) (DIPNR, 2004) has been used in this damage assessment. These guidelines include a template spreadsheet program that determines damage curves for three types of residential buildings;

- Single storey, slab on ground,
- Two storey, slab on ground, and
- Single storey, high-set.

The floor level survey data from the *North Warrnambool Flood Study for Merri River and Russell Creek* (GHD, 2003) did not specify the residential property construction. It has been assumed that all residential properties are slab on ground.

Damages are generally incurred on a property prior to any over floor flooding. There are two possibilities:

- The flooding overtops the representative ground level but does not necessarily reach the base of the house. When this representative ground level is exceeded by a depth of 10 cm, a nominal damage value of $3,222 (Dec 2009 dollars) has been adopted to represent garden damage.

- The flooding overtops the garden and also reaches the base of the house. The DECCW curves allow for a damage of $9,474 (Dec 2009 dollars) to be incurred when the water level reaches the base of the house (the base of the house is determined by the floor level less 0.5 m for slab on ground houses). This accounts for the garden damage as specified in the point above, but also includes some damage to cars and structures.

In summary, a cost of $3,222 (Dec 2009 dollars) was applied when only the property was overtopped by greater than 10 cm of depth of flood water. When the flooding reaches 0.5 m below the floor level of the house the DECCW damage curves (adjusted to current dollar values) have been adopted. This equates to $9,474 (Dec 2009 dollars) for flooding depths between 0.5 m below the floor height, when the flood water overtop the floor level the DECCW damage curves are used to determine the economic damage.

8.1.2 Other Parameters

There are a number of input parameters required for the DECCW curves, such as the area of the floor of houses in the floodplain and level of flood awareness. The damage assessment adopted values within the recommended range specified by the DECCW guidelines. The average house size for Warrnambool was estimated based on the approximate area of 706 properties from MapInfo. Each of these properties were represented by polygons and derived using ALS photography, the areas are approximate only. The average was determined excluding commercial properties and was approximately 200 m^2. This area reflects the ground floor only.

Within the catchment there were 189 houses which did not have floor levels recorded in the data obtained from the *North Warrnambool Flood Study for Merri River and Russell Creek* (GHD, 2003). In order to estimate the floor levels of these properties, the average floor level to building topography level was examined. The average floor height above the topography elevation was found to be 282 mm and this value was used to set the floor levels from the known building elevation (taken from the ALS data) for the properties with no floor height data.

Conservatively, the Effective Warning Time has been assumed to be zero as Russell Creek has no flow gauge. A long Effective Warning Time allows residents to prepare for flooding by moving valuable household contents (e.g. the placement of valuables on top of tables and benches).

The North Warrnambool catchment, while rural, has access to Colac, Geelong and Melbourne via multiple highways and as a result it is assumed that there are no post-flood inflation costs. These inflation costs are generally experienced in regional areas where re-construction resources are limited and large floods can cause a strain on these resources. For the local flooding assessed in this study it is unlikely that there would be large regional impacts (i.e. from Russell Creek). However, the Merri Creek flooding may cause this type of impact.

8.1.3 Average Weekly Earnings

The DECCW curves are derived for late 2001 and have been adjusted to represent December 2009 dollars.

General recommendations by DECCW are to adjust values in residential damage curves by the increase in Average Weekly Earnings (AWE), rather than by the inflation rate as measured by the Consumer Price Index (CPI). DECCW proposes that AWE is a better representation of societal wealth, and hence an indirect measure of the building and contents value of the home. The most recent data for AWE from the Australian Bureau of Statistics (ABS) was in February 2010. Therefore all ordinates in the residential flood damage curves were updated to the February 2010 dollars. In additional, all damage curves include GST as per the DECCW recommendations.

While not specified, it was assumed that these curves were derived in November 2001, which therefore assumes the use of the November 2001 AWE (issued quarterly) would be appropriate. November 2001 and February 2010 AWE statistics were obtained from the ABS website (www.abs.gov.au). The AWE figures and percentage adjustment factor is summarised in Table 8.1.

Table 8.1 – Residential damage curve adjustment factor

Month	Year	AWE
November	2001	$ 898.50
February	2010	$ 1,289.80
Change	43.6 %	

Consequently, all ordinates on the damage curves were increased by 43.6 %. It has been assumed that February 2010 values are representative of April 2010 dollars.

8.1.4 Commercial Damage Curves

Commercial damage curves are determined based on those included in the *FLDamage Manual* (Water Studies, 1992). FLDamage allows for three types of commercial properties;

- Low Value Commercial,
- Medium Value Commercial,
- High Value Commercial.

In Warrnambool in the Russell Creek and Merri River floodplains all commercial has been assumed to be low value commercial based on FLDamage. In determining these damage curves, it has been assumed that the effective warning time is approximately zero, and the loss of trading days has been approximated at 10.

The commercial damage curve is linked to the floor area of the property and the floor level survey has estimates of the floor area of the individual properties. These areas will be used to factor these curves, the curves have been determined for a standardised 100 m^2.

Within the catchment there were 2 commercial properties which did not have floor levels recorded in the data obtained from the *North Warrnambool Flood Study for Merri River and Russell Creek* (GHD, 2003). In order to estimate the floor levels of these properties, the average floor level to building topography level was examined. The average floor height above the topography elevation was found to be 420 mm and this value was used to set the floor levels from the known commercial building elevation (taken from the ALS data).

The CPI was used to bring the 1990 data to March 2010 (CPI was obtained from the ABS www.abs.com.au) . It was assumed that the Water Studies (1992) data was in June 1990 dollars. The CPI adjustment factor is shown in Table 8.2.

Table 8.2 – Commercial damage curve adjustment factor

Month	Year	CPI
June	1990	102.5
March	2010	171.0
Change	66.8 %	

Consequently, damages have been increased by 65.4% and GST has been included.

8.1.5 Industrial Damage Curves

No industrial buildings were identified in the Russell Creek and Merri River flood zones examined under this study.

8.1.6 Road damages

Road damage was assessed based on the Rapid Appraisal Method (RAM) which assigns a damage value for major roads, minor roads and unsealed roads. The RAM was developed in May 2000 and the damages are quoted in May 2000 dollars. To convert these to March 2010 dollars, the CPI was used to adjust for inflation. The adjustment factor is shown in Table 8.3.

Table 8.3 – Roads damage adjustment factor

Month	Year	CPI
May	2000	126.2
March	2010	171.0
Change	34.3 %	

The RAM uses a single estimate cost per km for roads which are inundated and includes:

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

The RAM estimates ogf the costs per km of inundated road are shown in Table 8.4. These unit damages were adjusted using the CPI adjustment factor. The RAM also states that the damages to roads and bridges generally outweighs the costs associated with other infrastructure such as water, electricity, gas and sewerage services and is a good approximation for the overall damage to the regional infrastructure.

Table 8.4 – Unit damages for roads and bridges (dollars per km inundated)

	Initial road repair	*Subsequent accelerated deterioration of roads*	*Initial bridge repair and increased maintenance*	Total cost applied per km to inundated roads (May 2000 $)	Total cost applied per km to inundated roads (Dec 2009 $)
Major sealed roads	*$ 32,000*	*$ 16,000*	*$ 11,000*	$ 59,000	**$ 79,237**
Minor sealed roads	*$ 10,000*	*$ 5,000*	*$ 3,500*	$ 18,500	**$ 24,846**
Unsealed roads	*$ 4,500*	*$ 2,250*	*$ 1,600*	$ 8,350	**$ 11,214**

8.1.7 Adopted Damage Curves

The adopted damage curves are shown in Figure 7.1. As described above, the commercial damage curve are standardised for a property of 100 m^2.

8.2 Annual Average Damage

Annual Average Damage (AAD) is calculated on a probability approach, using the flood damages calculated for each design event.

Flood damages (for a design event) are calculated using the 'damage curves' described in the sections above. These damage curves approximate the damage occurring on a property for varying depths of flooding. The total damages in the summation of the damage to all houses and properties within the flood extent for that design event.

The AAD attempts to quantify flood damage that a floodplain would receive on average during a single year. It does this by using a probability approach. A probability curve is drawn, based on the flood damages calculated for each design event. This is shown in Figure 7.2. For the example, the 100 year ARI design event has a 1% chance of occurring in any given year, and as such the 100 year ARI flood damage is plotted at this point on the AAD curve. AAD is then calculated by determining the area under the curve.

Further information on the calculation of AAD can be found in the *Floodplain Development Manual* (NSW Government, 2005).

8.3 Results

The results of the flood damage assessment are shown in Table 8.5. Based on the analysis as described in the above section the annual average damages (AAD) for the floodplain under existing conditions is approximately **$ 491,783**.

Table 8.5 – Summary of Economic Flood Damages

Site location	Properties with over floor flooding	Properties with over ground flooding	Total Damages ($ Dec 2009)
5 year ARI			
Residential	3	295	$ 682,503
Commercial	0	0	$ -
Road and infrastructure damage	-	-	$ 59,352
5 year ARI total	**3**	**295**	**$ 741,854**
10 year ARI			
Residential	7	360	$ 1,086,125
Commercial	0	0	$ -
Road and infrastructure damage	-	-	$ 97,090
10 year ARI total	**7**	**360**	**$ 1,183,215**
20 year ARI			
Residential	16	423	$ 1,534,937
Commercial	0	0	$ -
Road and infrastructure damage	-	-	$ 135,271
20 year ARI total	**16**	**423**	**$ 1,670,208**
50 year ARI			
Residential	44	595	$ 2,775,854
Commercial	3	3	$ 1,352,201
Road and infrastructure damage	-	-	$ 180,792
50 year ARI total	**47**	**595**	**$ 4,308,847**
100 year ARI			
Residential	146	842	$ 6,806,551
Commercial	5	5	$ 1,762,440
Road and infrastructure damage	-	-	$ 254,431
100 year ARI total	**151**	**842**	**$ 8,823,422**
200 year ARI			
Residential	241	1063	$ 11,837,078
Commercial	5	6	$ 2,053,254
Road and infrastructure damage	-	-	$ 313,854
200 year ARI total	**246**	**1064**	**$ 14,204,187**

8.4 Assumption and Qualifications

A significant assumption is the calculation of the AAD was the assumption that the damages below the 5 year ARI were extrapolated with the assumption that there are no damages at the 2 year ARI event. Assuming a different slope for this line or a different ARI for zero damages will result in a change in the AAD calculated value. A paper was presented at the 2006 Floodplain Management Conference (Thomson et al, 2006) highlighting the issues associated with this assumption. In addition the AAD was calculated up to the 200 year ARI event rather than the PMF and this may reduce the AAD.

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Figures

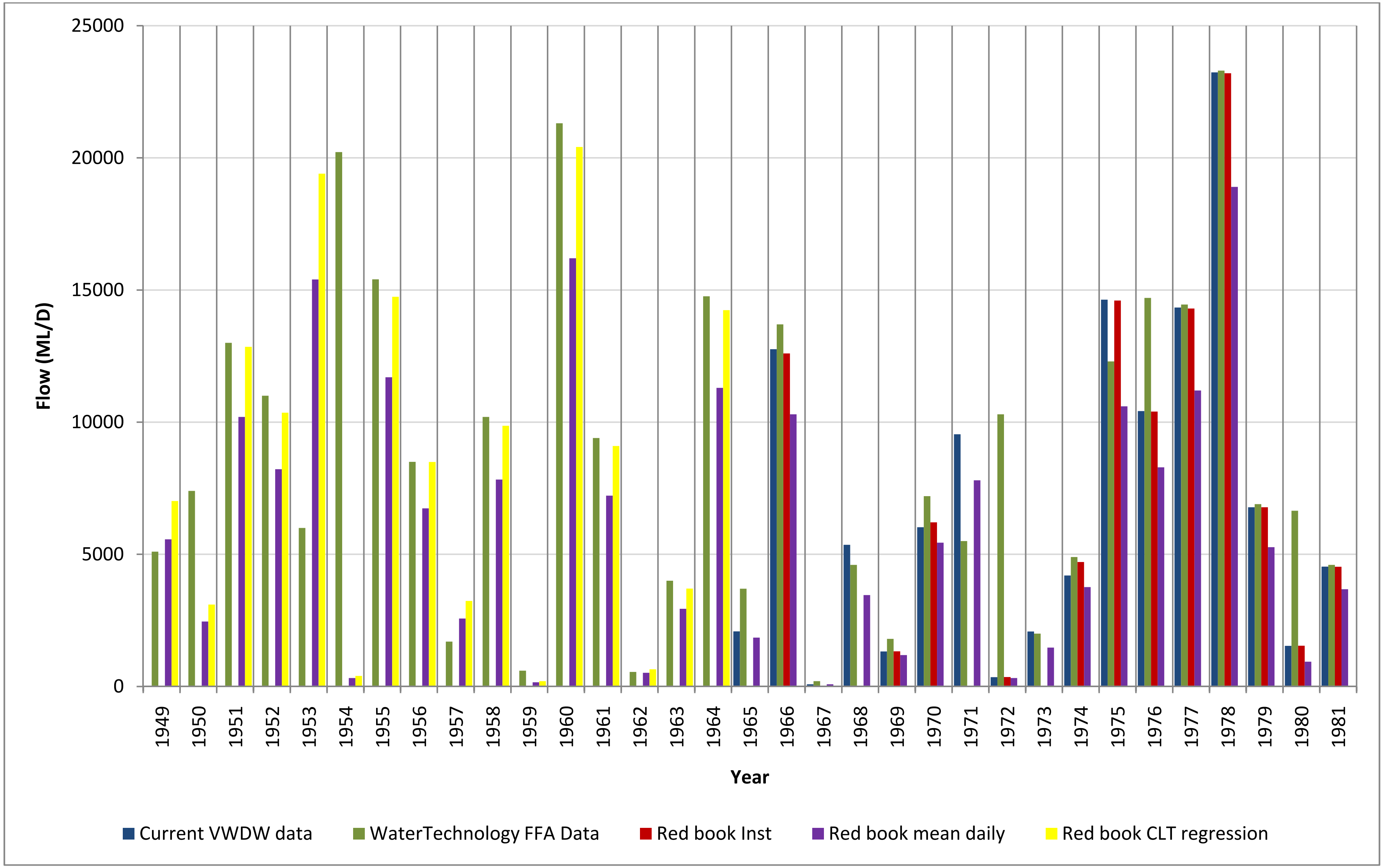

Figure 3.1 – Flow series from various data sets

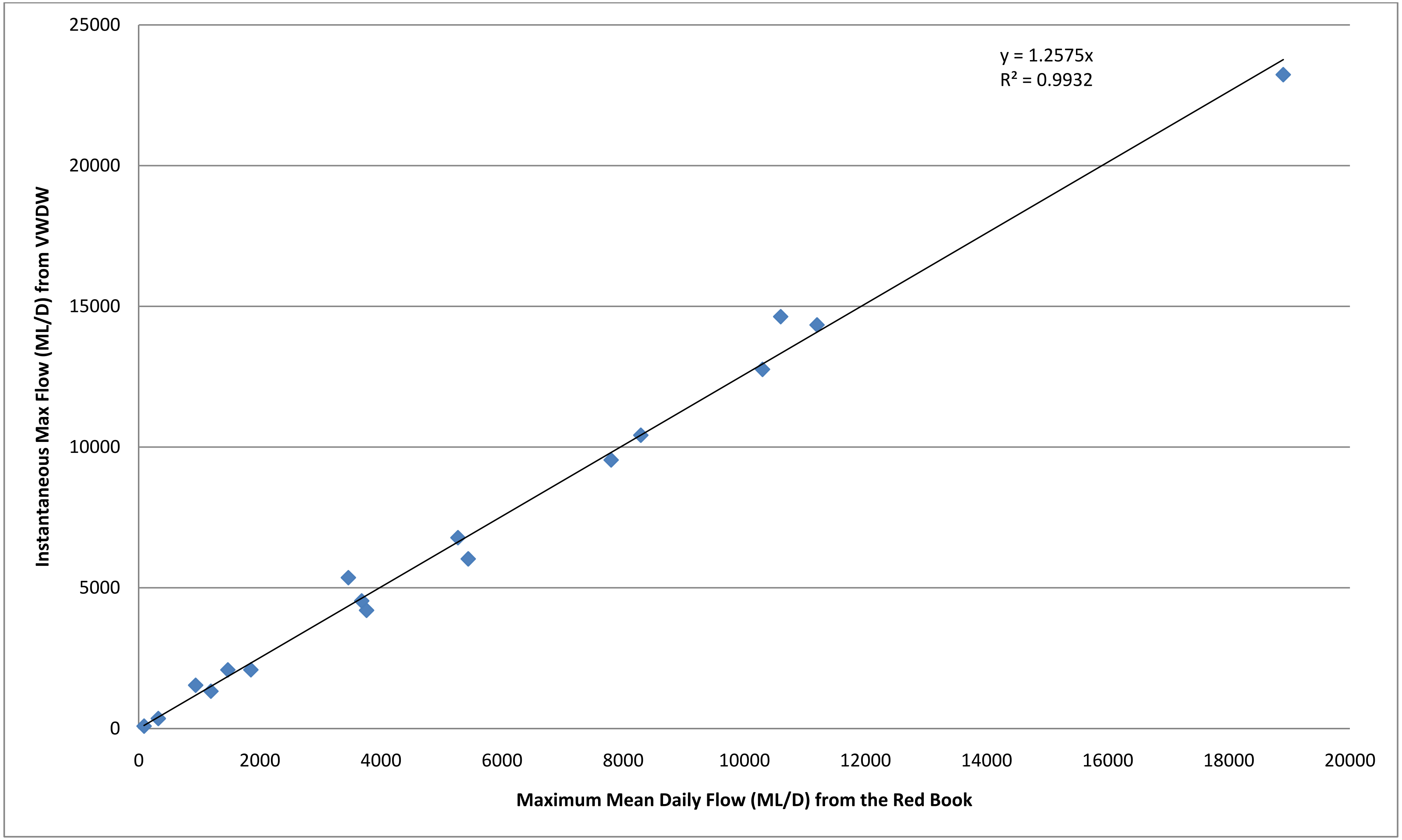

Figure 3.2 – Instantaneous max daily flow VWDW vs Max mean daily flow regression 'Red Book'

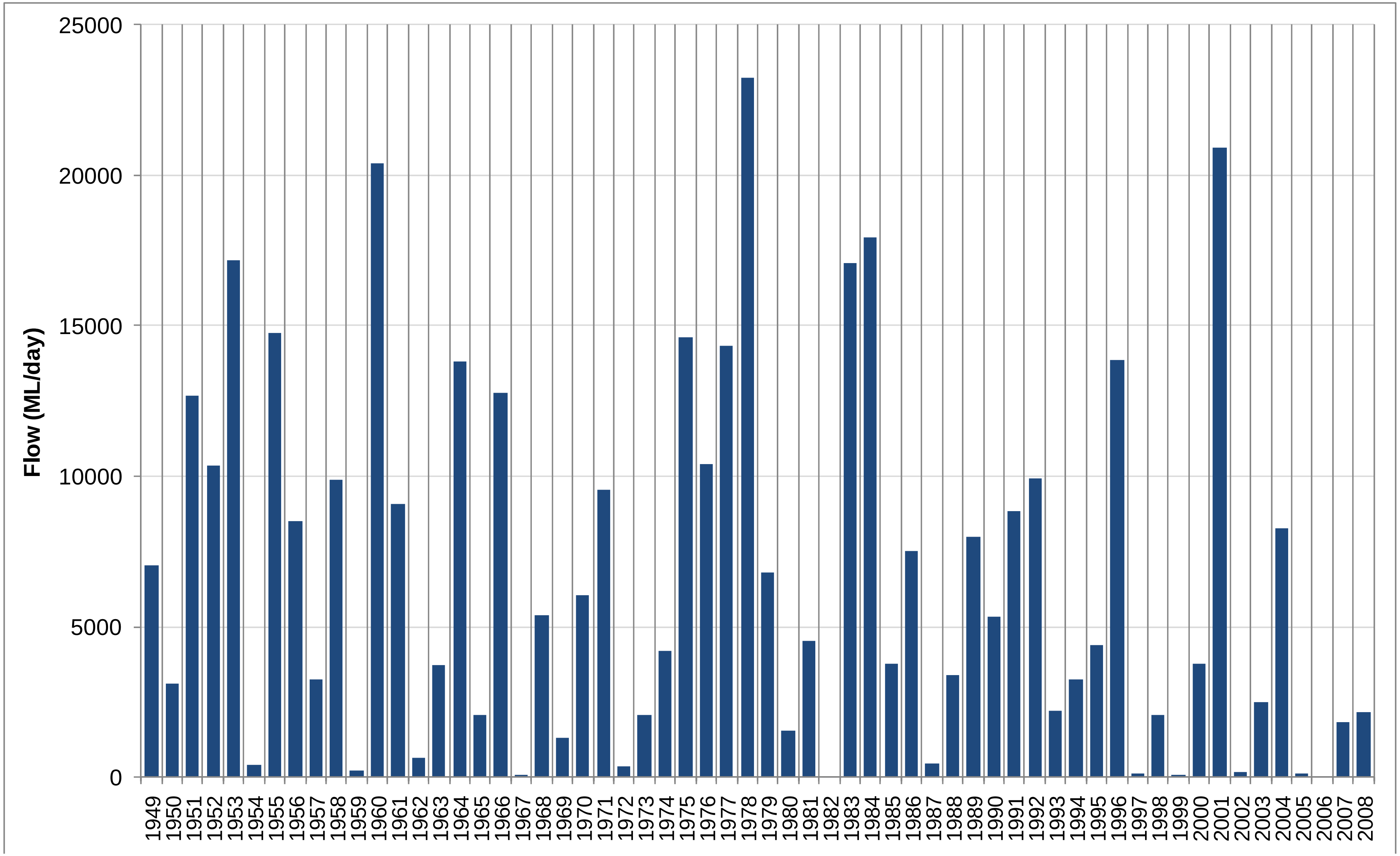

Figure 3.3 – Peak Annual Flows, Merri River @ Woodford

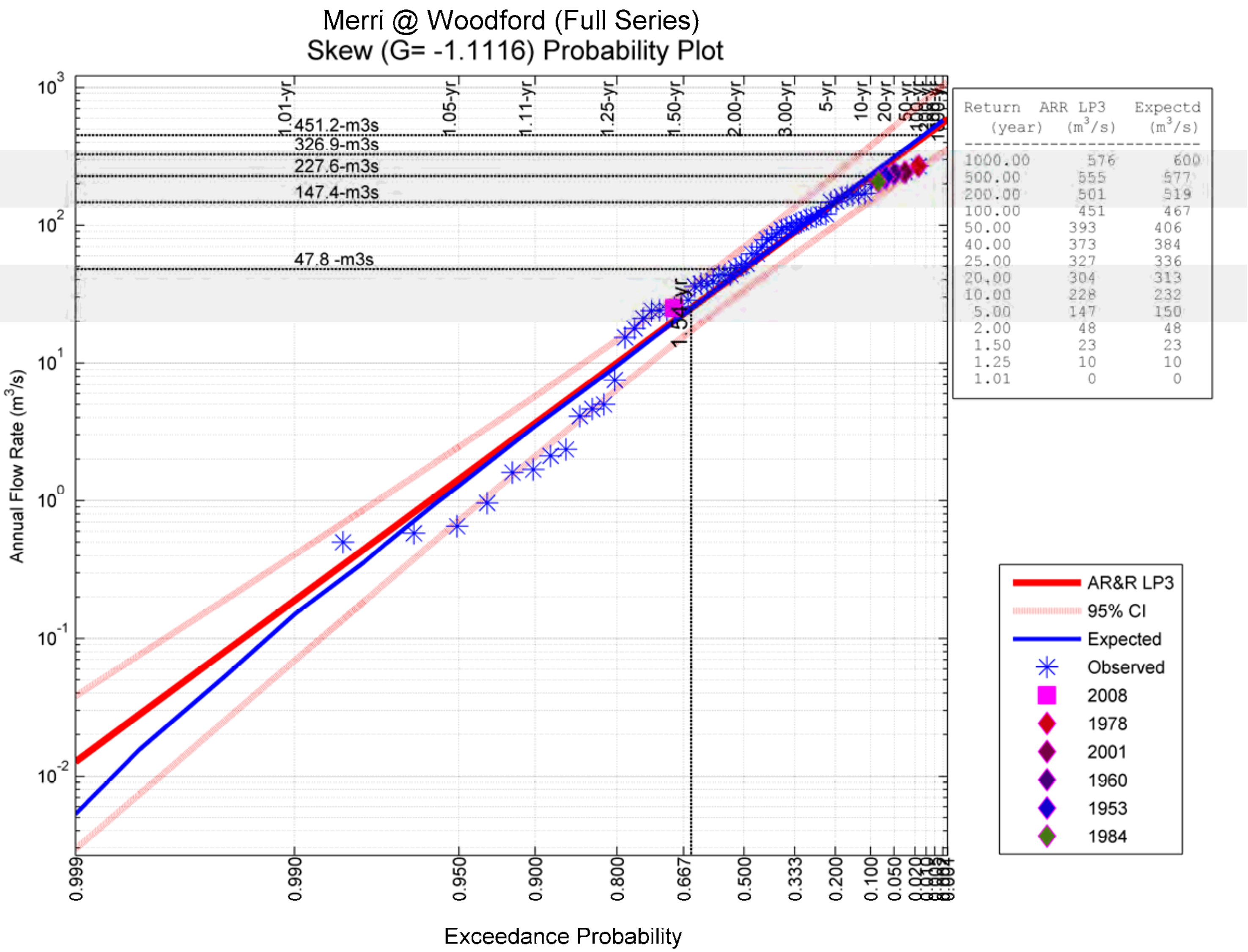

Figure 3.4 – FFA, Merri @ Woodford (1949-2008)

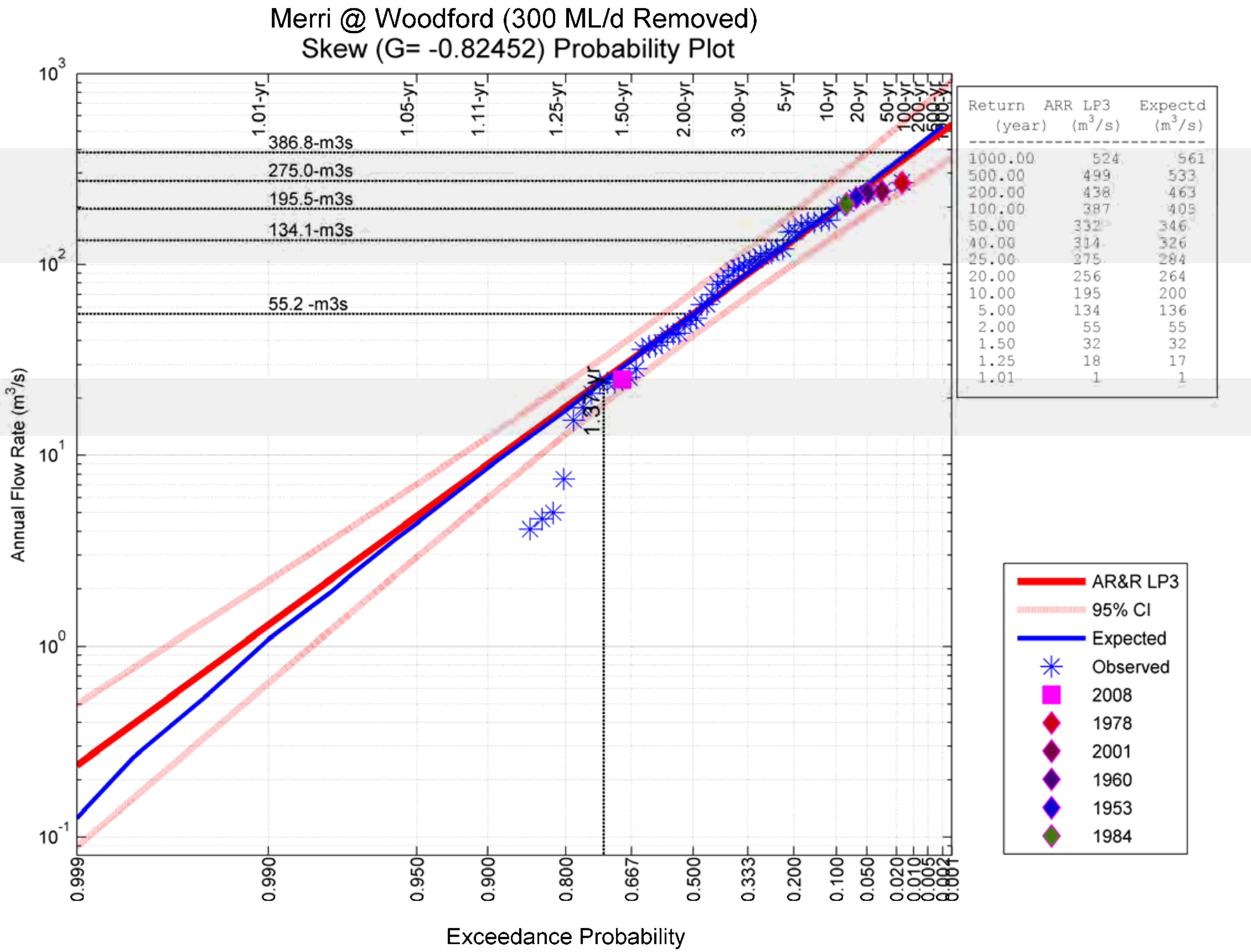

Figure 3.5 – FFA, Merri @ Woodford (1949-2008), Flow < 3.5 m^3/s removed

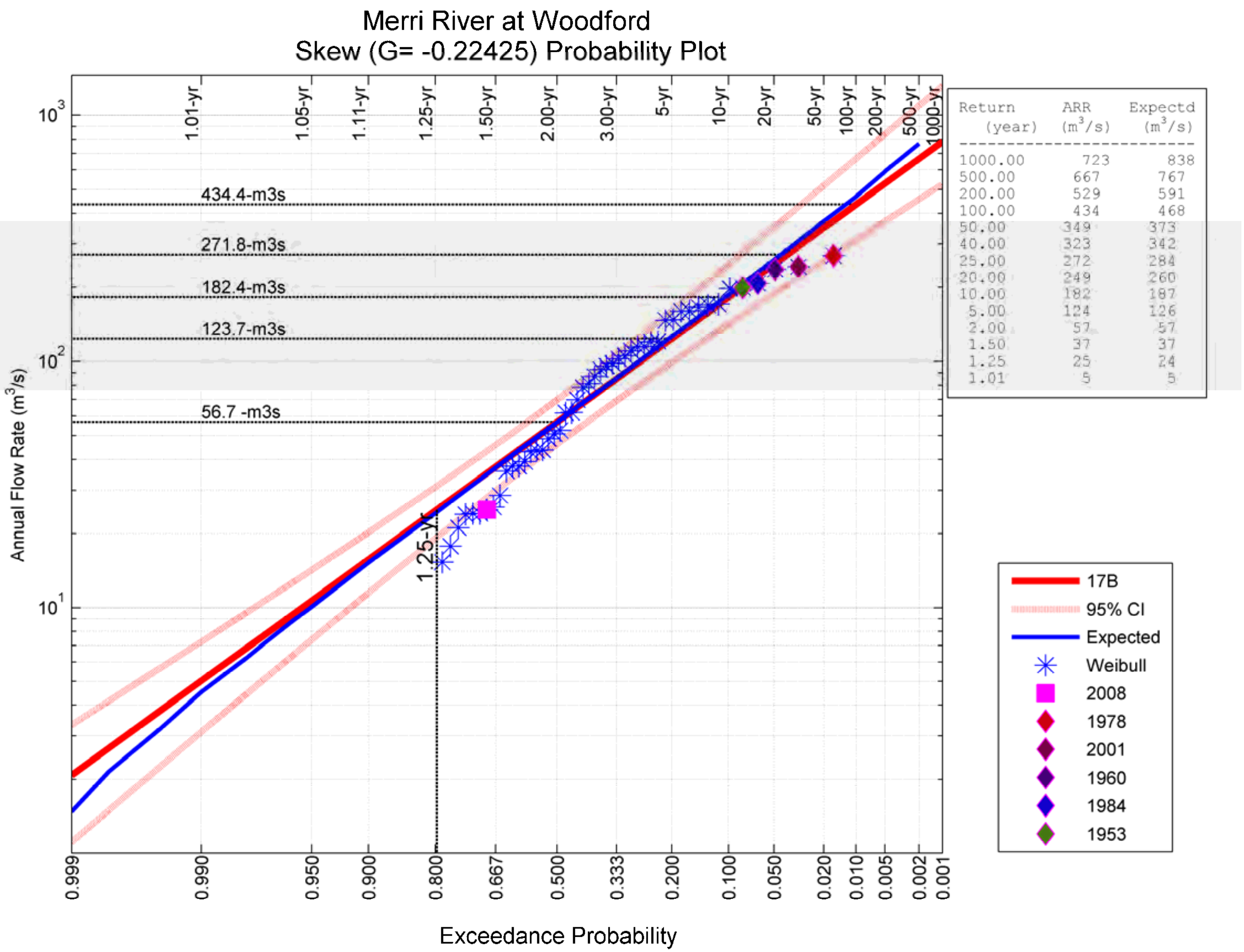

Figure 3.6 – FFA, Merri @ Woodford (1949-2008), Flow < 11.5 m^3/s removed

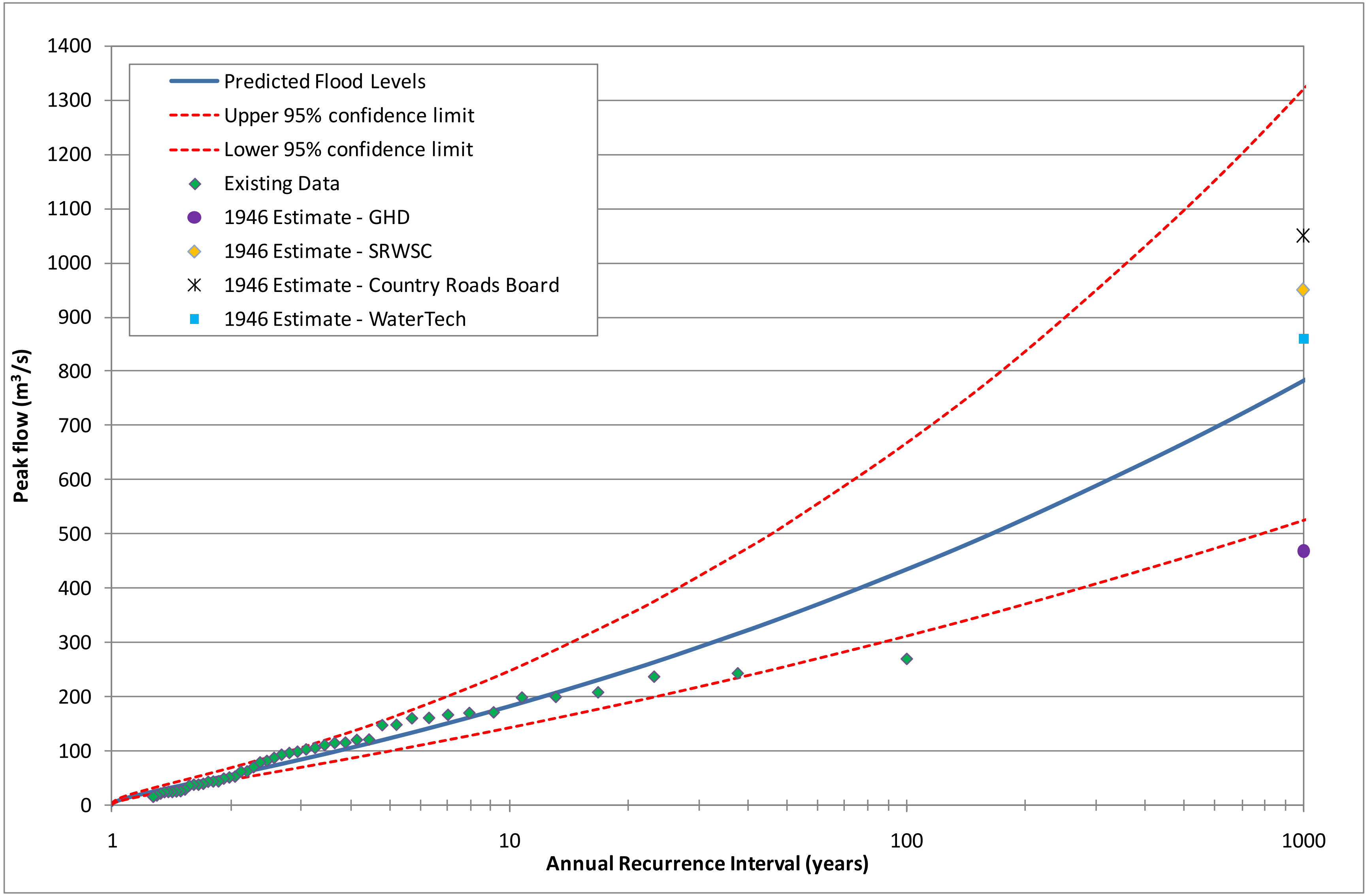

Figure 3.7 – FFA, Merri @ Woodford (1949-2008), Flow < 11.5 m^3/s removed, with 1946 estimates

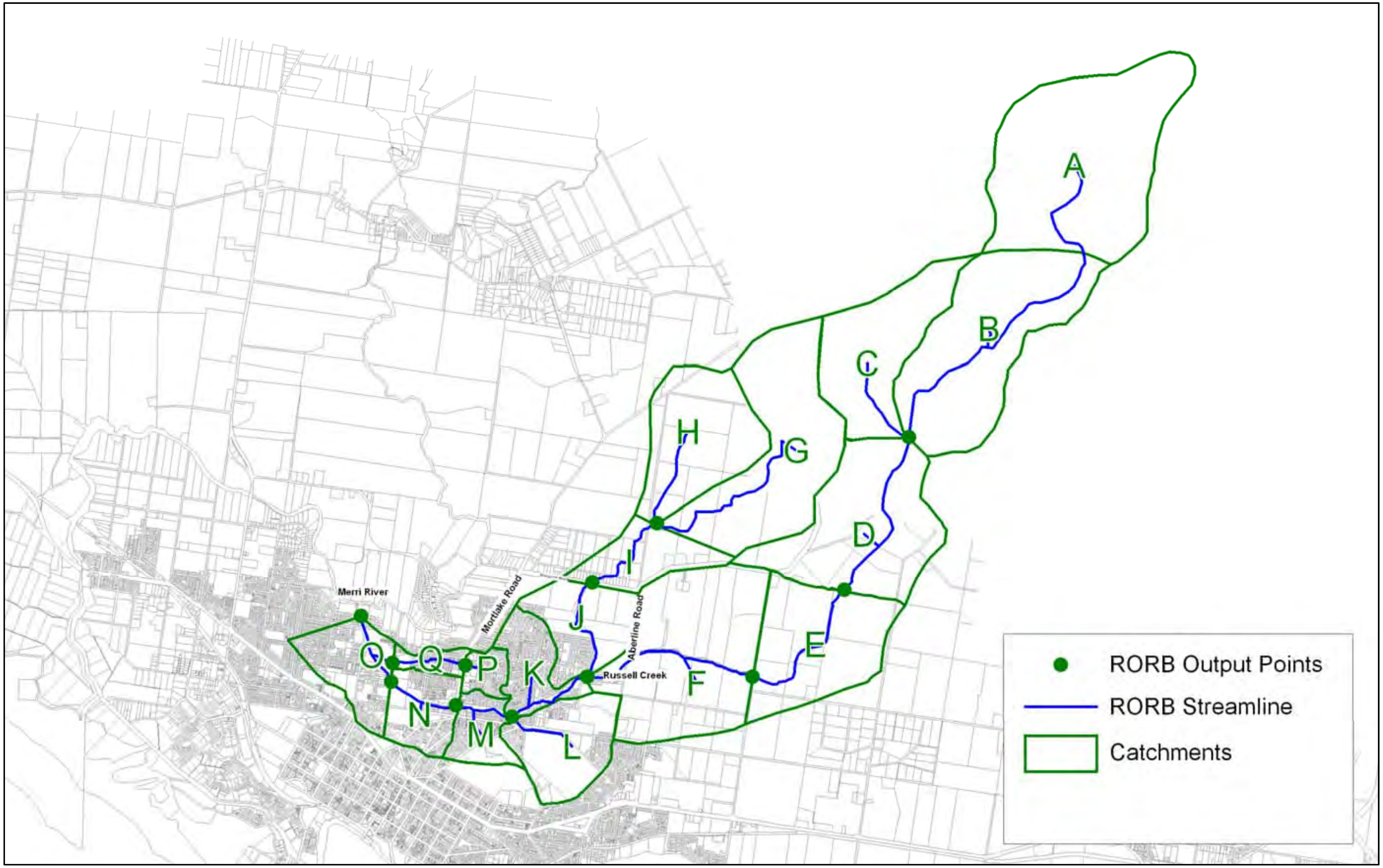

Figure 3.8 – Russell Creek RORB Model

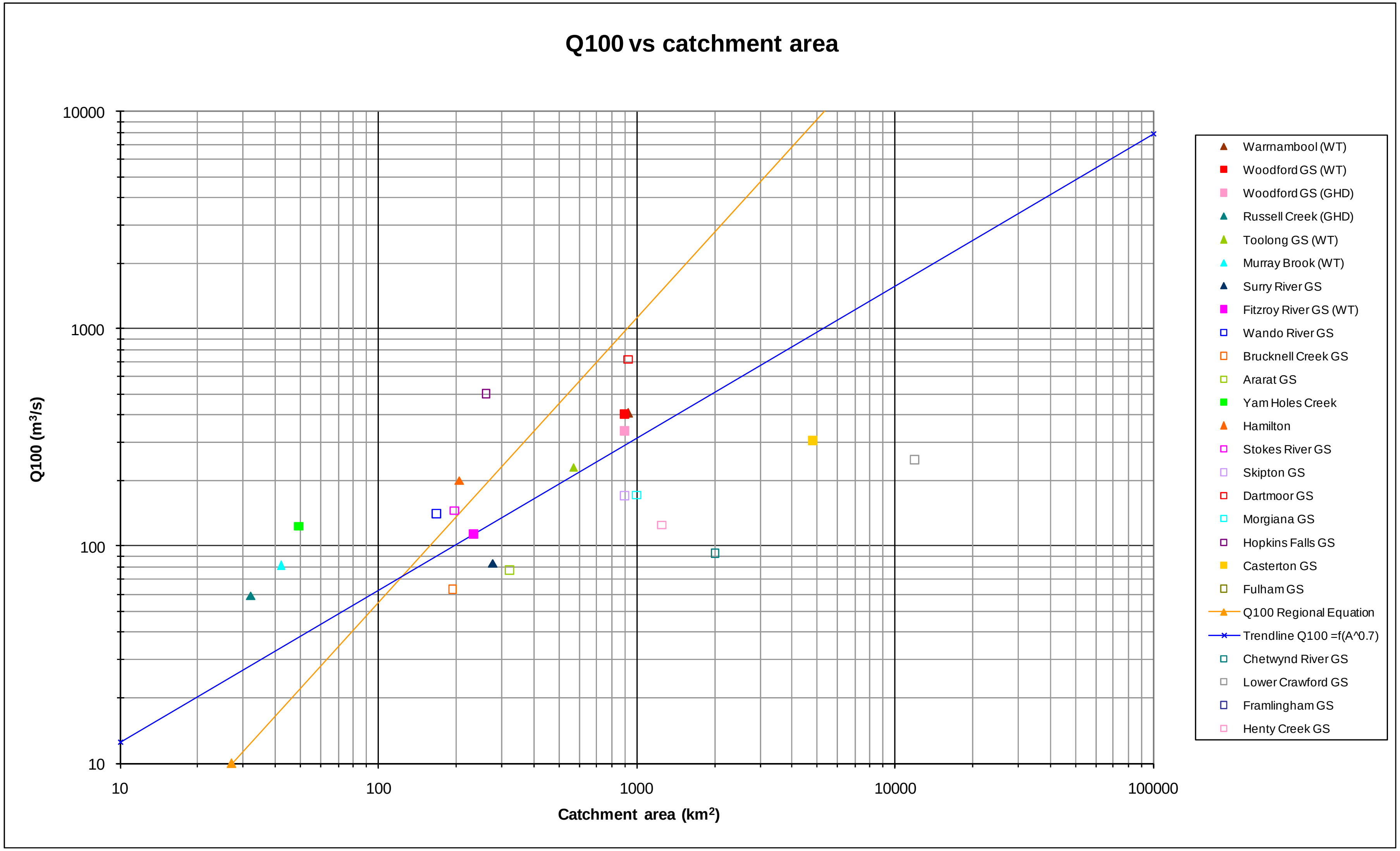

Figure 3.9 – Catchment Flow versus Catchment Size (GHCMA, 2010)

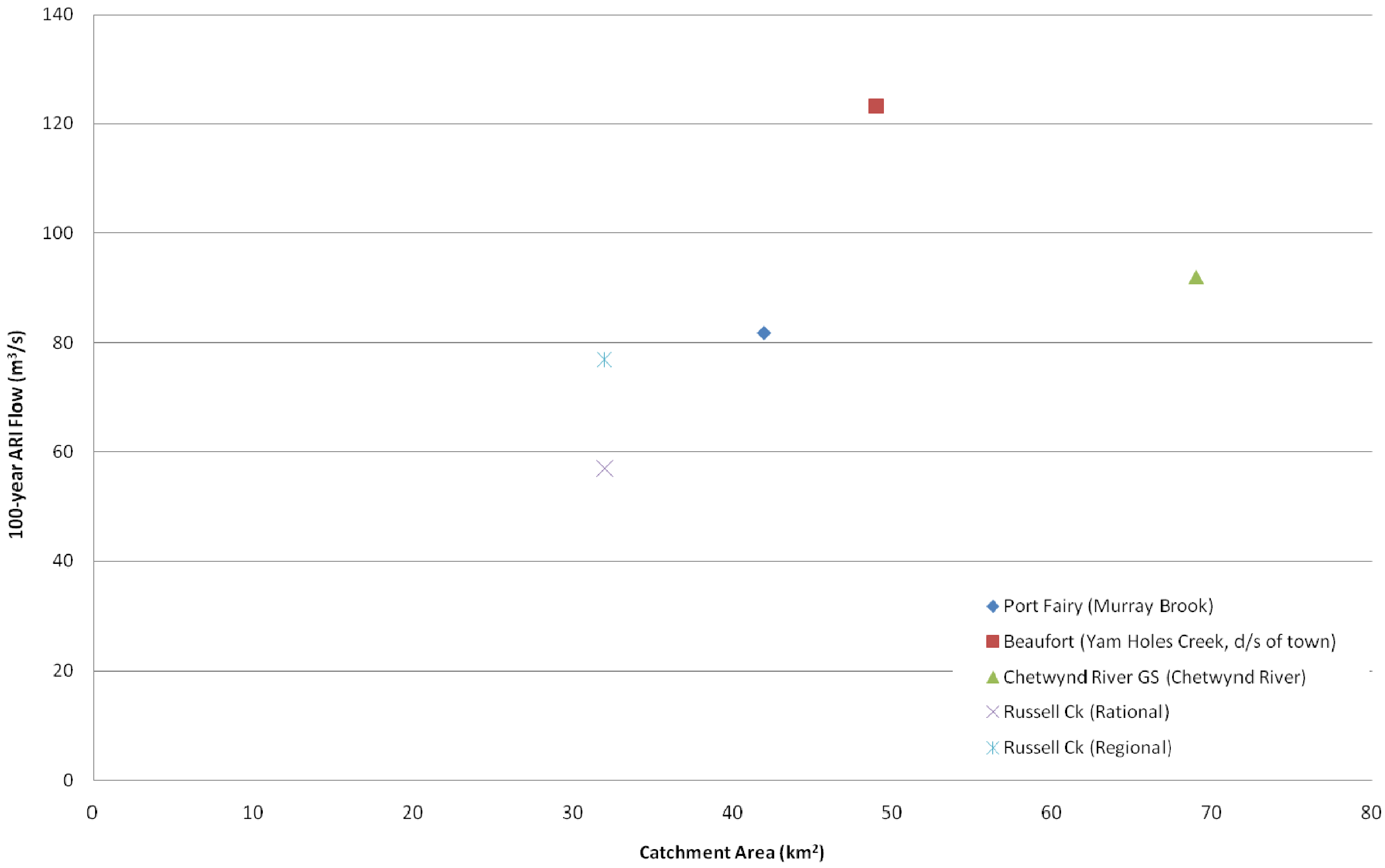

Figure 3.10 – Catchment Flow versus Catchment Size – catchments up to 80 km^2

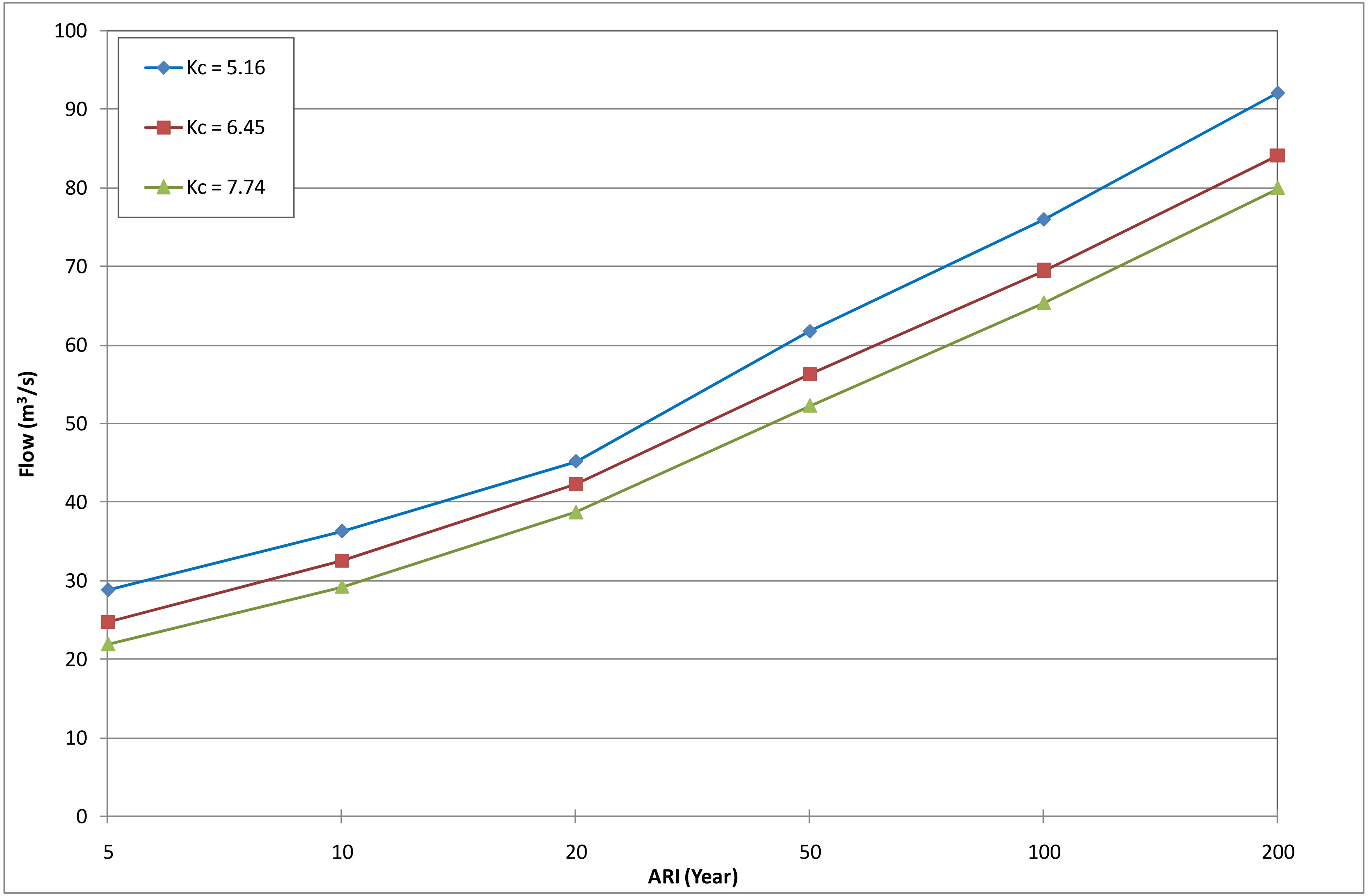

Figure 3.11 – **Russell Creek – Flow vs ARI for Differing k_c Values**

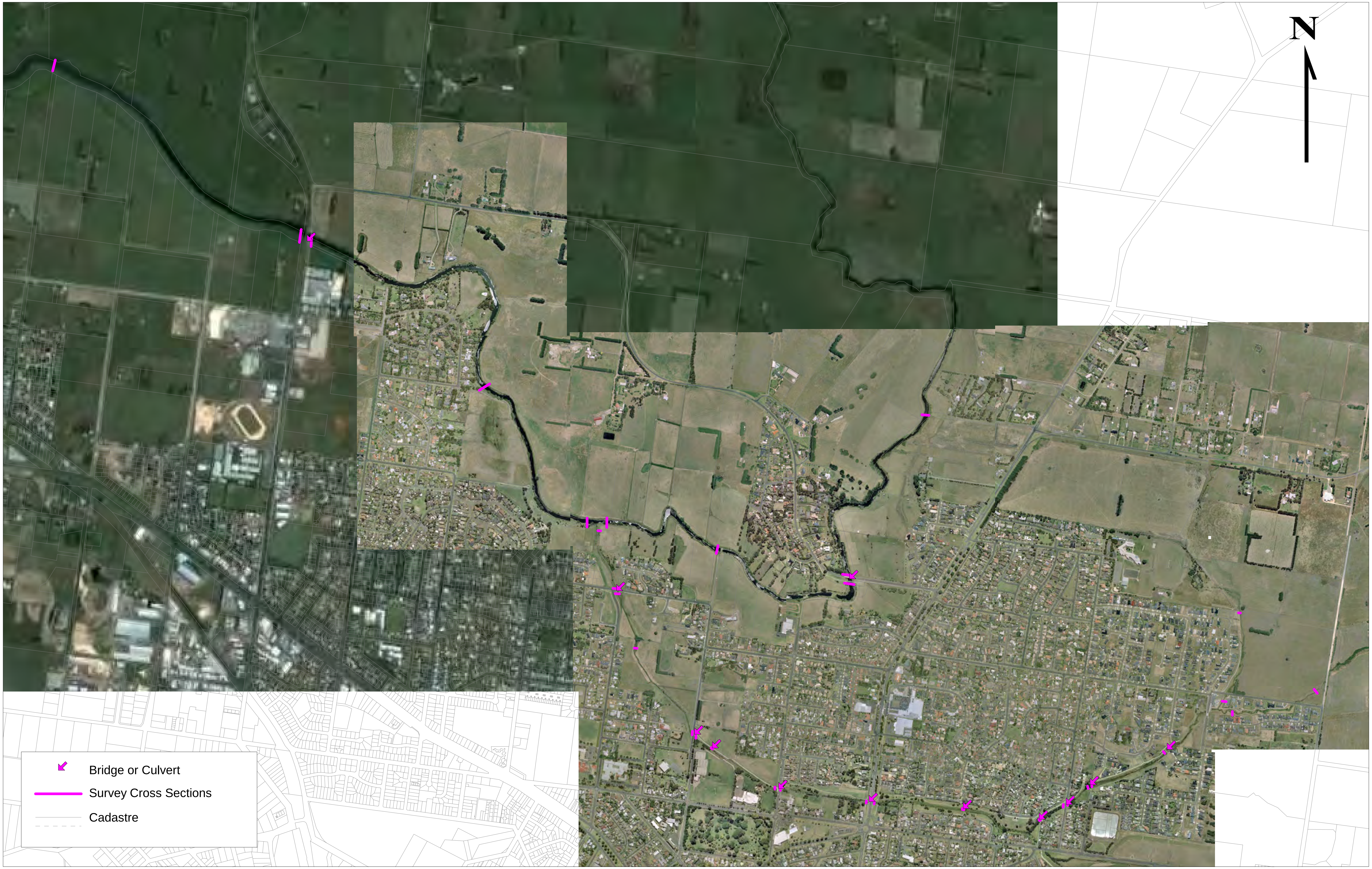

Figure 4.1 - Cross sections obtained from the field survey

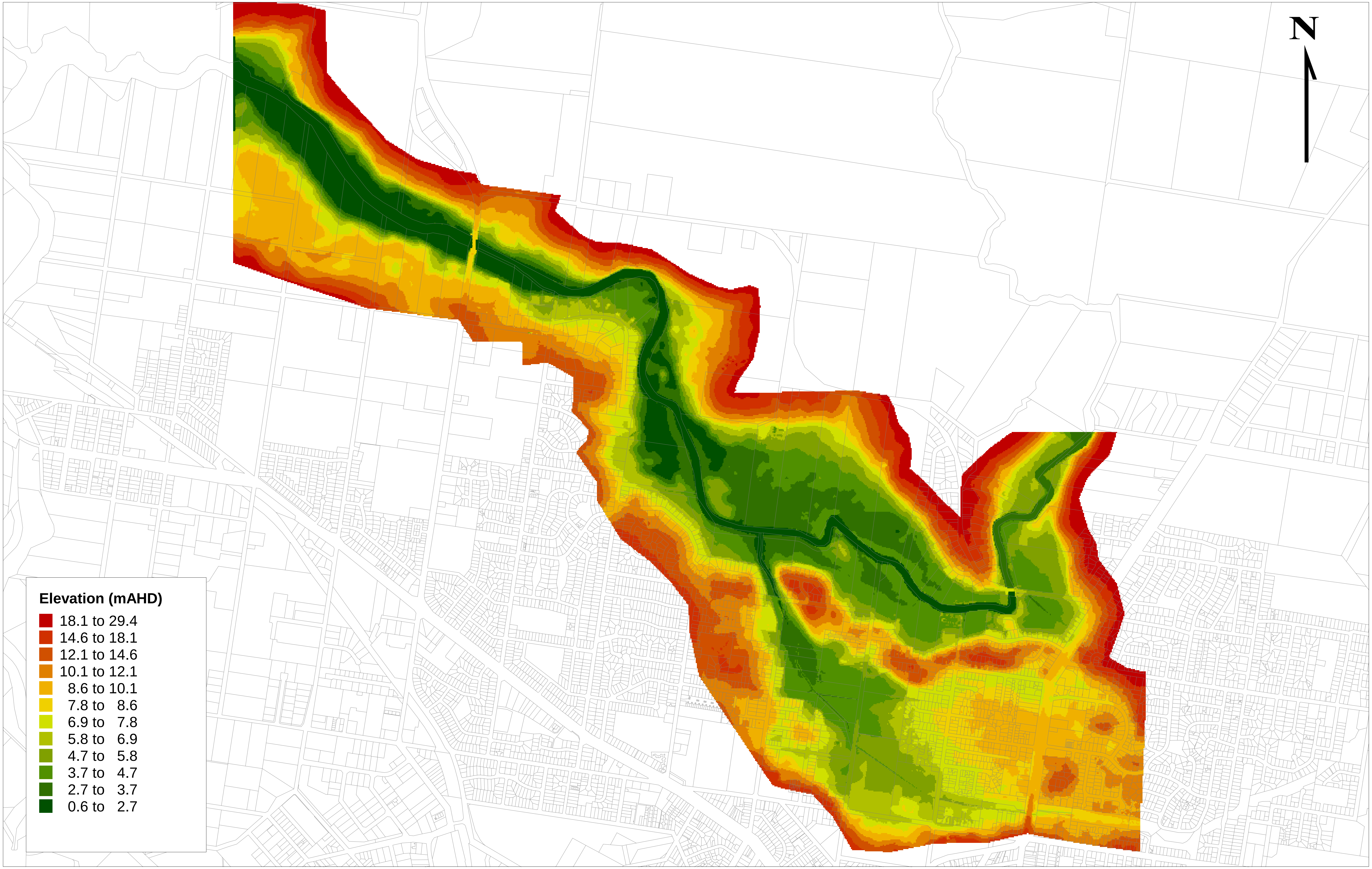

Figure 4.2 - The Merri River calibration topography

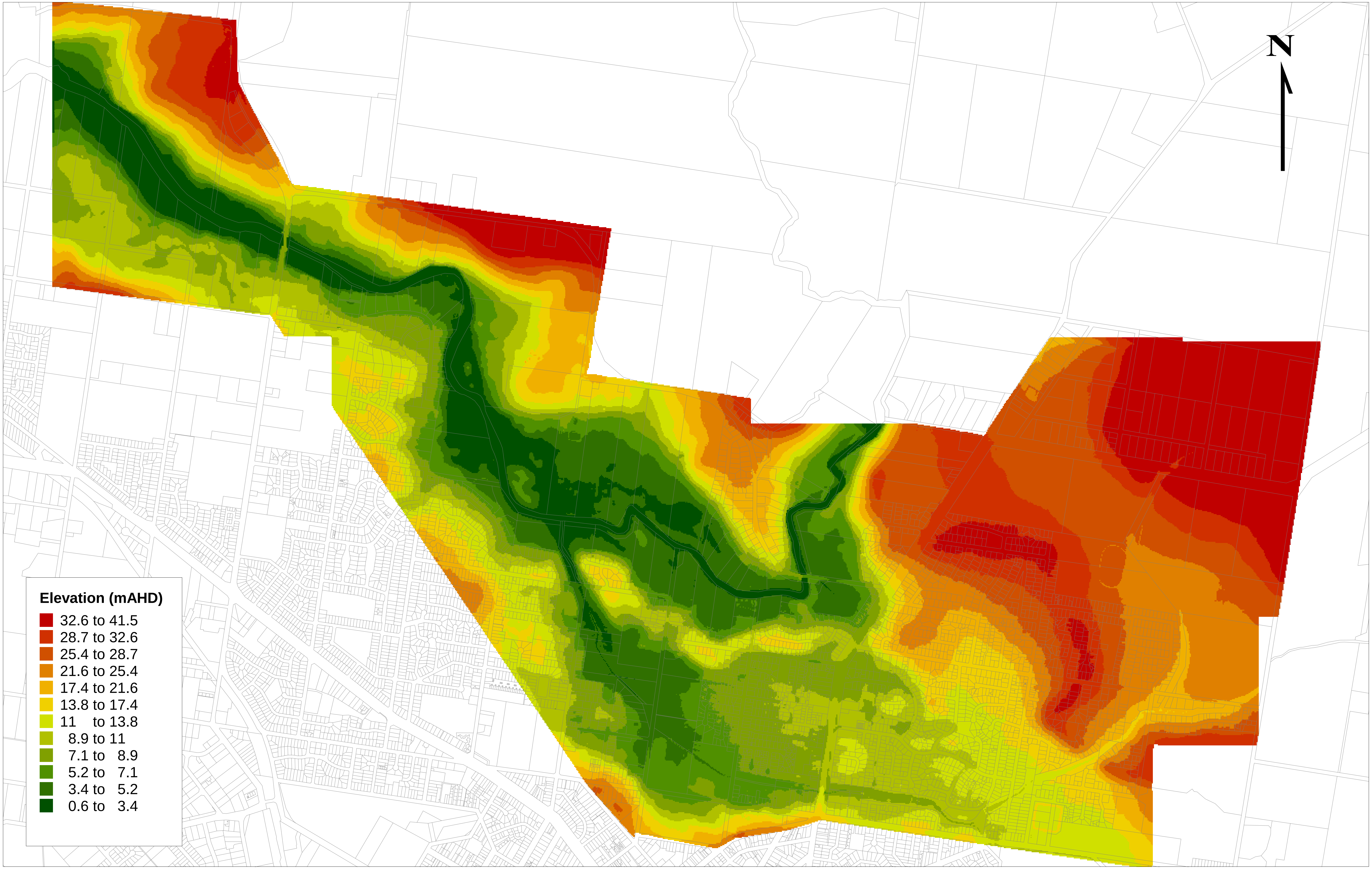

Figure 4.3 - The full topography layer for Merri River and Russel Creek

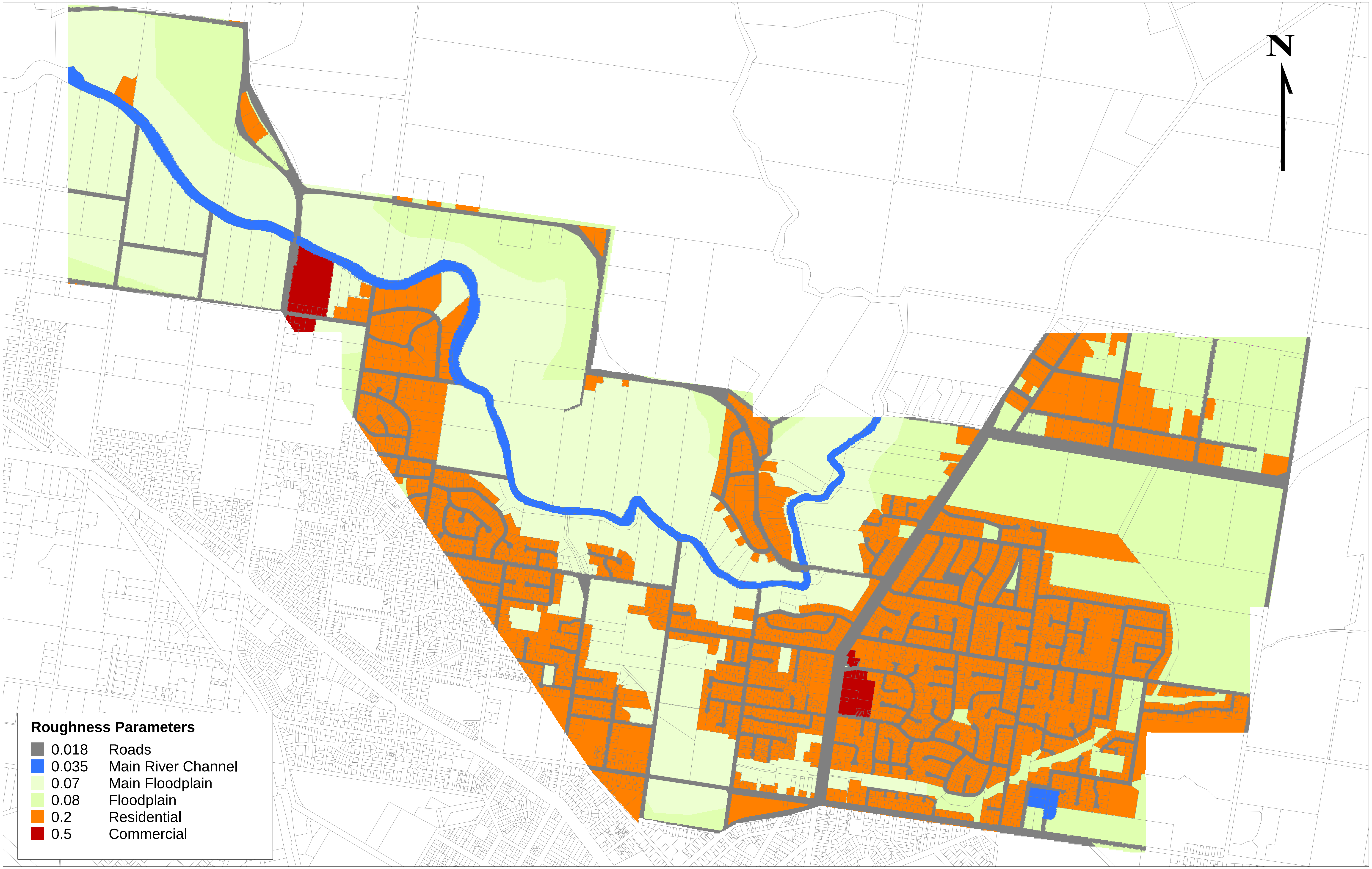

Figure 4.4 - The full roughness grid for Merri River and Russel Creek

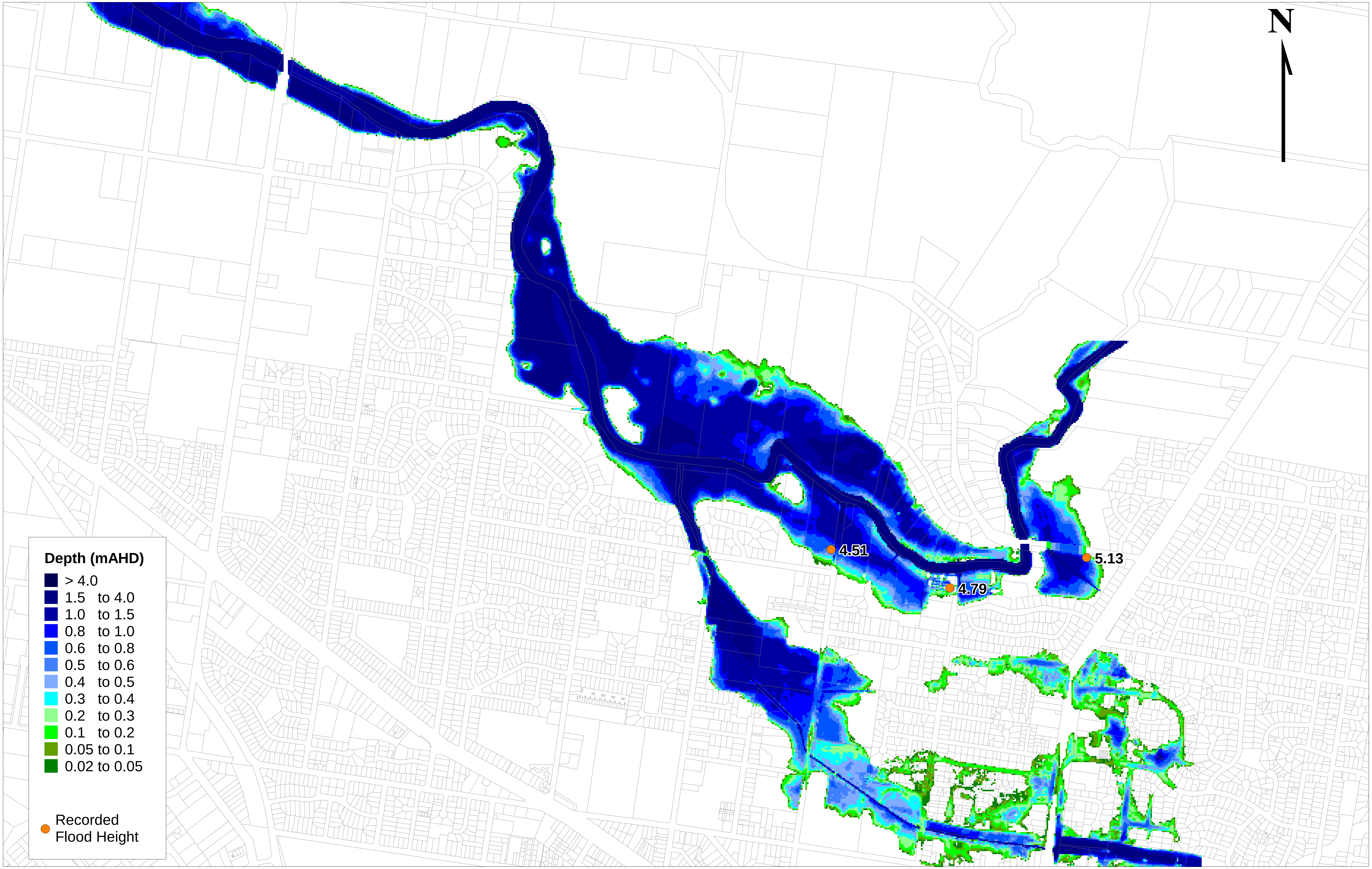

Figure 4.5 - Calibrated 1978 flood depths

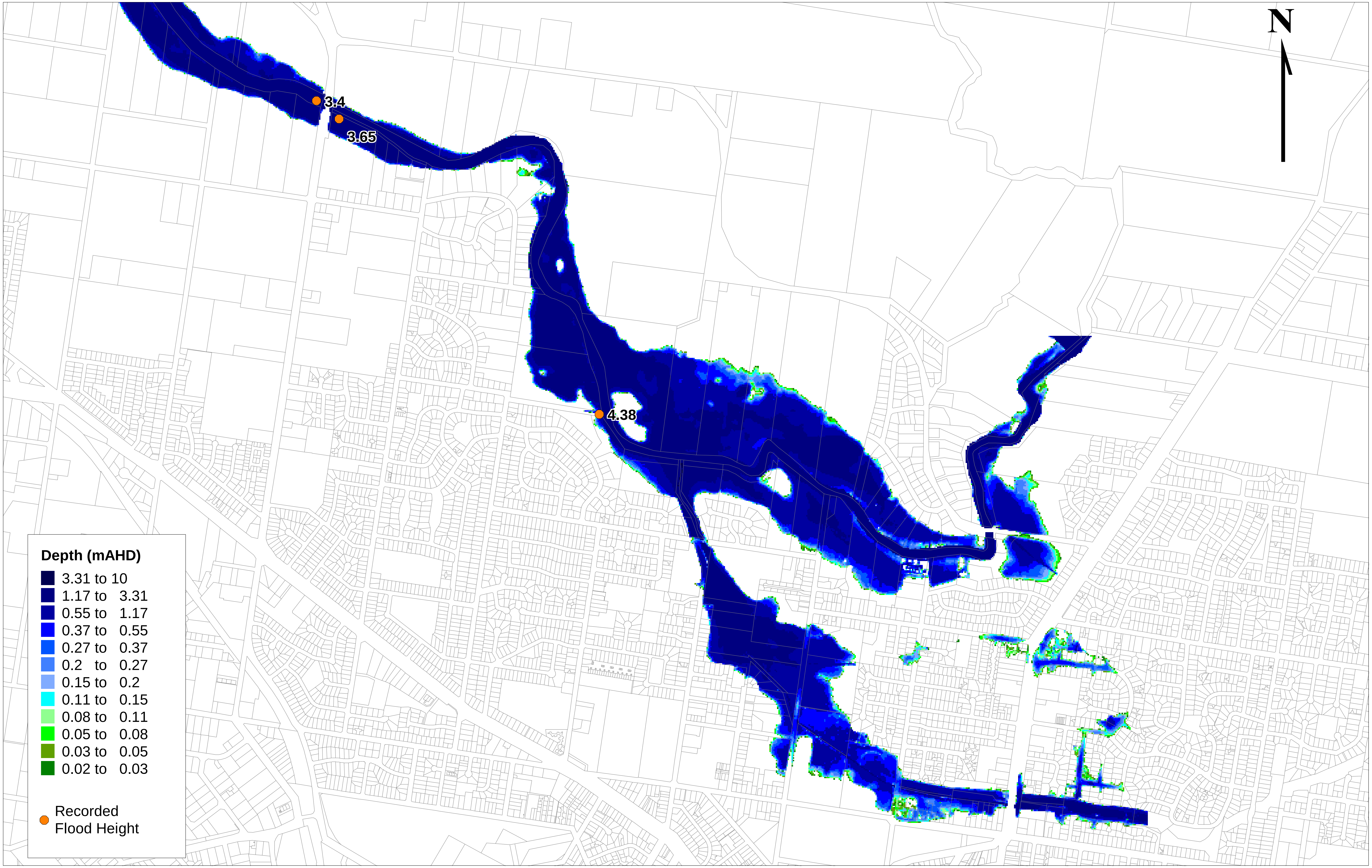

Figure 4.6 - Calibrated 2001 flood depths

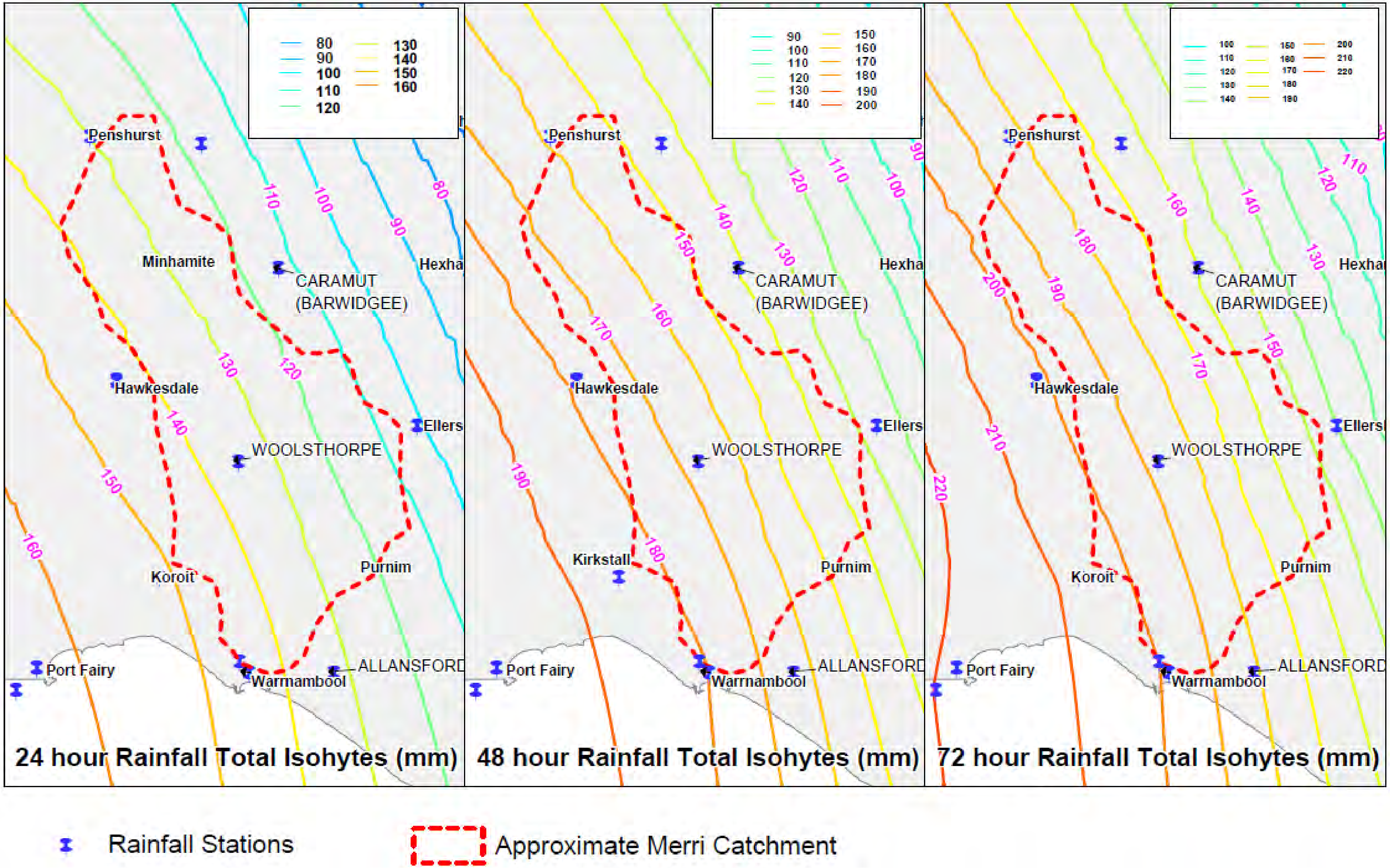

Figure 5.1 – Rainfall totals for the 24, 48 and 72 hour events in March 1946 derived from regional rainfall gauges

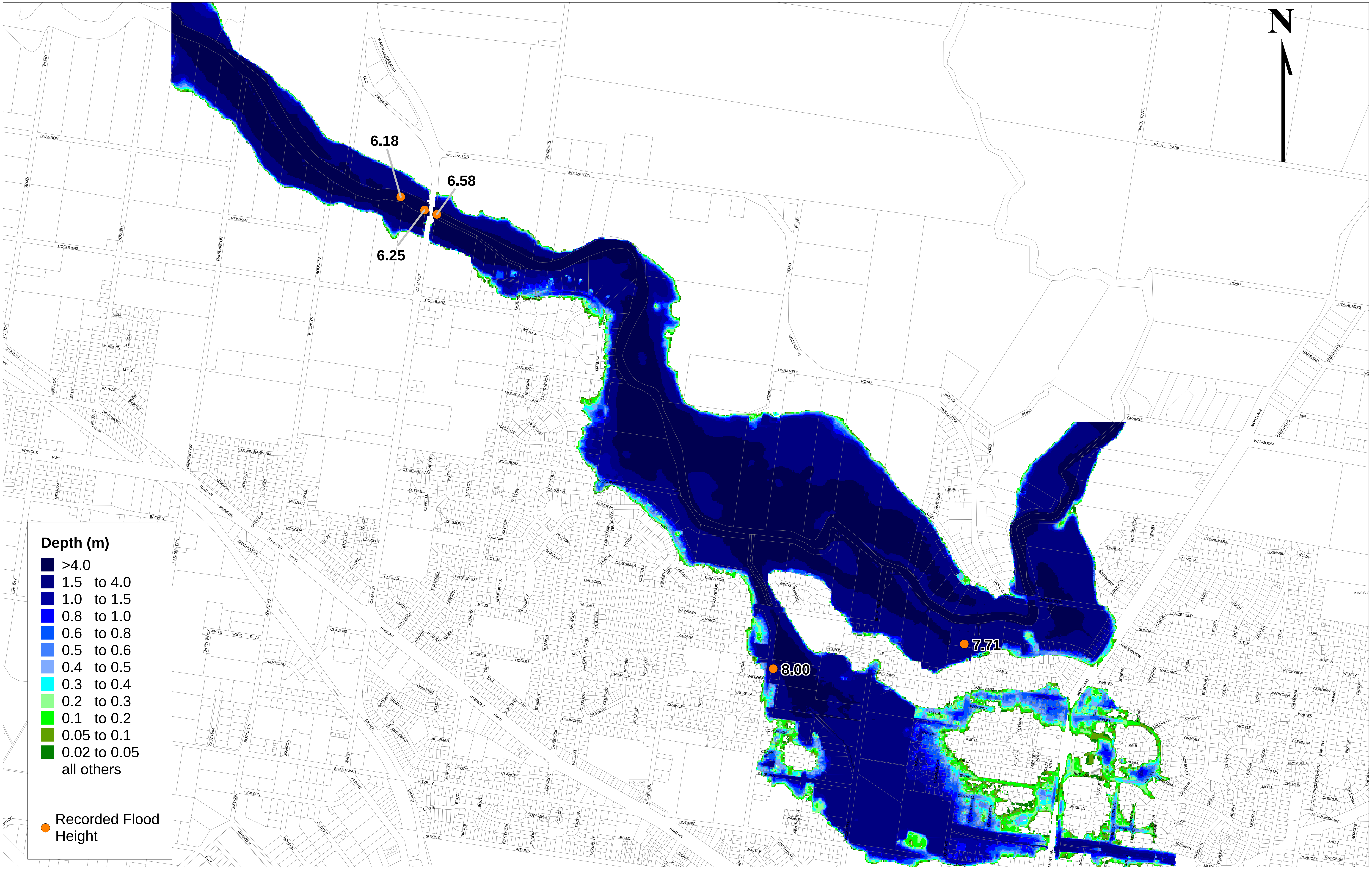

Figure 5.2 - Calibrated 1946 flood depths

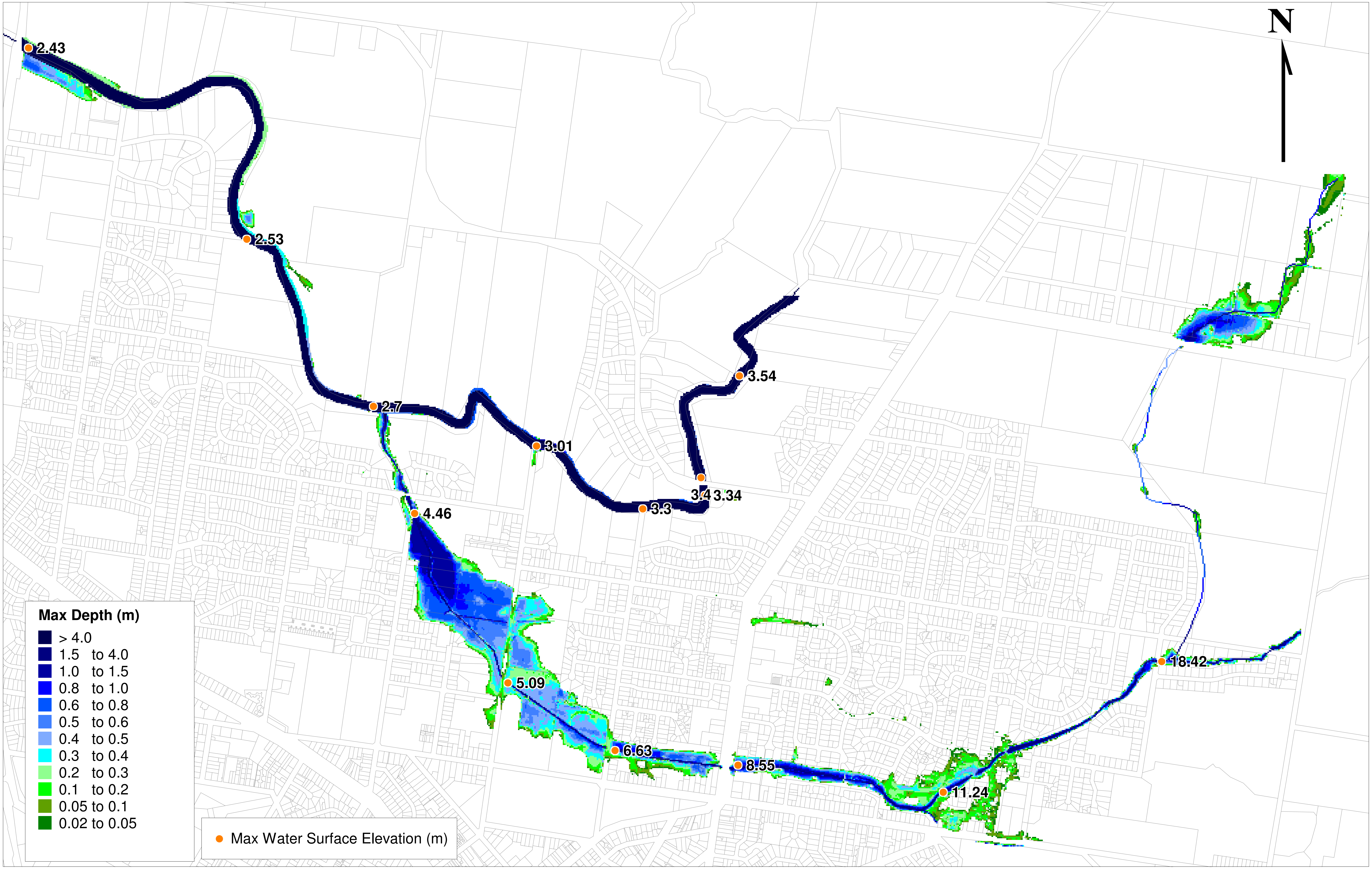

Figure 6.1 - 5 year ARI flood extents and depths

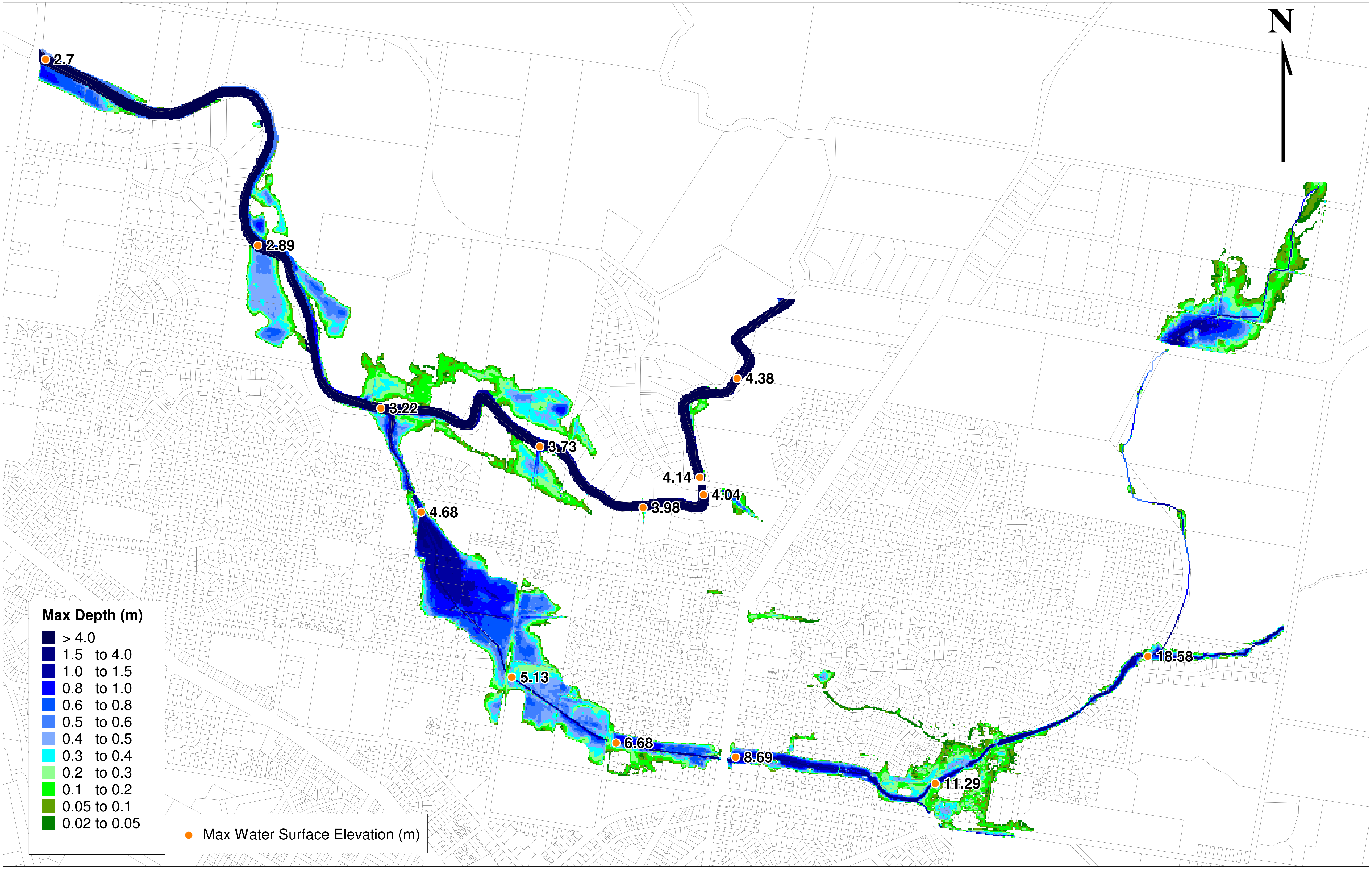

Figure 6.2 - 10 year ARI flood extents and depths

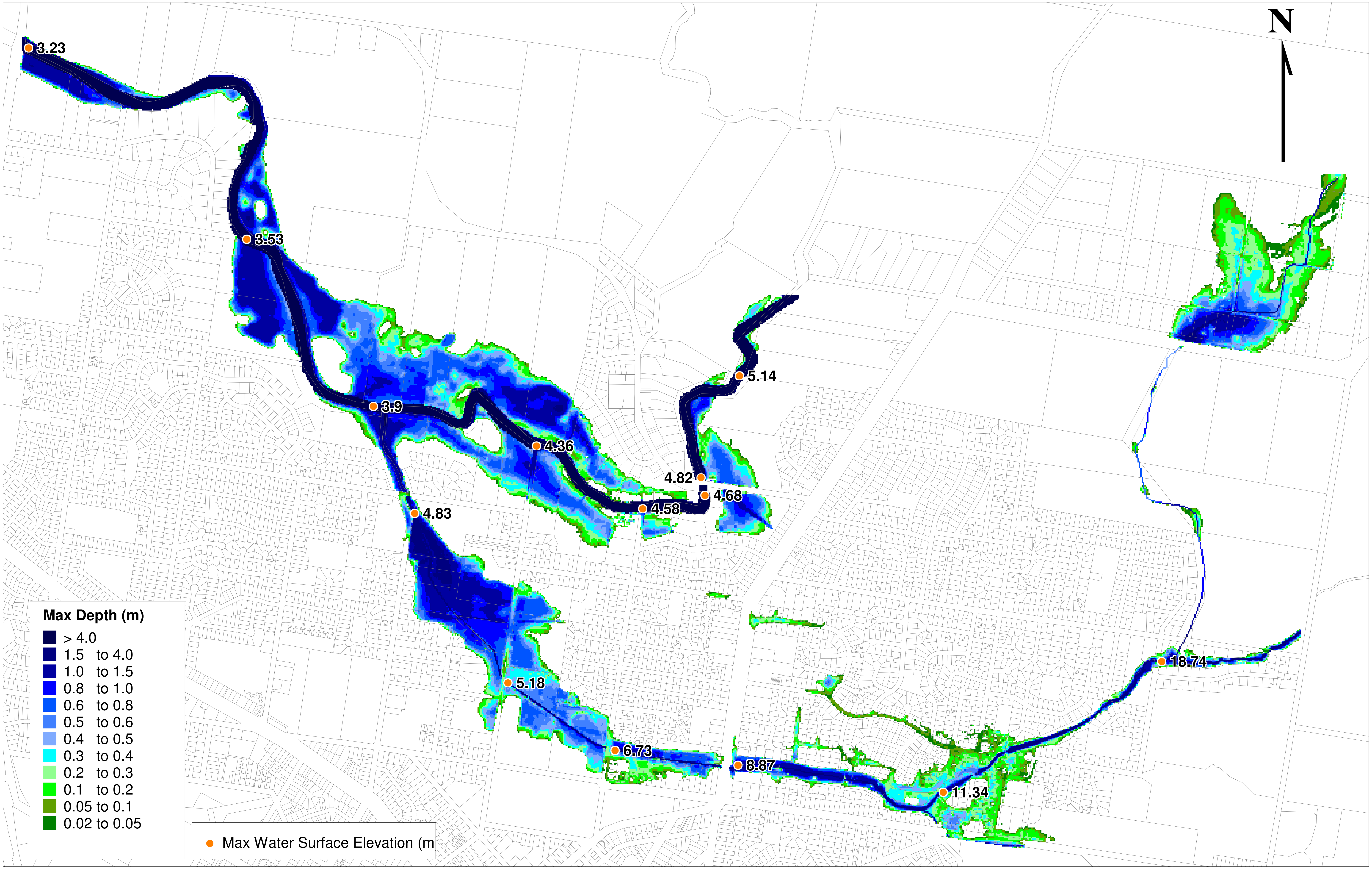

Figure 6.3 - 20 year ARI flood extents and depths

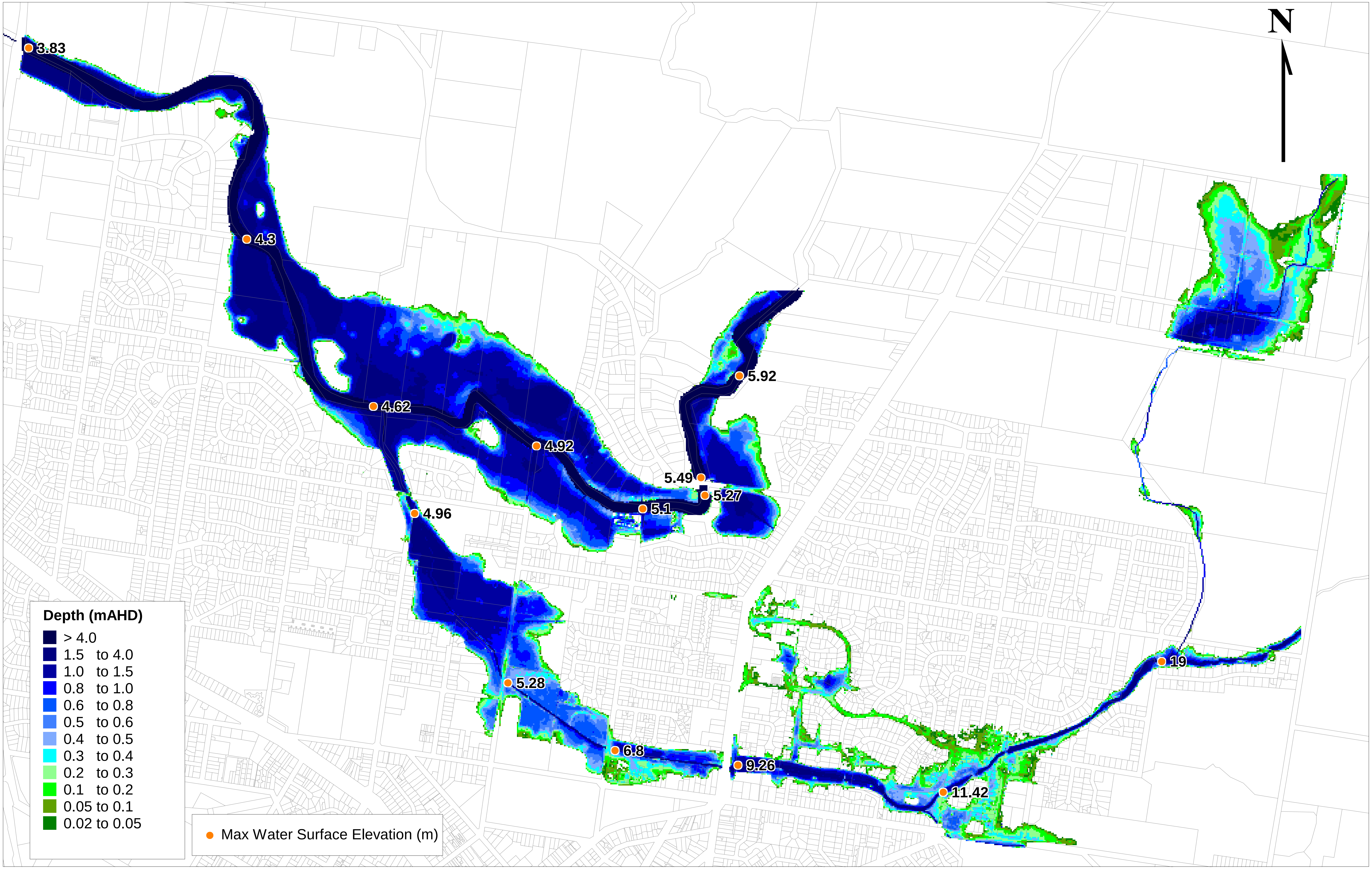

Figure 6.4 - 50 year ARI flood extents and depths

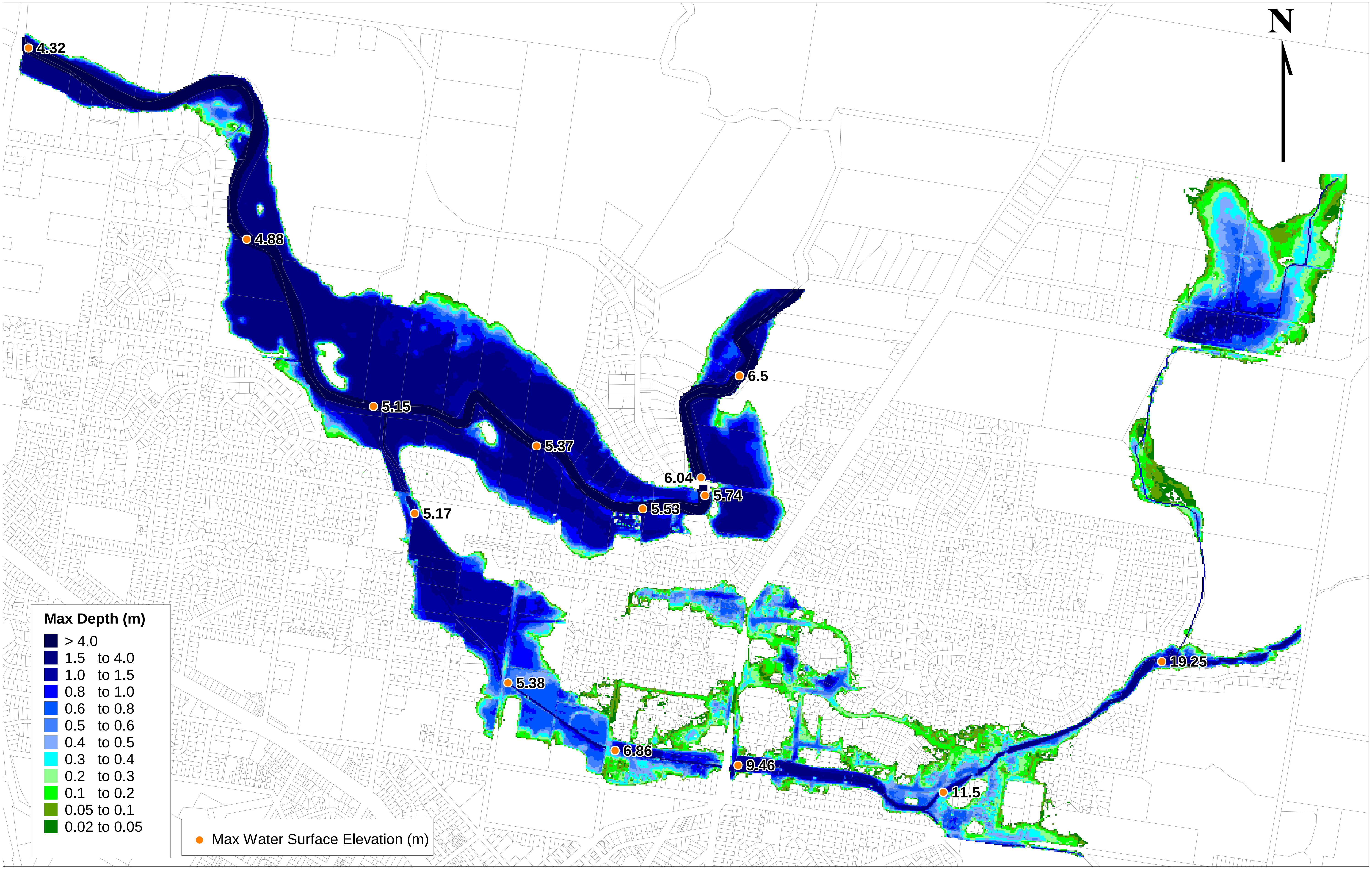

Figure 6.5 - 100 year ARI flood extents and depths

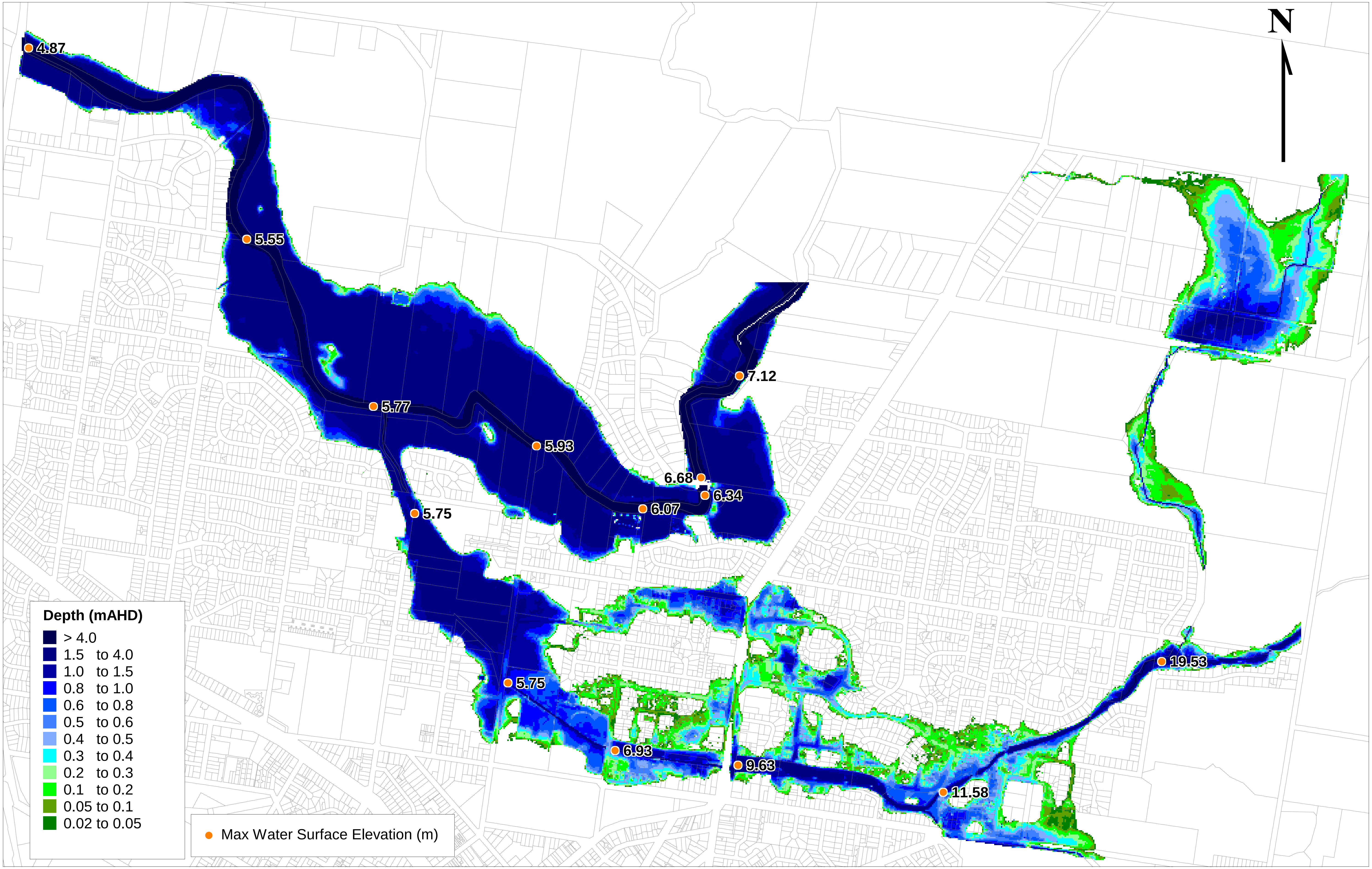

Figure 6.6 - 200 year ARI flood extents and depths

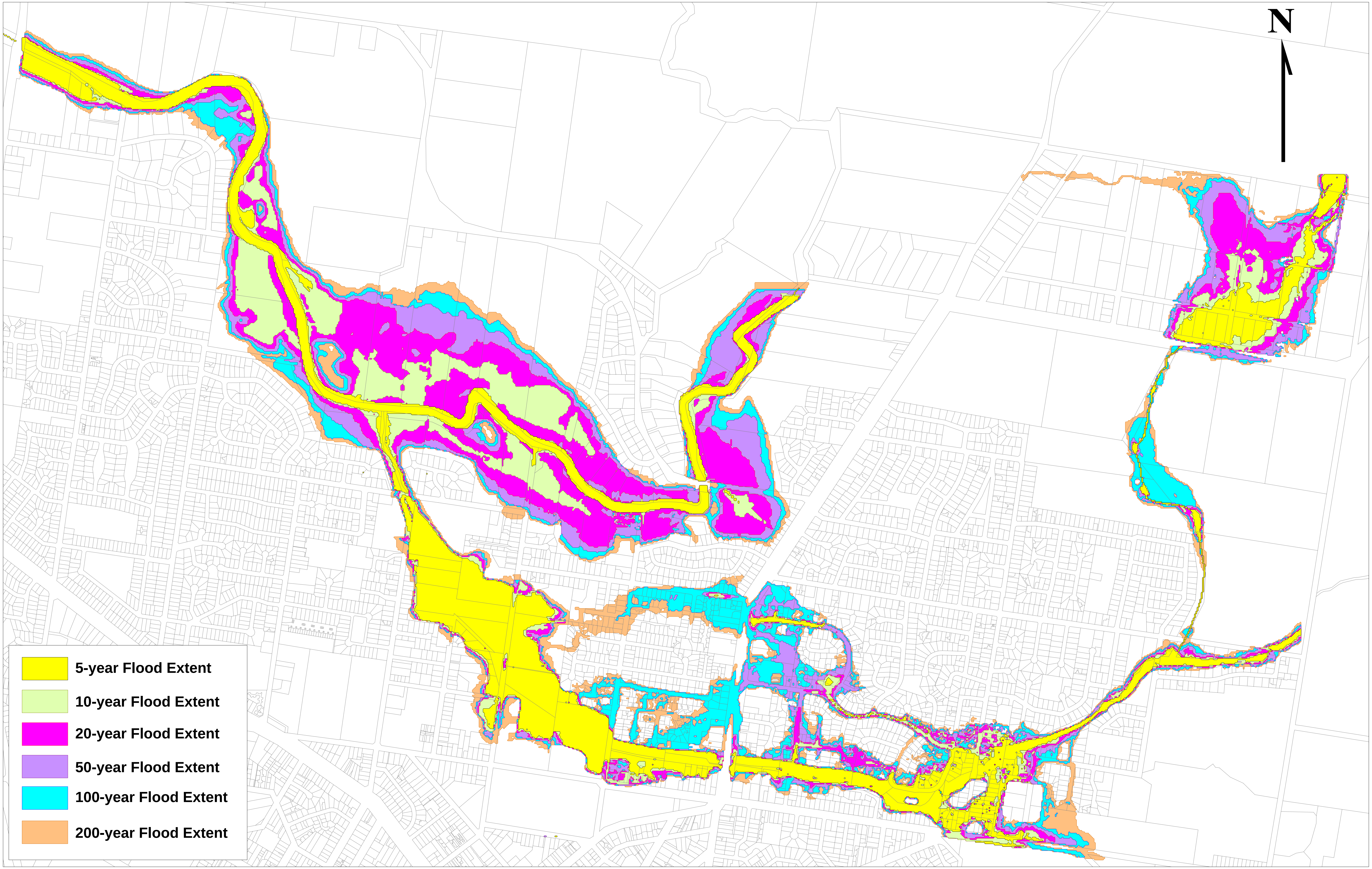

Figure 6.7 - Flood Extent Comparison

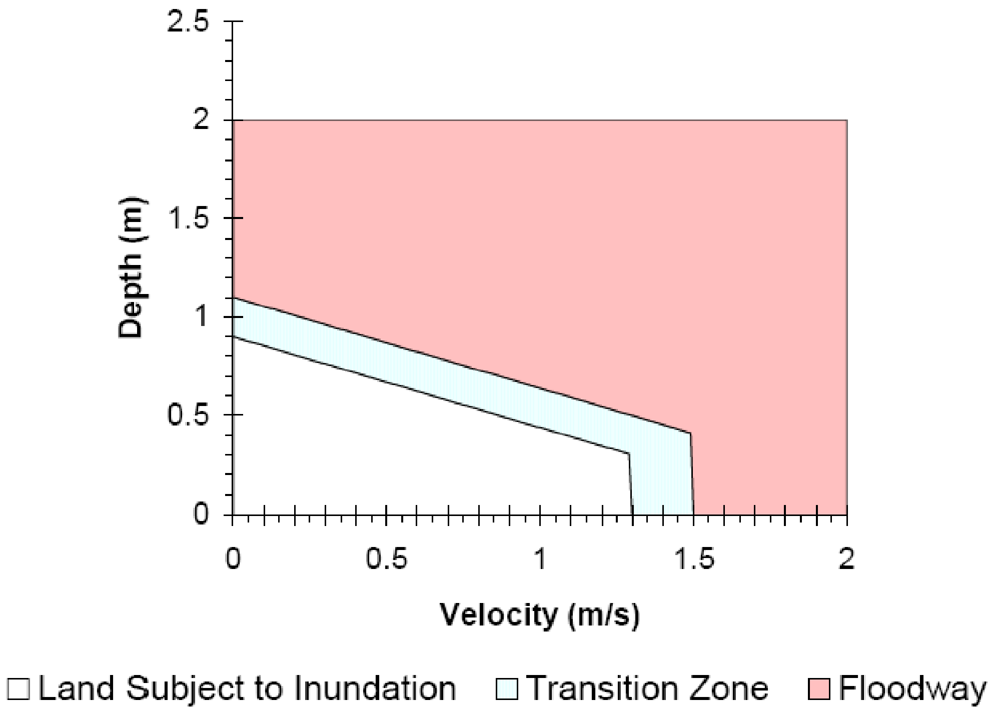

Figure 6.8 – Floodway overlay flood hazard criteria (NRE, 1998)

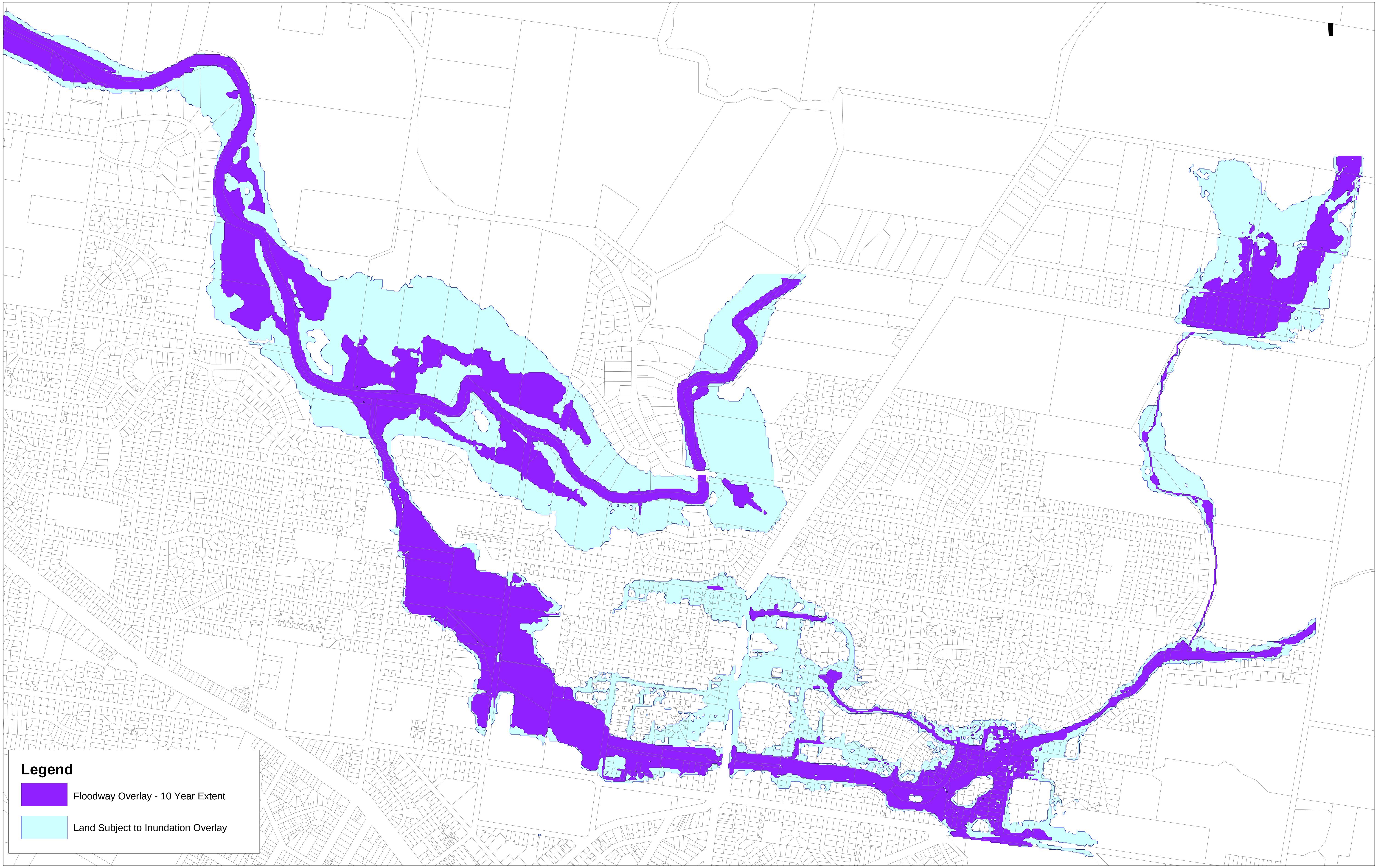

Figure 6.9 - Potential Floodway Overlay - 10 Year ARI Flood Extent

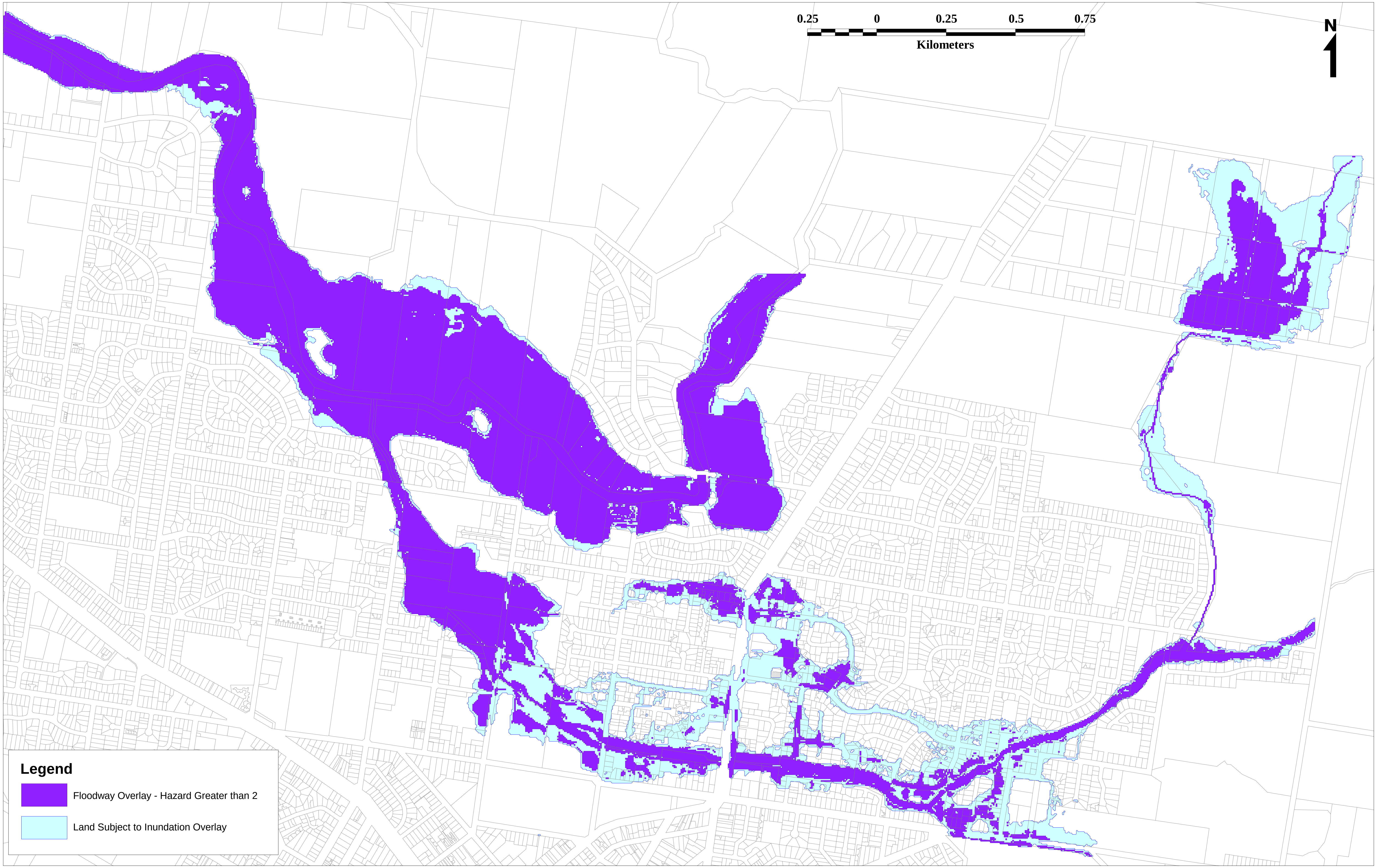

Figure 6.10 - Potential Floodway Overlay - Hazard Greater than 2

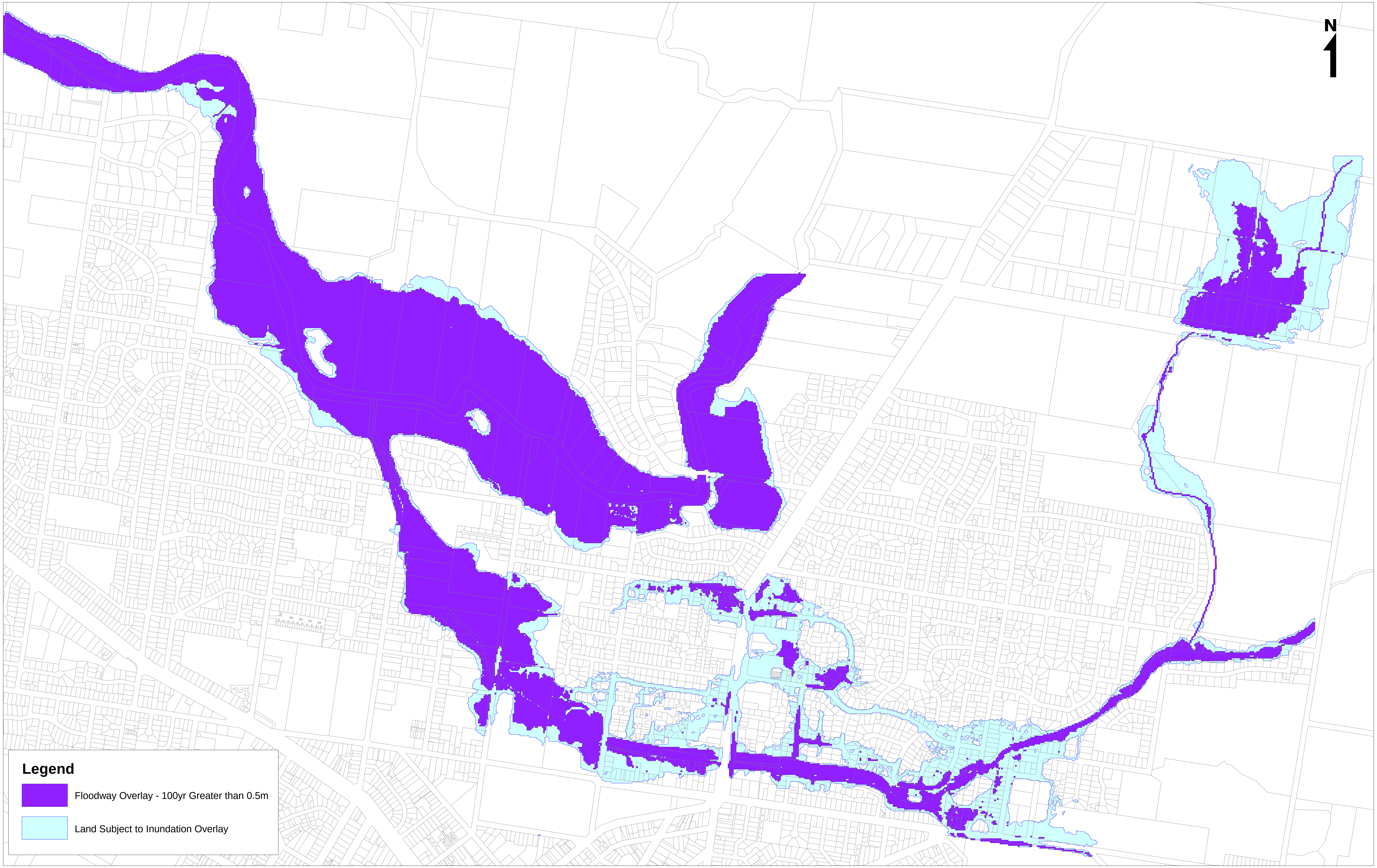

Figure 6.11 - Potential Floodway Overlay - 100yr Flood Depth Greater than 0.5m

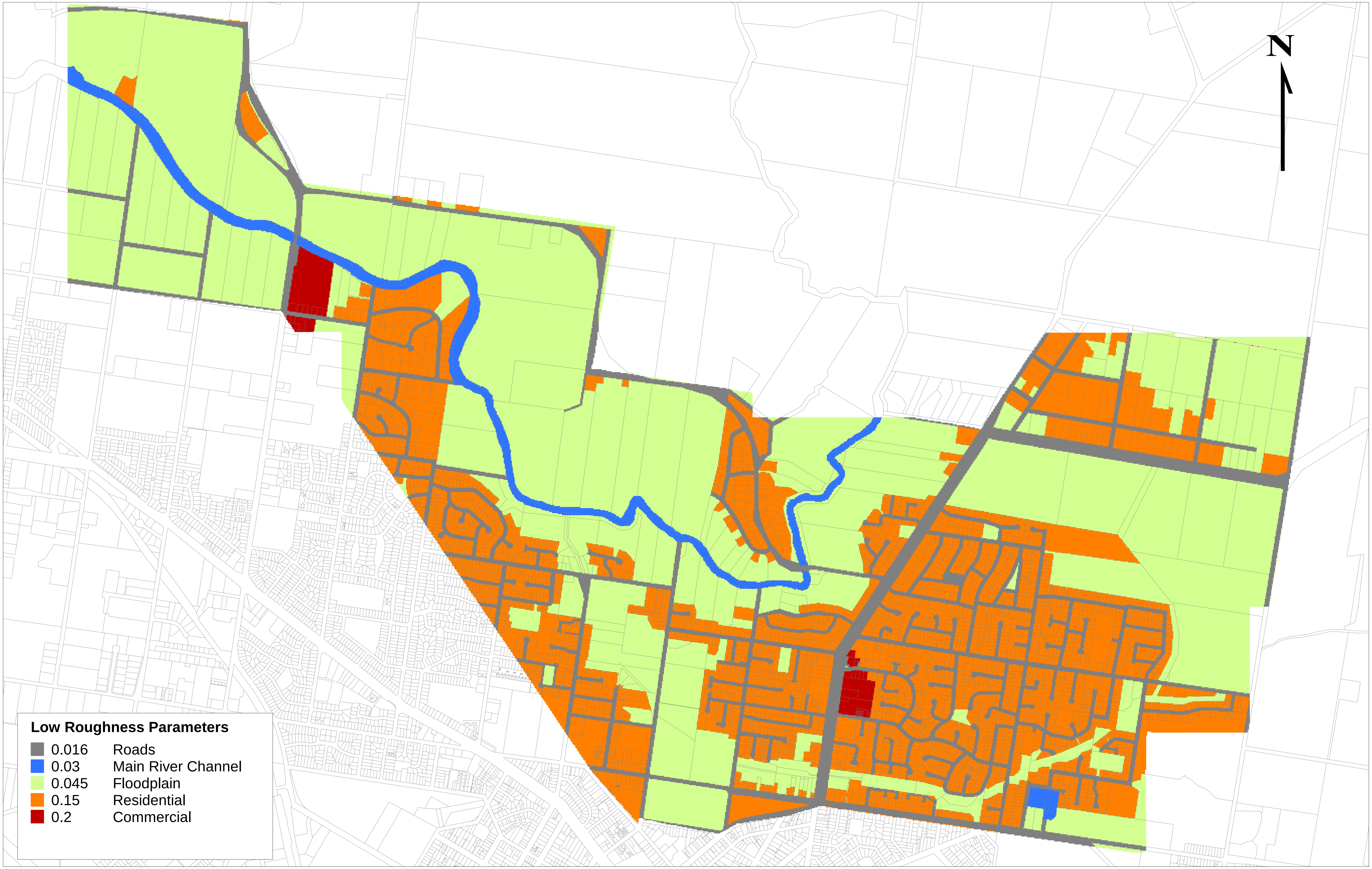

Figure 7.1 - The full roughness grid for Merri River and Russel Creek - Low roughness sensitivity analysis

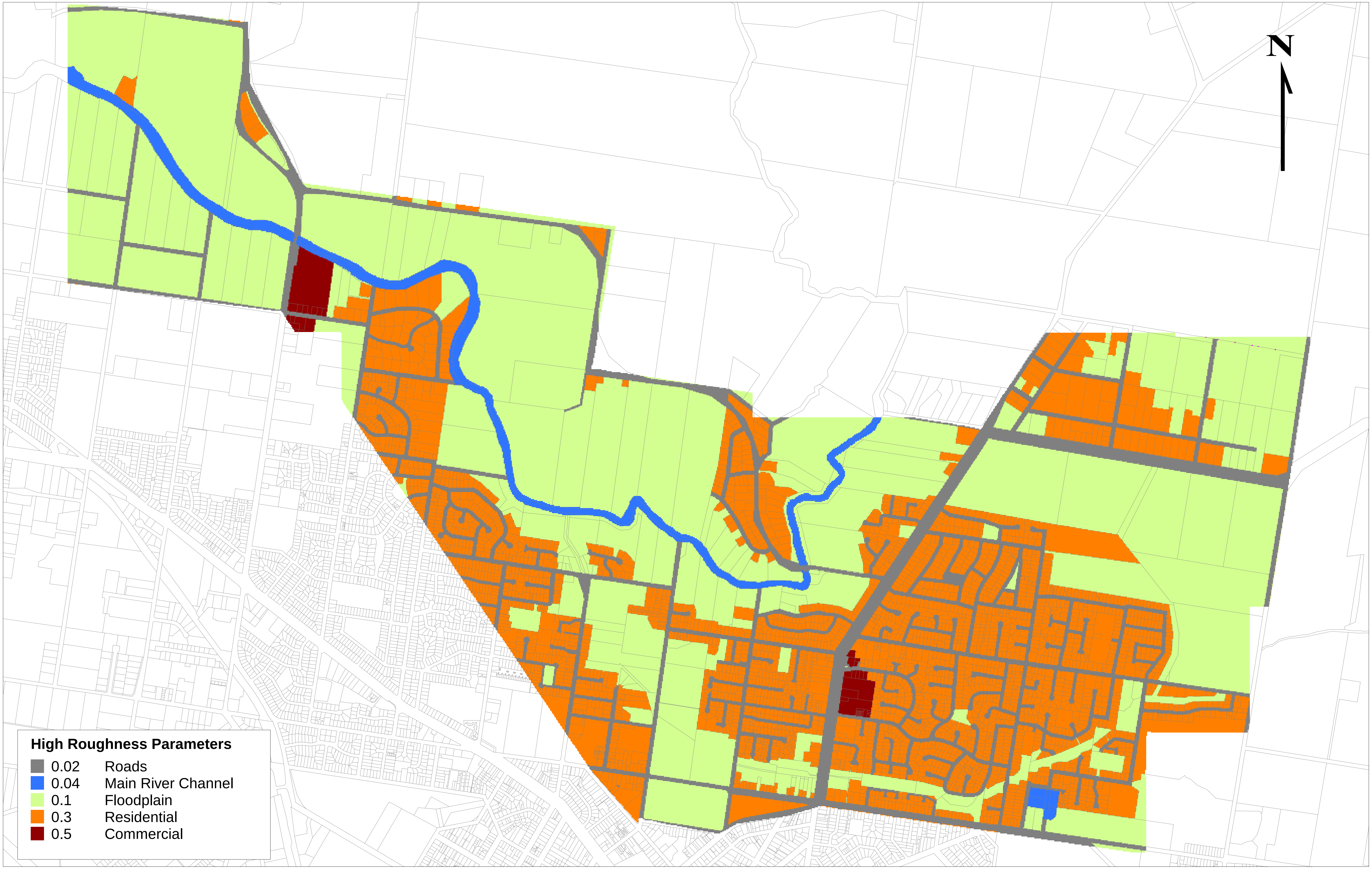

Figure 7.2 - The full roughness grid for Merri River and Russel Creek - High roughness sensitivity analysis

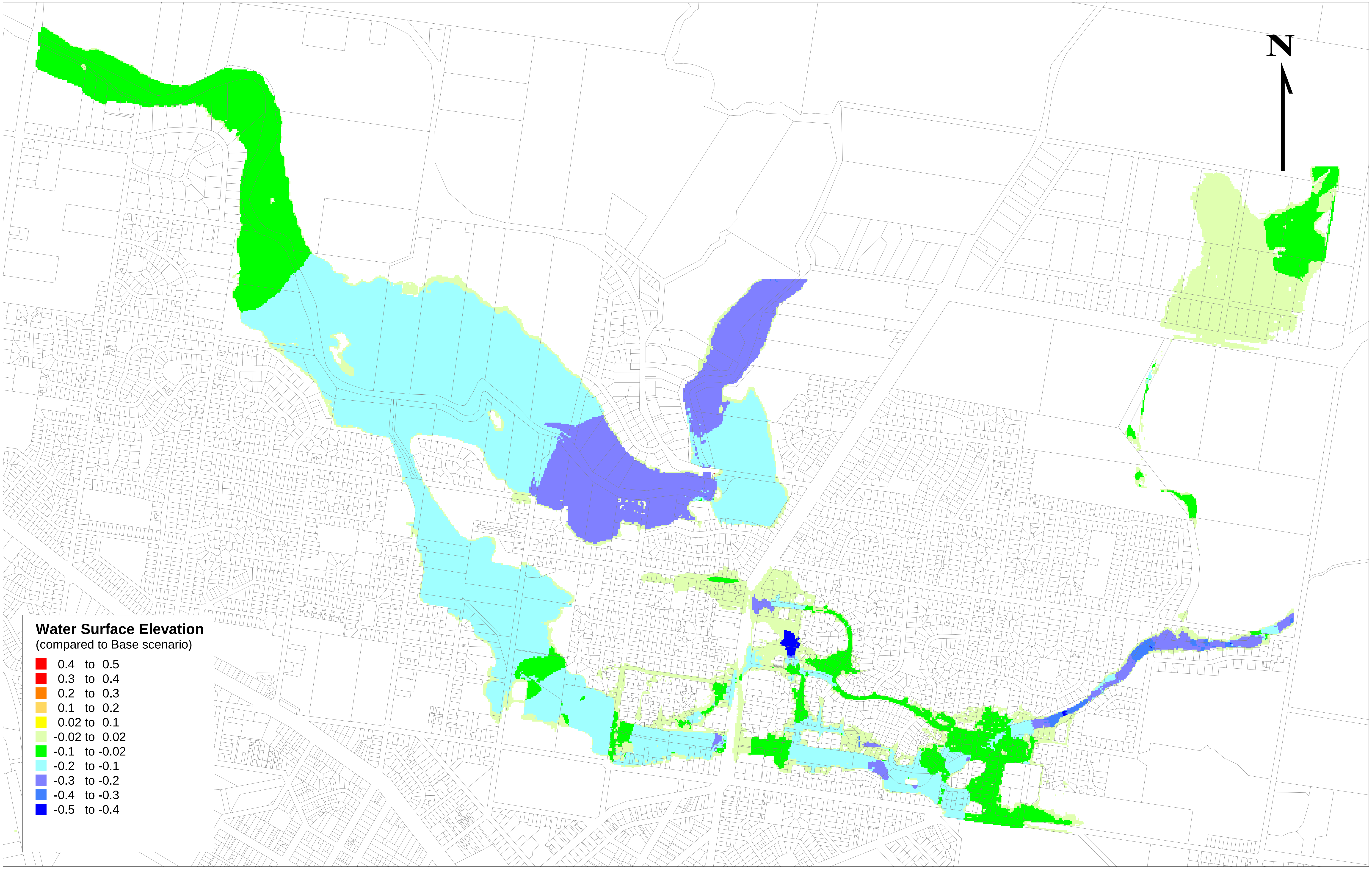

Figure 7.3 - Water surface elevation difference plot - difference of the low roughness scenario compared to the base case

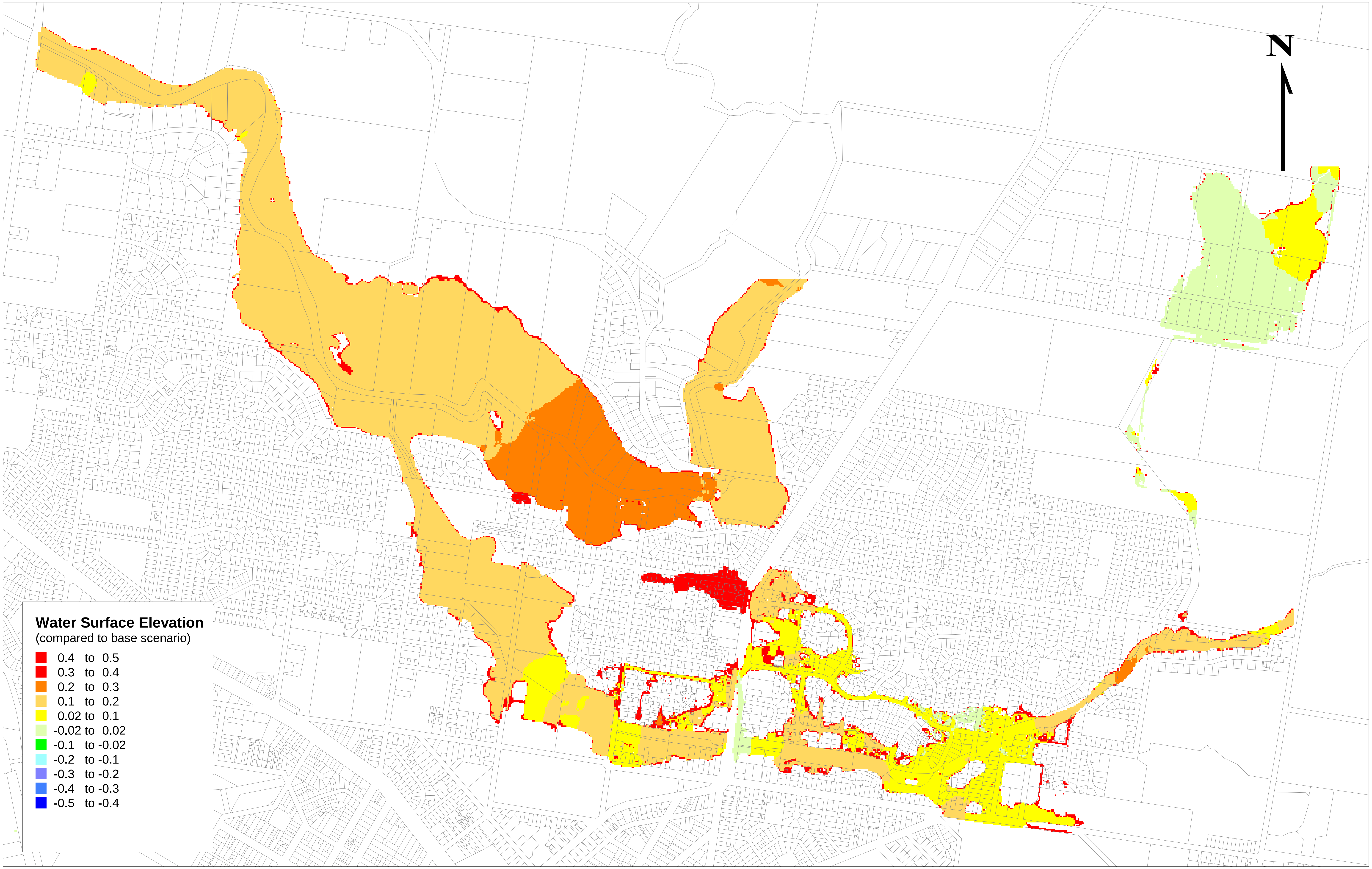

Figure 7.4 - Water surface elevation difference plot - difference of the high roughness scenario compared to the base case

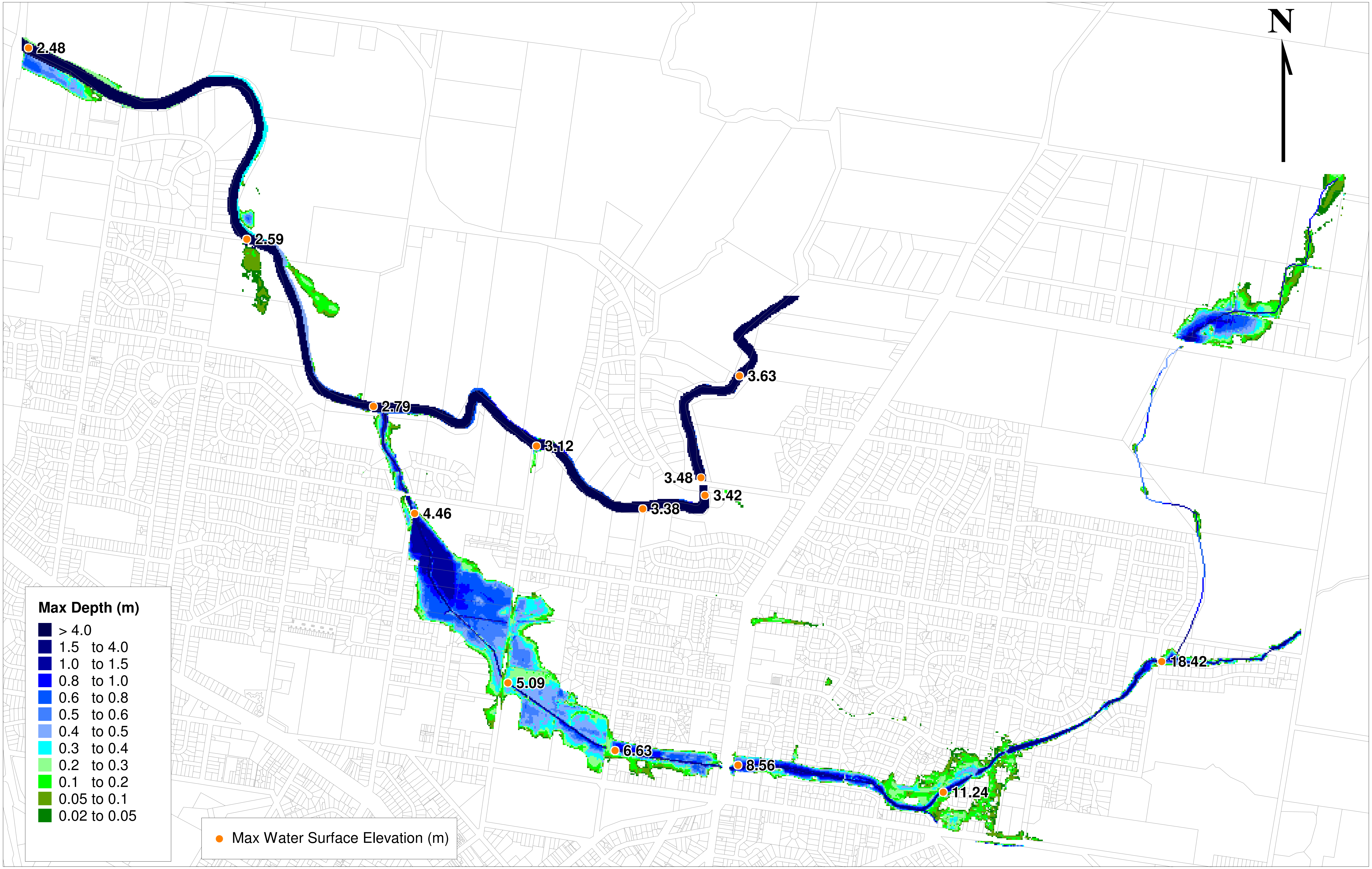

Figure 7.5 - 5 year ARI flood extents and depths using the Woodford non-transposed hydrograph

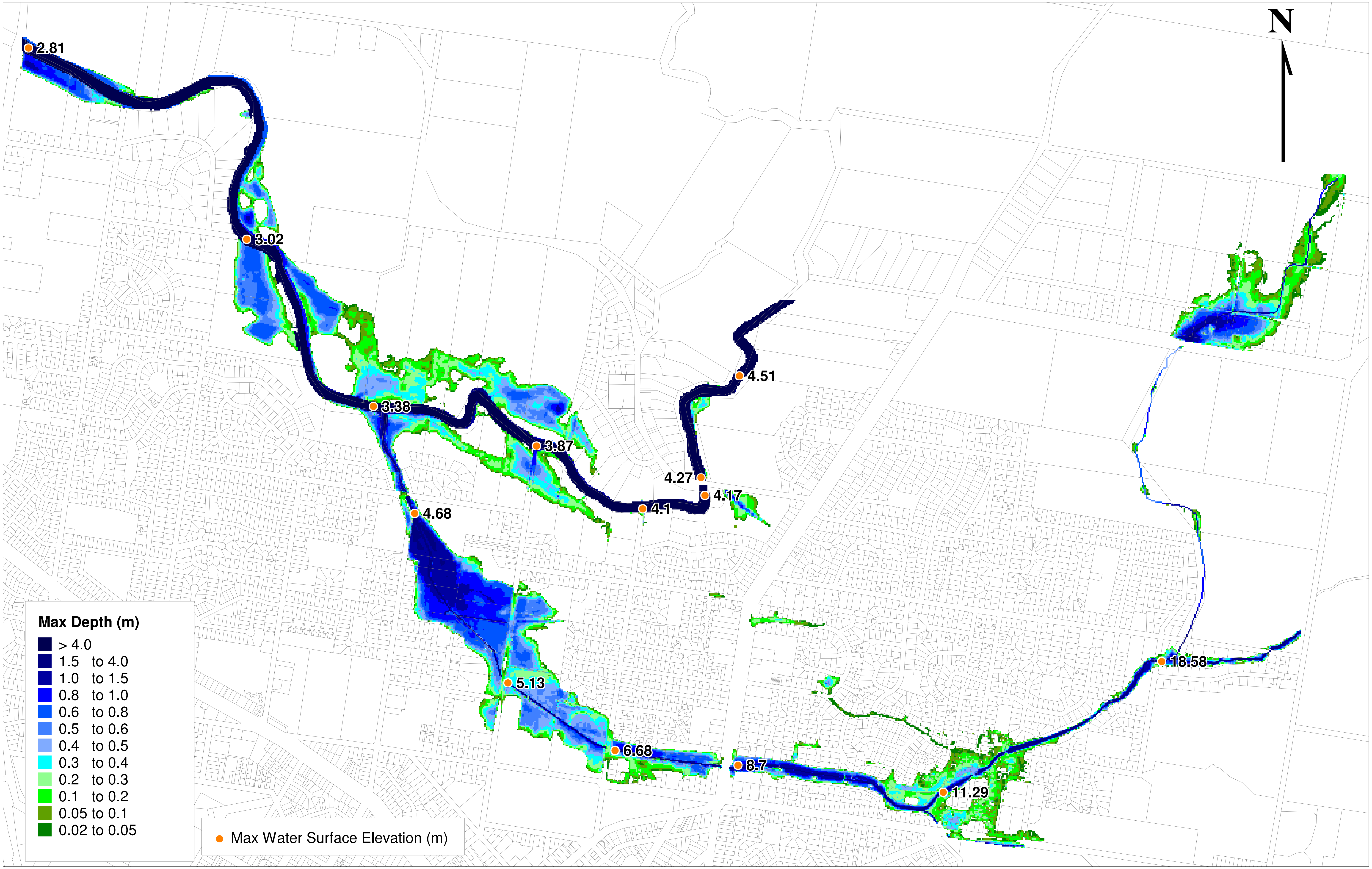

Figure 7.6 - 10 year ARI flood extents and depths using the Woodford non-transposed hydrograph

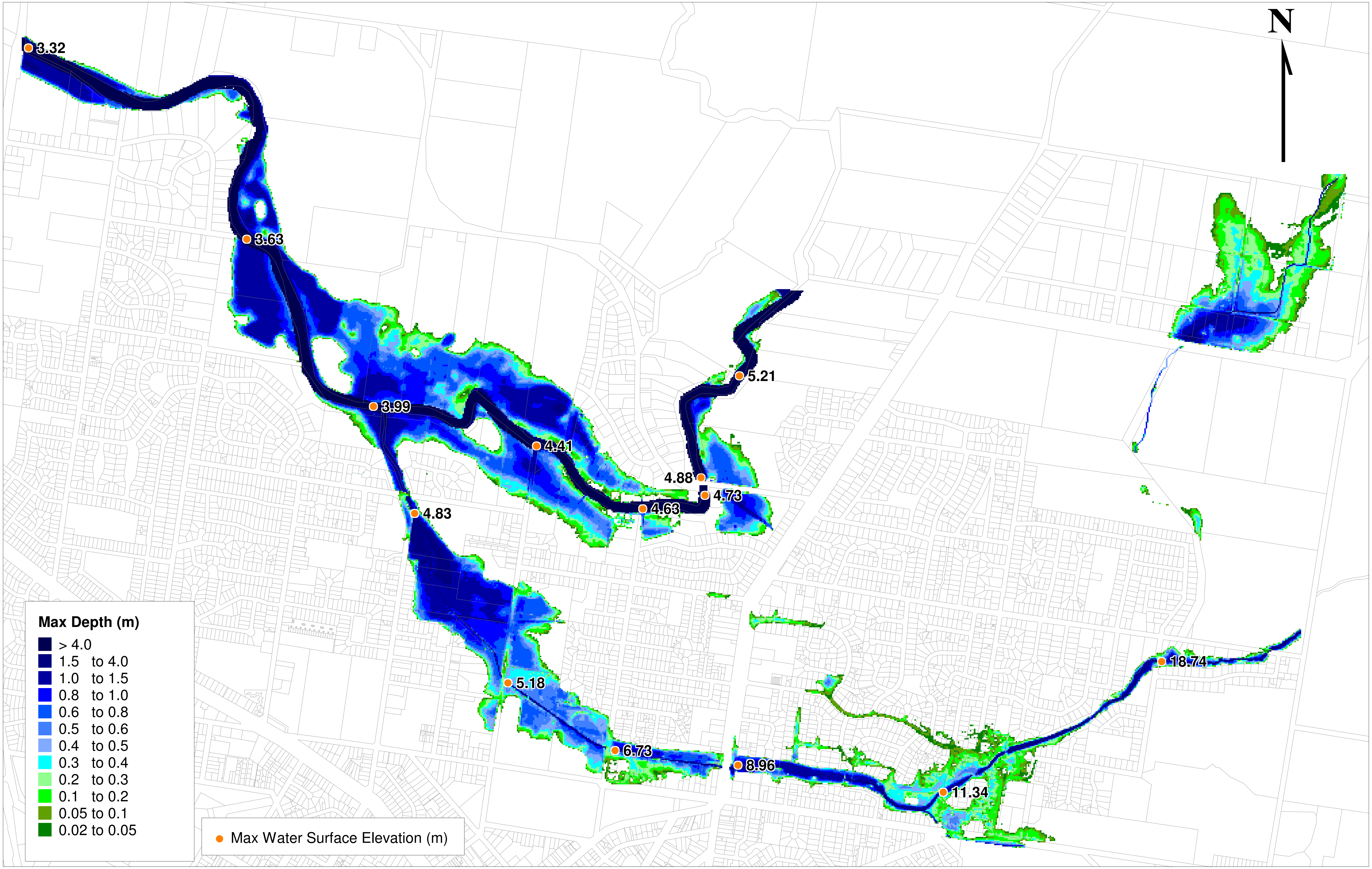

Figure 7.7 - 20 year ARI flood extents and depths using the Woodford non-transposed hydrograph

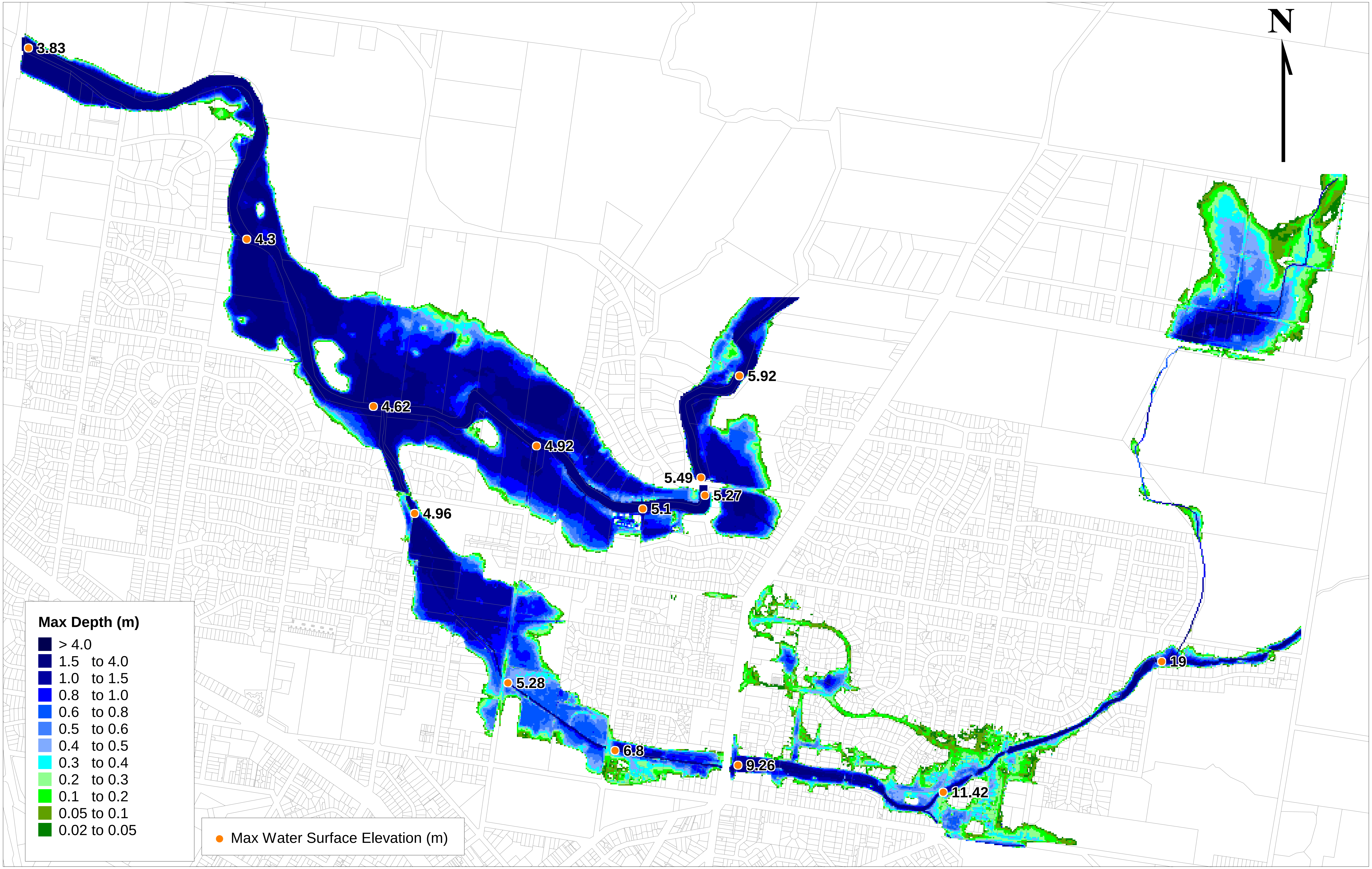

Figure 7.8 - 50 year ARI flood extents and depths using the Woodford non-transposed hydrograph

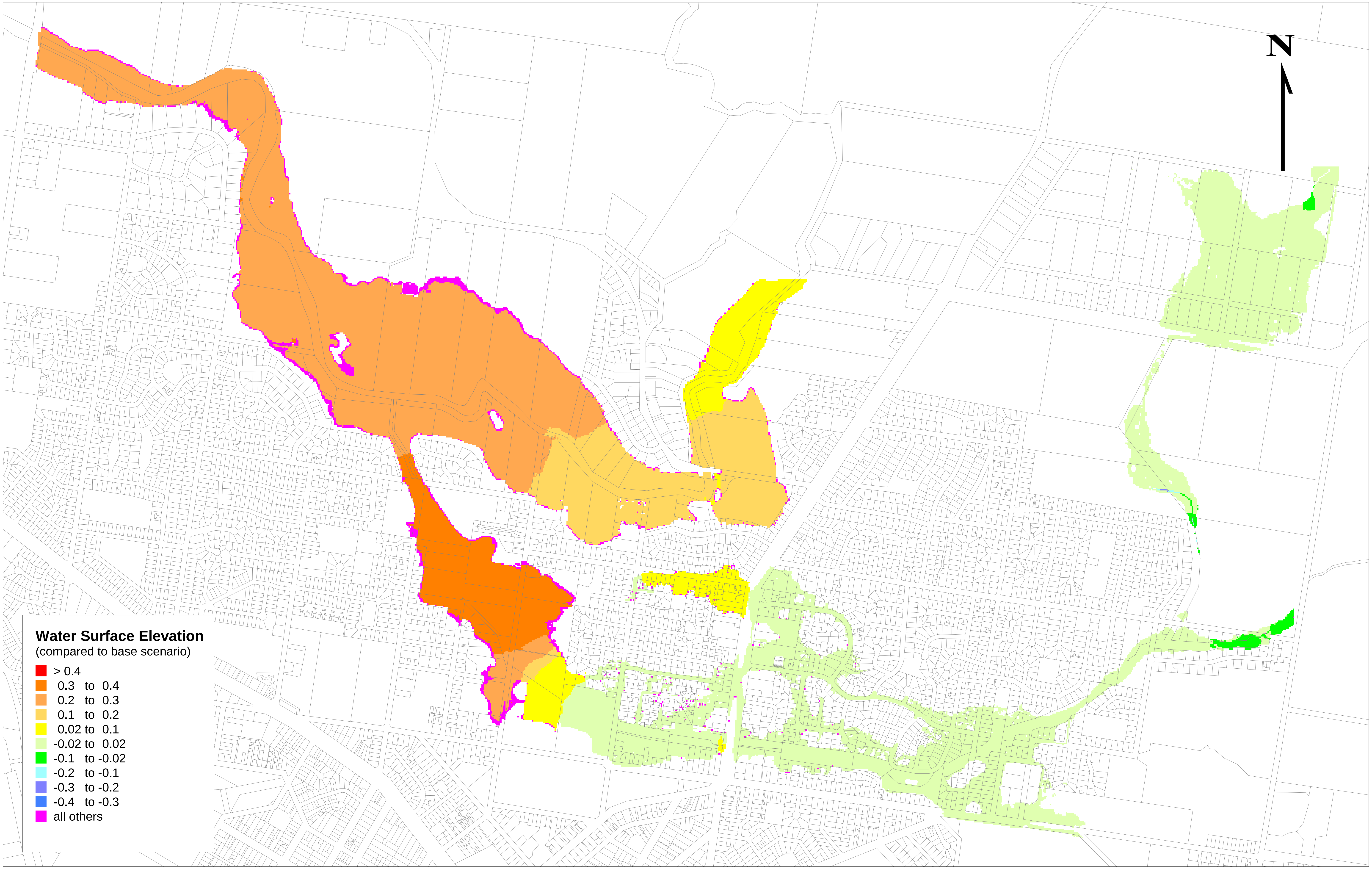

Figure 7.9 - Water surface elevation difference plot - difference of the intersecting peaks scenario compared to the base case

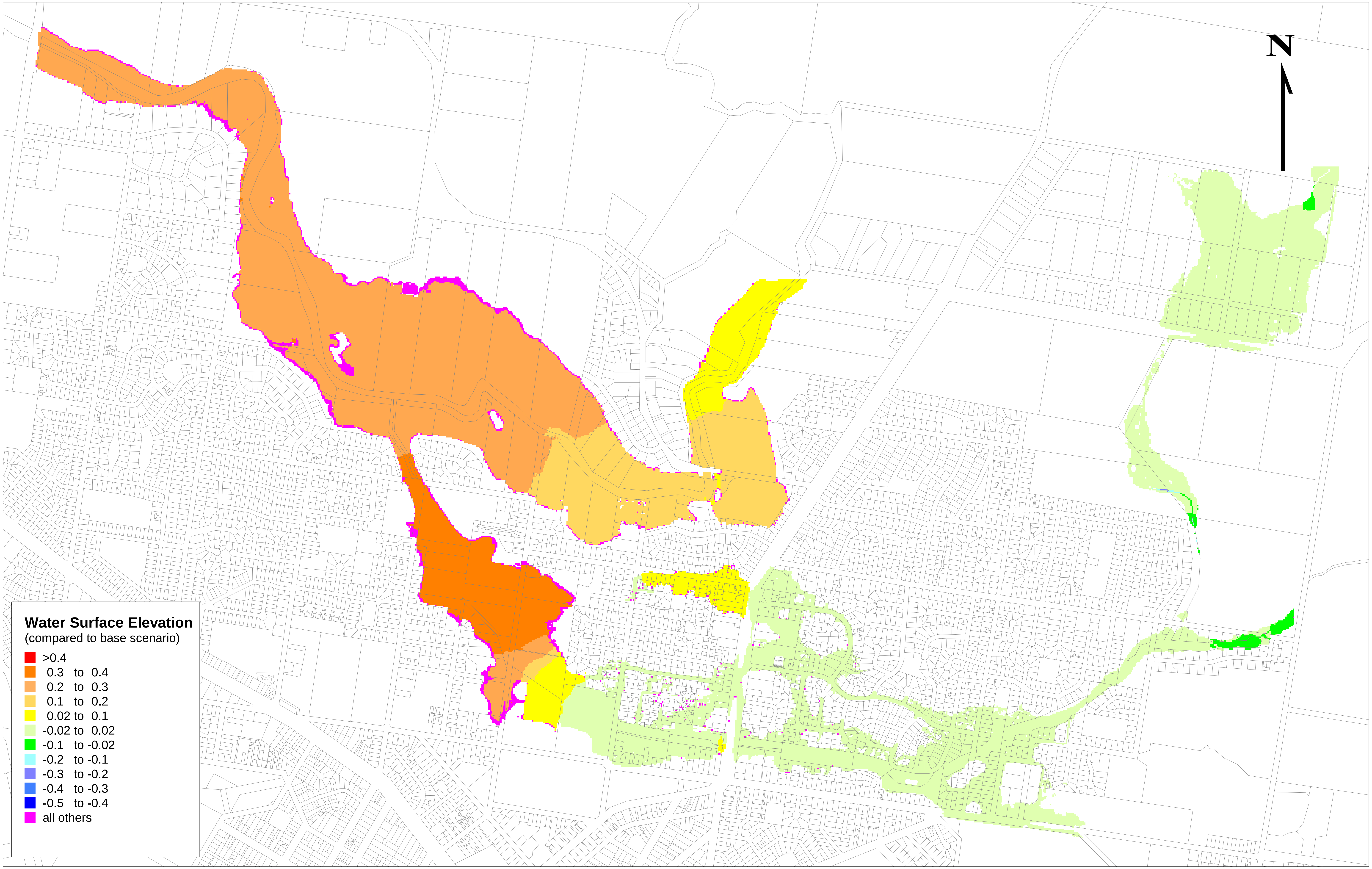

Figure 7.10 - Water surface elevation difference plot -difference of the Russel start Merri peak scenario compared to the base case

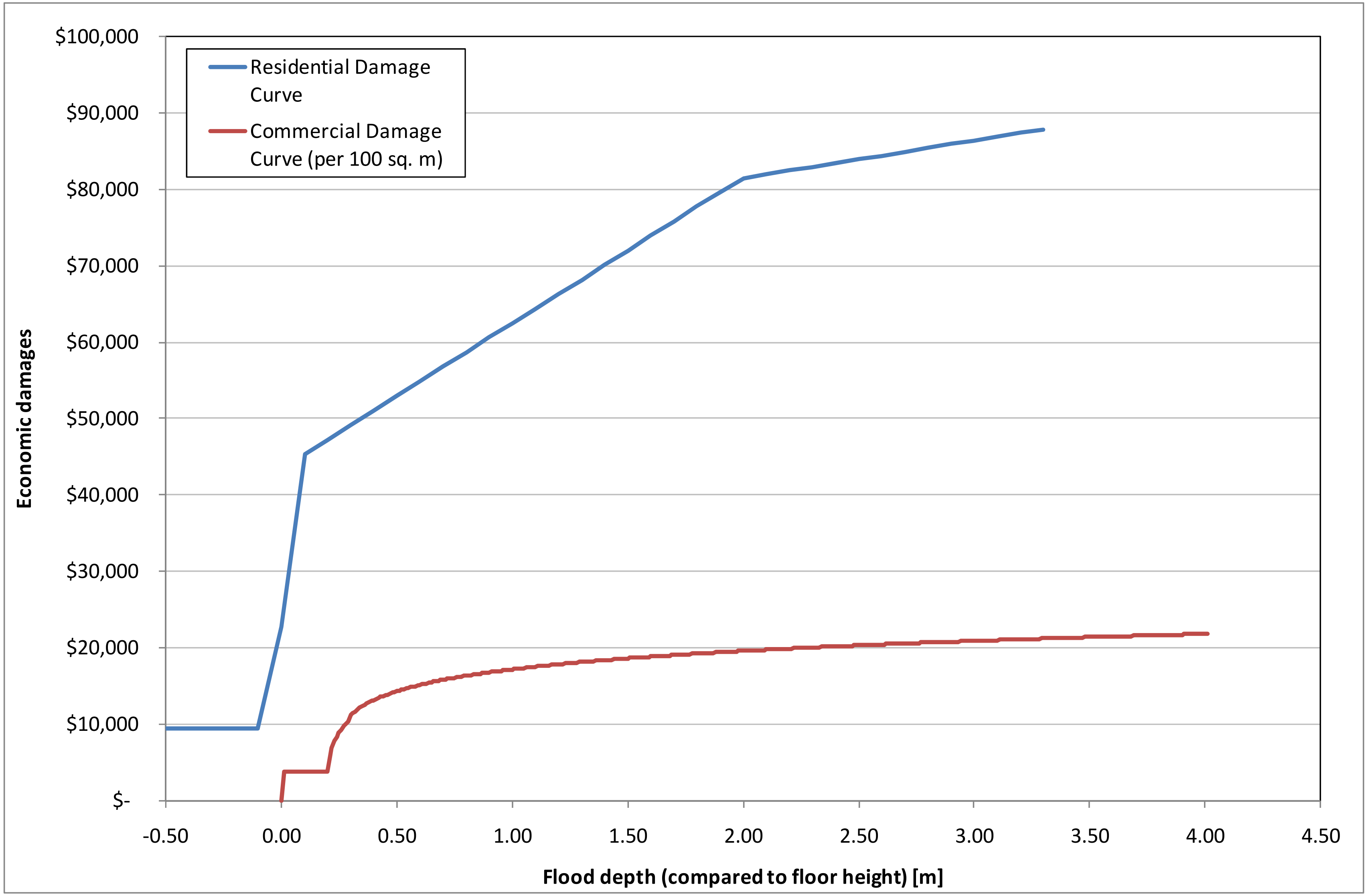

Figure 8.1 – Flood damage curves for residential and commecial properties

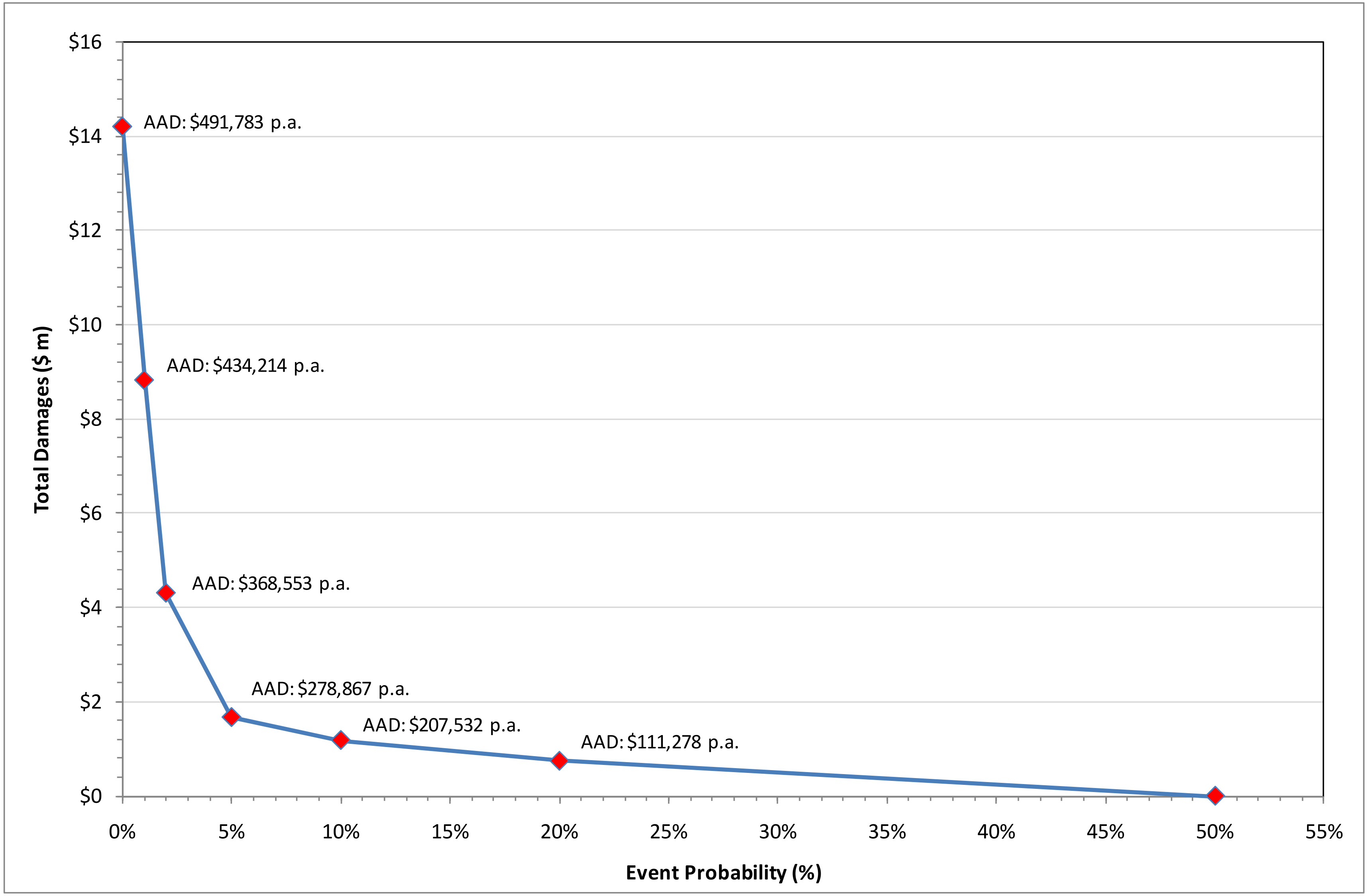

Figure 8.2 – Annual Average Damage (AAD) probability curve and incremental AADs for increasing events

Appendix A

RORB

A.1 Merri River RORB Catchment File

```
South Warrnambool
C Merri River RORB model developed with the aid of CatchmentSim
C Water Technology April 2004, updated 30 Sept 2004
C for Glenelg Hopkins CMA, South Warrnambool Flood Study
C only change from 040929 ver is extra output point at Denington....
C
1,          channel type flag - all reaches natural
1,11.75,-99,   sub-area 1.01
2,10.45,-99,   sub-area 1.02
3,          store h/g
1,12.61,-99,   sub-area 2.01
4,          add stored h/g from top of stack
2,9.12,-99,    sub-area 1.03
3,          store h/g
1,15.41,-99,   sub-area 3.01
2,11.94,-99,   sub-area 3.02
3,          store h/g
1,9.91,-99,    sub-area 4.01
4,          add stored h/g from top of stack
4,          add stored h/g from top of stack
2,11.32,-99,   sub-area 1.04
2,7.95,-99,    sub-area 1.05
3,          store h/g
1,14.35,-99,   sub-area 5.01
2,10.75,-99,   sub-area 5.02
2,15.75,-99,   sub-area 5.03
3,          store h/g
1,8.04,-99,    sub-area 6.01
2,7.05,-99,    sub-area 6.02
4,          add stored h/g from top of stack
4,          add stored h/g from top of stack
2,11.95,-99,   sub-area 1.06
3,          store h/g
1,10.02,-99,   sub-area 7.01
2,9.12,-99,    sub-area 7.02
4,          add stored h/g from top of stack
2,6.75,-99,    sub-area 1.07
3,          store h/g
1,12.31,-99,   sub-area 8.01
2,8.16,-99,    sub-area 8.02
4,          add stored h/g from top of stack
2,9.55,-99,    sub-area 1.08
3,          store h/g
1,12.50,-99,   sub-area 9.01
2,5.97,-99,    sub-area 9.02
4,          add stored h/g from top of stack
2,7.85,-99,    sub-area 1.09
3,          store h/g
1,4.89,-99,    sub-area 10.01
3,          store h/g
1,6.48,-99,    sub-area 11.01
3,          store h/g
1,8.07,-99,    subarea 12.01
4,          add stored h/g from top of stack
2,6.72,-99,    sub-area 11.02
3,          store h/g
1,4.54,-99,    sub-area 13.01
2,6.45,-99,    sub-area 13.02
2,7.35,-99,    sub-area 13.03
4,          add stored h/g from top of stack
2,6.23,-99,    sub-area 11.03
4,          add stored h/g from top of stack
4,          add stored h/g from top of stack
2,5.60,-99,    sub-area 1.10
```

3, store h/g
1,6.16,-99, sub-area 15.01
4, add stored h/g from top of stack
9,0,0,1,0,-99, Woodford inflow
7
Woodford Gauge
2,7.89,-99, sub-area 1.11
3, store h/g
1,5.59,-99, sub-area 16.01
4, add stored h/g from top of stack
2,7.58,-99, sub-area 1.12
7
Upstream of Russell Creek
3, store h/g
1,4.52,-99, sub-area 17.01 Russell Creek
4, add stored h/g from top of stack
2,5.32,-99, sub-area 1.13
3, store h/g
1,3.25,-99, sub-area 18.01
3, store h/g
1,5.06,-99, sub-area 19.01
3, store h/g
1,4.27,-99, sub-area 20.01
3, store h/g
1,4.43,-99, sub-area 21.01
4, add stored h/g from top of stack
4, add stored h/g from top of stack
4, add stored h/g from top of stack
4, add stored h/g from top of stack
7
Dennington
3, store h/g
1,3.0,-99, sub-area 22.01 Kelly Swamp
4, add stored h/g from top of stack
2,7.0,-99, sub-area 1.14
7
Outlet to sea
0, end of control vector
C sub-area areas
29.2700,21.1300,20.6700,22.8300,39.3000,50.7000,17.2600,39.5500,21.8700,20.5900,20.7000,33.7700,
15.8600,27.7900,36.4400,25.3200,21.1800,28.7600,16.4600,15.5400,29.0300,21.7600,19.3600,28.6600,
15.9700,25.1900,24.0109,22.760,28.4800,24.7200,23.9500,15.8100,12.0067,38.0733,20.6729,22.0477,
12.04500,25.4064,12.1368,13.1255,18.5498,8.0764,11.5622,6.6255,13.3647,-99, sq.km.
1,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,
0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,0.00,
0.00,0.00,0.00,0.00,0.00,0.00,0.00,-99, imp.prop.

A.2 Russell Creek RORB Catchment File

```
Russel Ck Drainage Investigation
C created at 15:21 on 19/1/07 by JLR of Cardno Lawson Treloar
C Reach type flag
1
C The Control Vector
1, 1.10, -99                  ,Gen rain from sub-area A
5, 3.17, -99                   ,Route to 1
3                             ,Store at 1
1, 1.47,-99                   ,Gen rain from sub-area B
4                             ,Add to h'graph
3                             ,Store at 1
1, 1.09,-99                   ,Gen rainfrom sub-area C
4                             ,Add previous h'graph to running
5, 1.43,-99                   ,Route to 2
3                              ,Store at 2
1, 0.78,-99                   ,Gen rain from sub-area D
4                             ,Add to h'graph
5, 1.66,-99                   ,Route to 3
3                             ,Store at 3
1, 0.90,-99                   ,Gen rain from sub-area E
4                             ,Add to h'graph
5, 0.91,-99                   ,Route to 4
3                             ,Store at 4
1, 1.39,-99                   ,Gen rain from sub-area F
4                             ,Add to h'graph
7
Aberline Rd
3                             ,Store at 4
1, 2.05,-99                   ,Gen rain from sub-area G
3                             ,Store at 5
1, 1.26,-99                   ,Gen rain from sub-area H
4                             ,Add to h'graph
7
Catchment G and H
5, 1.17,-99                   ,Route to 6
3                             ,Store at 6
11, 0.59,-99                  ,Gen rain from sub-area I
4                             ,Add to h'graph
7
Wangoom Rd
5, 1.24,-99                   ,route to 4
3                             ,store at 4
11, 0.75,-99                  ,Gen rain from sub-area J
4                             ,Add previous h'graph to running
7
Northern Subcatchment
4                             ,Add Aberline rd sub h'graph
5, 1.05,-99                   ,Route to 7
3                             ,Store at 7
11, 0.62,-99                  ,Gen rain from sub-area K
4                             ,Add previous h'graph to running
3                             ,Store at 7
11, 0.80,-99                  ,Gen rain from sub-area L
4                             ,Add previous h'graph to running
5, 0.86,-99                   ,Route to 8
3                             ,Store at 8
11, 0.42,-99                  ,Gen rain from sub-area M
4                             ,Add previous h'graph to running
7
Mortlake Rd
5, 1.01,-99                   ,Route to 9
3                             ,Store at 9
11, 0.55,-99                  ,Gen rain from sub-area N
4                             ,Add previous h'graph to running
```

```
5, 0.43,-99                          ,Route to O
7
All ex Urban North
3                                    ,Store at O
11, 0.30,-99                         ,Gen rain from sub-area P
5, 1.18,-99                          ,Route to 12
3                                    ,Store at 12
11, 0.75,-99                         ,Gen rain from sub-area Q
4                                    ,Add to h'graph
5, 0.28,-99                          ,Route to O
7
Urban North Subcatchment
4                                    ,Add to h'graph
5, 0.49,-99                          ,Route to 10
3                                    ,store at 10
11, 0.49,-99                         ,Gen rain from sub-area O
4                                    ,Add previous h'graph to running
7
Merri River
0
C Sub Area Data
C Areas, km**2, of subareas A,B....
4.68, 3.68, 2.19, 3.09, 2.33, 3.65, 3.29, 2.15, 0.99, 1.25, 0.86, 1.40, 0.61, 0.87,
1.08, 0.30, 0.25, -99
C Impervious Area Flag
1
0.05, 0.05, 0.05, 0.05, 0.05, 0.073, 0.05, 0.05, 0.489, 0.475, 0.52,
0.495, 0.52, 0.52, 0.52, 0.52, 0.52, -99
```

Flood Hydrographs

Appendix B

Flood Hydrographs

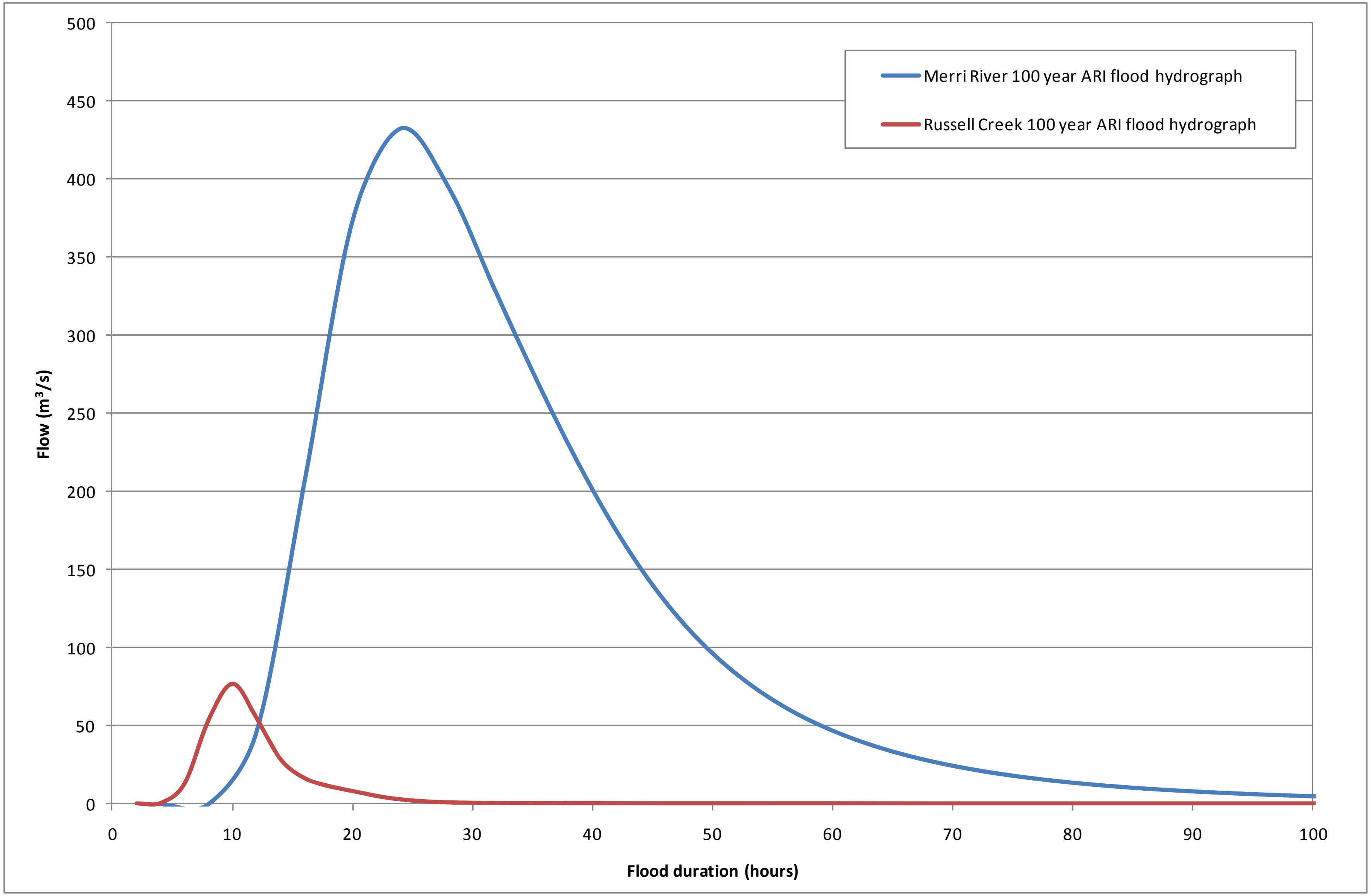

Figure B.1 Design hydrographs for Merri River at the upstream boundary of the hydraulic model

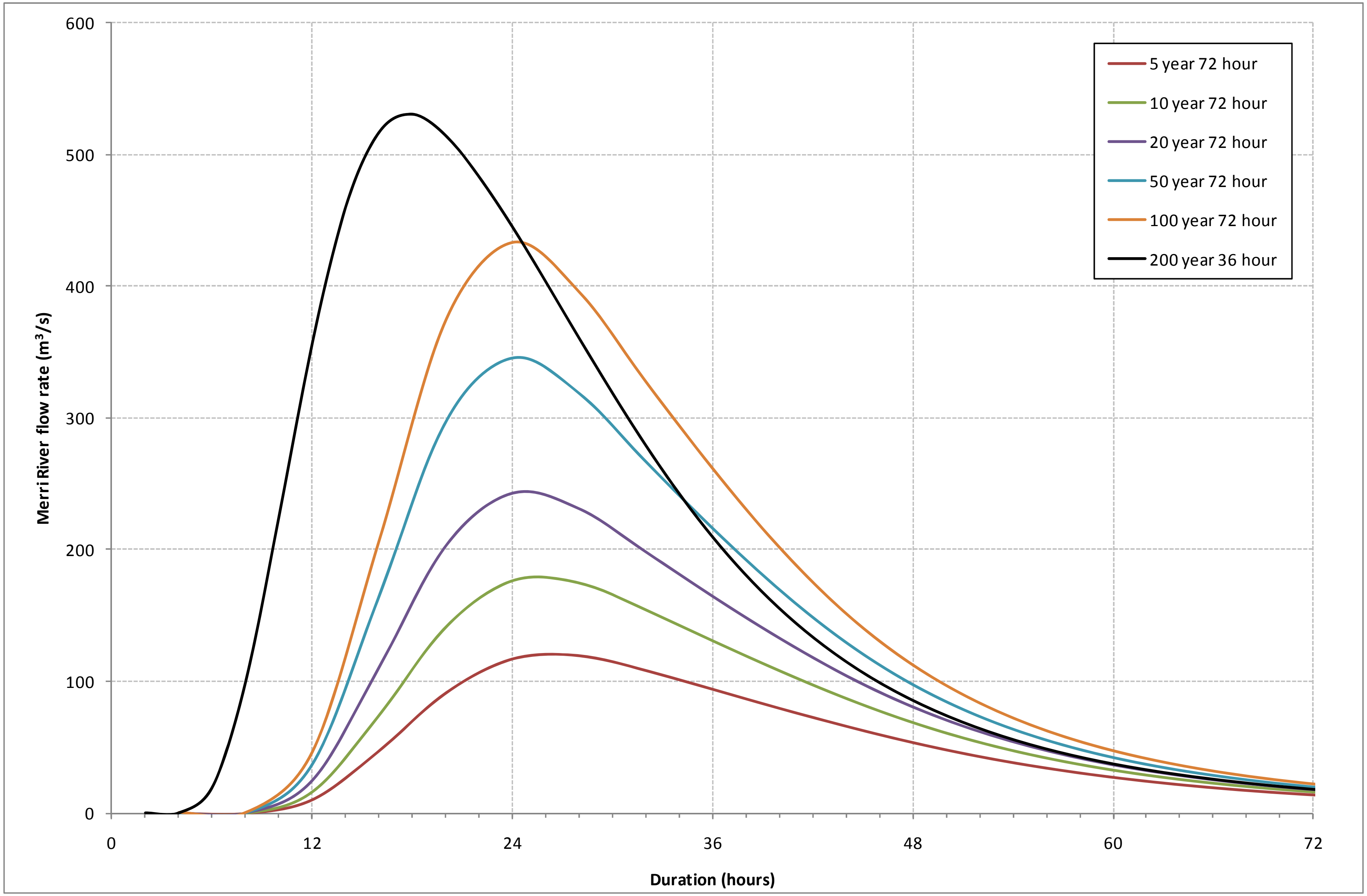

Figure B.2 Design hydrographs for Merri River at the upstream boundary of the hydraulic model

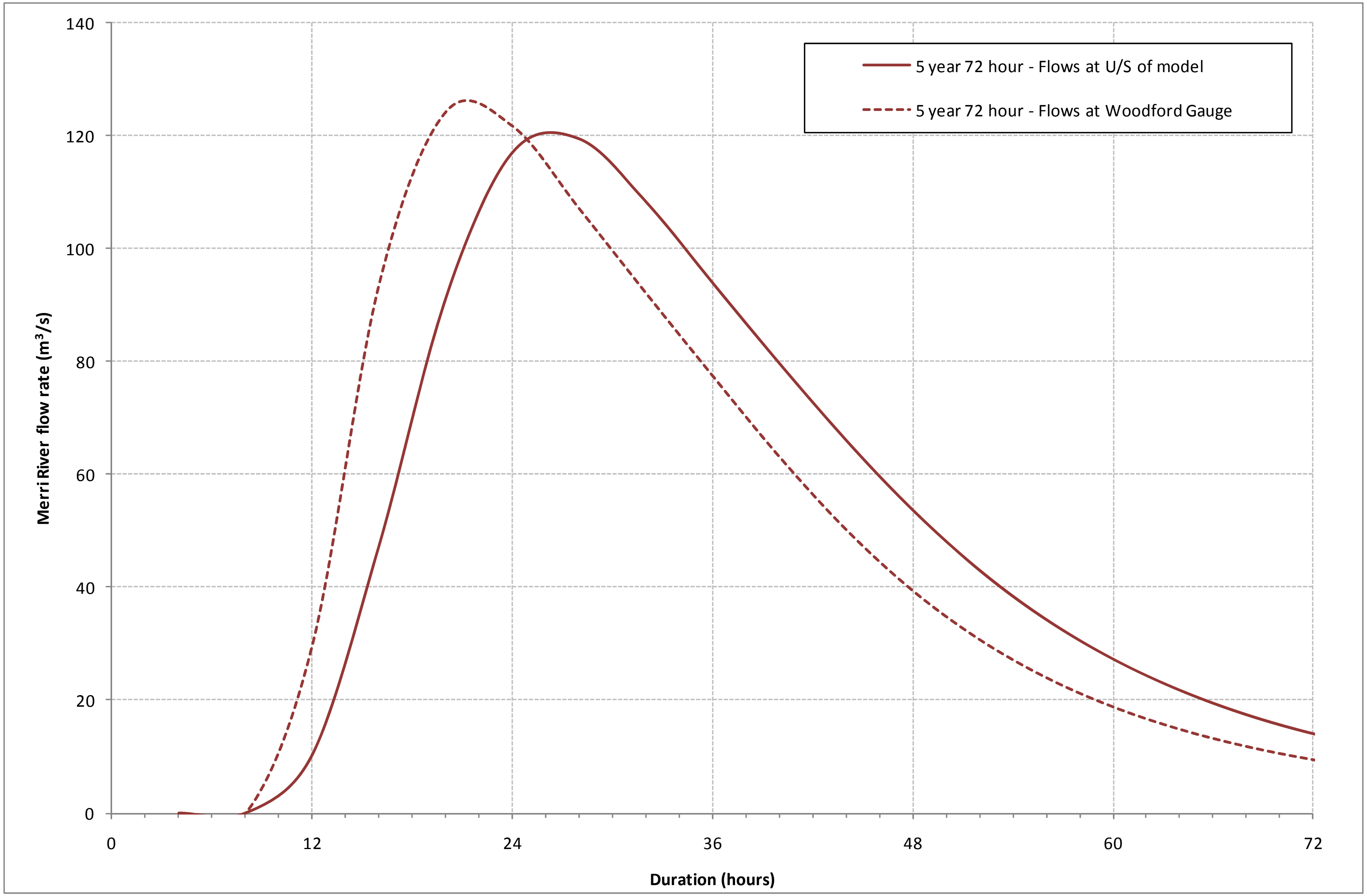

Figure B.3 Comparison of the 5 year hydrographs for Merri River between the Woodford gauge and the u/s boundary of the model

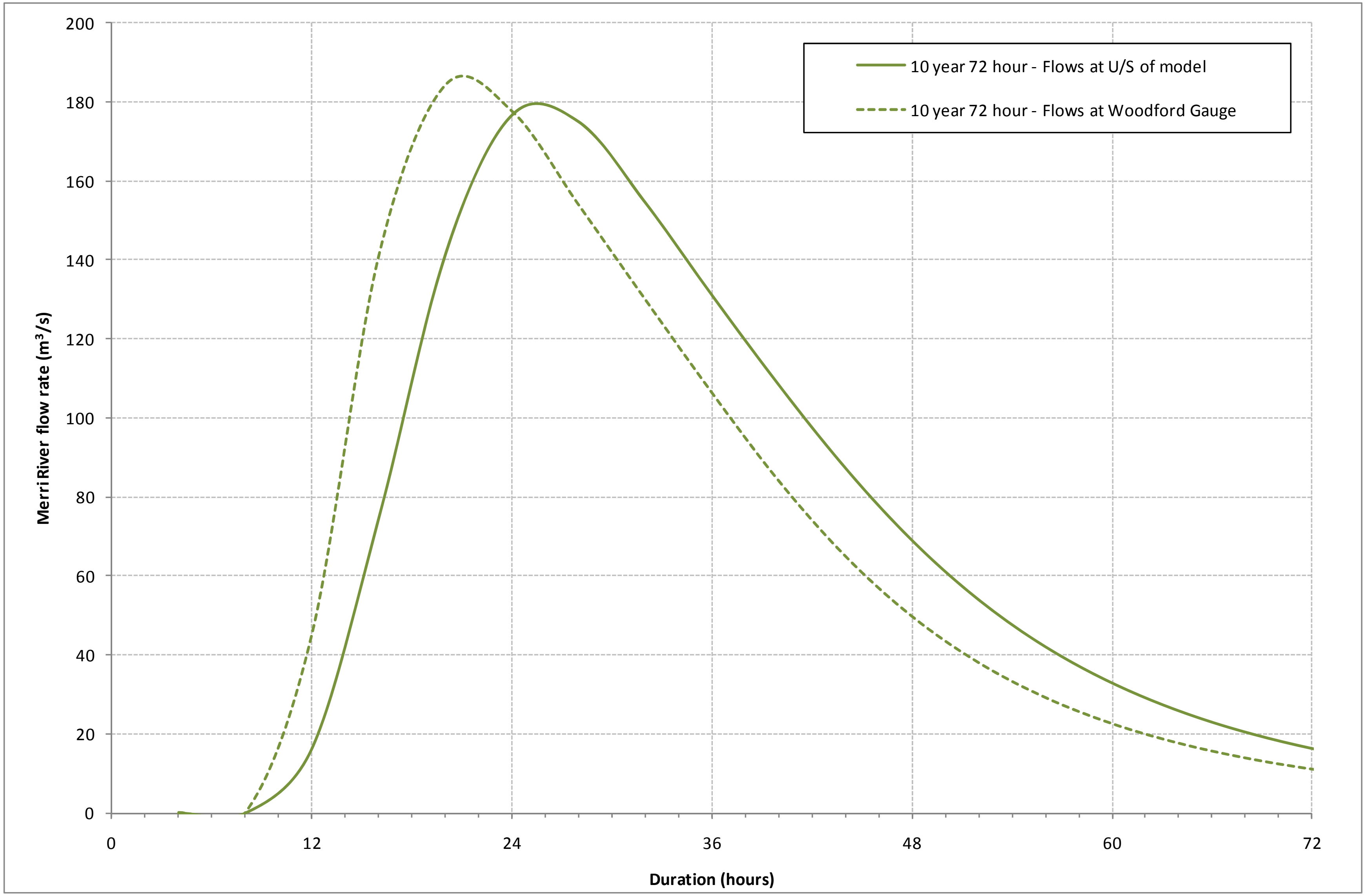

Figure B.4 Comparison of the 10 year hydrographs for Merri River between the Woodford gauge and the u/s boundary of the model

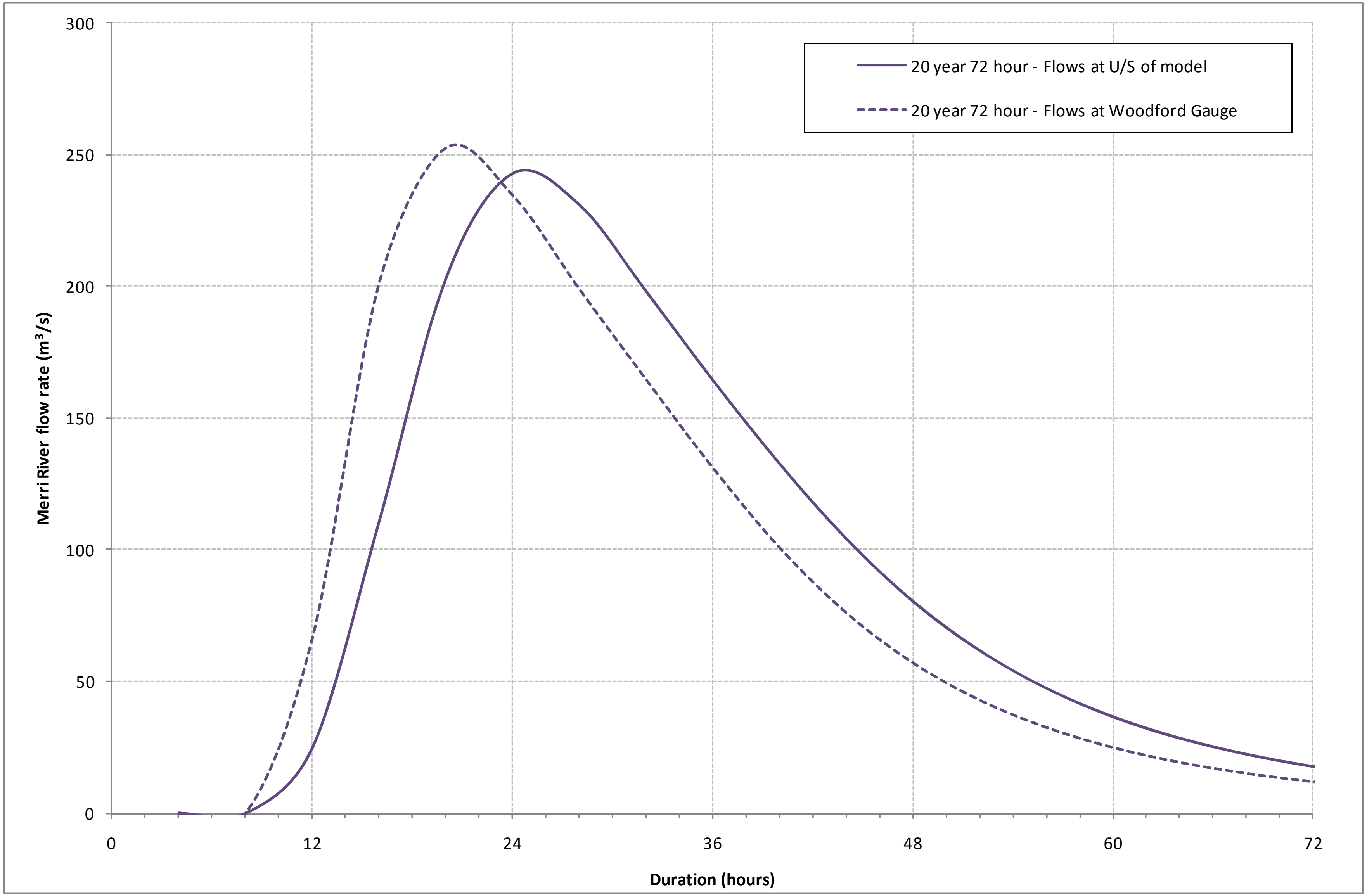

Figure B.5 Comparison of the 20 year hydrographs for Merri River between the Woodford gauge and the u/s boundary of the model

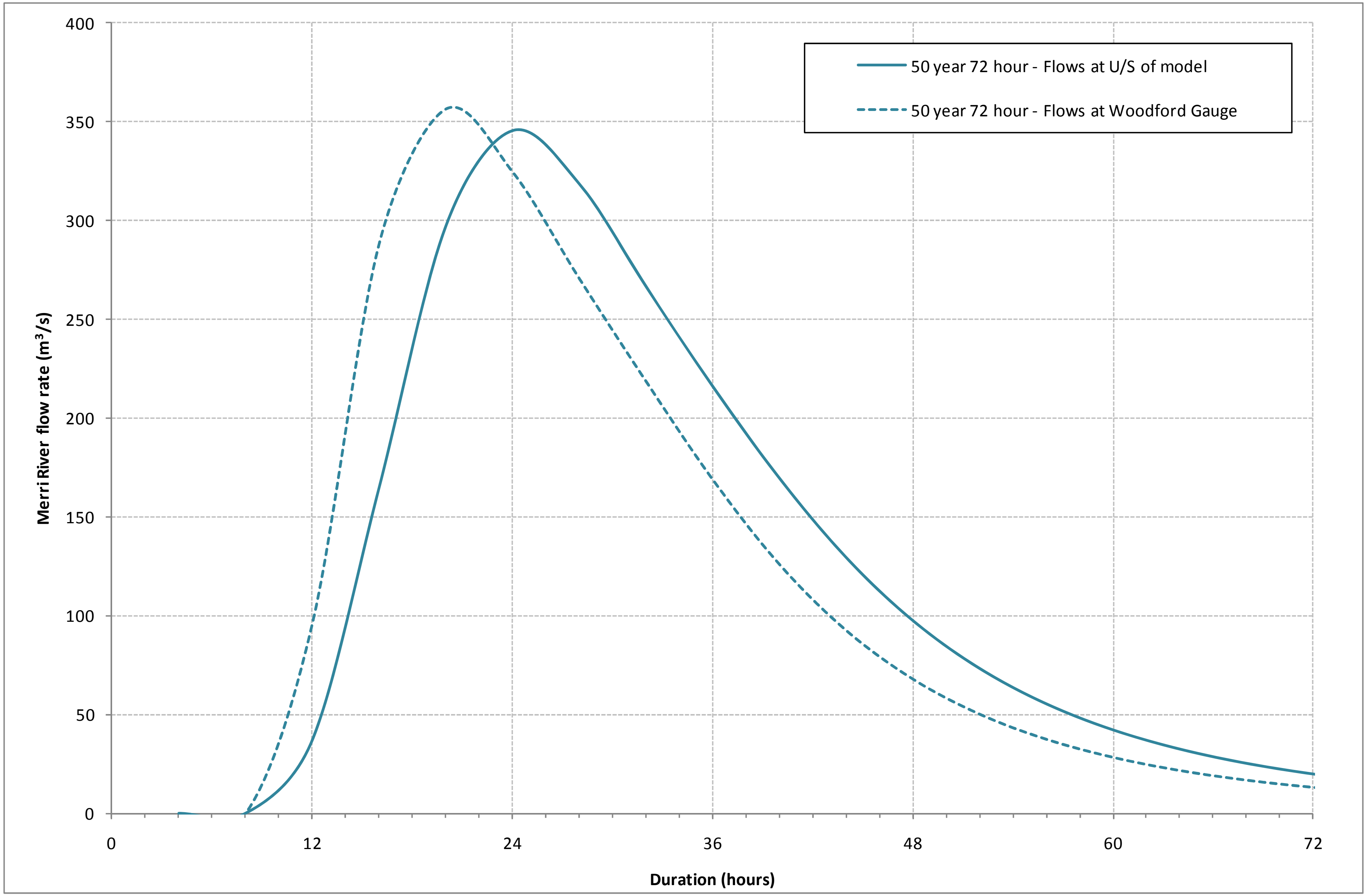

Figure B.6 Comparison of the 50 year hydrographs for Merri River between the Woodford gauge and the u/s boundary of the model

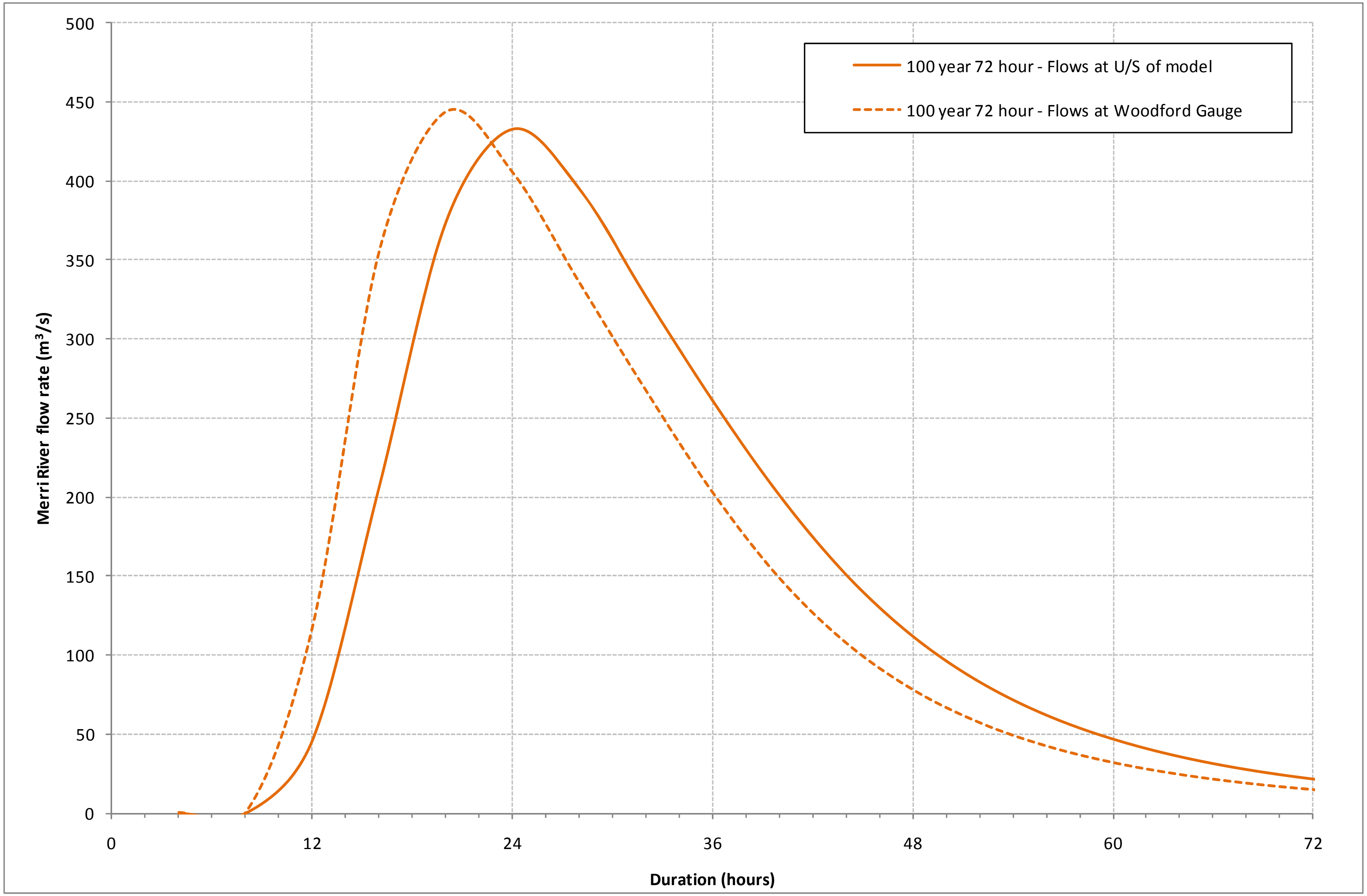

Figure B.7 Comparison of the 100 year hydrographs for Merri River between the Woodford gauge and the u/s boundary of the model

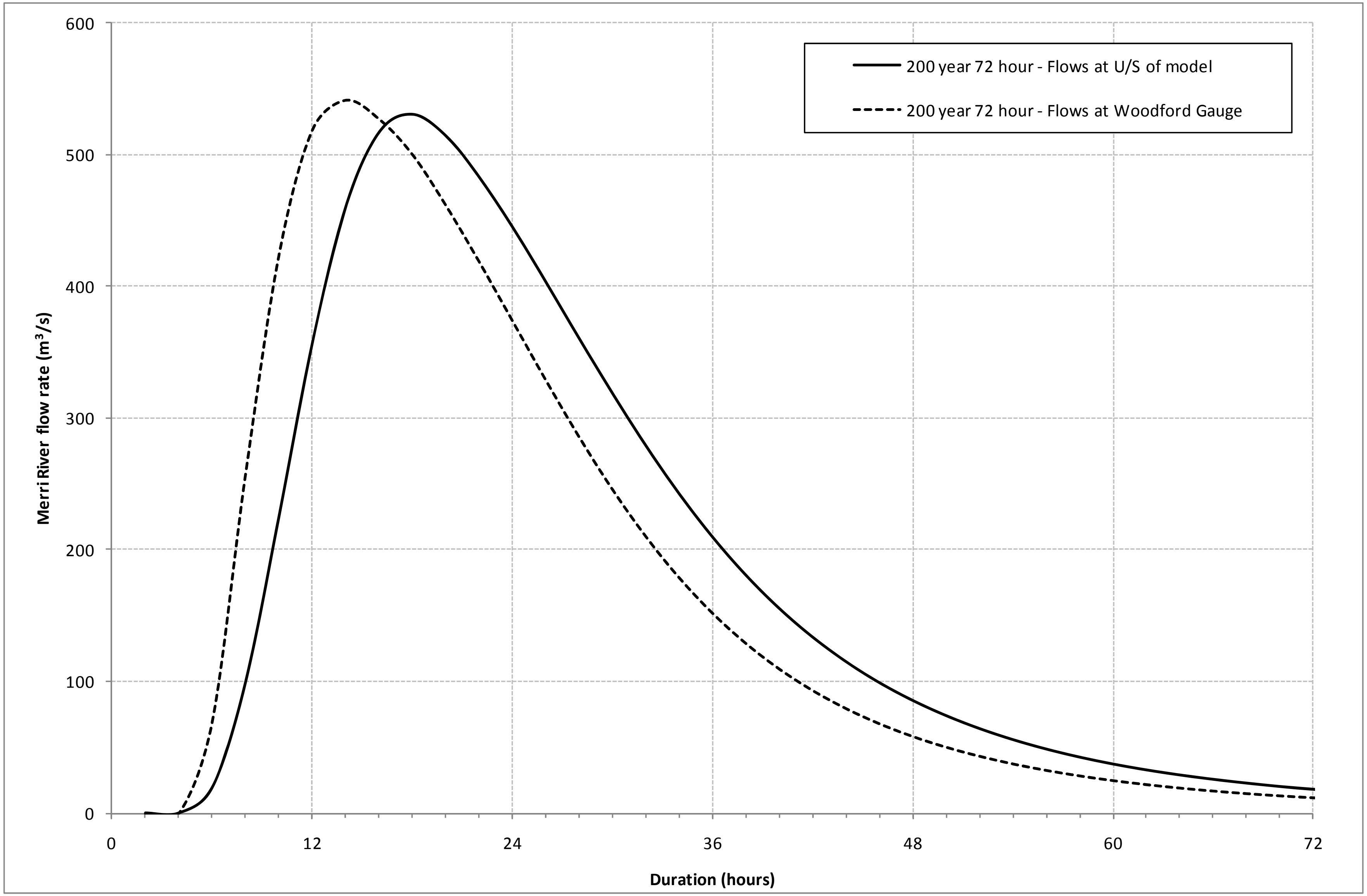

Figure B.8 Comparison of the 200 year hydrographs for Merri River between the Woodford gauge and the u/s boundary of the model

Appendix C

Rainfall Frequency Analysis

24 hr Rainfall Frequency Analysis						
	CARAMUT (BARWIDGEE)	ELLERSLIE POST OFFICE	HAWKESDALE POST OFFICE	KOROIT	PENSHURST (THE GUMS)	WOOLSTHORPE
ARI	Rainfall (mm)	Rainfall (mm)	Rainfall (mm)	Rainfall (mm)	Rainfall (mm)	Rainfall (mm)
5	47.7	51.0	48.0	46.9	49.9	45.5
10	58.1	62.9	58.4	58.4	61.4	55.0
20	69.1	75.5	69.2	71.5	73.8	64.8
25	72.9	79.7	72.8	76.0	78.1	68.1
40	81.1	89.0	80.6	86.4	87.4	75.3
50	85.3	93.5	84.5	91.7	92.1	78.8
100	98.8	108.3	97.0	109.8	107.5	90.2
200	113.8	124.5	110.4	130.7	124.6	102.4
500	136.0	147.9	129.7	163.4	150.2	120.1
1000	154.9	167.4	145.7	192.8	172.0	134.7
2000	175.8	188.7	162.9	226.9	196.2	150.6
10000	233.3	245.6	208.5	328.1	263.4	192.6

48 hr Rainfall Frequency Analysis						
	CARAMUT (BARWIDGEE)	ELLERSLIE POST OFFICE	HAWKESDALE POST OFFICE	KOROIT	PENSHURST (THE GUMS)	WOOLSTHORPE
ARI	Rainfall (mm)	Rainfall (mm)	Rainfall (mm)	Rainfall (mm)	Rainfall (mm)	Rainfall (mm)
5	60.7	65.6	63.7	61.1	61.7	59.5
10	72.1	79.7	76.6	75.3	74.5	72.2
20	83.6	94.2	89.6	91.5	87.7	85.7
25	87.3	99.0	93.9	97.2	92.1	90.3
40	95.3	109.5	103.0	110.0	101.7	100.4
50	99.2	114.6	107.4	116.6	106.4	105.4
100	111.6	131.2	121.5	139.0	121.7	122.0
200	124.5	149.0	136.2	164.9	138.1	140.4
500	142.6	174.5	157.0	205.7	161.9	167.6
1000	157.2	195.4	173.6	242.3	181.5	190.7
2000	172.5	218.0	191.2	284.7	202.8	216.4
10000	211.6	277.4	236.0	411.1	259.2	287.0

72 hr Rainfall Frequency Analysis						
	CARAMUT (BARWIDGEE)	ELLERSLIE POST OFFICE	HAWKESDALE POST OFFICE	KOROIT	PENSHURST (THE GUMS)	WOOLSTHORPE
ARI	Rainfall (mm)	Rainfall (mm)	Rainfall (mm)	Rainfall (mm)	Rainfall (mm)	Rainfall (mm)
5	66.2	73.3	70.4	69.1	68.7	66.0
10	78.3	88.1	84.8	84.5	82.2	79.7
20	90.5	103.0	99.7	102.0	95.9	94.3
25	94.5	107.9	104.6	108.2	100.4	99.3
40	103.1	118.4	115.4	122.1	110.2	110.4
50	107.2	123.4	120.7	129.2	115.0	115.9
100	120.6	139.8	137.9	153.6	130.3	134.3
200	134.7	156.9	156.3	181.9	146.6	154.7
500	154.5	181.1	182.9	226.4	169.8	185.2
1000	170.6	200.7	204.9	266.4	188.7	211.3
2000	187.7	221.4	228.7	312.9	209.0	240.5
10000	231.6	274.5	291.7	451.9	261.6	321.8

Appendix D

TBC