WOLLONDILLY AND MULWAREE RIVERS FLOOD STUDY

GOULBURN MULWAREE COUNCIL

FINAL REPORT

SEPTEMBER 2016

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AEP	Annual Exceedance Probability
AHD	Australian Height Datum
ARI	Average Recurrence Interval
ALS	Airborne Laser Scanning
BOM	Bureau of Meteorology
CMA	Central Mapping Authority
DECC	Department of Environment and Climate Change
DNR	Department of Natural Resources
DRM	Digital Rainfall Method
DTM	Digital Terrain Model
GIS	Geographic Information System
GPS	Global Positioning System
IFD	Intensity, Frequency and Duration of Rainfall
mAHD	meters above Australian Height Datum
PMF	Probable Maximum Flood
SRMT	Shuttle Radar Mission Topography
TUFLOW	one-dimensional (1D) and two-dimensional (2D) flood and tide simulation software program (hydraulic computer model)
WBNM	Watershed Bounded Network Model (hydrologic computer model)

TERMINOLOGY USED IN REPORT

Australian Rainfall and Runoff have produced a set of draft guidelines for appropriate terminology when referring to the probability of floods. In the past, AEP has generally been used for those events with greater than 10% probability of occurring in any one year, and ARI used for events more frequent than this. However, the ARI terminology is to be replaced with a new term, EY.

Annual Exceedance Probability (AEP) is expressed using percentage probability. It expresses the probability that an event of a certain size or larger will occur in any one year, thus a 1% AEP event has a 1% chance of being equalled or exceeded in any one year. For events smaller than the 10% AEP event however, an annualised exceedance probability can be misleading, especially where strong seasonality is experienced. Consequently, events more frequent than the 10% AEP event are expressed as X Exceedances per Year (EY). Statistically a 0.5 EY event is not the same as a 50% AEP event, and likewise an event with a 20% AEP is not the same as a 0.2 EY event. For example an event of 0.5 EY is an event which would, on average, occur every two years. A 2 EY event is equivalent to a design event with a 6 month average recurrence interval where there is no seasonality, or an event that is likely to occur twice in one year.

While AEP has long been used for larger events, the use of EY is to replace the use of ARI, which has previously been used in smaller magnitude events. The use of ARI, the Average Recurrence Interval, which indicates the long term average number of years between events, is now discouraged. It can incorrectly lead people to believe that because a 100-year ARI (1% AEP) event occurred last year it will not happen for another 99 years. For example there are several instances of 1% AEP events occurring within a short period, for example the 1949 and 1950 events at Kempsey.

Where the % AEP of an event becomes very small, for example in events greater than the 0.02 % AEP, the ARR draft terminology suggest the use of 1 in X AEP so a 0.02 % AEP event would be the same as a 1 in 5,000 AEP.

The PMF is a term also used in describing floods. This is the Probable Maximum Flood that is likely to occur. It is related to the PMP, the Probable Maximum Precipitation.

This report has adopted the approach of the ARR draft terminology guidelines and uses % AEP for all events greater than the 10% AEP and EY for all events smaller and more frequent than this.

EY	AEP (%)	AEP (1 in x)	ARI	Use
6	99.75	1.002	0.17	
4	98.17	1.02	0.25	
3	95.02	1.05	0.33	WSUD
2	86.47	1.16	0.50	
1	63.21	1.58	1.00	
0.69	50.00	2	1.44	
0.5	39.35	2.54	2.00	Stormwater/pit and pipe design
0.22	20.00	5	4.48	
0.2	18.13	5.52	5.00	
0.11	10.00	10	9.49	
0.05	5.00	20	20	
0.02	2.00	50	50	
0.01	1.00	100	100	
0.005	0.50	200	200	Flooding
0.002	0.20	500	500	
0.001	0.10	1000	1000	
0.0005	0.05	2000	2000	Limit CRC FORGE
0.0002	0.02	5000	5000	Extreme risk /Dams
PMF	**1×10^{-5} AEP - 1×10^{-7} AEP**			

A copy of the draft terminology is available at: http://www.arr.org.au/arr-guideline/draft-chapters/

FOREWORD

The NSW State Government's Flood Prone Land Policy provides a framework to ensure the sustainable use of floodplain environments. The Policy is specifically structured to provide solutions to existing flooding problems in rural and urban areas. In addition, the Policy provides a means of ensuring that any new development is compatible with the flood hazard and does not create additional flooding problems in other areas.

Under the Policy, the management of flood liable land remains the responsibility of local government. The State Government co-funds floodplain risk management studies, plans and measures to alleviate existing problems and provides specialist technical advice to assist Councils in the discharge of their floodplain management responsibilities.

The Policy provides for technical and financial support by the Government through five sequential stages:

1. Data Collection
- Data requirements for an ensuing flood study are assessed. Existing data sets are assessed for usability and existing reports collected and summarised.

2. Flood Study
- Determine the nature and extent of the flood problem.

3. Floodplain Risk Management
- Evaluates management options for the floodplain in respect of both existing and proposed development.

4. Floodplain Risk Management Plan
- Involves formal adoption by Council of a plan of management for the floodplain.

5. Implementation of the Plan
- Construction of flood mitigation works to protect existing development, use of Local Environmental Plans to ensure new development is compatible with the flood hazard.

The Goulburn Mulwaree Flood Study (the Study) presented herein constitutes the first and second stages of the NSW Floodplain Risk Management Program for the township of Goulburn. The Study takes into account flooding from both the Wollondilly and the Mulwaree Rivers.

WMAwater has been engaged by Goulburn Mulwaree Council to prepare this Study under the guidance of the Goulburn Floodplain Risk Management Working Party (GFRMWP).

1. INTRODUCTION AND BACKGROUND

This Study has been prepared by WMAwater on behalf of the Goulburn Mulwaree Council (Council). The main objective of this study is to define mainstream flood behaviour at Goulburn due to the Wollondilly and Mulwaree Rivers. The Study has examined past flood events in addition to undertaking a flood assessment for a range of design storms under existing conditions. The findings in this report provide material to inform Council with regards to managing existing and future flood risk due to mainstream flooding at Goulburn.

1.1. Objectives

The information and results obtained from this study define existing flood behaviour for the Wollondilly and Mulwaree Rivers and provide a firm basis for the development of a subsequent Floodplain Risk Management Study and Plan (FRMS&P).

Primarily, the study was developed in order to meet the objective of defining design flood behaviour (0.2 EY, 20%, 10%, 2%, 1%, 0.5% AEP events and the Probable Maximum Flood (PMF)) for mainstream flooding at Goulburn and to produce:

- Flood levels, extents, velocities and flows for the full range of modelled design events;
- Provisional hazard and preliminary hydraulic category maps for the 1% AEP and PMF events;
- Flood emergency response classification of communities;
- Analysis on the sensitivity of flood behaviour to changes in flood producing rainfall events due to climate change; and
- A modelling system to be used in the subsequent FRMS&P to test proposed flood risk management strategies.

1.2. The Study Area

Goulburn is located in the Southern Tablelands of NSW approximately 220 km south-west of Sydney in the Goulburn Mulwaree Council Local Government Area (LGA). The township is located immediately upstream (west) of the confluence of the Wollondilly and Mulwaree Rivers. Historically, flooding due to both rivers has been experienced at Goulburn. The study area's upstream boundary on the Wollondilly River is located approximately 1 km upstream of Rossi Weir and the upstream boundary along the Mulwaree River is roughly 4 km upstream of the Hume Highway. The downstream boundary is located approximately 8 km downstream of the confluence of these two rivers. Figure 1 displays the Goulburn study area.

The Wollondilly River originates in the southern areas of the Greater Blue Mountains, which is characterised by steep rugged terrain and a well-defined/confined floodplain. The Wollondilly River flows south and then east towards Goulburn and has a catchment area of 708 km^2 upstream of Goulburn at Rossi Weir. The Mulwaree River approaches Goulburn from the south and has an upstream catchment area of 770 km^2 at the Hume Highway. The Mulwaree River catchment is comparatively flat with a wide/dispersed floodplain. A Digital Elevation Model

(DEM) of the region is presented in Figure 2, which illustrates the relatively sharp relief and constrained floodplain on the Wollondilly River and the flatter, wide floodplain of the Mulwaree River.

The city of Goulburn has a population of approximately 23,000 (2014 census). Land use in Goulburn is predominantly composed of low-density residential and commercial development. Numerous areas of open space along the floodplains of both rivers are present, such as Leggett Park (Wollondilly River) and Goulburn Golf Course (Mulwaree River).

1.3. Flood History

Historically, mainstream flooding due to the Wollondilly and Mulwaree Rivers at Goulburn has been a relatively infrequent occurrence, however the impact of flooding is significant to the community. Historic newspaper articles indicate that major flood events, known to have caused flooding of properties at Goulburn, occurred in April 1870, July 1900, June 1925, June 1950, October 1959, November 1961 and August 1974. More recently, significant flooding in Goulburn has occurred in August 1990, December 2010, March 2012 and June 2012.

No long term gauge is available in the region on either river which means that no complete series of annual maximum flood events can be defined. This leads to difficulties in determining event magnitude and event rank.

Keeping this limitation in mind, the following sections investigate the flood history of these two rivers. It must be noted that the magnitude of flooding in these rivers is weakly correlated with significant variation in event magnitude occurring in each river during the same event. This means that a very large flood event occurring in the Wollondilly River does not always mean that a large flood event will occur in the Mulwaree River, and vice versa. For example the 2010 flood event has an estimated probability of approximately 1% AEP on the Wollondilly River, whilst the Mulwaree River flood is estimated to have a probability of approximately 10% AEP. Historically, little information about each river's respective magnitude is provided with information often provided about Goulburn flooding as a whole. The following sections have attempted to provide individual information on historic flood event's rank and magnitude for each river.

1.3.1. Wollondilly River Flood History

The earliest reported flood of significance on the Wollondilly River occurred during April 1870 and led to the destruction of the Marsden's Crossing Bridge (near the existing Marsden Bridge on Fitzroy Street). The bridge was built three feet higher than the highest previously recorded flood however the 1870 event overtopped the bridge and destroyed it. This flood was reported to be six feet higher than any previously known flood on the Wollondilly River at Goulburn (The Empire, 3rd May 1870). Following the 1870 event, a number of smaller, yet significant, flood events occurred in the early 20th Century. The June 1925 flood on the Wollondilly River was noted to be the worst experienced in 37 years, with the evacuation of residents from West Grove (the area near Braidwood Road) (Barrier Miner, 22nd June 1925).

Numerous flood events occurred in the period of 1950 to 1980 with the largest occurring in

November 1961, followed by the October 1959 and August 1974 flood events. Prior to the 2010 flood event, the 1961 flood event was the largest historic flood event to be experienced on the Wollondilly River at Goulburn.

The largest flood event to have occurred on the Wollondilly River at Goulburn since at least 1870, and presumably since settlement of the city, is the December 2010 flood event. The Goulburn Local Flood Plan (SES, October 2011) indicated that approximately ten properties were inundated over floor during the event. The 2010 flood event is determined to have been between 0.9 m and 0.1 m higher than the 1961 flood event dependant on location. This large range of differences in peak flood level between the two events is due to changes on the floodplain between 1961 and 2010. For example, numerous bridges have been constructed or replaced during this period with the impact of these structures noted in the variance in peak flood level between the two events. Also, surveyed peak flood level marks for the 1961 event (Reference 1) situated in the vicinity of Marsden Bridge (constructed in 1981) and at Marsden Weir are approximately 0.3 to 0.9 m lower than the 2010 event, where as in the significantly flood affected area downstream of Victoria Street Bridge (constructed in 1968) surveyed peak flood levels for the 1961 event are only 0.1 m lower. The impact of changes to these bridges on peak flood levels is increased peak flood levels upstream and decreased to flood levels downstream. Changes in vegetation density likely also played a significant role in these anomalies.

The 1986 Flood Study (Reference 1) also assessed flood history at Goulburn and derived a partial series of floods at Marsden Weir from 1870 to 1977. This data was constituted from a combination of available gauge data (1962 to 1977) and transformed levels from observed data at the Marsden Bridge situated 200 m downstream of the weir. Table 1 presents this data with additional level and discharge estimates for the 2010, 2012 and 2013 events obtained from the calibrated hydraulic model (see Section 7.1.1).

An additional significant flood event occurred in 1990 with the community consultation process (see Section 3) indicating that it was approximately 0.3 m lower than the 2010 flood event downstream of Victoria Street. The magnitude of the event at Marsden weir cannot be estimated with any certainty and thus it has not been included in Table 1.

Table 1: Peak Flood Levels and Discharges: Wollondilly River at Marsden Weir Goulburn

Date	Peak Gauge Height (m)	Peak Discharge (m^3/s)	Comments
1870 (Nov.)	3.13	820	Adjusted from Marsden Bridge Level
1900	2.37	630	Adjusted from Marsden Bridge Level
1925	2.02	490	Adjusted from Marsden Bridge Level
1943	2.20	560	Adjusted from Marsden Bridge Level
1950	2.29	600	Adjusted from Marsden Bridge Level
1952	2.48	675	Adjusted from Marsden Bridge Level
1959 (Oct.)	3.13	820	Adjusted from Marsden Bridge Level
1959 (July)	1.92	450	Adjusted from Marsden Bridge Level
1961 (Nov.)	3.24	900	Adjusted from Marsden Bridge Level
1962 (Sept.)	0.85	74	Gauge Records
1963 (Aug.)	1.62	300	Gauge Records
1964 (Oct.)	0.90	85	Gauge Records
1965 (Oct.)	0.45	17	Gauge Records
1966 (Nov.)	0.68	44	Gauge Records
1967 (Sept.)	1.01	112	Gauge Records
1968 (Aug.)	0.50	22	Gauge Records
1969 (Oct.)	1.29	170	Gauge Records
1970 (Sept.)	0.89	85	Gauge Records
1971	1.30	180	Gauge Records
1972	0.44	10	Gauge Records
1973 (Sept.)	0.46	21	Gauge Records
1974 (Aug.)	2.54	720	Gauge Records
1975 (July)	1.32	185	Gauge Records
1976 (Sept.)	1.75	350	Gauge Records
1977	0.27	7	Gauge Records
2010 (Dec.)	4.13	1058	Results from Calibrated Hydraulic Model
2012 (March)	1.67	311	Results from Calibrated Hydraulic Model
2013 (June)	1.68	316	Results from Calibrated Hydraulic Model

Note: gauge zero = 630.46 mAHD

1.3.2. Mulwaree River Flood History

Information about historic flooding on the Mulwaree River at Goulburn is scarce with no available stream gauge to provide reliable flood data and only anecdotal information available. Notwithstanding, an attempt has been made to identify and rank significant historic flood events.

The April 1870 flood event was again the first flood of significance on the Mulwaree River since settlement of Goulburn. The event was reported to have been approximately two feet higher than the previous large flood in 1864 (The Empire, Sydney NSW, 3rd May 1870).

After the 1870 event no recorded large flood events occurred until July 1900, when The Sydney Morning Herald (6th July 1900) reported that the Mulwaree Rivers overtopped its banks and inundated a number of residential properties as well as the Goulburn Brewery.

There is no record of any large flood events occurring in the Mulwaree River catchment after

1900 (minor flooding noted in May 1925, Sydney Morning Herald 13th May 1925) up until the October 1959 flood event. The 1986 Goulburn Flood Study (Reference 1) provides surveyed peak flood levels for this event (in mAHD) that were used in the Reference 1 model calibration. Examination of available records indicates that the 1959 event was the largest Mulwaree River flood on record, and was at approximately 0.3 m higher than the second largest event occurring in August 1974. Comparison of the Reference 1 study 1959 event peak flood levels to the current study design results indicates that this event had an AEP between 1% and 0.5% with the difference in flood level between these two events ranging between approximately ± 0.3 m. Of note is that the 1959 flood event in the neighbouring Yass River catchment, situated west of the Mulwaree catchment, also had a magnitude of approximately 1% AEP.

In more recent history, the August 1990 flood at Goulburn was noted to have caused flooding to residential properties, the race track and the golf course (The Canberra Times, 3rd August 1990) as well as the forced the evacuation of 40 families. The community consultation process indicated that the 1990 flood event was approximately 0.2 - 0.3 m higher than the recent 2010 flood event on the Mulwaree River. The 2012 and 2013 flood events are noted to have been minor with no affectation of residential properties.

Table 2 presents peak flood levels (in mAHD) at a number of locations on the Mulwaree River obtained from a variety of sources. Comparison of these levels provides an indication of the magnitude of historic flood events on the Mulwaree River.

Table 2: Peak Flood Levels (mAHD) – Mulwaree River Goulburn

Location	Event	Peak Flood Level (mAHD)	Rank	Comments
Fitzroy Bridge	1959	630.08*	1	Bridge Design Plans
	1974	630.00	2	Observed Level (1986 Flood Study)
	2010	629.22	3	Results from Calibrated Hydraulic Model
	2012	626.39	4	Results from Calibrated Hydraulic Model
	2013	626.45	5	Results from Calibrated Hydraulic Model
Upstream of Golf Course, Eleanor Street	1959	630.67	1	Observed Level (1986 Flood Study)
	1974	630.32	2	Observed Level (1986 Flood Study)
	2010	629.32	3	Results from Calibrated Hydraulic Model
Corner Hercules & Genelg Street	1959	630.66	1	Observed Level (1986 Flood Study)
	1974	630.35	2	Observed Level (1986 Flood Study)
	2010	629.34	3	Results from Calibrated Hydraulic Model
Northern end of Emma Street	1974	630.32	1	Community consultation
	1990	629.52	2	Community consultation
	2010	629.36	3	Community consultation

* Noted as the maximum recorded flood level at Fitzroy Bridge on Department of Main Roads NSW design plans. The Fitzroy Bridge approaches are large and provide a significant restriction to flow. If the 1959 event had occurred post construction of the bridge it is likely that the peak flood level would be significantly higher.

2. AVAILABLE DATA

Various items of data salient to the study have been collected and reviewed. Most datasets were sourced from Council, NSW Office of Water (NOW), the Bureau of Meteorology (BOM), the Roads and Maritime Services (RMS), Sydney Catchment Authority (SCA) and the Office of Environment and Heritage (OEH) and supplemented by additional survey where required. The community consultation process also provided some data based on the residents knowledge of the local area (see Section 3). The key focus of the exercise was to collect data suitable for the model build and the calibration/validation process. This section provides a summary of the various forms of data utilised in the study.

2.1. Relevant Studies

2.1.1. Water Resources Commission of NSW, Goulburn Flood Study 1986 (Reference 1)

The Water Resources Commission of NSW completed a Flood Study for Goulburn in 1986. Flood frequency analysis was conducted for the partial series presented in Table 1 (excluding the 2010, 2012 and 2013 events) for flows on the Wollondilly River at the Marsden Weir. A 100 year ARI flow of 900 m³/s was calculated, however due to the relatively short recorded period and lack of confidence in event flow estimates, the Marsden Weir FFA derived flows were not used for design flows. Instead, Regional Flood Frequency Analysis (RFFA) of nearby gauged catchments (Lachlan Basin and Hawkesbury Basin) was used and translated into design flows for the Wollondilly and Mulwaree Rivers with the results of the partial series FFA conducted at Marsden Weir used to verify the RFFA results. The 1986 Flood Study design flows are presented in Table 3.

Table 3: 1986 Flood Study Design Flows - RFFA

Event (ARI)	RFFA Flow (m³/s)
5	293
10	447
20	638
50	957
100	1270

Hydraulic modelling was undertaken with HEC-2 and the floodplain topography was defined using a series of surveyed cross-sections across the channel and floodplain. This model was calibrated against surveyed levels from the 1974 flood event and validated using the 1961 and 1959 flood events. Peak flood profiles for the 100 year ARI and 20 year ARI were derived from the calibrated hydraulic model and the flood extent for these events inferred from available topographic data.

2.1.2. SMEC, Wollondilly River and Mulwaree Chain of Ponds Floodplain Risk Management Study and Plan (2003) – Reference

SMEC completed a Floodplain Risk Management Study and Plan in 2003 based on the results of the 1986 Flood Study by the Water Resources Commission of NSW (see Section 2.1.1). As part of this study, the hydraulic model and flood frequency analysis was updated and several mitigation strategies were recommended for implementation in Goulburn.

The RFFA was modified in the 2003 SMEC Study using additional gauges in surrounding catchments and extending the period of record. The design flows were revised as per those outlined in Table 4, with an increase in flow of 145 m³/s noted for the 1% AEP event

Table 4: 2003 FRMS&P Revised Design Flows - RFFA

Event (AEP)	RFFA Flow (m³/s)
20%	258
10%	428
5%	648
2%	1,026
1%	1,415
0.5%	1,868
0.2%	2,519
Extreme Flood	4,244

*Note that these flows were obtained from the hydraulic model results presented in Appendix D of the Reference 2 study. The main body of the report indicates that the 1% AEP flow 'adopted for the Wollondilly River and Mulwaree Ponds at Goulburn' is 2,185 m³/s, which is presumably a typing error.

The 2003 SMEC Study also converted the HEC-2 hydraulic model to a HEC-RAS model and the downstream boundary was extended.

This Study investigated a number of mitigation options, recommending channel improvements particularly along the Mulwaree River. The study also suggests options such as voluntary purchase, voluntary house raising, flood proofing of buildings and improving flood access and emergency response as viable solutions to the flood problem at Goulburn.

Comparison of the current study's results to the Reference 2 study results is presented in Section 7.1.4.

2.2. Model Build and Calibration Data

Topographical and survey data provide a basis for both the hydrologic and hydraulic models in terms of catchment delineation and properties. Furthermore, in a hydraulic model this data is vital for model configuration. Structures such as bridges and culverts need to be realistically represented to reproduce accurate hydraulic properties. This information has been obtained from a variety of sources including Council, OEH, RMS and survey where information was not available.

Additional information used to ensure the models accuracy through calibration/validation was also obtained from a variety of sources including Council, OEH, SES, NOW, SCA and BoM. Information such as historic rainfall (see Section 2.4.1) and flow data (see Section 2.5) have been used to calibrate/validate the hydrologic model, and surveyed peak flood level marks (see Section 2.6) and stream gauge data have been used to calibrate/validate the hydraulic model.

The topographical and survey data used to construct and calibrate both the hydrologic and hydraulic models is outlined in Sections 2.2.1 to 2.6.

2.2.1. ALS Data

Airborne Laser Scanning (ALS) data of the study area was provide by LPI (via Council) and was used to define ground surface elevation. The ALS data was flown in 2011. ALS provides ground level spot heights from which a Digital Elevation Model (DEM) has been constructed. For the purpose of this study a two metre DEM grid was constructed and this data, in combination with channel cross section survey (see Section 2.2.2), formed the foundation of the 2D hydraulic model build process (see Section 6). The DEM for the study area is presented in Figure 2.

The accuracy of the ground definition of the ALS data can be adversely affected by the nature and density of vegetation and/or the presence of steeply varying terrain. It should be noted however, that the quality of the ALS data within the study area is generally high and has been verified using local benchmarks.

2.2.2. River Bathymetry Survey

The DEM generated from the ALS data mentioned in Section 2.2.1 does not define the in-bank bathymetry below the water level at the time survey was flown. To determine the in-bank conveyance below the water level, bathymetry survey was carried out. Bathymetry survey was undertaken by a qualified hydrosurvey firm (Southern Cross Consulting Surveyors) who produced a dataset of approximately 200 cross-sections for portions of the Wollondilly and Mulwaree Rivers to define in-bank bathymetry (displayed in Figure 1).

The cross sections were used to generate a DEM of the Rivers bathymetry (within the river banks). The bathymetry was then combined with the ALS data (see Section 2.2.1) to create a DEM of the combined in-bank and floodplain. This combined DEM was used for modelling purposes.

2.2.3. 90 m SRTM Data

For the wider catchment, the Consortium for Spatial Information's 90m-SRTM DEM data, which is a 90 m resolution DEM from the Shuttle Radar Topographic Mission has been used. This data has been used in catchment delineation for the hydrologic model and is displayed in Figure 2. Whilst not of a comparable accuracy or resolution to the ALS data the SRTM data is adequate for catchment delineation work.

2.2.4. Hydraulic Structure Data

Structures such as bridges, weirs and road/rail crossings can impact on flood behaviour. In Goulburn a number of structures on both the Wollondilly and Mulwaree River floodplains were identified as having the potential to impact significantly on flood behaviour.

Design plans for the structures listed below were obtained from Council, the Bridge Engineering Section of the RMS and the Australian Rail Track Corporation (ARTC):

1. Rossi Bridge over the Wollondilly River

2. Victoria Street Bridge which crosses the Wollondilly River
3. Sewer Aqueduct crossing the Wollondilly River
4. Hume Highway Bypass bridges crossing the Mulwaree River
5. Lansdowne Bridge over the Mulwaree River
6. Park Road Culverts over the Mulwaree River
7. The Railway Viaduct across the Mulwaree River
8. Sydney Road Bridge across the Mulwaree River

The locations (numbered accordingly) of these structures are presented in Figure 1.

2.2.4.1. Hydraulic Structure Survey

Where design bridge plans were not available, hydraulic structure survey was conducted. These structures are listed below with their locations (numbered accordingly) displayed in Figure 1:

9. Rossi Weir across the Wollondilly River
10. Marsden Weir crossing the Wollondilly River, 200 m upstream of Fitzroy Street
11. Marsden Bridge which crosses the Wollondilly River at Fitzroy Street
12. Kenmore Bridge which crosses the Wollondilly River along Tarlo Street
13. Railway Bridge over the Wollondilly River 200 m downstream of Tarlo Street
14. Weir Structure across the Mulwaree River (580 m downstream of Bungonia Road)
15. Bridge crossing the Mulwaree River along Braidwood Road
16. Weir Structure across the Mulwaree River (880 m upstream of Braidwood Road)
17. Weir/Causeway across the Wollondilly River along Murrays Flat Road

Survey for each structure was undertaken by Southern Cross Consulting Services (SCCS) so that the conveyance capacity and other details of these structures could be accurately modelled. The following features were surveyed for each bridge:

- Creek cross section survey at upstream face;
- Creek cross section survey at downstream side offset a few meters from structure;
- Pier locations and width;
- Level of deck underside at each creek side (and middle if curved bridge deck);
- Level of deck top at each creek side (and middle if curved bridge deck); and
- Level of fence/railing top at each creek side (and middle if curved bridge deck).

For each weir the following data was requested:

- The crest level
- Spillway characteristics; and
- Details of any additional culverts such as internal dimensions of circular culverts (diameter) and rectangular box culverts (width, height) and upstream and downstream levels of culvert inverts

2.3. Sooley and Pejar Dams

The Wollondilly catchment upstream of Goulburn has two significant storages; Sooley Dam and Pejar Dam. Pejar Dam is situated 70 km upstream of Goulburn and has an upstream catchment of 143 km^2 and a capacity of 9.0 GL. Sooley Dam has an upstream catchment of 126 km^2 and a capacity of 6.0 GL.

For the purpose of this Study, it has been assumed that these dams do not impact on Wollondilly River design flows at Goulburn. The same assumption was also made in the Reference 1 Goulburn Flood Study (1986). While these dams have the potential to attenuate minor flows depending on available airspace and event volume, they are unlikely to impact significantly on flood events at Goulburn for the following reasons:

- The dams are situated well upstream of Goulburn;
- Their upstream catchments areas are small relative to the entire upstream catchment of the Wollondilly River at Goulburn;
- The dams are primarily used for water supply and therefore are kept as full as possible;
- Flood events tend to occur during wet sequences, such as the period of 2010 to 2013 where these dams were close to 100% capacity the majority of the time;
- Floods are typically preceded by significant antecedent rainfall which is likely to fill the dams prior to a flood event.

The above reasons indicate that the dams will typically be near 100% capacity at the onset of flood causing rainfall. Table 5 presents the dams' pre-event minimum percentage storage capacity for downstream flood peak attenuation to occur. This was determined by examining the volume of the rising limb of the each dam's flood hydrograph for the 36 hour duration event (the Wollondilly River at Goulburn critical duration, see Section 5.8.1) and comparing this to the storage capacity. If the volume of the rising limb of a flood hydrograph is greater than the dam's available airspace then the flood peak will pass through the dam unattenuated. The results indicate that at the onset of a 0.2 EY event, Sooley Dam can be at 77% capacity and not attenuate peak flows. As event magnitude increased the required dam airspace to attenuate the flood peak is decreased. For example, during the 1% AEP event, Sooley Dam can be at only 38% capacity and the peak flow will pass through unattenuated.

Table 5: Minimum Storage Capacity (%) of Dams pre Event for Flood Peak Attenuation to Occur

Event	Dam Capacity at Start of Event (%)	
	Sooley Dam	Pejar Dam
0.2 EY	77	82
10% AEP	70	77
5% AEP	59	69
2% AEP	47	61
1% AEP	38	55
0.5% AEP	29	48

2.4. Rainfall Data

2.4.1. Historic Rainfall Data

The rainfall data described in the following sections pertains to information that was used in calibration/validation of the hydrologic and hydraulic model. Calibration/validation events were selected based on available pluviometer rainfall data (Section 2.3.1.1), daily read rainfall data (Section 2.3.1.2), stream gauge data (Section 2.4) and peak flood level data (Section 2.5). Selected events had all data requirements from these data sets. The hydrologic model was calibrated to the December 2010 event (see Section 5.5) and then verified using the March 2012 and June 2013 flood events (see Section 5.5.2). The hydraulic model was calibrated to the December 2010 event and validated using the March 2012 and June 2013 events (see Section 6.6.1).

Due to a lack of suitable rainfall data from any one source, a combination of pluviometer rainfall data (Section 2.3.1.1) and daily read rainfall data (Section 2.3.1.2) has been used to create rainfall inputs for the Wollondilly and Mulwaree catchments. Section 2.3.1.3 outlines the process of merging these data sets for use in the hydrologic model.

2.4.1.1. Pluviometer Rainfall Data

Pluviometer rainfall data (high temporal resolution rainfall data) is advantageous as it contains information on both a storms temporal pattern and total rainfall depth. Ten pluviometer rainfall gauges were identified within the catchment and have been used in the current study. Of the ten gauges, Council have commissioned eight of these which provides high definition data of the catchment's rainfall temporal patterns. Such high detail is not typically available for modelling. A summary of the pluviometer rainfall gauge details is presented below in Table 6.

Table 6: Available Pluviometers in the Goulburn area and their available record period

Name	Catchment	Distance to Catchment Centroid	Available Record Period		Owner of Pluviometer
			Start	End	
Pejar Dam	Wollondilly	5 km			Council
Pejar Creek	Wollondilly	7 km	01/08/2002	-	Council
Wollondilly River Upstream of Pejar Dam	Wollondilly	8 km	01/08/2002	-	Council
Sooley	Wollondilly	15 km	01/08/2002	-	Council
Sooley Dam	Wollondilly	16 km	01/01/2006	-	Council
Cardross	Wollondilly	16 km	07/08/2002	-	Council
Bumana	Wollondilly	17 km	15/08/2002	-	Council
Rossiville Weir	Wollondilly	18 km	01/08/2002	-	Council
Goulburn (Springfield)	Mulwaree	5 km	03/03/2009	-	BOM

In using the pluviometer data in model calibration, rainfall has been applied using an inverse distance weighting relationship for each sub-catchment upstream of the Murrays Flat gauge. This process involves analysing the proximity of each gauge to each sub-catchment and applying a weighting in the hydrologic model based on these distances i.e. the closest gauge was given the greatest weighting and the furthest gauge was given the smallest. Use of the inverse distance weighting relationship to incorporate pluviometer rainfall data aimed to account for spatio-temporal variation rainfall across the Wollondilly and Mulwaree catchments.

It is important to note that the Goulburn (Springfield) gauge is the only pluviometer rainfall gauge in the Mulwaree catchment, and thus the temporal representation of rainfall in this catchment is not as high as that in the Wollondilly catchment. This gauge was not operational during the period between January 2011 and May 2013. As a result, for the December 2012 flood event, only the Wollondilly rainfall gauges were available for use in the Mulwaree catchment.

2.4.1.2. Daily Read Rainfall Data

Daily read rainfall gauges do not adequately define the shorter duration intensities that are responsible for flooding Goulburn and (in isolation) are therefore not suitable for use in hydrologic/hydraulic model calibration or validation. However due to the spatial distribution of gauges, daily read rainfall data has been used to estimate total rainfall depths and rainfall spatial distribution across the catchment.

Regional daily read gauges were investigated to determine catchment rainfall depths for the three calibration/validation events. Table 7 presents the daily read rainfall gauges used, catchment location and distance to the catchment centroid. Table 8 displays the rainfall depths obtained at each of these gauges for the 2010, 2012 and 2013 flood events. The locations of the daily read gauges are displayed in Figure 3.

Rainfall depths for the region were created by interpolating (Nearest Neighbour) between neighbouring gauges. The estimated rainfall distribution for the 2010, 2012 and 2013 calibration events are presented in Figure 6 to Figure 12. Utilising these rainfall distribution grids, unique rainfall depths for each sub-catchment within the Goulburn sub-catchments were able to be calculated for the hydrologic model calibration/validation events. This allowed for modelling of the spatial variation in rainfall across the catchment.

Table 7: Daily Rainfall Gauges Used in this Study

ID	Name	Distance (km)*	Closest Catchment
63032	Golspie (Ayrston)	40 km	Wollondilly
63307	Taralga (Kiriwin)	38 km	Wollondilly
68008	Bundanoon Bowling Club	69 km	Wollondilly
68085	Nerriga (Tolwong)	43 km	Mulwaree
68100	Bundanoon (Plattwood)	70 km	Wollondilly
70011	Bungendore Post Office	38 km	Mulwaree
70012	Bungonia (Inverary Park)	27 km	Mulwaree
70025	Crookwell Post Office	20 km	Wollondilly
70036	Lake Bathurst (Somerton)	8 km	Mulwaree
70040	Goulburn (Cherryton)	7 km	Wollondilly
70055	Goulburn (Kippilaw)	15 km	Wollondilly
70060	Lower Boro (Calderwood)	24 km	Mulwaree
70063	Marulan (George St)	43 km	Wollondilly
70069	Crookwell (Gundowringa)	9 km	Wollondilly
70071	Goulburn (Pomeroy)	5 km	Wollondilly
70077	Goulburn (Springfield)	5 km	Mulwaree
70097	Breadalbane (Old Post Office)	20 km	Wollondilly
70105	Mount Fairy (Merigan)	25 km	Mulwaree
70111	Biala (Alvison)	28 km	Wollondilly
70119	Big Hill (Glen Dusk)	42 km	Wollondilly
70131	Woodhouselee (Leeston)	11 km	Wollondilly
70135	Mummell (Kangaroobie)	8 km	Wollondilly
70137	Gurrundah (Wandonga)	12 km	Wollondilly
70143	Brayton (Longreach)	37 km	Wollondilly
70147	Goulburn (Hillwood)	19 km	Wollondilly
70213	Gurrundah (Ashwell)	14 km	Wollondilly
70263	Goulburn Tafe	23 km	Mulwaree
70269	Marulan (Johnniefelds)	41 km	Wollondilly
70290	Collector (Winderadeen)	23 km	Mulwaree
70325	Wollondilly (River View)	57 km	Wollondilly

* Distance has been determined as the shortest distance from the Wollondilly or Mulwaree catchments centroid to each gauge.

Table 8: Recorded Daily Rainfall

ID	Name	Rainfall (mm)									
		Dec 2010		Feb/Mar 2012					June 2013		
		8th	9th	28th	29th	1st	2nd	3rd	23rd	24th	25th
63032	Golspie (Ayrston)	2	95	41.6	68.8	21	14.2	7	-	-	-
63307	Taralga (Kiriwin)	2.6	74.4	55.4	64	54.6	12.6	10	54.5	35	15.5
68008	Bundanoon Bowling Club	4.8	39.8	57.6	69.8	19	15.8	10.2	99.4	127	37
68085	Nerriga (Tolwong)	28.2	60.8	44.6	62	18.6	6.6	9	146	39.2	25
68100	Bundanoon (Plattwood)	4	42	-	-	-	-	-	-	-	-
70011	Bungendore Post Office	26	4.4	33.4	49.4	33	2	49	42.2	0.6	10.6
70012	Bungonia (Inverary Park)	-	-	56.8	69.4	19	5.6	12.8	117.8	23	19.6
70025	Crookwell Post Office	42.6	39	75	80	31.6	18	13	73.2	21.2	7.2
70036	Lake Bathurst (Somerton)	27.5	23	50	69	18	4.8	30.2	60	15	14
70040	Goulburn (Cherryton)	40	101	55	79	13	9.2	16.1	91	29	8.4
70055	Goulburn (Kippilaw)	40	71	64.4	65.6	17	7	20	106	12.4	12.4
70060	Lower Boro (Calderwood)	13.4	21.6	41.6	62.6	15.6	3.2	22.4	63.6	19	19.4
70063	Marulan (George St)	-	-	46	102.6	7.4	12.2	11.6	98.8	62	18
70069	Crookwell (Gundowringa)	32	67	-	-	-	-	-	60	13.4	10.2
70071	Goulburn (Pomeroy)	46	99	55.8	92.2	19.4	6.2	19.4	84	27	8
70077	Goulburn (Springfield)	60	60	60	80	17.2	4	25.2	80	27	7.6
70097	Breadalbane (Old Post Office)	136.4	18.8	63	64	18.2	5.6	25.2	98	4.4	8.4
70105	Mount Fairy (Merigan)	17.8	16	31.6	22.8	26.2	5.4	30.4	33	56	17
70111	Biala (Alvison)	52.6	63.4	75.2	61.4	27.6	16.2	11.2	102	13	9
70119	Big Hill (Glen Dusk)	5.3	42.9	79	61.4	29.6	8.6	7.2	55.4	49.4	15.2
70131	Woodhouselee (Leeston)	38.5	80.5	-	-	-	-	-	-	-	-
70135	Mummell (Kangaroobie)	40	133.6	-	-	-	-	-	-	-	-
70137	Gurrundah (Wandonga)	52	112	70	71.2	18.5	5.9	18.5	-	-	-
70143	Brayton (Longreach)	68	51	70	69.5	10.5	9.5	8.5	73	46	7.5
70147	Goulburn (Hillwood)	56.4	39	74.4	69.8	55.4	12.4	11	81.2	34.6	15.4
70213	Gurrundah (Ashwell)	-	-	-	-	-	-	-	72	23	9.6
70263	Goulburn Tafe	30.4	90	57.6	67.6	15.6	6.4	18.2	94.8	17	10.6
70269	Marulan (Johnniefelds)	37.6	86	-	-	-	-	-	80.4	52	12.2
70290	Collector (Winderadeen)	30.4	18	52	68.6	43	1.8	35.6	38	17.6	6.6
70325	Wollondilly (River View)	2	14.8	65.6	64.4	22	8.2	9.2	35.4	42.6	21

2.4.1.3. Rainfall Data Merge

Rainfall data mentioned in Sections 2.4.1.1 and 2.4.1.2 was used to create rainfall data sets with 30 minute temporal resolution for input into the hydrologic model. The catchment weighted average rainfall depth was determined from the spatial rainfall patterns mentioned in Sections 2.4.1.2 and this depth was applied to the temporal patterns using the methods described in Sections 2.4.1.1. Figure 13 presents rainfall hyetographs for the December 2010, March 2012 and June 2013 historic events using the average rainfall depth and average temporal patterns across the Goulburn catchment. The Figure 13 hyetographs are for display purposes only and in the hydrologic model, each sub-catchment has been assigned its own unique rainfall depth and associated hyetograph depending on the rainfall depth and temporal pattern determined in Section 2.4.1.1 and 2.4.1.2. This allows for spatio-temporal variation of rainfall across the catchment.

2.4.2. Design Rainfall Data

Design rainfall data is an important input parameter into a hydrologic model to determine design flows. The design rainfall depths are used in conjunction with design rainfall temporal patterns to create design storms. In current practise, design rainfalls are based on Australian Rainfall and Runoff 1987 (ARR87) design rainfall data. However this data is in the process of being revised with new Intensity-Frequency-Duration (IFD) relationships available as part of the ARR revision (ARR2013). ARR87 IFD data has been used for the current study.

2.4.2.1. Design Rainfall Data

ARR87 design rainfall for the region was obtained from the Bureau of Meteorology (BoM) and spatial variation in design rainfall has been accounted for in the current study. Temporal patterns (ARR87) are for Zone I and were obtained from Australian Rainfall and Runoff (Reference 3).

Table 9 presents the ARR87 design rainfall depths for the Wollondilly and Mulwaree catchment's critical durations (36 and 48 hours respectively, see Section 5.8).

Table 9: ARR1987 Design Rainfall – Wollondilly and Mulwaree catchments – Critical Duration

Event	Wollondilly Catchment (36 hour)		Mulwaree Catchment (48 hour)	
	Average (mm)	Max (mm)	Average (mm)	Max (mm)
0.2 EY	95	102	107	110
10% AEP	106	116	121	126
5% AEP	122	135	140	147
2% AEP	142	160	166	175
1% AEP	158	180	186	197

2.4.2.2. Probable Maximum Precipitation

The Wollondilly and Mulwaree catchments upstream of Goulburn have catchment areas of less than 1,000 km^2. PMP depth calculation for these catchments is therefore calculated by the Generalised Short Duration Method (GSDM) (Reference 4). The PMP for both catchments simultaneously was determined by the Generalised Southeast Australia Method (GSAM) (Reference 5) as the catchment area upstream of the confluence of these two rivers exceeds 1,000 km^2. Figure 14 and Figure 15 displays the PMP spatial rainfall distribution and the rainfall depths allocated to each GSDM ellipsoid for the Wollondilly and Mulwaree Catchments respectively for the critical duration of 6 hours (see Section 5.8).

2.5. Stream Gauge Data

Table 10 details all stream gauges in the Wollondilly and Mulwaree catchments upstream of the Murrays Flat Gauge. Of these gauges, only four are suitable for model calibration/validation. Gauges used in model calibration/validation are displayed as black text in Table 10 with the locations presented in Figure 1. Flood levels, flows, rating curves, cross-sections and other details for these stream gauges were obtained from Council and the SCA.

Further details on the gauges used in model calibration/validation are presented in Section 2.5.1.

Table 10: Stream Gauges in the Goulburn Area

Site Number	Name	Available Record Period			Owner of Pluviometer
		Start	End	Years	
212012	Goulburn*	01/01/1960	01/12/1977		NoW
212047	Cardross*	19/08/1985	05/08/1997		NoW
21210060	Whiteheads Creek (G1)**	11/05/1987	-		NoW
21210061	Whiteheads Creek (G2)**	09/02/1987	-		NoW
21210062	Whiteheads Creek (G3)**	11/05/1987	-		NoW
21210063	Whiteheads Creek (G4)**	11/05/1987	-		NoW
21210064	Whiteheads Creek (G5)**	16/08/1988	-		NoW
21210065	Whiteheads Creek (G6)**	18/08/1988	-		NoW
2122711	**Murrays Flat**	**16/08/1990**	**Ongoing**	**25**	**SCA**
2122725	**The Towers**	**06/06/1990**	**Ongoing**	**25**	**SCA**
2122712	Rossi Weir*	01/01/1991	01/01/1994		Sydney Water
570027	Murrays Flat (Wollondilly River)*	05/08/1999	-		BOM
70320	Towrang Bridge (Wollondilly River)*	01/01/1985	-		BOM
212027	DS Pejar Dam*	01/01/1973	14/12/1982		NoW
	Sooley**	01/08/2002	Ongoing		Council
	Sooley Dam**	01/01/2006	Ongoing		Council
	Cardross******	**07/08/2002**	**Ongoing**	**13**	**Council**
	Pejar Creek**	01/08/2002	Ongoing		Council
	Pejar Dam**	01/08/2002	Ongoing		Council
	Bumana**	15/08/2002	Ongoing		Council
	Rossiville Weir***	**01/08/2002**	**Ongoing**	**13**	**Council**
	Wollondilly River US of Pejar Dam**	01/08/2002	Ongoing		Council
570068	Pejar Dam*	02/2016	Ongoing		Council
570069	Sooley Dam*	02/2016	Ongoing		Council
570070	Marsden Weir*	02/2016	Ongoing		Council
570066	Landsdowne Bridge*	02/2016	Ongoing		Council
570067	Inveralochy Bridge*	02/2016	Ongoing		Council

* Not used in calibration as data is not available for relevant storm events
** Not used in calibration due to its small upstream catchment / minor tributary stream gauge
*** Only used in the hydraulic model calibration as no gaugings or ratings are available.
**** Only used in hydrologic model calibration as gauge is situated outside of the hydraulic model domain.

2.5.1. Available Stream Gauges

Despite the vast amount of stream gauge and rainfall data available in the Goulburn catchment, there are a number of factors which determine the suitability of a stream gauge for model calibration. For example, a stream gauge's period of record must coincide with a significant flood event suitable for model calibration and typically the period of record of a nearby pluviometer to ensure that a storm event's temporal pattern is available. In addition, stream gauges on tributaries with small catchments upstream of Goulburn are generally not useful for mainstream model calibration. Finally, a rating curve derived from flow gaugings is crucial to determine reliable flow estimates and flood hydrographs. Taking the above into account it was found that three gauges in the catchment were suitable for use in hydrologic model calibration. These gauges are:

- Cardross (Wollondilly River 11 km upstream from Goulburn);
- The Towers (Mulwaree 4.5 km upstream from Goulburn); and
- Murrays Flat (7 km downstream of the Wollondilly/Mulwaree Rivers Confluence).

Ratings curves for the above listed gauges have been verified using the hydraulic model.

For hydraulic model calibration/validation, The Towers and Murrays Flat gauges (mentioned above), are situated in the hydraulic model domain along with the Rossiville Weir Gauge. The Rossiville Weir Gauge has not been gauged and has no available rating curve and is therefore not suitable for hydrologic model calibration. Time-varying stage hydrographs provided by these gauges have been used to calibrate/validate the hydraulic model.

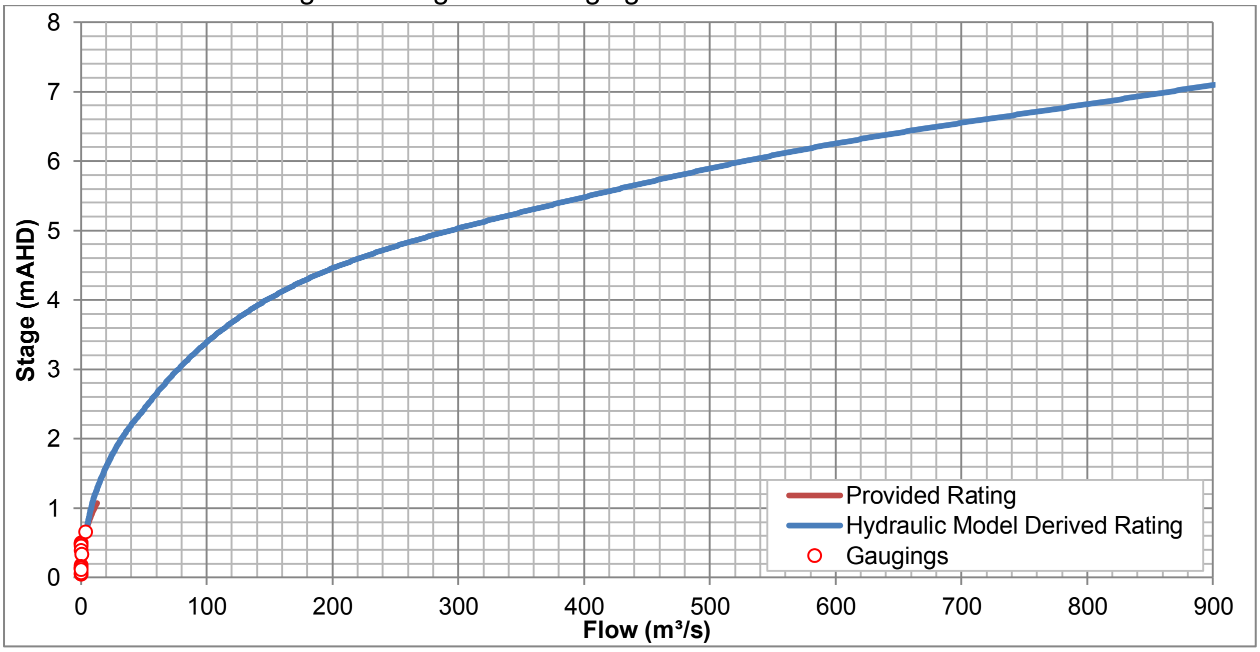
2.5.2. Rating Analysis

2.5.2.1. Cardross Gauge Rating

Flow gaugings have been undertaken at the Cardross gauge since its installation in 2002. However, the maximum gauging was performed at a stage of 0.657 m whilst the largest flood within the available record period (December 2010) achieved a peak stage of 6.9 m. Accordingly, the available rating curve for this gauge only provides flows for a maximum stage of 1 m and is therefore unsuitable for determining flood event flows.

A hydraulic model at the Cardross gauge was established to extrapolate the rating for high flows. This model was created separate to the main hydraulic model described in Section 6 due to the distance upstream. The model was calibrated to match available gaugings. Chart 1 presents the provided rating (red), the model derived rating (blue) and available gaugings. It can be seen that only low flow gaugings are available. As such the accuracy of the model derived rating is unable to be verified.

Chart 1: Cardross Gauge – Ratings and Gaugings

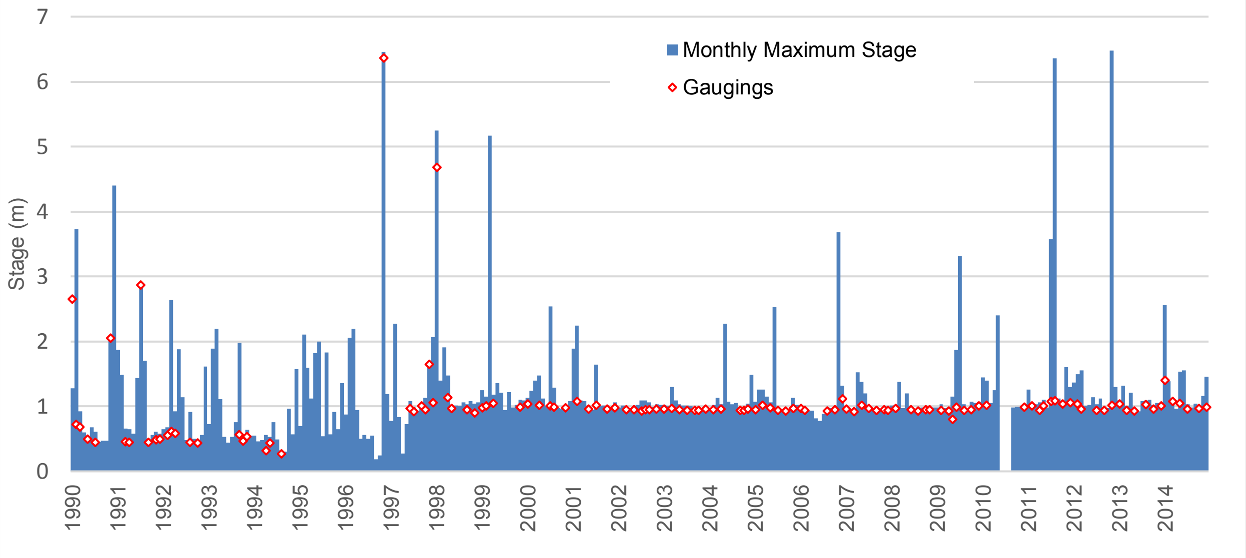


Gauge Zero = 639.83 mAHD

2.5.2.2. Murrays Flat Gauge Rating

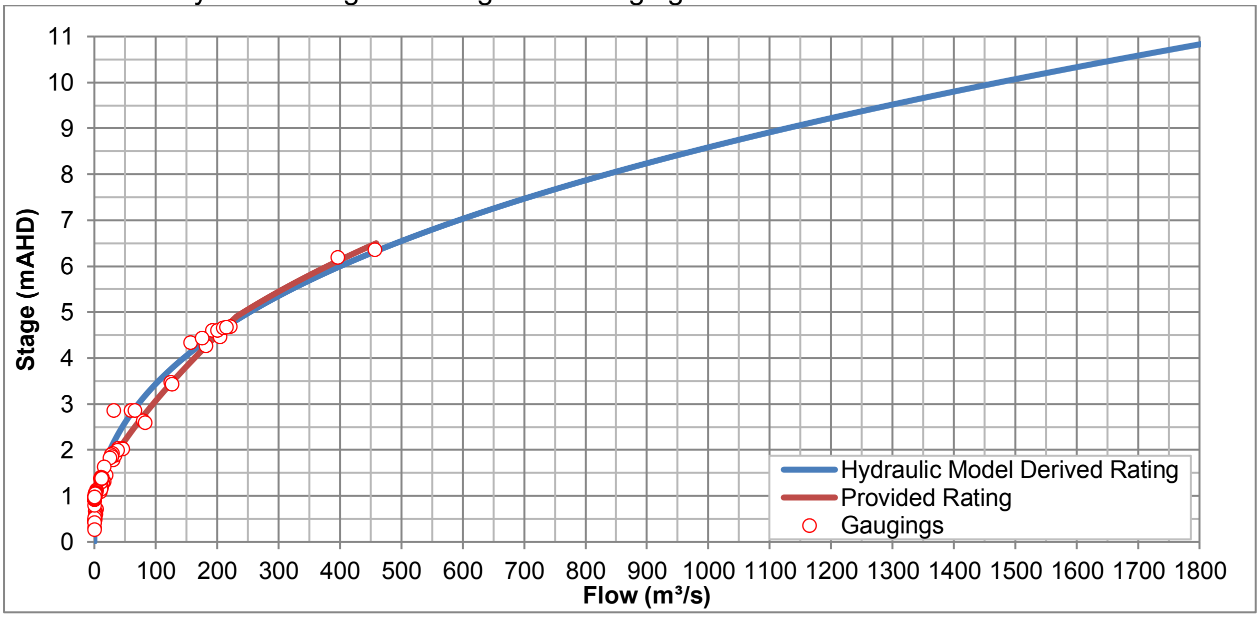
Flow gaugings have been undertaken at the Murrays Flat gauge since its installation in 1990. Chart 2 displays the maximum stage recorded each month for the gauge's period of record (16/08/1990 to 24/07/2015). The Murrays Flat gauge has a gauging taken during a flood event in June 1997, however the gauge failed during the largest flood event in this period, the December 2010 flood.

Chart 2: Monthly Maximum Stage and Gaugings Level



The rating in the hydraulic model at the Murrays Flat gauge was extended above the maximum gauged flow of 450 m^3/s. Chart 3 displays a comparison of the hydraulic model derived rating and the provided rating. The Murrays Flat gauge was included in the Goulburn hydraulic model domain and the stage hydrograph at the gauge was used in the model calibration (see Section 7.1.1).

Chart 3: Murrays Flat Gauge – Ratings and Gaugings



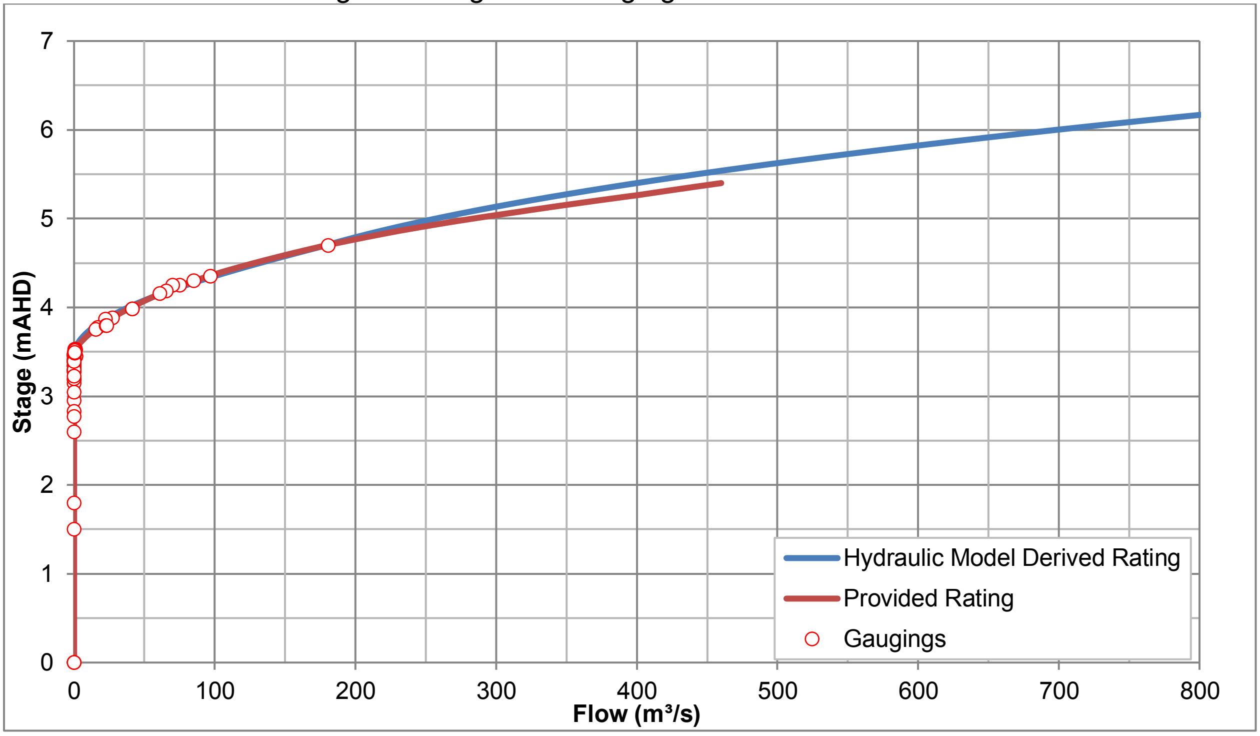
Gauge Zero = 615.47 mAHD

2.5.2.3. The Towers Gauge Rating

On the Mulwaree River, The Towers Gauge has had flow gaugings undertaken periodically since 1990.

The maximum recorded gauging was performed at a stage of 4.7 m and a flow of 180 m^3/s. The hydraulic model was used to verify and extrapolated the rating curve beyond the maximum gauging. The model was calibrated to match flow gaugings to verify the accuracy of the rating for flows greater than the maximum gauged flow.

Chart 4: The Towers Gauge – Ratings and Gaugings



2.5.3. Annual Series Data

Table 10 presents the available record period for the three gauges suitable for hydrology (see Section 2.5.1). Of note are the relatively short and incomplete record periods. A summary of the annual series for each gauge is presented in the following sections.

It should be noted that the partial series for the Marsden Weir Gauge obtained from Reference 1 Flood Study is presented in Section 1.3.1.

2.5.3.1. Murrays Flat Annual Series

The Murrays Flat gauge has 24 years of available record with a notable gauge failure during the 2010 flood event and no data available for the 1990 flood event which occurred prior to installation of the gauge. All flows have been determined based on the model derived rating (see Section 2.5.2.2) using the annual peak stages provided by the Sydney Catchment Authority with the exception of the 2010 event flow which has been estimated from the hydrologic model calibration (see Section 5.6).

The 1990 flood event is presumably the largest to occur at the Murrays Flat Gauge since 1990 based on Golden Valley Gauge records downstream. Both the 1990 and 2010 events are substantially larger than the next largest events that occurred in 1997, 2012 and 2013 (ranging between 465 – 485 m³/s).

Table 11: Murrays Flat Gauge Annual Maximum Flow

Year	Stage (m)	Flow (m³/s)	Year	Stage (m)	Flow (m³/s)	Year	Stage (m)	Flow (m³/s)
1990	-	-	**1999**	5.17	275	**2008**	1.37	8
1991	4.40	184	**2000**	1.48	10	**2009**	1.15	3
1992	2.86	64	**2001**	2.54	47	**2010***	~10.0*	~1,465*
1993	2.20	33	**2002**	1.64	15	**2011**	1.26	5
1994	1.98	25	**2003**	1.30	6	**2012**	6.36	465
1995	2.10	29	**2004**	2.28	36	**2013**	6.47	485
1996	2.20	33	**2005**	1.48	10	**2014**	2.55	48
1997	6.45	482	**2006**	2.53	47			
1998	5.25	286	**2007**	3.69	119			

* 2010 event flow estimated from the calibrated hydrologic model. Stage derived from the rating described in Section 2.5.2.2.

2.5.3.2. The Towers Annual Series

The Towers gauge has 26 years of available record. All flows have been determined based on the model derived rating (see Section 2.5.2.3) using the annual peak stages provided by the Sydney Catchment Authority.

The 1990 flood event is the largest on record at The Towers Gauge achieving a peak flow of 383 m³/s. The associated gauge level of the 1990 event was recorded at 2.23 m, however this event occurred prior to construction of the weir downstream (see Section 2.2.4). Using the model derived rating, the 1990 flood event would have achieved a peak stage of 5.36 m at the gauge under existing conditions. Other notable events occurred in 1991 (254 m³/s) and 2010 (3.50 m, 194 m³/s).

Table 12: The Towers Gauge Annual Maximum Flow

Year	Stage (m)	Flow (m³/s)	Year	Stage (m)	Flow (m³/s)	Year	Stage (m)	Flow (m³/s)
1990*	2.23	383	**1999**	3.52	23	**2008**	3.49	0
1991*	3.35	254	**2000**	3.54	10	**2009**	2.93	0
1992	3.57	23	**2001**	3.54	39	**2010**	3.50	194
1993	3.52	18	**2002**	3.49	13	**2011**	3.55	3
1994	3.50	19	**2003**	3.51	3	**2012**	4.35	104
1995	3.50	6	**2004**	3.30	0	**2013**	3.56	89
1996	3.78	25	**2005**	3.49	13	**2014**	3.53	7
1997	3.58	80	**2006**	3.49	18	**2015**	3.51	1
1998	3.55	88	**2007**	3.56	39			

*Prior to construction of the weir downstream of the gauge (see Section 2.2.4).

2.6. Peak Flood Level Marks

Peak flood level marks have been obtained from various sources and have been used for model calibration/validation. The following sections provide information on available peak flood levels for various events.

2.6.1. Surveyed Peak Flood Levels

Southern Cross Consulting Services (SCC) were commissioned to survey a set of peak flood level marks for historic flood events for use in hydraulic model calibration (see Section 7.1.1).

The peak flood levels used in model calibration were obtained via the community consultation process (Section 3). Questionnaire responses (Section 3.1) and reports during the community information session (Section 3.2) were examined and a list of people who had witnessed flooding was compiled. WMAwater engineers met with 23 local residents to obtain 39 peak flood level marks from several flood events in Goulburn. SCC was then commissioned to survey the observed peak flood levels to mAHD for use in model calibration. Details of the floodmarks are presented below in Table 13 and the locations of those floodmarks used for calibration are displayed in Figure 1.

The December 2010, March 2012 and June 2013 flood events were used for model calibration as the survey points were well spread throughout Goulburn and these events had suitable rainfall and flow data. Peak flood level marks were also obtained for the 1974 and 1990 flood events, however pluviometer rainfall data (see Section 2.4.1.1) and Goulburn Stream Gauge data (see Section 2.5.3) was not available for calibration of these events.

Table 13 provides an indication of the accuracy of the surveyed floodmarks and provides comments. It should be noted that the most reliable method of determining peak flood level is generally from eye witness accounts of the maximum height of flooding on a fixed manmade object such as a home, shed or fence post. Estimates of flood extent where the witness indicates how far the flood encroached on their land are generally less accurate, particularly if no fixed objects are close by. In these cases peak flood levels were often compared to the model results as flood extent indicators. The problems with determining peak flood levels are often not able to be identified with any certainty until the floodmarks have been surveyed and comparisons to ground levels in mAHD and estimated peak flood extents have been reviewed.

Peak flood levels that are assessed as having a ‘Poor’ accuracy rating, as presented in Table 13, have either been used to examine flood extents or not been used in model calibration.

Table 13: Peak Flood Level Survey Marks for Calibration (continued over page)

Id	X	Y	Surveyed Peak Flood Level	Storm Event*	Accuracy**	Indicator Type	Comment
1	747953	6152909	633.87	2010	Good	Flood depth	Depth matched (within 0.1 m)
2	747974	6152920	631.28	2012	Poor	Flood extent	Flood extent matched
3	747967	6152873	633.51	2010	Poor	Flood extent	Flood extent matched
4	748000	6152890	631.14	2012	Poor	Flood extent	Flood extent matched
5	748002	6152909	630.11	2013	Poor	Flood depth	Flood extent matched
6	747995	6152837	633.82	2010	Good	Flood depth	Depth matched (within 0.1 m)
7	747991	6152845	634	2010	Good	Flood depth	Depth matched (within 0.1 m)
8	749452	6152229	631.42	2010	Good	Flood depth	Depth matched (within 0.1 m)
9	749429	6152230	631.69	1990*	Average	Flood depth	Not modelled
10	749453	6152238	631.2	2012	Poor	Flood extent	Overland flow flood mark
11	749442	6152246	631.26	2010	Good	Flood depth	Depth matched (within 0.1 m)
12	749472	6152374	631.28	2010	Good	Flood depth	Depth matched (within 0.1 m)
13	749456	6152384	631	1990*	Good	Flood depth	Not modelled
14	749482	6152442	631.36	2010	Good	Flood depth	Depth matched (within 0.1 m)
15	749484	6152449	634.78	1974*	Poor	Flood depth	Not modelled
16	749453	6152298	631.28	2010	Good	Flood depth	Depth matched (within 0.1 m)
17	749457	6152288	631.26	2010	Good	Flood depth	Depth matched (within 0.1 m)
18	749464	6152293	630.54	2012	Poor	Flood extent	Overland flow flood mark
19	749491	6152304	631.17	2010	Good	Flood depth	Depth matched (within 0.1 m)
20	749539	6152301	631.29	2010	Good	Flood depth	Depth matched (within 0.1 m)
21	749566	6152292	631.26	2010	Good	Flood depth	Depth matched (within 0.1 m)
22	749628	6152274	631.88	2010	Poor	Flood depth	Depth not matched – surrounding flood marks matched
23	749588	6152247	631.54	2010	Poor	Flood depth	Depth matched (within 0.2 m)
24	749590	6152223	631.67	2010	Poor	Flood depth	Depth not matched – surrounding flood marks matched
26	749679	6152422	631.26	2010	Good	Flood depth	Depth matched (within 0.1 m)
27	749679	6149849	630.45	2010	Poor	Flood depth	Depth not matched – surrounding floodmarks matched
28	749891	6150242	629.41	2010	Good	Flood depth	Depth matched (within 0.1 m)
29	749843	6150247	627.71	2012	Poor	Flood extent	Overland flow flood mark
30	749893	6150264	630.51	2010	Poor	Flood depth	Depth not matched – surrounding floodmarks matched
31	749887	6150274	629.57	2010	Average	Flood depth	Depth matched (within 0.2 m)
32	749802	6150308	629.36	2010	Good	Flood depth	Depth matched (within 0.1 m)

Id	X	Y	Surveyed Peak Flood Level	Storm Event*	Accuracy**	Indicator Type	Comment
33	749802	6150308	629.52	1990*	Good	Flood depth	Event not modelled
34	749802	6150308	630.32	1974*	Good	Flood depth	Event not modelled
35	749110	6150037	629.34	2010	Good	Flood depth	Depth matched (within 0.1 m)
36	749999	6150157	629.32	2010	Good	Flood depth	Depth matched (within 0.1 m)
37	749432	6150030	628.11	2012	Good	Flood depth	Depth matched (within 0.1 m)
38	749433	6149598	628.58	2012	Good	Flood extent	Extent matched
39	749451	6149599	629.06	2010	Average	Flood depth	Depth matched (within 0.3 m)

*Note: due to insufficient pluviometer and flow data, flood levels from the 1974 and 1990 events have not been calibrated to.
**Note: the accuracy of the peak flood level marks has been estimated from witness statements.

2.6.1.1. Floodmark 5 Comments

Floodmark 5 was the only flood depth obtained for the June 2013 event. While this flood mark was not matched, a good match was achieved at the Rossi weir gauge a short distance upstream. This flood mark does provide a good indication of flood extent which has been matched in the hydraulic model.

2.6.1.2. Floodmarks 10, 18 and 29 Comments

During the process of collecting floodmarks, it was observed that occasionally residents had identified areas affected by overland flow flooding rather than flooding from the Wollondilly or Mulwaree Rivers. Photographs taken from a helicopter during these events indicated that the areas where these flood marks are situated were not flood affected by riverine flooding (the focus of the current study). Therefore these observed flood marks are likely due to observations of overland flow flooding.

2.6.1.3. Floodmarks 22 and 24 Comments

In the hydraulic model calibration for the December 2010 event, ten of the 13 flood marks in the Avoca street area were matched to within 0.1 m accuracy. Floodmarks 22 and 24 were not matched however are surrounded by accurately matched flood levels. Therefore, it was concluded that these floodmarks do not accurately reflected flood levels in this area.

2.6.1.4. Floodmark 27 Comments

Floodmark 27 was recorded as a 2010 event peak flood level on the Mulwaree River. This flood mark was over 1 m higher than other surrounding flood marks for the same event. Accordingly, it was concluded that this flood mark does not accurately reflect true flood levels in this area.

2.6.2. Southeast Study

A report relating to the magnitude of the 2010 flood event at Goulburn, undertaken by Southeast Engineering and Environmental, provided a number of peak flood levels that were used for model calibration. Table 14 provides these flood level marks and comments made as part of the Southeast study.

Table 14: Southeast Study – Peak Flood Levels

Address	Level (mAHD)	Details from Surveyor
Marsden Weir / Fitzroy St Bridge	634.15	Estimated HWL Dec 2010 – RH Bank 50 m, upstream of Bridge over Wollondilly River – Fitzroy Street – See NSW SES Photo
Goulburn Golf Club	629.35	HWL Dec 2010, mark on steel column of verandah
Cnr Kenmore and Derwent	631.2	HWL Dec 2010, debris shown in NSW SES Photo
79 Fitzroy Street	633.72	HWL Dec 2010 – As advised by Owner – 100 m downstream of Fitzroy St Bridge, top of rear Colourbond fence – See NSW SES Photo
54 Avoca Street	631.23	HWL Dec 2010, see NSW SES Photo
58 Avoca Street	631.27	HWL Dec 2010, see NSW SES Photo
20 Bellevue Street	631.11	HWL Dec 2010, as advised by Owner

2.6.3. 1986 Goulburn Flood Study

The 1986 Goulburn Flood Study provides numerous peak flood level marks for various events. The study was undertaken prior to 2010 and therefore peak flood level marks suitable for model calibration have not been obtained. However, historic event peak flood level marks have been examined and compared to the current study design results as a means for verifying the 1% AEP peak flood levels. This comparison is presented in Section 7.1.3.

3. COMMUNITY CONSULTATION

Community consultation is an important element of the Flood Study ultimately facilitating community engagement and acceptance of the overall project. Consultation work was undertaken to assess the flood experience of the community and gather additional data.

3.1. Questionnaire Distribution

A community questionnaire survey was undertaken during August 2015. 300 surveys were distributed to residents near flood affected areas in the study area and a total of 46 responses were received (see Figure 16). This equates to a return rate of 23% which is high compared to other Flood Studies in rural NSW. However it must be noted that these were targeted questionnaires aimed at people likely to have been flood affected and accordingly the views expressed by this sample may not accurately reflect that of the total population. A summary of the questionnaire results is presented in Figure 17a – d.

The majority (93%) of respondents were from residential dwellings with one respondent noted as business and another two respondents as ‘other’ which was generally undeveloped land or farmland (see Figure 17a).

The majority of respondents have lived in the region for more than five years and would have therefore experienced the 2010, 2012 and 2013 flood events. 57% of respondents have lived in the area longer than 30 years. This indicates that flood awareness of respondents (and likely the general community) should be relatively high (see Figure 17b).

85% of respondents were ‘very aware’ of flooding from the Wollondilly River at Goulburn and 76% were ‘very aware’ of flooding from the Mulwaree River at Goulburn (see Figure 17c).

Six respondents noted that they had been flooded above floor level in the past and another eleven respondents had been flooded in their yard (see Figure 17d).

Roads that were notably affected by flooding include:

- Avoca Street;
- Hercules Street;
- Bellevue Street; and
- Kenmore Street.

A copy of the distributed Community Consultation Newsletter and Questionnaire is contained in Appendix B.

3.2. Community Workshop

A Community Workshop was held for the general public and was advertised via mailed community newsletters, local newspapers and Council’s website.

The workshop was held in Goulburn Civic Centre on the 21st October 2015. Approximately 15 people attended the meeting which was aimed to both provide information about the study and obtain information on their flood experienced.

A number of attendees did have valuable information about flood behaviour on both rivers, including peak flood level marks.

3.3. Community Follow-up Meetings

For a three day period immediately after the community workshop described above, WMAwater engineers visited local residents for one on one follow-up meetings. Approximately 25 residents were visited with a large number amount of valuable flood related information provided. In particular, 39 peak flood level marks were obtained and surveyed for model calibration (see Section 2.6).

3.4. Public Exhibition of the Draft Final Report

The Goulburn Flood Study Draft Final report was on public exhibition for a period of 4 weeks in July and August 2016. Hard copies of the report were available at the Customer Service desk in the Civic Centre. The report was also available online on Council's website during this period.

4. MODELLING APPROACH

In order to accurately model flood behaviour of the Wollondilly and Mulwaree Rivers at Goulburn, the development of hydrologic and hydraulic models was required. The overall modelling approach was to establish a hydrologic model in conjunction with a 1D/2D hydraulic model (see Diagram 1). The hydrologic model is used to generate flow hydrographs for input to the hydraulic model. The 1D/2D hydraulic model then utilises flows from the hydrologic model to calculate flood levels and velocities in the region. The mainstream hydrologic model used was the Watershed Bounded Network Model (WBNM). The hydraulic model used was TUFLOW, a 1D/2D fully dynamic fixed grid based model.

The hydrologic model was calibrated to the December 2010 event (see Section 5.5) and then verified using the March 2012 and June 2013 flood events (see Section 5.5.2) to flows recorded at the four stream gauges described in Section 2.5.1. Additional verification of the hydrologic model was then undertaken by comparing hydrologic model derived design flows to Flood Frequency Analysis (FFA) undertaken at the Marsden Weir, The Towers and Murrays Flat gauges (see Section 5.3), noting that the available record period for these gauges is short and therefore not ideal for determining design flows.

Flows from the historic events modelled in WBNM were then input into the TUFLOW hydraulic model which was calibrated to the December 2010 event and validated with the March 2012 and June 2013 events (see Section 7.1.1).

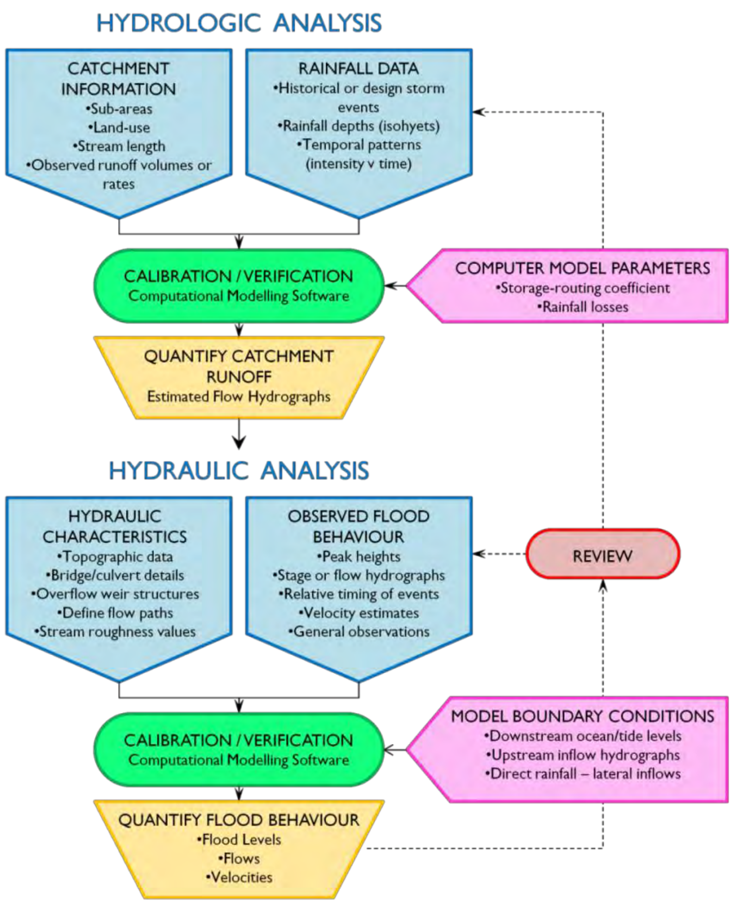


Diagram 1: Flood Study Process

5. HYDROLOGY

5.1. Background

The key purpose of this study is to define design flood behaviour for the Study Area described in Section 1.2 (see Figure 1). To achieve this goal the development of design flows (described in the ensuing sections) for input into a 1D/2D hydraulic model (see Section 6.2.5.1) was required.

5.2. Introduction

There are two basic approaches to undertaking design flood analysis:
- The rainfall runoff routing approach (hydrologic modelling); and
- The flood frequency approach (also called FFA).

Both approaches have advantages and disadvantages however for the current study the balance was very much in favour of using the rainfall/runoff routing approach for the Goulburn catchment. For a FFA approach, a nearby stream gauge must have an adequate length and quality of observed record and accuracy of the rating curve (see Section 2.5.3). As described below, the study area does not meet this criteria.

The Goulburn catchment does have some stream gauge data at Marsden Weir, Murrays Flat and The Towers (see Section 2.5.3). However the length of record at the Murrays Flat and The Towers is insufficient to have significant confidence in the quality of the 1% AEP flow estimates. The accuracy of the Marsden Weir flow estimates is unknown, again reducing confidence in the 1% AEP flow estimate derived from FFA.

Instead of FFA, a hydrologic model (see Section 5.4) has been used to determine flows for input into the hydraulic model. The model was calibrated to the December 2010 flood event (see Section 5.6), validated using the 2012 and 2013 historic events (see Section 5.5), and then verified to FFA (see Section 5.6) to provide context for the design flow estimates,.

These analyses constitute the hydrological analysis component of the study and aim to describe the probability of a given discharge occurring in the Study Area. Calculated design flows (as time varying hydrographs) are then input into the hydraulic model so that design flood levels, extents and hazard can be determined.

5.3. Flood Frequency Analysis

5.3.1. Overview

FFA uses the record of past flooding at a site to determine design event discharge. By fitting a probability distribution to a series of historical floods, the AEP of a given discharge can be determined. The two principles underlying the analysis are that previous floods will re-occur with the same frequency in the future and that the flood record is an accurate representation of the general flooding behaviour, i.e. of adequate sample size. As mentioned in Section 2.5.3, the annual series length is generally short and accordingly, hydrologic modelling is the preferred method of determining design flows. However, FFA has been used to verify design flows from the hydrologic model (Section 5.6) and to inform design initial losses (see Section 5.7.2).

The FFA undertaken as part of this study uses the annual series data presented in Section 2.5.3 and follows methods prescribed by Australian Rainfall & Runoff (AR&R). To compensate for the short record periods and improve design flow estimates, the Australian Rainfall and Runoff Revision, Project 5, Regional Flood Methods (P5) (Reference 6) covariants have been incorporated in the FFA. The P5 covariants have been developed for regional flood frequency estimation (RFFE) which provides design flow estimates for catchments. The covariants include regional estimates of statistical flow parameters such as mean, standard deviation and skew which when incorporated into the FFA for a gauge with a relatively short record period provides higher quality design flow estimates.

Bayesian analysis was undertaken, which consisted of fitting a probability distribution to a series of annual peak discharges. This method is recommended by AR&R and avoids the issues associated with using peak flood levels, which can be strongly influenced by changes to the floodplain.

The analysis was made up of two stages: constructing a time series of flood events at each gauge and applying a probability distribution to this time series. The first stage involved determining what data was available for analysis and what is the appropriate data for the FFA (this is covered in Sections 2.5.3 and 5.3.2) and the second stage involves fitting a probability distribution to the data set to determine design flows (see Section 5.3.3).

5.3.2. Adopted Data Set

FFA has been performed on the highest recorded value of discharge for each year of available record at the gauges described in Section 2.5.3, as well for the incomplete annual series at the Marsden Weir gauge described in Section 1.3.1.

Censored data has been included in a number of annual series to incorporate events of unknown flow or to extend the period of record where possible. The following assumptions in regards to censored data have been made:

- **Marsden Weir –** All events less than 600 m³/s have been included as censored data. To extend the record period an assumption of event magnitude for years of missing data had to be made. Due to the large gaps in the record period prior to 1959 and post 1977, the presence of significant flood events could not be ruled out. However, it was considered unlikely any events exceeding 600 m³/s would have been experienced without alternative records of the event occurring being located. Accordingly, 600 m³/s was set as the threshold for censored events.
- **Murrays Flat –** The 2010 and 1990 flood events were included as censored events with flows exceeding 1,465 m³/s (the 2010 flow calculated in the hydraulic model). Based on the Golden Valley gauge situated downstream and other anecdotal information the 1990 flood event was larger than the 2010 event downstream of the Wollondilly and Mulwaree rivers confluence.

- **The Towers –** No censored data was included in FFA.

The regional P5 Log-Pearson III covariants that were input into the FLIKE software are presented in Table 15 to Table 16.

Table 15: Project 5 Regional Log-Pearson III Covariants – Murrays Flat Gauge Catchment

Parameter	Mean	Standard Deviation	Correlation	
Mean of Log Flow	5.323	0.648	1	
Standard Deviation of Log Flow	2	0.89	0.167	-0.33
Skew of Log Flow	3	0.098	0.027	0.17

Table 16: Project 5 Regional Log-Pearson III Covariants – The Towers Gauge Catchment

Parameter	Mean	Standard Deviation	Correlation	
Mean of Log Flow	5.821	0.428	1	
Standard Deviation of Log Flow	2	0.881	0.138	-0.33
Skew of Log Flow	3	0.092	0.026	0.17

5.3.3. Probability Distribution

A Bayesian maximum likelihood approach was used to fit a specified probability distribution to the annual maximum series. Two probability distributions were used; the Log-Pearson III (LP3), which is commonly used in FFA, and the Generalized Extreme Value (GEV) distribution, which is a more recently developed family of probability distributions that combine the Gumbel, Frechet and Weibull families of distributions. These distributions were used in combination with P5 covariant data. It was found that the LP3 distribution fitted the data better than the GEV distribution and as such was used in preference. Flike (file version 5) was used to apply the Bayesian maximum likelihood approach.

5.3.4. FFA Flow Results

Figure 18 to Figure 20 present the frequency plots for the Marsden Weir, The Towers and Murrays Flat Stream Gauges respectively along with the hydrologic model derived design flows (see Section 5.7.4). The FFA derived design flows are presented in Table 17.

Table 17: FFA Design Flow Estimates

Event	Flow (m^3/s)		
	Marsden Weir	The Towers	Murrays Flat
0.2 EY	345	90	230
10%	450	160	450
5%	580	270	790
2%	790	410	1,510
1%	1010	560	2,330

As mentioned the available record period is insufficient to have significant confidence in the quality of design flow estimates. The available data period and selection of censored events impacts on which FFA derived design flows are most accurate. The width of the 90% confidence limits displayed in Figure 18 to Figure 20 also indicates estimated accuracy of design flows. A summary of the accuracy of FFA derived design flows is presented below:

- **Marsden Weir –** 144 years of record has been incorporated into the analysis providing reasonable flow estimates for the 1% and 2% AEP events. However, the low flows (less than 600 m³/s) have been included as censored data and accordingly FFA design flow estimates for events from the 5% AEP and smaller are not reliable.
- **The Towers –** 26 years of record has been used in the analysis with no censored data included. FFA design flow estimates are more reliable for the smaller events (0.2EY – 5%) as it can be assumed that the gauge has recorded a number of events of this magnitude.
- **Murrays Flat –** 25 year of record has been used in the analysis with two censored events for the large 1990 and 2010 floods. FFA design flow estimates are more reliable for the smaller events (0.2EY – 5%).

Acknowledging that there are issues with the FFA (related to a lack of available data period), hydrologic model design flows have been compared to the FFA results as a means of verification of the hydrologic model (see Section 5.6).

5.4. Hydrologic Model

For the current study, hydrologic modelling of the Wollondilly and Mulwaree Rivers at Goulburn was undertaken using WBNM. WBNM is a widely used hydrologic model which has been substantially tested on Australian catchments.

WBNM has numerous variables that impact on the calculated catchment discharge. This includes input rainfall, rainfall losses (initial and continuing), the WBNM routing parameter 'C' and the non-linearity parameter 'm'. For the current study, input rainfall data for historic events and design rainfalls are described in Sections 2.4.1 and 2.4.2 respectively and model losses are described in Section 5.7.2. The non-linearity parameter 'm' has been set as default (0.77) which is in agreement with ARR guidelines (Reference 3). The routing parameter 'C' has been varied during model calibration to match modelled and observed flow estimates at the Cardross, The Towers and Murrays Flat stream gauges (Section 5.5) for the 2010 flood event. The selected parameters have been validated using the 2012 and 2013 flood events (Section 5.5). Further information on the WBNM routing parameter is contained in Section 5.4.1.

5.4.1. WBNM Routing Parameter 'C'

WBNM uses a routing parameter (also referred to as the 'C' parameter) to calculate the catchment response time for intra-catchment runoff and channel flow. The WBNM routing parameter is important in determining the timing of runoff from a catchment which influences the shape of the hydrograph as well as the catchments channel routing properties that affect routing speed and attenuation. The general relationship is that a decrease in the lag parameter will result in an increase in flood peak discharge (Reference 13) and as such a smaller 'C' value will typically produce shorter lag times and less attenuation.

In catchments for which reliable gauge data is available, the WBNM model should be calibrated against recorded flood data in order to ensure that the adopted routing parameter is representative of the catchment being modelled. This has been undertaken for the current study.

For ungauged catchments Reference 13 recommends a routing parameter value of 1.6. This was determined in studies undertaken on ten catchments in eastern NSW, and an additional 54 catchments across Queensland, NSW, Victoria and South Australia. This is based on the average calculated C parameter from numerous storm events on each of these calibrated catchments. However, variance in the C parameter across these catchments is relatively large with the sample having a minimum C value of 0.7 and maximum of 2.8 (standard deviation of 0.5).

WBNM routing parameter is a function of the catchment's channel and floodplain. Typically, steeper catchments with narrow floodplains, such as the Wollondilly River catchment, have lower 'C' values. As mentioned previously, a lower 'C' parameter will lead to faster flood travel times and less attenuation. The topography of the Mulwaree River catchment, which is significantly flatter with a poorly defined channel and wider floodplain than the Wollondilly River catchment, is typical of a catchment with a higher 'C' parameter.

The selected 'C' parameters determined by model calibration for the two catchments (Wollondilly River and the Mulwaree River) are presented in Section 5.6.

5.4.2. Hydrologic Catchment Delineation

Hydrologic model delineation was determined by interpretation of aerial imagery, ALS and 90 m SRTM data (see Sections 2.2.1 and 2.2.3).

The hydrologic model layout for Goulburn is presented in Figure 2 and summary of the hydrologic catchment properties is displayed in Table 18.

Table 18: Goulburn Hydrology - Catchment Properties

Number of Catchments	Total Area (km^2)	Average Area (km^2)	Minimum Area (km^2)	Maximum Area (km^2)
70	1581	23	4.4	45

5.4.3. Percentage Imperviousness

The model percentage imperviousness was based on inspection of aerial photography for each sub-catchment. The majority of the catchments have a percentage imperviousness of zero as they are predominately natural/rural in nature. The maximum assigned percentage impervious was 40% which accounts for the Goulburn City centre.

5.5. Hydrologic Model Calibration/Validation

To calibrate the hydrologic model, the rainfall for the December 2010 flood event (see Section 2.4.1) was input into the model and model parameters were adjusted to match modelled flows to observed flows at the gauges described in Section 2.5.1. Specifically, the parameters that were adjusted to obtain calibration of the model were:

- Rainfall losses;
    - initial losses; and

  - o continuing losses; and the
- WBNM routing parameter ‘C’.

After the model was calibrated, rainfall for the March 2012 and June 2013 flood events were then input into the calibrated WBNM model (without changing the calibrated model parameters) and the modelled flows were again compared to the observed flow data to validate the hydrologic model. All three events were run without changing the hydrologic model parameters with the exception of the initial losses. It is considered acceptable to change initial losses as these are defined by antecedent rainfall conditions and can vary greatly between events.

The selected routing parameters are presented in Table 19, and Table 20 presents the loss parameters used for the historic events in the hydrologic model. It should be noted that the same routing parameters were used in all three events. The same continuing loss of 1.95 mm/hr has been used throughout the model domain for each of the three events. It must be noted that a continuing loss of 1.95 mm/hr has only been selected as it provided the best match to observed data during the calibration/validation process and that the significance of this number’s precision (i.e. number of decimal places) is not an indication of the accuracy of this value. A continuing loss of 2 mm/hr could just as easily have been selected.

Table 19: Historic Event – Hydrologic Model Routing Parameters

Event	Wollondilly Routing Parameter	Mulwaree Routing Parameter
December 2010	1.2	2.0
March 2012	1.2	2.0
June 2013	1.2	2.0

Table 20: Historic Event – Hydrologic Model Losses Validation

Event	Initial Loss (mm)		Continuing Loss (mm/hr)
	Wollondilly Catchment	Mulwaree Catchment	
December 2010	55	52	1.95
March 2012	75	68	1.95
June 2013	68	66	1.95

Figure 21 presents the results for the hydrologic model calibration to the 2010 event and Figure 22 and Figure 23 presents the results for validation of the hydrologic model to the 2012 and 2013 events respectively. These figures display comparisons of the modelled and observed flow hydrographs for each event at the gauges mentioned in Section 2.5.1.

Table 21 to Table 23 present a summary of the accuracy of the modelled flows compared to the observed flows based on the author’s opinion from examining the peak flow, timing of the peak, total event volume and hydrograph shape.

Table 21: Summary of 2010 Event Hydrologic Model Calibration

Stream Gauge	Peak Flow	Peak Timing	Event Volume	Hydrograph Shape
Cardross	Excellent	Good	Good	Good
The Towers	Excellent	Excellent	Good	Good
Murrays Flat*	-	-	-	-

*Note that the Murrays Flat gauge failed during the 2010 flood event

Table 22: Summary of 2012 Event Hydrologic Model Validation

Stream Gauge	Peak Flow	Peak Timing	Event Volume	Hydrograph Shape
Cardross	Excellent	Good	Good	Good
The Towers*	Good	Poor	Poor	Poor
Murrays Flat	Excellent	Excellent	Average	Average

*Note that the Goulburn (Springfield) pluviometer gauge (see Section 2.4.1.1) was not operational during the 2012 flood event. This is the only pluviometer rainfall gauge available in the Mulwaree Catchment and accordingly model results for this event and catchment are affected by a lack of suitable temporal pattern.

Table 23: Summary of 2013 Event Hydrologic Model Validation

Stream Gauge	Peak Flow	Peak Timing	Event Volume	Hydrograph Shape
Cardross	Excellent	Excellent	Excellent	Good
The Towers	Good	Good	Poor	Poor
Murrays Flat	Average	Average	Good	Good

Table 24 and Table 25 provide a numeric assessment of the hydrologic model's performance at the Cardross and The Towers Stream Gauges respectively. The difference in flood peak in m³/s and as a percentage is presented along with the difference in flood peak timing and event volume.

Table 24: Numeric Summary of Cardross Gauge Model Calibration/Validation Results

Event	Peak Flow (m³/s / %)	Peak Timing (h : mm)	Event Volume (%)
2010	+3 / 0%	+ 1 : 20	+ 3
2012	+1 / 0%	+ 1 : 45	- 30
2013	-2 / -1	- 1 : 00	- 6

Table 25: Numeric Summary of The Towers Gauge Model Calibration/Validation Results

Event	Peak Flow (m³/s / %)	Peak Timing (h : mm)	Event Volume (%)
2010	- 2 / - 1%	- 0 : 25	+ 23
2012*	+19 / +19%	+ 6 : 25	+ 8
2013	-1 / -1 %	- 0 : 35	+ 94

*No pluviometer data in the Mulwaree Catchment for the 2012 Event

Generally the model performed well for all historic events for the four examined criteria, and in particular for peak flow which is the most important aspect for the current study.

5.6. Verification of the Hydrologic Model to FFA

The hydrologic model is designed such that design flows are intended to have the same AEP as the AEP of the selected design rainfall. To ensure this, the hydrologic model has been verified to the FFA results (Section 5.3.4).

The verification was undertaken by comparing FFA results to flows from the calibrated hydrologic model. The hydrologic model flows were derived by applying ARR87 design rainfall depths (Section 2.4.2.1), ARR87 temporal patterns for the derived critical durations (Sections 5.8), design continuing losses (Section 5.7.2) and ARR2013 Aerial Reduction Factors (ARF) for the upstream catchment areas of each gauge (Section 5.7.3).

Initial losses were adjusted (but kept consistent over the entire catchment area) to match hydrologic model flows to FFA results for the smaller events (0.2EY – 5% AEP) at The Towers and Murrays Flat gauges. As previously noted, FFA flow estimates for smaller events for these gauges are reliable.

Figure 18 to Figure 20 present the frequency plots for the Marsden Weir, The Towers and Murrays Flat Stream Gauges respectively along with the hydrologic model derived design flows (see Section 5.7.4). Hydrologic model derived flows are typically a good match to the flows derived by the FFA, acknowledging that there are limitations associated with the available data suitable for FFA. This reinforces the impression of the hydrologic model's suitability for determining design flows for the Wollondilly and Mulwaree Rivers and indicates that a high degree of confidence can be had in hydrologic model calibration/validation.

5.7. Design Flow Hydrologic Modelling

5.7.1. Design Rainfall

Hydrologic modelling has been undertaken with ARR87 design rainfall and ARR87 temporal patterns (see Section 2.4.2) for events ranging between the 0.2EY and the 0.5% AEP. The PMP rainfall discussed in Section 2.4.2.2 has been used to derive the PMF flows.

5.7.2. Design Loss Parameters

ARR 1987 (Reference 3) suggests the losses presented in Table 26 for ungauged NSW catchments.

Table 26: Suggested losses for ungauged NSW catchments (Reference 3)

Location	Initial Loss (IL)	Continuing Loss (CL)
East of Western Slopes	10-35 mm	2.5 mm/h
Arid Zone, mean Annual rainfall <300mm	15 mm/h	4 mm/h

The current study selected design loss parameters are based on the model calibration/validation and verification process. Specifically, the continuing losses were identified in the model calibration/validation process (Section 5.5) and the initial losses were determined in the model verification process (Section 5.6). The determined losses have been applied to design flood modelling of all events with the exception of the PMF.

Table 27 presents the continuing losses used in design event modelling. A continuing loss of 1.95 mm/hr has been selected which is slightly less than the ARR87 recommended loss of 2.5

mm/hr (Table 18). The model calibration/validation indicates that a continuing loss of 1.95 mm/hr is more suitable for the Goulburn catchment (see Section 5.6). As mentioned previously the number's precision (i.e. number of decimal places) is not an indication of the accuracy of this value. A continuing loss of 2 mm/hr could just as easily be selected. PMP rainfall continuing losses are based on Reference 16 and are presented in Table 27. No continuing losses have been applied for the impervious areas.

Table 27: Adopted Design Event Continuing Losses

Event	Pervious Continuous Loss (mm/h)	Impervious Continuous Loss (mm/h)
0.2 EY – 0.5% AEP	1.95	0
PMF	1	0

Initial losses have been determined through the model verification process and are consistent with an initial loss model proposed by Walsh (Reference 15) in which initial losses vary dependant on event AEP. The approach derived design initial losses on a probabilistic basis using streamflow data from gauged catchments in conjunction with design rainfall data. The initial losses determined in the current study are presented in Table 28. For impervious regions of the catchments, an initial loss of 1.5 mm has been assigned to account for ponding.

Table 28: Adopted Design Event Initial Loss Model - Reference 15

ARI (years)	5	10	20	50	100
Initial Loss (mm)	45	45	45	20	20

5.7.3. Aerial Reduction Factors

The aerial reduction factors (ARF) published in ARR87 (Reference 3) are based on American data and have now been superseded by application of the CRC-Forge method developed with Australian data (Reference 17 and 18). The following equations have been utilised in the current study along with applicable regional parameters from Reference 17 (displayed in Table 29) to determine the ARF for the Goulburn design hydrology.

Equation 1: Short duration aerial reduction factor equation (less than 18 hours)

$$ARF = min\{1, [1 + a(Area^b + c) + d(Area^e)(f - log_{10}Duration)]\}$$

Equation 2: Long duration aerial reduction factor equation (18 to 120 hours)

$$ARF = min\{1, [1 + a(Area^b + clog_{10}Duration)Duration^d + eArea^f Duration^g (0.3 + log_{10}AEP)]\}$$

Where:

$Duration\ =\ storm\ duration\ (h)$

$Area = area\ of\ interest\ (sq.km)$

$AEP\ =\ Annual\ exceedance\ probablity\ as\ a\ fraction\ between\ 0.5\ and\ 0.0005$

Table 29: Parameters for ARF equations

Region	Duration	a	b	c	d	e	f	g
NSW (GSAM)	>18h	-0.23	0.183	-0.91	-0.43	0.00048	0.38	0.21

5.7.4. Hydrologic Model Design Flow Results

Design flows derived from the hydrologic model for input into the hydraulic model are presented below. Specifically, the design flows on Wollondilly River at Marsden Weir and Mulwaree River at Eastgrove for each catchment's respective critical duration are presented in Table 30 and Table 31 respectively. ARFs consistent with the 725 km^2 catchment at Marsden Weir and 794 km^2 catchment at Eastgrove have been used (see Section 5.7.3). Calculated design flows (as time varying hydrographs) have been input into the hydraulic model to define design flood behaviour.

Table 30: Wollondilly River Design and Historic Event Flows

Event (AEP)	0.2 EY	2012	10%	2013	5%	2%	2010	1%	0.5%	PMF
Marsden Weir Flow (m³/s)	203	311	312	312	487	935	1,060	1,114	1,298	11,032

Table 31: Mulwaree River Design and Historic Event Flows

Event (AEP)	0.2 EY	2012	2013	10%	2010	5%	2%	1%	0.5%	PMF
Eastgrove Flow (m³/s)	152	160	179	232	301	347	616	762	912	6,119

Flows for the 2010, 2012 and 2013 events are also presented in the above tables. Examining these tables provides an indication of event magnitude for these historic events. On the Wollondilly River the 2012 and 2013 events have a probability of 10% AEP, whilst the 2010 event was approximately a 1% AEP flood. On the Mulwaree River the 2012 and 2013 events were slightly larger than a 0.2EY event and the 2010 event has an AEP of between 10% - 5%.

5.7.4.1. Comparison of Design Flows to Previous Studies

Design flows calculated in the current study differ significantly compared to those determined in the 2003 SMEC study. The design flows determined in the current study are lower, particularly in the Mulwaree catchment. Table 32 presents the flows calculated in the current study for both the Mulwaree and Wollondilly catchments as well as the flow determined in the 2003 SMEC study.

Table 32: Current Study / 2003 SMEC Study Design Flow Comparison

Event (ARI)	Wollondilly River Flow (m³/s)		Mulwaree River Flow (m³/s)	
	2003 SMEC Study	Current Study	2003 SMEC Study	Current Study
20%	258	203	258	152
10%	428	312	428	232
5%	648	487	648	347
2%	1,026	935	1,026	616
1%	1,415	1,114	1,415	762
0.5%	1,868	1,298	1,868	912
Extreme Flood	4,244	11,032	4,244	6,119

The RFFA methodology used in calculation of design flows in the 2003 SMEC study followed best engineering practise and utilised the best data available at the time of the study. The method used data from 19 regional stream gauges situated within 110 km of Goulburn with an average record length of 37 years. A relationship was formed between catchment area, 0.5EY flow and the 1% AEP flow for these regional stream gauges to extrapolate design flow estimates to Goulburn for both the Wollondilly and Mulwaree Rivers. As both of these catchments are of similar sizes, the SMEC 2003 study design flows were determined to be the same for both rivers.

For the current study, large amounts of rainfall and stream gauge data (see Section 2.4 and 2.5) as well as recent flood events in 2010, 2012 and 2013 provided a plethora of data which allowed rigorous calibration/validation and verification of a hydrologic model (see Section 5.5 and 5.6). Such data was not available at the time of the SMEC 2003 study. The revised modelling highlighted that a catchment's discharge is not only related to its size, but also other factors such as the size of the channel, width of the floodplain and other topographical features (see Section 5.4.1). As mentioned, the Mulwaree River catchment has a poorly defined channel and wide floodplain which leads to increased attenuation and reduced discharge. On the contrary, the Wollondilly River catchment has a well-defined channel and a confined floodplain which leads to higher discharges. The current study methodology takes these catchment characteristics into account when determining design flows which leads to the difference in design flows for each catchment identified by the current study but not calculated in the SMEC 2003 study.

In summary the design estimates from the current study are to be preferred as the overall methodology allowed the incorporation of significantly more local data and does not rely of extrapolating regional data for use at Goulburn.

A comparison between 1% AEP levels derived in the current study and in 2003 SMEC Study is presented in Section 7.1.4.

5.8. Goulburn Critical Duration Assessments

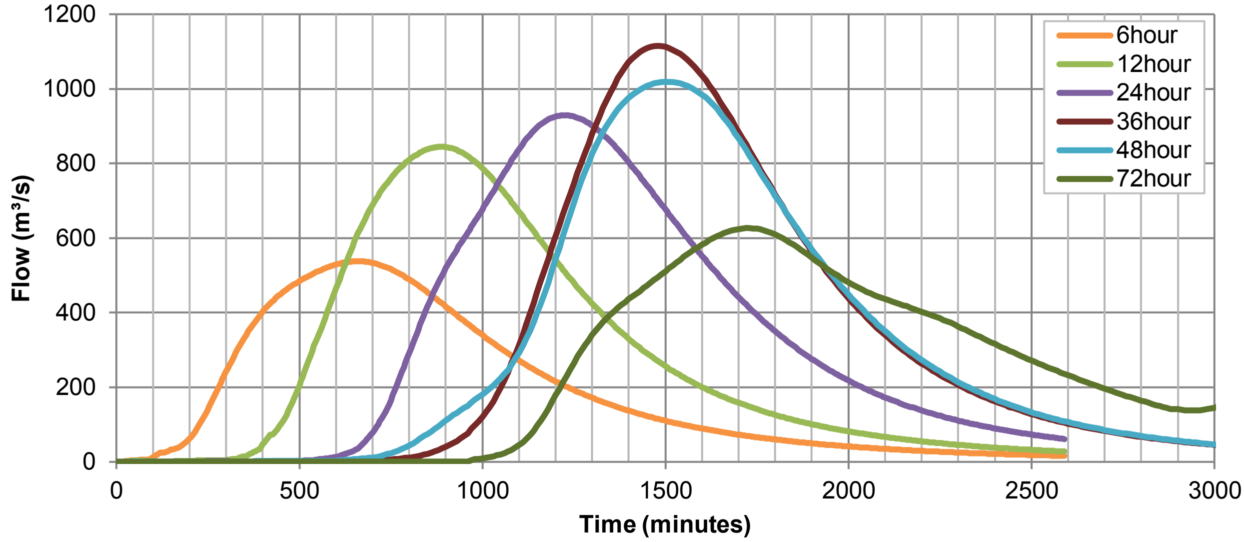
A series of critical duration assessments have been undertaken to determine which storm duration is responsible for generating the largest flow at the following locations in the study area:

- Marsden Weir on the Wollondilly River;
- Eastgrove along the Mulwaree River; and
- The confluence of the Wollondilly and Mulwaree Rivers.

5.8.1. Wollondilly River Critical Duration Assessment

A critical duration assessment was carried out for flows at Marden Weir on the Wollondilly River. The Goulburn 1% AEP design events were used to determine the critical duration at the Marsden Weir with the assumption that the critical duration remains constant for events of all AEP (with the exception of the PMF). The flow hydrographs for the 1% AEP events of varying durations at the Marsden Weir are presented in Chart 5. The 36 hour duration event was found to be critical along the Wollondilly River at Marsden Weir.

Chart 5: Marden Weir, Wollondilly River - Critical Duration – 1% AEP Flow Hydrographs

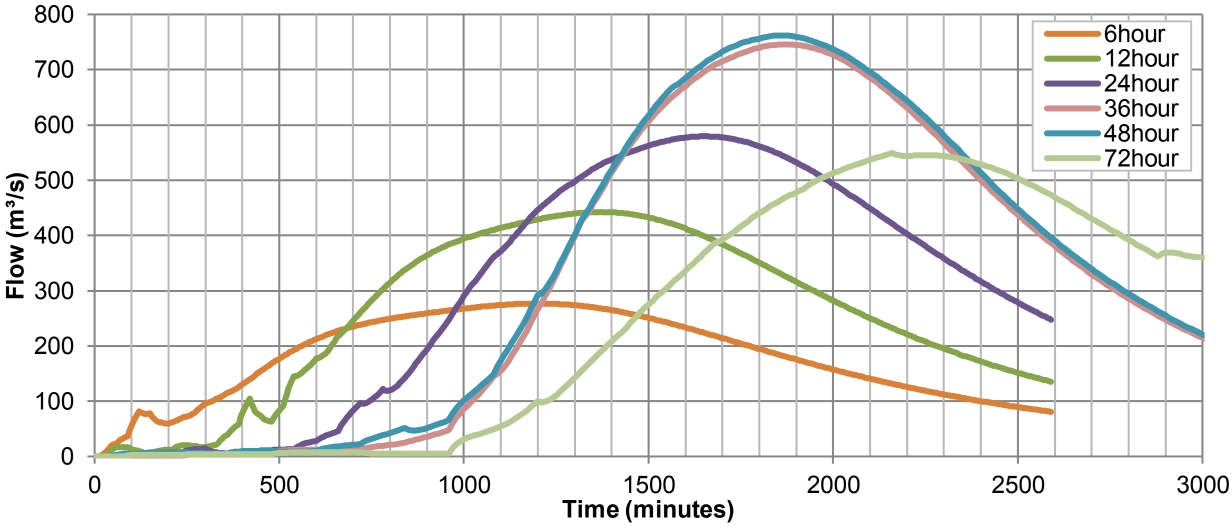


A critical duration assessment using the same process was undertaken for the PMP with the critical duration of the PMF for the Wollondilly River found to be 6 hours.

5.8.2. Mulwaree River Critical Duration Assessment

A critical duration assessment was undertaken for the Mulwaree River at Eastgrove using the 1% AEP design events. Chart 6 displays the flow hydrographs for the 1% AEP of varying durations at Eastgrove. The 48 hour duration event was found to be critical along the Mulwaree River.

Chart 6: Mulwaree River at Eastgrove – Critical Duration – 1% AEP Flow Hydrographs

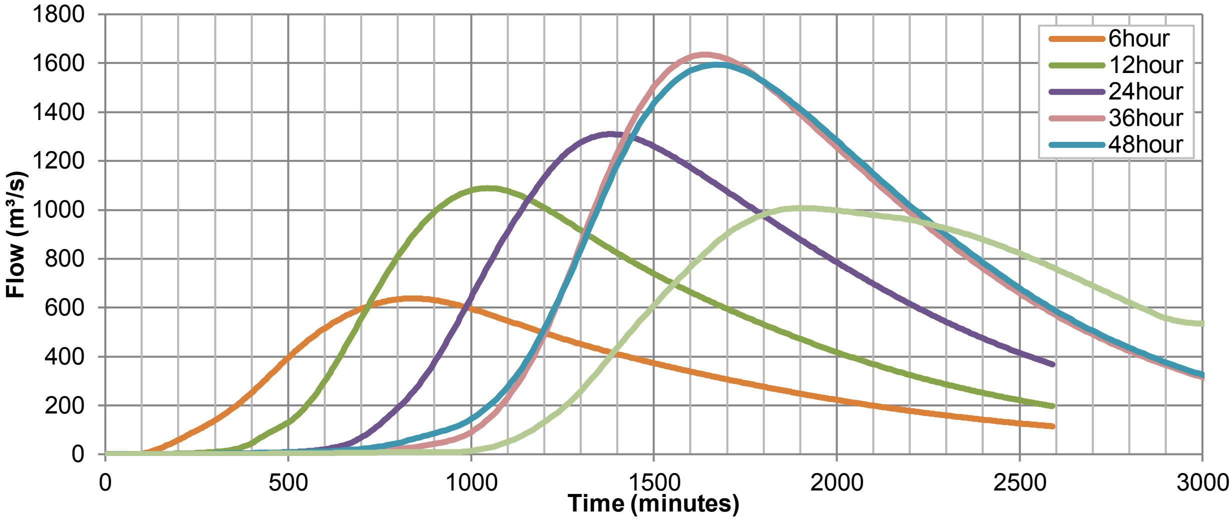


A critical duration assessment using the same process was undertaken for the PMP with the critical duration of the PMF for the Mulwaree River found to be 6 hours.

5.8.3. Downstream Wollondilly River Critical Duration Assessment

A critical duration assessment was undertaken for the 1% AEP event at the confluence of the Wollondilly and Mulwaree Rivers. Chart 7 presents the flow hydrographs for the 1% AEP events of varying durations. It was found that the 36 hour event was critical downstream of the confluence of these two rivers.

Chart 7: Wollondilly River at Murrays Flat - Critical Duration 1% AEP Flow Hydrographs



5.9. Hydrologic Sensitivity Analysis

Sensitivity analysis was carried out in order to assess the effect that adjusting hydrologic model parameters has on model results. Comparisons were carried out in the hydraulic models to determine impacts on peak flood levels for the 1% AEP design flood event.

The following hydrologic parameters were tested:

- An increase in rainfall losses of 20% (both initial and continuing losses);
- A decrease in rainfall losses of 20% (both initial and continuing losses);
- An increase in routing parameter ‘C’ of 20%;
- A decrease in routing parameter ‘C’ of 20%;
- Increases in rainfall of 10%, 20% and 30%.

All hydrologic model sensitivity analysis results are presented in Section 7.1.5 with the exception of increases to rainfall which is covered in the section on climate change (Section 7.1.5.2).

6. HYDRAULIC MODELLING

6.1. Introduction

The hydraulic model uses flow inputs (discharge hydrographs generated by a hydrological model) to calculate flood levels, depths and velocities. The hydrodynamic modelling program TUFLOW (Reference 19) has been used in this study. TUFLOW is a finite difference grid based 1D/2D hydrodynamic model which uses the St Venant equations in order to route flow according to gravity, momentum and roughness.

TUFLOW is ideally suited to this study because it facilitates the identification of potential flood problem areas as well as inherently representing the available floodplain storage within the 2D model geometry. In addition to this, TUFLOW allows for the utilisation of breaklines at differing resolution to the main grid. Breaklines are used to ensure the correct representation of features which may affect flooding (features such as roads, embankments, etc.) which is especially important in an urban environment.

Importantly, TUFLOW models can clearly define spatial variations in flood behaviour across the study area. Information such as flow velocity, flood levels and hydraulic hazard can be readily mapped in detail across the model extent. This information can then be easily integrated into a GIS based environment enabling outcomes to be efficiently incorporated into Council's planning activities (in for example waterRIDE or Mapinfo).

6.2. Model Build Process

Model construction begins with the DEM (constructed from the ALS mentioned in Section 2.2.1 combined with channel cross section survey described in Section 2.2.2) which defines at high resolution a catchment's topographical characteristics. Finer features (drainage channel and levees) that have significant impacts on flows may then be incorporated via additional spatial layers of information. Numerous spatial layers are applied to the model with the aim of closely replicating the catchment's true hydraulic conditions.

6.2.1. Model Domain and Grid Size

The Goulburn hydraulic model extent covers an area of 78 km² and is displayed in Figure 24. Ground elevations in the model were informed by the DEM described in Section 2.2.1 and displayed in Figure 2.

The selection of grid size for use in a hydraulic model is based on ensuring hydraulic features are adequately defined whilst not creating excessively long model run times. An important feature of a hydraulic model (depending on site characteristics and applicable flood mechanism) is the capacity to model channel in-bank conveyance accurately. Emulation of in-bank capacity is key to correctly model the study area and as such the conveyance characteristics of the in-bank, based on the model, have been compared to cross-sections achieved by survey (see Section 2.2.2). This conveyance comparison can be seen in Figure 25 and Figure 26 for the

Wollondilly and Mulwaree Rivers respectively. It indicates that a 10 m grid adequately defines the in-bank conveyance for both the Wollondilly and Mulwaree Rivers. The locations of the selected cross sections are displayed on Figure 1 along with the corresponding cross section numbers.

Accordingly, a 10 m finite difference grid was utilised for the Goulburn study area. The selected grid size allowed for reasonable run times whilst adequately defining in-bank conveyance in 2D.

6.2.2. Breaklines

Flow paths, levee banks, weirs, railway lines and road embankments are hydraulic features that have a significant impact on flood behaviour, especially in relatively flat regions such as the areas on the Mulwaree River floodplain. Such features have been represented in the model by breaklines with crest and invert heights determined by analysis of the ALS data (see Section 2.2.1). The locations of these various hydraulic features are displayed in Figure 24.

6.2.3. Roughness Values

As mentioned in the previous section various hydraulic characteristics are combined with the model grid in order to inform the final hydraulic model properties. This is equally true for cell roughness estimates. The Manning's 'n' values for each grid cell were estimated based on established references and previous studies and were then confirmed by calibration of the hydraulic model. Values were applied to the 2D overland area based on land use information as shown in Table 33 below.

Table 33: Mannings 'n' values

Land Use	Manning's 'n'
Open Areas (grazing, cropping etc.)	0.06
Golf Courses and Playing Fields	0.05
Low Density Vegetation	0.07
Riparian Vegetation	0.10
Creek in-bank	0.05
Urban	0.06
Roads	0.02
Railway	0.03
Buildings	3.00

Sensitivity testing of the applied roughness values has been carried out. See Section 6.4 for the results of this analysis.

It should be noted that these roughness values are within the range of those recommended by Chow (1959) and Henderson (1966) as well as the revised ARR guidelines (Project 15: Two Dimensional Modelling in Urban and Rural Floodplains). They are also comparable to the roughness values used in the Yass Flood Study (Reference 10).

6.2.4. Hydraulic Structures

Numerous hydraulic structures such as bridges and weirs situated along the Wollondilly and

Mulwaree Rivers have been identified in the Goulburn study area. Structure information was sourced from Council, RMS, survey commissioned as part of this study or measured by WMAwater engineers during a site visit (see Section 2.2.4). Details of these structures were input into the model as 2D elements with the locations of these structures displayed in Figure 24. Further information on these hydraulic structures are presented in Section 2.2.4.

6.2.4.1. Blockage

Structure blockage can significantly affect peak flood levels both upstream and downstream of a structure. Blockage of hydraulic structures can occur with the transportation of materials by flood waters, which in the vicinity of Goulburn is most likely vegetation such as logs and fallen trees.

No specific information related to blockage of hydraulic structures has been obtained and blockage is unlikely due to the size of the bridge structures in the study area.

The current study follows recommendations in accordance with the ARR Blockage Guidelines (Reference 9) which notes that bridges with diagonal spans exceeding 6 m are not likely to block during a flood event. All bridge crossings in the study area are greater than 6 m and therefore assumed not susceptible to blockage.

6.2.5. Boundary Conditions

6.2.5.1. Inflows

A calibrated/validated/verified hydrologic model (see Section 5) was used to produce design flows for the 0.2 EY, 10%, 5%, 2%, 1%, 0.5% AEP and the PMF events. These design flows were used as inflows for the hydraulic model at the upstream boundaries and for internal sub-catchments, to define design flood behaviour such as peak flood levels and velocities.

6.2.5.2. Downstream Boundary

The downstream boundary of the 2D model is 9 km downstream of the Wollondilly and Mulwaree Rivers confluence. The boundary is situated immediately downstream of a constriction in the Wollondilly River floodplain which is an important control on water levels upstream (for larger events).

The downstream boundary consists of a fixed water level boundary in a 1D channel located 10 km downstream of the 2D model extent. The boundary has no impact on flood behaviour within the study area.

6.3. Hydraulic Model Calibration

Hydraulic model calibration was undertaken for historic events using flows from the calibrated hydrologic model (Section 5.5), stream gauge data (see Section 2.5) and peak flood level information (see Section 2.6).

Calibration of the hydraulic models generally consisted of matching surveyed peak flood levels (see Section 2.6) to the modelled levels. Hydraulic model calibration was performed on the December 2010 event and validated to the March 2012 and June 2013 flood events with calibration results contained in Section 7.1.1.

6.4. Hydraulic Sensitivity Analysis

Sensitivity analysis was carried out in order to assess the effect that adjusting hydraulic model parameters has on model results. Sensitivity was determined by investigating impacts on peak flood levels for the 1% AEP design flood event.

The following hydraulic model parameters were tested:

- An increase in Manning's n roughness of 25%;
- A decrease in Manning's n roughness of 25%;
- Grid size reduced to 5 m.

All sensitivity analysis results are presented in Section 7.1.5.

6.4.1. Joint Event Modelling

6.4.1.1. General Information

A 'joint' event is where two flood mechanisms (independent or otherwise) interact in order to produce the flood levels, extents, flows and depths characterising the flood event. Often in joint events there is a relatively small proportion of the overall results (temporally or spatially) which are impacted by the conjunction of events. A good example of a joint event is where two rivers, fed by flows from individual upstream catchments, meet a significant distance downstream and have the potential to influence one another's flood behaviour. An example of this is at the confluence of the Wollondilly and Mulwaree Rivers. Flooding at the junctions of these water courses can be caused by individual flooding in either watercourse or a combination of flooding in both systems occurring simultaneously.

A key issue in joint modelling is independence, as in, are the two mechanisms being modelled independent or otherwise. Where events are independent coordinating them to produce peak flood behaviour can be inappropriate. For example if a 1% AEP flood in one river has no correlation with a 1% AEP event in the other, then combining them together to maximise flood levels will not produce 1% AEP flood behaviour but instead 0.01% AEP flood behaviour (i.e. a 10,000 year ARI event versus the intended 1% AEP event).

The NSW Government has previously provided some guidance on the joint modelling of non or weakly correlated phenomena (NSW, 2009). These guidelines recommend that when modelling flooding associated with various semi-dependant sources that the following approach should be taken:

- Model the 1% AEP flood in the first system with the 5% AEP flood in the second system; then
- Model the 5% AEP flood in the first system with the 1% AEP flood in the second

system; and
- Take a peak envelope of the results to produce your 1% AEP scenario.

This approach was utilised in the Yass Flood Study (Reference 10). It has also been examined in the current study for interactions between flooding in the Wollondilly and Mulwaree River catchments.

6.4.1.2. Approach Adopted

As discussed, flooding at Goulburn can result from both the Wollondilly and Mulwaree Rivers. When examining Goulburn flood history (Section 1.3) it was noted that the magnitude of flooding occurring in these rivers often differed greatly for the same event. This is due to the large size of the two catchments and distance between them meaning that the same flood producing rainfall does not typically occur in both catchments simultaneously. However, a flood event on one river is generally associated with some degree of flooding in the other.

Sensitivity analysis indicates that flood levels in the Mulwaree River are sensitive to flooding in the Wollondilly River with increases in flood levels occurring as far upstream as the Hume Highway. Accordingly, a peak flood envelope method has been employed in the current study to account for these impacts.

The joint probability method employed was based on the method outlined in Section 6.4.1.1 and consisted of modelling the 5% AEP local event in conjunction with the 1% AEP mainstream event and vice versa. In addition to this the 1% AEP flood in both catchments has been modelled simultaneously using an ARF consistent with the area upstream of the Wollondilly/Mulwaree River confluence.

7. RESULTS AND ANALYSIS

7.1. Hydraulic Model Results

A summary of the hydraulic model results is contained in the following sections. Hydraulic model results provide peak flood levels, depths and extents for the calibration/validation historic events (see Section 7.1.1) and design floods (see Section 7.1.2). For historic events, calibration/validation involved matching modelled flood levels to observed flood levels, and flood behaviour to information obtained from the Community Consultation process (Section 3). All design results are displayed for the critical durations determined in Section 5.8 for the Goulburn study area.

7.1.1. Hydraulic Model Calibration/Validation Results

7.1.1.1. Hydraulic Model Calibration

The December 2010 event was modelled in the hydraulic model using flows determined from the hydrologic model (see Section 5.4). Figure 27 presents the modelled December 2010 flood event depths and extent (raster) as well as a comparison of observed peak flood levels to modelled levels (displayed as red points). An absolute average error of 0.08 m was achieved which gives confidence in the performance of both the hydraulic and hydrologic models. This calibration is based on comparison of modelled and surveyed peak flood levels and extents at 21 locations throughout the domain.

Furthermore, the December 2010 event was also calibrated to the observed stage hydrographs at the Rossi Weir and The Towers Stream Gauges (see Figure 30). The general shape of the modelled stage hydrographs were a good match to that observed. At Rossi Weir the peak stage was matched to within 0.02 m and at The Towers stream gauge the peak flood level was matched to within 0.04 m.

7.1.1.2. Hydraulic Model Validation

The March 2012 and June 2013 events were modelled in the calibrated hydraulic model using flows determined via the hydrologic model (see Section 5.4). These models were validated using surveyed flood depths, extents and comparisons to modelled and observed stage hydrographs at the Rossi Weir, The Towers and Murrays Flat Stream Gauges (see Figure 31 and Figure 32).

<u>March 2012 Flood Event</u>

Figure 28 presents the modelled March 2012 flood event depths and extents as well as a comparison of observed and modelled peak flood levels (displayed as a red point) and flood extents (displayed as yellow points). The only available peak flood level was matched in the hydraulic model with 0.1 m accuracy and three flood extents marks were also matched.

Figure 31 displays the observed and modelled stage hydrographs at the Rossi Weir, The Towers and Murrays Flat Stream Gauges. The peak stage was matched to within 0.01 m at the Rossi Weir gauge and matched exactly at the Murrays Flat gauge. At the Towers Gauge the comparison of modelled to observed stage does not provide a good match, however as noted previously there was no pluviometer rainfall data available in the Mulwaree catchment for input in to the hydrologic model for the 2012 event. Notwithstanding, the difference in peak stage between modelled and observed is less than 0.1 m.

June 2013 Flood Event

The modelled flood event depths and extents for the June 2013 event are shown in Figure 29 however unfortunately there are no peak flood levels for comparison. The observed and modelled stage hydrographs at Rossi Weir, The Towers and Murrays Flat are shown in Figure 32. The peak stage at Rossi Weir was matched to within 0.08 m, at The Towers was exactly matched and Murrays Flat matched to within 0.09 m. The shape and fit of the modelled stage hydrographs to the observed is generally good.

7.1.1.3. Discussion of Calibration/Validation Results

The results of the December 2010 calibration event are considered to be very good, with 21 peak flood levels and extents matched throughout the domain with an average absolute error of 0.08 m. Whilst only limited peak flood level information was available for validation of the hydraulic model in the March 2012 and June 2013 events, the results of the model validation were also good with stage hydrographs matched to within 0.1 m accuracy at the peak. The results indicate that a high degree of confidence can be had in the hydraulic model and subsequent design results.

7.1.2. Hydraulic Model Design Results

Design results are the peak flood envelope of the Wollondilly River and Mulwaree River events, which in reality are unlikely to occur simultaneously.

Design flood maps (Figure 33 to Figure 39) have been produced to display flood affected regions for the various design events. It should be noted that inundation patterns and/or peak flood levels shown for design events are based on best available estimates of flood behaviour within the catchment. Inundation extents and patterns may vary depending on the actual rainfall event, local tributary flows, the relative timing of flows and local influences such as changes in topography and road works etc.

A summary of the provided results are displayed below with further details in the following sections:

- Peak flood depths and levels for the design flood events (PMF, 5Y ARI, 10%, 5%, 2%, 1% and 0.5% AEP) are presented in Figure 33 to Figure 39;
- Wollondilly River and Mulwaree River flood profiles for each design flood event are presented in Figure 40 and Figure 41 respectively; and
- Design event peak flood flows, levels and velocities at key locations (see Appendix C).

Figure 33 to Figure 39 present design flood depths in various shades of blue. Lighter shades of blue indicate shallower flood depths and the darker blues indicate deeper flood depths. The depth of flooding is indicated on the colour pallet on each of these figures. Additionally, peak flood levels in Australian Height Datum (m) are presented as flood level contours at 1 m increments. The design peak flood level is displayed as contours throughout the study area, with flood levels typically ranging between 635 mAHD to 626 mAHD for the 1% AEP event. It should be noted that Council have been provided with digital data that describe flood behaviour and site specific information pertaining to flooding should be requested from Council.

Table 34 displays the peak flood heights and flows at Marsden Weir for the range of design flood events.

Table 34: Marsden Weir – Design Peak Flood Heights and Flows

Event*	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	PMF
Peak Flood Level (m)	632.0	632.2	632.6	634.3	634.9	635.5	646.4
Event Peak Flow (m^3/s)	204	312	489	935	1,181	1,297	10,913

* Event probability is displayed as AEP. Please see the Terminology Section at the beginning of this report for conversion to ARI.

Table 35 displays the peak flood heights and flows at Eastgrove for the range of design flood events.

Table 35: Eastgrove – Design Peak Flood Heights and Flows

Event*	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	PMF
Peak Flood Level (m)	627.5	627.8	628.1	629.8	630.4	631.0	640.3
Event Peak Flow (m^3/s)	153	234	353	621	783	911	5,705

* Event probability is displayed as AEP. Please see the Terminology Section at the beginning of this report for conversion to ARI.

Figure 40 and Figure 41 display peak flood profiles for the Wollondilly and Mulwaree Rivers respectively. On the Wollondilly River, the difference in peak flood level between events of varying magnitude is of note, with the 1% AEP being 4 – 5m higher than the 0.2EY in many locations. Additionally, the PMF is 8 – 12 m higher than the 1% AEP event. The large difference in flood level experienced between events is due to the constrained floodplain of the Wollondilly River. The Mulwaree River floodplain is not constrained, however Mulwaree flood levels at Goulburn scale dramatically due to a constriction of the Wollondilly River floodplain immediately downstream of the confluence. The back watering effects of this constriction are clearly visible on the peak flood profiles with larger events being considerably more affected than smaller events.

Appendix C presents peak flood flows, levels and velocities for the various design events at key locations on both Rivers. The locations where these flood characteristics were recorded are presented as 'Analysis Points' in Figure 44. Flow measurements were recorded for a cross section of the river, perpendicular to flow direction at the location of each Analysis Point.

7.1.3. Design Results Comparison to Historic Events

Figure 42 presents the difference in peak flood level between various historic flood events and the current study 1% AEP flood. Four large floods have been compared, namely the 1870 (green), 1959 (red), 1961 (purple) and 1974 (orange) events.

Results indicate that the 1% AEP flood on the Wollondilly River achieves a flood level approximately 0.3 – 1.0 m higher than the 1961 flood event which is the second largest flood to have occurred apart from the 2010 flood event (see Section 1.3.1). It is noted from the hydraulic model calibration that the 2010 flood event is 0.15 m – 0.4 m lower than the 1% AEP event. There are two available points for comparison of the 1870 flood event. The 1% AEP is noted to be 1.2 m higher at the Marsden Weir gauge than the 1870 flood, however downstream of the Kenmore Bridge the 1870 flood event was approximately 0.1 m higher than the 1% AEP event. This is likely due to changes on the floodplain such as the construction of the Kenmore Bridge or the railway bridge immediately downstream.

On the Mulwaree River, the 1% AEP flood is noted to be smaller (0.1 m – 0.3 m lower) than the 1959 flood event upstream of the railway viaduct, downstream of which the 1% AEP becomes larger (0.1 m – 0.3 m higher) than the 1959 event. This change in flood level is likely due to the backwatering effect of the Wollondilly River on peak flood levels in the Mulwaree River.

It is important to note that the 1959 storm event produced a 1% AEP flood in the neighbouring Yass River catchment. Examination of daily read rainfall data indicates that more rainfall was experienced in the Mulwaree River catchment for the same event. The 1959 event (21st October) catchment average rainfall for both the Yass catchment (114.5 mm) and Mulwaree catchment (149.7 mm) has been calculated based on available daily read rainfall data. It was noted that the Mulwaree catchment average rainfall was approximately 30% higher than that experienced in the Yass River catchment. Additionally the 1% AEP 24 hour point rainfall depth in the Mulwaree catchment is 144.2 mm. When factoring in ARF, the potential for embedded bursts and that the catchment average rainfall determined above is restricted (i.e. recorded as the period between 9 am on consecutive days, not the greatest 24 hour rainfall total for the period) this tends to indicate that the 1959 flood event was significantly larger than a 1% AEP event which adds robustness to the current study results.

7.1.4. Design Results Comparison to Previous Studies

A comparison of the current study and 2003 SMEC Study 1% AEP flood levels are presented in Table 36 for the Wollondilly River and Table 37 for the Mulwaree River. Figure 43 presents the spatial difference in peak flood level between the 2003 SMEC Study and the current study 1% AEP peak flood levels.

As expected the current study results are lower than the 2003 SMEC Study results which is due to the use of smaller design flows as noted in Section 5.7.4.1. The Wollondilly River is on average 1.1 m lower in the current study and the Mulwaree River is on average 1.4 m lower.

As mentioned, the design estimates from the current study are to be preferred as the overall methodology allowed the incorporation of significantly more local data and does not rely of extrapolating regional data for use at Goulburn.

Table 36: Comparison of Current and 2003 SMEC Study 1% AEP Flood Levels – Wollondilly

Structure No.	Location	Current Study (mAHD)	2003 SMEC Study (mAHD)	Difference (m)
10	Marsden Weir	634.9	636.0	1.1
11	Marsden Bridge	634.7	635.6	0.9
2	Victoria Street Bridge	632.0	632.8	0.8
12	Kenmore Bridge	630.6	631.9	1.3
n/a	Mulwaree Confluence	629.9	631.1	1.2

Table 37: Comparison of Current and 2003 SMEC Study 1% AEP Flood Levels – Mulwaree

Structure No.	Location	Current Study (mAHD)	2003 SMEC Study (mAHD)	Difference (m)
4	Hume Highway Bypass	631.0	632.4	1.4
5	Landsdowne Bridge	630.6	632.1	1.5
7	Railway Viaduct	630.3	631.8	1.5
8	Sydney Road Bridge	630.1	631.5	1.4
n/a	Wollondilly Confluence	629.9	631.1	1.2

7.1.5. Sensitivity Analysis Results

Sensitivity analysis was carried out in order to assess the effect that adjusting model parameters (Manning's 'n', flow and grid size) had on design model results. Comparisons were carried out using peak flood levels for the 1% AEP design event. The location of the sensitivity 'Analysis Points' are presented in Figure 44.

7.1.5.1. Model Parameter Sensitivity

Table 38 and Table 39 display a comparison of peak flood levels throughout the model domain for the various sensitivity runs presented in Sections 5.9 and 6.4 for the Wollondilly and Mulwaree Rivers respectively. The location of the sensitivity 'Analysis Points' are presented in Figure 44.

The model's sensitivity to the selected hydrologic lag parameter was examined. By decreasing the lag parameter by 20% an average increase in peak flood level of 0.6 m was experienced on both the Wollondilly and Mulwaree Rivers. A decrease of approximately 0.5 m in peak flood level is associated with increasing the lag parameter by 20%. This indicates that the system is highly sensitive to the hydrologic model lag parameter. As mentioned in Section 5.4.1, the hydrologic model lag parameter impacts on the magnitude of the flow. Changes in flow lead to large changes in peak flood level at Goulburn due to the confined nature of the Wollondilly River channel and its backwatering effects on the Mulwaree River upstream of the confluence.

The model's sensitivity to the selected losses was examined by increasing and decreasing both the initial and continuing losses by 20%. Decreasing the applied losses led to an increase of 0.33 m and 0.36 m in peak flood level on the Wollondilly and Mulwaree Rivers respectively. This indicates that the system is sensitive to the selected loss parameters, again due to the impact that changes losses has on the magnitude of the flow.

Roughness values presented in Table 33 were increased and decreased by 20% to test model sensitivity. An increase in roughness led to an average increase in peak flood level of 0.6 m on the Wollondilly and 0.5 m on the Mulwaree Rivers. This indicates that the model results are highly sensitive to the selected roughness values.

Table 38: Model Parameter Sensitivity Analysis Results – Wollondilly River

ID	Location	Hydrologic Lag Parameter		Hydrologic Losses		Manning's 'n'	
		-20%	+20%	-20%	+20%	-20%	+20%
W01	Rossi Bridge Upstream	0.82	-0.59	0.38	-0.37	-0.43	0.65
W02	Rossi Bridge Downstream	0.60	-0.52	0.30	-0.31	-0.37	0.50
W03	Rossi Gauge	0.59	-0.49	0.30	-0.29	-0.30	0.50
W04	Rossi Weir Upstream	0.67	-0.41	0.38	-0.23	-0.26	0.57
W05	Rossi Weir Downstream	0.64	-0.71	0.31	-0.45	-0.97	0.53
W06	Marsden Weir Upstream	0.72	-0.61	0.37	-0.35	-0.66	0.60
W07	Marsden Weir Downstream	0.74	-0.62	0.38	-0.36	-0.67	0.62
W08	Marsden Bridge Upstream	0.71	-0.60	0.37	-0.35	-0.60	0.59
W09	Marsden Bridge Downstream	0.72	-0.58	0.38	-0.33	-0.60	0.60
W10	Behind Properties on Fitzroy Street	0.65	-0.54	0.32	-0.31	-0.63	0.54
W11	Victoria Bridge Upstream	0.65	-0.48	0.35	-0.30	-0.54	0.58
W12	Victoria Bridge Downstream	0.56	-0.46	0.32	-0.28	-0.54	0.50
W13	Avoca St Mid	0.56	-0.46	0.32	-0.29	-0.53	0.49
W14	Kenmore Bridge Upstream	0.62	-0.49	0.36	-0.37	-0.56	0.57
W15	Kenmore Bridge Downstream	0.62	-0.49	0.37	-0.36	-0.56	0.57
W16	Crookwell Rail Bridge Upstream	0.63	-0.51	0.37	-0.37	-0.57	0.58
W17	Crookwell Rail Bridge Downstream	0.63	-0.51	0.37	-0.37	-0.57	0.58
W18	Sewer Aqueduct Upstream	0.68	-0.60	0.40	-0.39	-0.55	0.62
W19	Sewer Aqueduct Downstream	0.67	-0.60	0.41	-0.39	-0.55	0.61
W20	Wollondilly/Mulwaree Confluence	0.68	-0.60	0.41	-0.39	-0.56	0.62
W21	Murrays Flat Gauge	0.71	-0.66	0.45	-0.45	-0.49	0.67
	Average	**0.60**	**-0.55**	**0.36**	**-0.35**	**-0.55**	**0.58**

Table 39: Model Parameter Sensitivity Analysis Results – Mulwaree River

ID	Location	Hydrologic Lag Parameter		Hydrologic Losses		Manning's 'n'	
		-20%	+20%	-20%	+20%	-20%	+20%
M01	The Towers Gauge	0.40	-0.22	0.18	-0.13	-0.11	0.29
M02	The Towers Weir Upstream	0.30	-0.13	0.13	-0.07	0.03	0.21
M03	The Towers Weir Downstream	0.30	-0.22	0.13	-0.09	-0.23	0.21
M04	Thornes Bridge Upstream	0.23	-0.20	0.11	-0.11	-0.16	0.17
M05	Thornes Bridge Downstream	0.22	-0.17	0.11	-0.10	-0.15	0.16
M06	Hume Highway Bridge 2 Upstream	0.55	-0.26	0.28	-0.16	-0.22	0.44
M07	Hume Highway Bridge 2 Downstream	0.59	-0.32	0.32	-0.21	-0.32	0.48
M08	Hume Highway Bridge 3 Upstream	0.59	-0.39	0.31	-0.25	-0.34	0.48
M09	Hume Highway Bridge 3 Downstream	0.60	-0.41	0.33	-0.27	-0.39	0.49
M10	Hume Highway Bridge 4 Upstream	0.56	-0.31	0.30	-0.20	-0.27	0.46
M11	Hume Highway Bridge 4 Downstream	0.60	-0.38	0.33	-0.25	-0.37	0.49
M12	Lansdowne Bridge Upstream	0.67	-0.50	0.37	-0.33	-0.43	0.55
M13	Lansdowne Bridge Downstream	0.68	-0.52	0.38	-0.33	-0.45	0.56
M14	Goulburn Brewery	0.71	-0.60	0.40	-0.38	-0.52	0.60
M15	Park Road Roundabout	0.74	-0.64	0.43	-0.41	-0.55	0.63
M16	Park Road Upstream	0.74	-0.64	0.43	-0.41	-0.55	0.63
M17	Park Road Downstream	0.74	-0.64	0.43	-0.41	-0.55	0.63
M18	Goulburn Golf Club Upstream	0.75	-0.64	0.43	-0.41	-0.55	0.63
M19	Goulburn Golf Club Downstream	0.75	-0.65	0.43	-0.41	-0.56	0.63
M20	May Street Bridge Upstream	0.75	-0.65	0.43	-0.42	-0.56	0.64
M21	May Street Bridge Downstream	0.75	-0.65	0.43	-0.41	-0.56	0.64
M22	Railway Viaduct Upstream	0.74	-0.64	0.43	-0.41	-0.56	0.64
M23	Railway Viaduct Downstream	0.73	-0.64	0.43	-0.41	-0.57	0.63
M24	Sydney Road Bridge Upstream	0.70	-0.61	0.42	-0.40	-0.57	0.62
M25	Sydney Road Bridge Downstream	0.68	-0.60	0.41	-0.39	-0.57	0.61
	Average	**0.60**	**-0.47**	**0.33**	**-0.30**	**-0.40**	**0.50**

The Wollondilly and Mulwaree River system at Goulburn is highly sensitive to changes in both flow (due to changes in lag parameter and losses) and roughness due to the unique topography of the area. The Wollondilly River floodplain is constrained which leads to changes in flow to cause significant impacts on peak flood level. The conveyance of the Wollondilly River causes Mulwaree River flows to back water thus increasing peak flood levels upstream of the confluence.

Model results were shown to be insensitive to grid size with typically less than 0.1 m difference displayed in peak flood level when comparing the current study 10 m grid results to those produced when using a 7.5 m grid.

7.1.5.2. Climate Change Sensitivity

Intensive scientific investigation is ongoing to understand the impact that human activity has and will continue to have on the climate. Since the 1950s, unprecedented warming has occurred to the atmosphere and oceans, with global snow and ice diminishing, sea level rising and concentrations of greenhouse gases increasing (IPCC Fifth Assessment Synthesis Report 2014). One direct impact of a changing climate with relevance to this flood study is the potential for heavier rainfall, leading to increased flood levels in Goulburn.

As rainfall intensity increases have the potential to increase flood levels in the Wollondilly and Mulwaree catchments, the New South Wales Government recommends investigating vulnerabilities of such increases through sensitivity analysis (DECC 2009). Recommended sensitivity analysis involves a 10%, 20% and 30% increase of peak rainfall and storm volume. The results of this rainfall sensitivity modelling for the 1% design rainfall event are presented in

Table 40 and Table 41 for the Wollondilly and Mulwaree Rivers respectively.

Results show that the system is highly sensitivity to increases in rainfall intensity with an average increase in peak flood level of 0.6 m associated with a 10% increase in rainfall. For 20% and 30% increases in rainfall, peak flood levels are expected to further increase by on average 1.1 m and 1.6 m respectively.

Table 40: Climate Change Sensitivity Analysis Results – Wollondilly River

ID	Location	Rainfall Increase		
		+10%	+20%	+30%
W01	Rossi Bridge Upstream	0.65	1.27	1.81
W02	Rossi Bridge Downstream	0.50	1.00	1.50
W03	Rossi Gauge	0.50	0.97	1.45
W04	Rossi Weir Upstream	0.57	1.06	1.56
W05	Rossi Weir Downstream	0.53	1.07	1.61
W06	Marsden Weir Upstream	0.60	1.17	1.74
W07	Marsden Weir Downstream	0.62	1.20	1.77
W08	Marsden Bridge Upstream	0.59	1.17	1.77
W09	Marsden Bridge Downstream	0.60	1.18	1.77
W10	Behind Properties on Fitzroy Street	0.54	1.08	1.59
W11	Victoria Bridge Upstream	0.58	1.18	1.92
W12	Victoria Bridge Downstream	0.50	0.98	1.53
W13	Avoca St Mid	0.49	1.05	1.63
W14	Kenmore Bridge Upstream	0.57	1.20	1.85
W15	Kenmore Bridge Downstream	0.57	1.14	1.68
W16	Crookwell Rail Bridge Upstream	0.58	1.15	1.69
W17	Crookwell Rail Bridge Downstream	0.58	1.15	1.67
W18	Sewer Aqueduct Upstream	0.62	1.20	1.75
W19	Sewer Aqueduct Downstream	0.61	1.17	1.72
W20	Wollondilly/Mulwaree Confluence	0.62	1.17	1.72
W21	Murrays Flat Gauge	0.67	1.27	1.84
	Average	**0.58**	**1.13**	**1.69**

Table 41: Climate Change Sensitivity Analysis Results – Mulwaree River

ID	Location	Rainfall Increase		
		+10%	+20%	+30%
M01	The Towers Gauge	0.29	0.58	0.81
M02	The Towers Weir Upstream	0.21	0.44	0.68
M03	The Towers Weir Downstream	0.21	0.43	0.72
M04	Thornes Bridge Upstream	0.17	0.33	0.51
M05	Thornes Bridge Downstream	0.16	0.31	0.51
M06	Hume Highway Bridge 2 Upstream	0.44	0.96	1.50
M07	Hume Highway Bridge 2 Downstream	0.48	1.01	1.56
M08	Hume Highway Bridge 3 Upstream	0.48	1.01	1.55
M09	Hume Highway Bridge 3 Downstream	0.49	1.03	1.59
M10	Hume Highway Bridge 4 Upstream	0.46	0.97	1.50
M11	Hume Highway Bridge 4 Downstream	0.49	1.03	1.59
M12	Lansdowne Bridge Upstream	0.55	1.13	1.70
M13	Lansdowne Bridge Downstream	0.56	1.14	1.72
M14	Goulburn Brewery	0.60	1.19	1.78
M15	Park Road Roundabout	0.63	1.24	1.84
M16	Park Road Upstream	0.63	1.24	1.84
M17	Park Road Downstream	0.63	1.24	1.84
M18	Goulburn Golf Club Upstream	0.63	1.25	1.85
M19	Goulburn Golf Club Downstream	0.63	1.25	1.85
M20	May Street Bridge Upstream	0.64	1.25	1.85
M21	May Street Bridge Downstream	0.64	1.25	1.85
M22	Railway Viaduct Upstream	0.64	1.25	1.85
M23	Railway Viaduct Downstream	0.63	1.24	1.83
M24	Sydney Road Bridge Upstream	0.62	1.22	1.80
M25	Sydney Road Bridge Downstream	0.61	1.19	1.71
	Average	**0.50**	**1.01**	**1.51**

Flood impact maps that present the expected difference in 1% AEP peak flood levels with potential increases in rainfall of 10%, 20% and 30% associated with climate change are presented in Figure 45 to Figure 47 respectively.

7.1.5.3. Summary of Sensitivity Analysis Findings

The sensitivity analysis results indicate that the Wollondilly/Mulwaree River system at Goulburn is highly sensitive to selected model parameters. This is due to the unique characteristics of the floodplain at Goulburn, and in particular, the constriction of the floodplain immediately downstream of the confluence of these two rivers. This constriction causes a modest increase in flow to result in large increase in peak flood level due to backwatering effects.

It is recommended that the system’s sensitivity to selected model parameters is examined as part of any future FRMS&P to provide insight into freeboard estimates. The Floodplain Development Manual’s (Reference 16) recommended freeboard of 0.5 m is potentially unsuitable for use in Goulburn due to the large variability in design flood level based on model parameter selection.

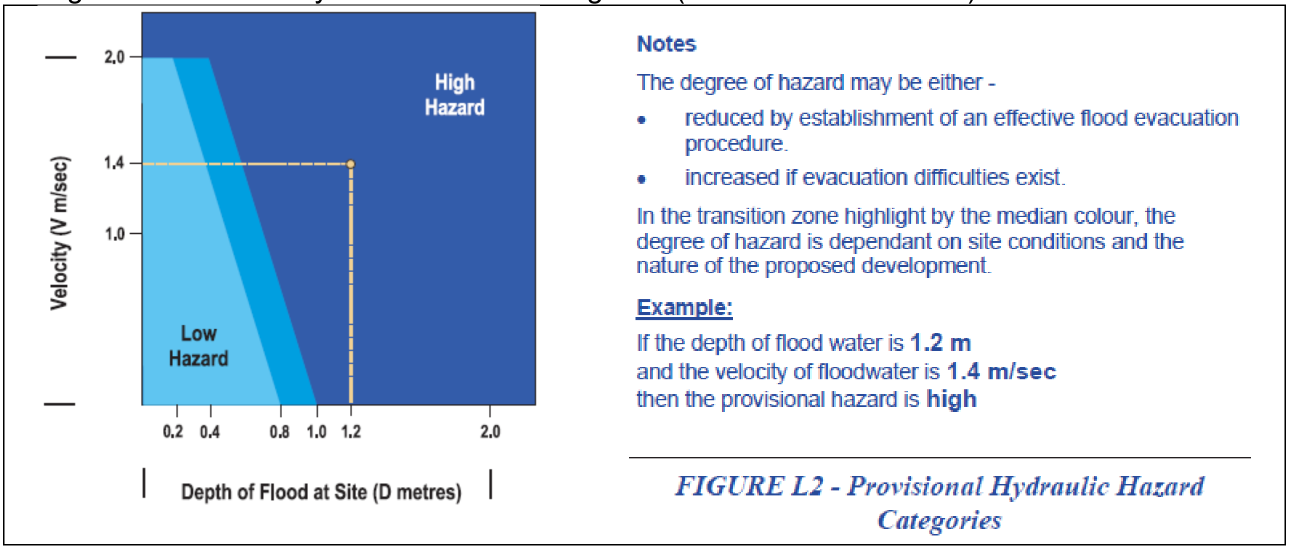
7.2. Preliminary Hazard Classification

The risk to life and potential damages to buildings during floods varies both in time and place across the floodplain. In order to provide an understanding of the effects of a proposed development on flood behaviour and the effects of flooding on development and people, the floodplain can be sub-divided into hydraulic and hazard categories.

Hazard is a measure of the overall harm caused by flooding and should consider a number of factors including the depth of flooding, velocity of flood waters, access to escape routes, duration etc. In the first instance provisional hazard categories can be defined based on the depth and velocity of floodwaters. Provisional flood hazard categories were defined in this study in accordance with the *Floodplain Development Manual - Figure L2* (Reference 16) as indicated below.

The hazards are provisional because they only consider the hydraulic aspects of flood hazard. High and low provisional hazard areas were defined for the 1% AEP and PMF events and are presented in Figure 48 and Figure 49 respectively. The *Floodplain Development Manual* (Reference 16) requires that other factors be considered in determining the “true” hazard such as size of flood, effective warning time, flood readiness, rate of rise of floodwaters, depth and velocity of flood waters, duration of flooding, evacuation problems, effective flood access, type of development within the floodplain, complexity of the stream network and the inter-relationship between flows.

Image 1: Provisional Hydraulic Hazard Categories (source: Reference 16)



7.3. Preliminary Hydraulic Categorisation

The 2005 NSW Government's Floodplain Development Manual (Reference 16) defines three hydraulic categories which can be applied to different areas of the floodplain; namely floodway, flood storage or flood fringe. Floodway describes areas of significant discharge during floods, which, if partially blocked, would cause a significant redistribution of flood flow. Flood storage areas are used for temporary storage of floodwaters during a flood, while flood fringe is all other flood prone land.

There is no single definition of these three categories or a prescribed method to delineate the flood prone land into them. Rather, their categorisation is based on knowledge of the study area, hydraulic modelling and previous experiences.

For this study, hydraulic categories were defined by the following criteria, which correspond in part with the criteria proposed by Howells et. al. (2003):

Floodway:		**Velocity x Depth > 0.25 m^2/s AND Velocity > 0.25 m/s**
	OR	**Velocity > 1 m/s AND Depth > 0.15 m**
Flood Storage:		Land outside the floodway where Depth > 0.5m
Flood Fringe		Land outside the floodway where Depth < 0.5m

The preliminary hydraulic categories for the 1% AEP and PMF events are presented in Figure 50 and Figure 51 respectively.

7.4. Hotspots

It is standard practice to identify flooding hotspots as part of the Flood Study and provide some detailed information for flood mechanisms impacting on these locations. A hotspot is identified as an area of interest from a flooding perspective. For example, locations where many residences are liable to flooding might be defined as hotspots as might other locations where key drainage assets are not meeting design standards or where key infrastructure, such as a highway, is flood affected. These Hotspots are often also NSW SES points of interest that are useful for NSW SES flood intelligence.

7.4.1. Hotspot 1: Avoca Street

Hotspot #1 is situated on the southern back of the Wollondilly River, downstream of Victoria Street. The community consultation process found that properties along Avoca, Bellevue and Derwent Streets experienced over floor and yard flooding in the recent December 2010 flood event.

Analysis of design results indicate that this area is first inundated during events larger than the 5% AEP event and flood depths exceeding 1 m are experienced in residential lots in the 1% AEP event. Peak flood levels taken on the eastern side of the PCYC on Avoca Street for various design events are in presented in Table 42.

Table 42: Avoca Street Flood Levels

Event	Level (mAHD)
Ground Level	**629.3**
0.2 EY	N/A
10% AEP	N/A
5% AEP	629.3
2% AEP	630.9
1% AEP	631.5
0.5% AEP	632.0
PMF	640.6

The 1% AEP hazard mapping (see Figure 48) indicates that large areas of high hazard flow are experienced in the region which poses a significant risk to both life and property.

7.4.2. Hotspot 2: Fitzroy Street Downstream of Marsden Bridge

Hotspot # 2 is situated on the southern back of the Wollondilly downstream of Marsden Bridge. The community consultation process found that a number of properties experienced yard and in some cases over floor flooding in the December 2010 event due to Wollondilly River flooding.

Table 43 presents the design flood levels at the back fence line of these properties, indicating that the first threat of flooding occurs in the 5% AEP event.

Table 43: Fitzroy Street Properties Flood Levels

Event	Level (mAHD)
Ground Level	**631.7**
0.2 EY	N/A
10% AEP	N/A
5% AEP	631.8
2% AEP	633.6
1% AEP	634.1
0.5% AEP	634.6
PMF	643.6

In the 1% AEP event, properties at Hotspot #2 experience high hazard flood waters with flood depths up to 2.5 m and velocities in excess of 1 m/s at the rear of the lots. The front of the lots on Fitzroy Street are either low hazard or not flooded, providing residents with an adequate escape route in the 1% AEP event. However, events larger than the 1% AEP are of concern due to floodwaters overtopping Fitzroy Street. This causes a high hazard environment due to the high velocities associated with flow overtopping the road and the potential for access to these properties to be cut. This risk is further increased for larger events which should be investigated as part of the FRMS&P.

7.4.3. Hotspot 3: Park Road, Eastgrove

Hotspot #3 is situated at Eastgrove, which is located on the eastern bank of the Mulwaree River. Eastgrove has a history of flooding, most recently in December 2010. In the 1% AEP event this area experiences high hazard flows with flood depths exceeding 1.5 m at properties on Hercules Street and 2.5 m at a property on Emma Street. The design flood levels at the Mulwaree River crossing at Park Road are shown in Table 44.

Table 44: Park Road Flood Levels

Event	Level (mAHD)
Ground Level	**627.0**
0.2 EY	627.6
10% AEP	627.8
5% AEP	628.1
2% AEP	629.8
1% AEP	630.4
0.5% AEP	631.0
PMF	640.3

Park Road is a key access route to the Goulburn Township which is well-used. This River crossing is inundated by flood waters relatively frequently causing Eastgrove residents to access Goulburn via Hetherington Street or Memorial Road. A significant amount of risk is present if people attempt to use the crossing during flood and as such management of this during flooding should be a priority.

The results of the Community Consultation process indicated that a number of properties in this area have experienced flooding particularly on Hercules Street, Emma Street and Eleanor

Street.

7.4.4. Hotspot 4: Braidwood Road

Hotspot #4 is situated on the western bank of the Mulwaree River between the Goulburn Recreation Area (racecourse) and the railway embankment. The area has a mixture of residential and non-residential land uses and flooded in events larger than the 5% AEP. Numerous properties are flooded in the 1% AEP flood event with flood depths exceeding 1 m in so areas. The fringes of the flood extent tend to be low hazard, however high hazard flooding does affect some lots.

7.5. Flood Emergency Response Planning

To assist in the planning and implementation of response strategies, the NSW SES in conjunction with OEH has developed guidelines to classify communities according to the impact that flooding has upon them. These Emergency Response Planning (ERP) classifications (Reference 21) consider flood affected communities as those in which the normal functioning of services is altered, either directly or indirectly, because a flood results in the need for external assistance. This impact relates directly to the operational issues of evacuation, resupply and rescue. Based on the guidelines, communities are classified as either; Flood Islands; Road Access Areas; Overland Escape Routes; Trapped Perimeter Areas or Indirectly Affected. The ERP classification can identify the type and scale of information needed by the NSW SES to assist in emergency response planning (refer to Table 45). Section 7.5.1 provides a description of each of the ERP Classification definitions.

Table 45: Emergency Response Planning Classifications of Communities

Classification	Response Required		
	Resupply	Rescue/Medivac	Evacuation
High flood island	Yes	Possibly	Possibly
Low flood island	No	Yes	Yes
Area with rising road access	No	Possibly	Yes
Area with overland escape routes	No	Possibly	Yes
Low trapped perimeter	No	Yes	Yes
High trapped perimeter	Yes	Possibly	Possibly
Indirectly affected areas	Possibly	Possibly	Possibly

Key considerations for flood emergency response planning in these areas include:

- Cutting of external access isolating an area;
- Key internal roads being cut;
- Transport infrastructure being shut down or unable to operate at maximum efficiency;
- Flooding of any key response infrastructure such as hospitals, evacuation centres, emergency services sites;
- Risk of flooding to key public utilities such as gas, power, sewerage; and
- The extent of the area flooded.

Figure 54 shows the ERP classifications for Goulburn for the PMF event. This has been determined by examining design flood results up to and including the PMF. This figure shows that areas outside of the Wollondilly and Mulwaree River floodplains but within the in township of Goulburn are indirectly flood affected as access to basic services (hospital etc.) is available, however sewerage may be affected. To the east of the Mulwaree River in Eastgrove and surrounds, areas not flooded are classified as ‘High Trapped Perimeter Area’ as road access to the region will be restricted during a PMF event thus removing access to basic services.

Areas on the Wollondilly and Mulwaree River floodplains have various classifications but are typically classed as ‘Rising Road Access’ where road access to the Goulburn township is not cut. To the west of the Wollondilly/Mulwaree River confluence in the township of Goulburn, high risk ERP classifications including ‘Low Flood Island’, ‘High Flood Island’ and ‘Overland Refuge Area on High Flood Island’, exist due to flood waters breaking through town from the Wollondilly towards the Mulwaree River causing road access to be cut early on which is followed by subsequent flooding of the area. The Mulwaree River floodplain is classified as ‘Low Trapped Perimeter Area’, ‘Low Flood Island’ and ‘Overland Refuge Area on High Trapped Perimeter Area’, again due to flooding of access roads.

This same process has been performed for the 5% and 1% AEP events with the ERP classification presented in Figure 52 and Figure 53 respectively.

7.5.1. ERP Classification Definitions

High Flood Island - The flood island includes enough land higher than the limit of flooding (i.e. above the PMF) to cope with the number of people in the area. During a flood event the area is surrounded by floodwater and property may be inundated. However, there is an opportunity for people to retreat to higher ground above the PMF within the island and therefore the direct risk to life is limited. The area will require resupply by boat or air if not evacuated before the road is cut. If it will not be possible to provide adequate support during the period of isolation, evacuation will have to take place before isolation occurs.

Low Flood Island - The flood island is lower than the limit of flooding (i.e. below the PMF) or does not have enough land above the limit of flooding to cope with the number of people in the area. During a flood event the area is isolated by floodwater and property will be inundated. If floodwater continues to rise after it is isolated, the island will eventually be completely covered. People left stranded on the island may drown and property will be inundated.

High Trapped Perimeter Area - The inhabited or potentially inhabited area includes enough land to cope with the number of people in the area that is higher than the limit of flooding (i.e. above the PMF). During a flood event the area is isolated by floodwater and property and may be inundated. However, there is an opportunity for people to retreat to higher ground above the PMF within the area and therefore the direct risk to life is limited. The area will require resupply by boat or air if not evacuated before the road is cut. If it will not be possible to provide adequate support during the period of isolation, evacuation will have to take place before isolation occurs.

Low Trapped Perimeter Area - The inhabited or potentially inhabited area is lower than the limit of flooding (i.e. below the PMF) or does not have enough land above the limit of flooding to cope with the number of people in the area. During a flood event the area is isolated by floodwater and property may be inundated. If floodwater continues to rise after it is isolated, the area will eventually be completely covered. People trapped on the island may drown.

Areas with Overland Escape Route - are those areas where access roads to flood free land cross lower lying flood prone land. Evacuation can take place by road only until access roads are closed by floodwater. Escape from rising floodwater is possible but by walking overland to higher ground. Anyone not able to walk out must be reached by using boats and aircraft. If people cannot get out before inundation, rescue will most likely be from rooftops.

Areas with Rising Road Access - are those areas where access roads rising steadily uphill and away from the rising floodwaters. The community cannot be completely isolated before inundation reaches its maximum extent, even in the PMF. Evacuation can take place by vehicle or on foot along the road as floodwater advances. People should not be trapped unless they delay their evacuation from their homes. For example people living in two storey homes may initially decide to stay but reconsider after water surrounds them.

Indirectly Affected Areas - are areas which are outside the limit of flooding and therefore will not be inundated nor will they lose road access. However, they may be indirectly affected as a result of flood damaged infrastructure or due to the loss of transport links, electricity supply, water supply, sewage or telecommunications services and they may therefore require resupply or in the worst case, evacuation

Overland Refuge Areas - are areas that other areas of the floodplain may be evacuated to, at least temporarily, but which are isolated from the edge of the floodplain by floodwaters and are therefore effectively flood islands or trapped perimeter areas. They should be categorised accordingly and these categories used to determine their vulnerability.

8. CONCLUSION

The Goulburn Flood Study presented herein has been prepared by WMAwater on behalf of the Goulburn Mulwaree Council (Council) and constitutes the first and second stages of the NSW Floodplain Risk Management Program for Goulburn. The Study considered flooding in Goulburn from the Wollondilly River and Mulwaree River.

As part of this study hydrologic and hydraulic models were developed and calibrated/validated to historic flood information. The calibrated/validated models have been used to define design flood behaviour.

The information and results obtained from this study define design flood behaviour at Goulburn and provide a firm basis for the development of a subsequent Floodplain Risk Management Study and Plan (FRMS&P).

9. ACKNOWLEDGEMENTS

WMAwater would like to acknowledge the assistance of Goulburn Mulwaree Council staff in carrying out this work as well as The Office of Environment and Heritage.

Goulburn Mulwaree Council has prepared this document with financial assistance from the NSW Government through its Floodplain Management Program. This document does not necessarily represent the opinions of the NSW Government or the Office of Environment and Heritage.

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Figures

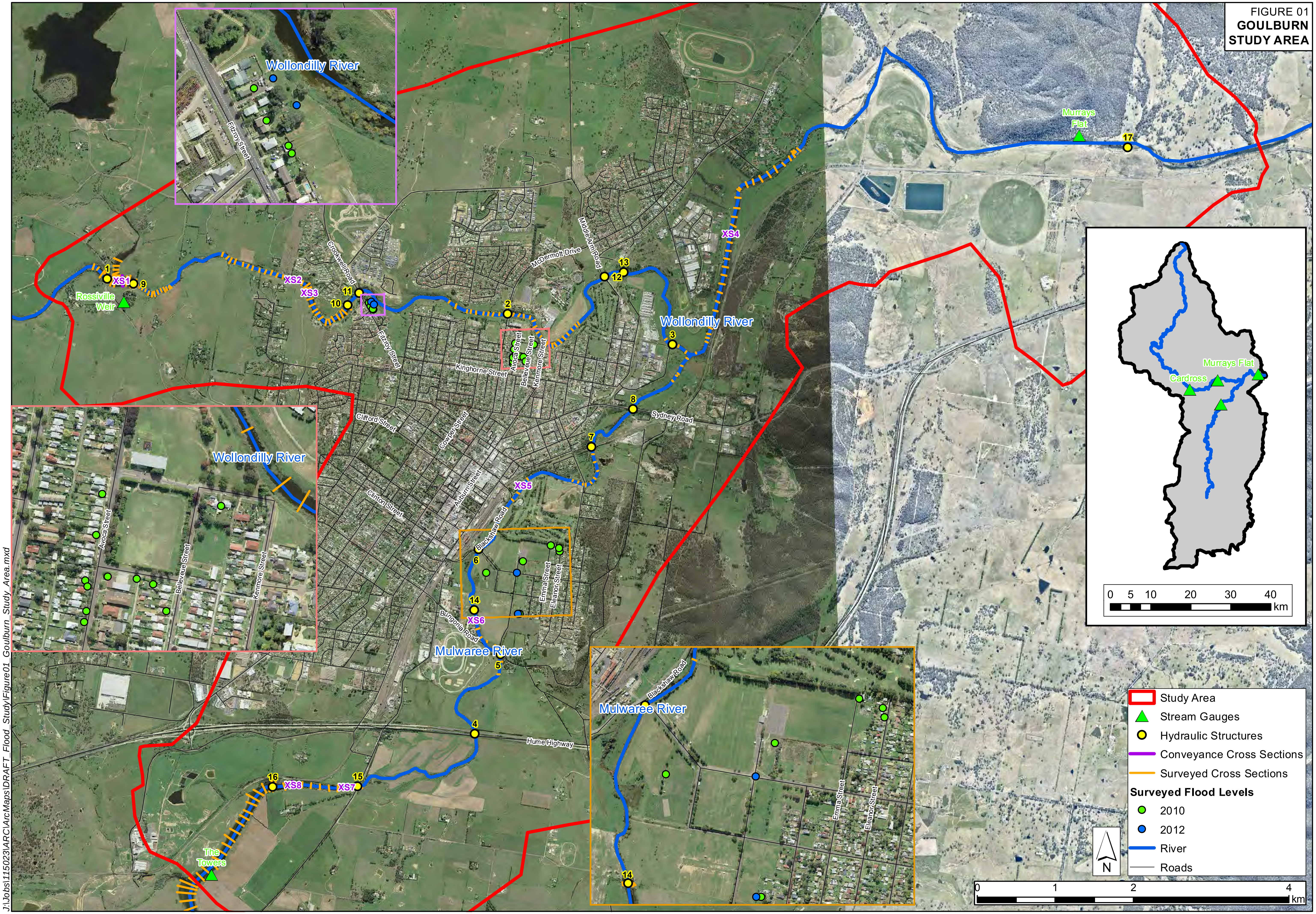

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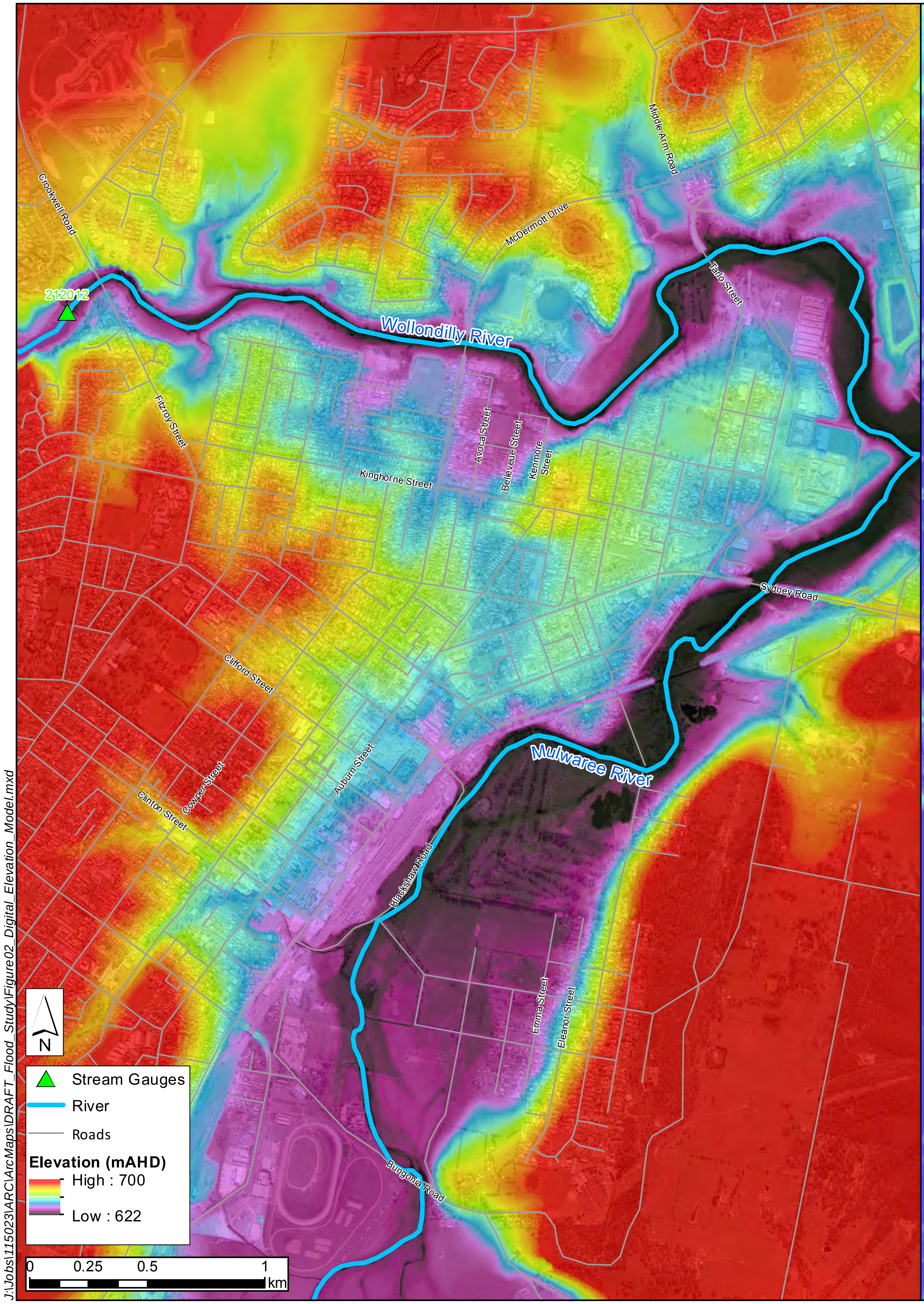
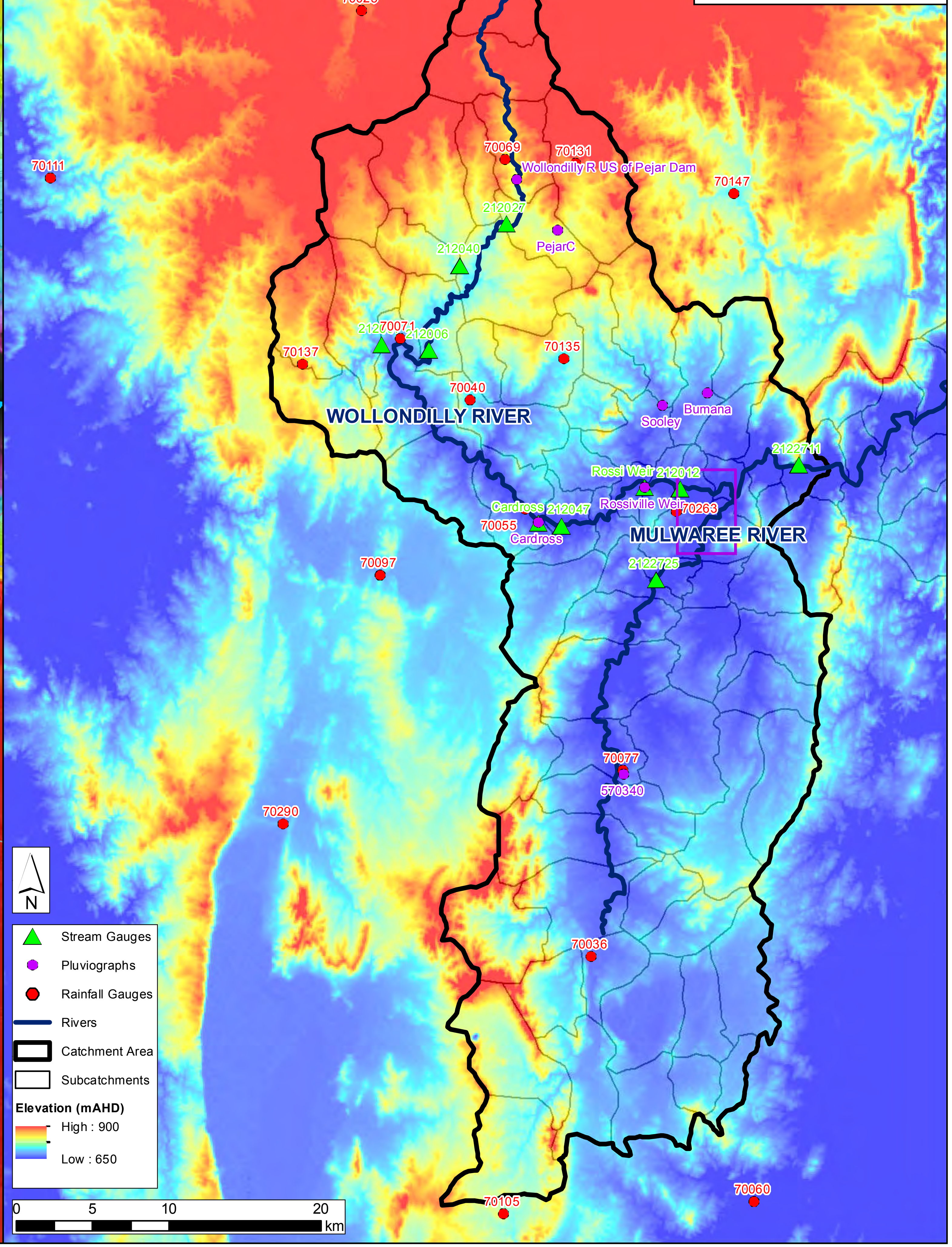
FIGURE 02
DIGITAL ELEVATION MODEL





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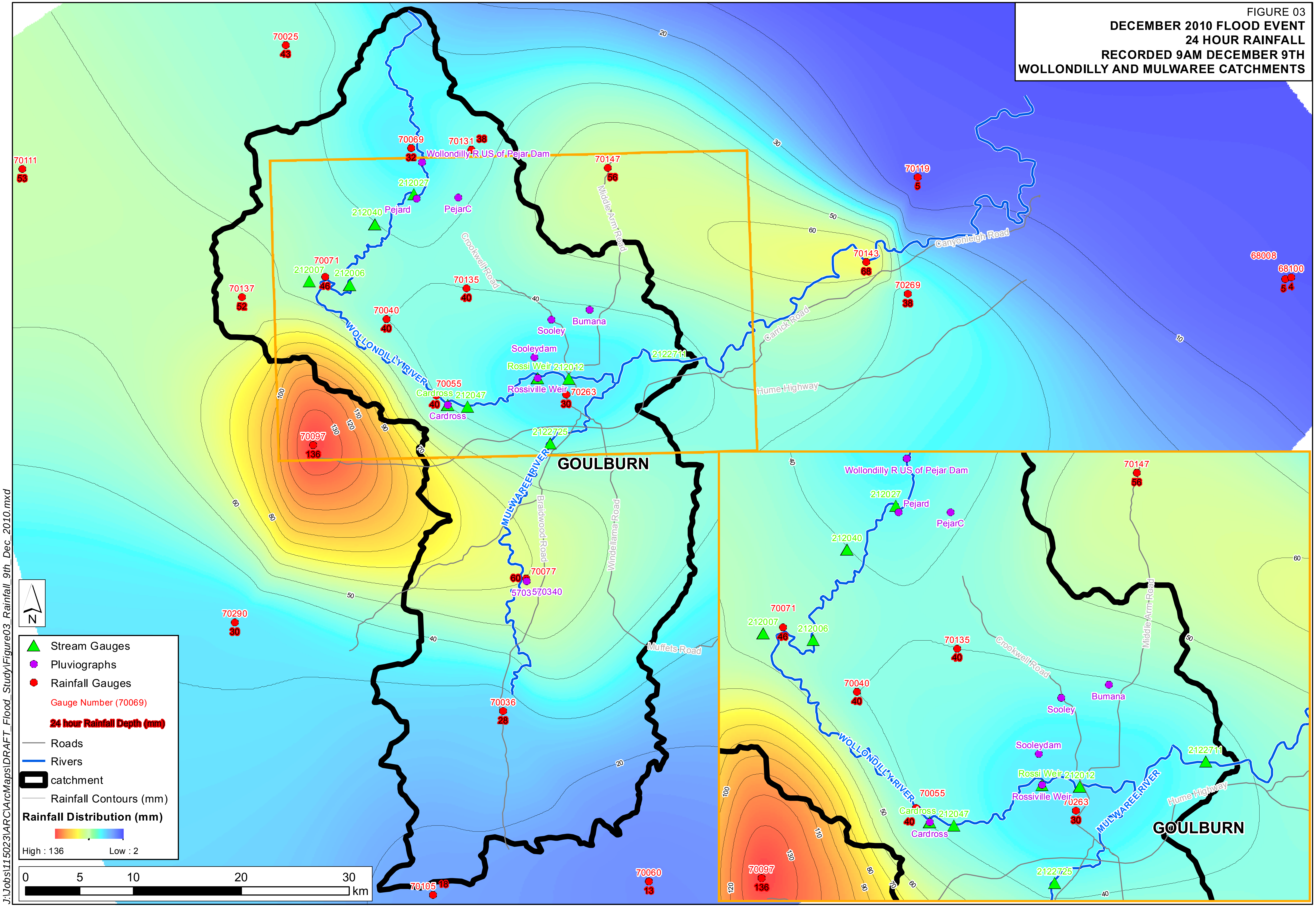
FIGURE 03
DECEMBER 2010 FLOOD EVENT
24 HOUR RAINFALL
RECORDED 9AM DECEMBER 9TH
WOLLONDILLY AND MULWAREE CATCHMENTS
GOULBURN
WOLLONDILLY RIVER
MULWAREE RIVER
Wollondilly R US of Pejar Dam
Pejard
PejarC
Bumana
Sooley
Sooleydam
Rossi Weir
Rossiville Weir
Cardross
Crookwell Road
Middle Arm Road
Canyonleigh Road
Carrick Road
Hume Highway
Braidwood Road
Windellama Road
Muffets Road
Stream Gauges
Pluviographs
Rainfall Gauges
Gauge Number (70069)
24 hour Rainfall Depth (mm)
Roads
Rivers
catchment
Rainfall Contours (mm)
Rainfall Distribution (mm)
High : 136
Low : 2
0 5 10 20 30 km
N
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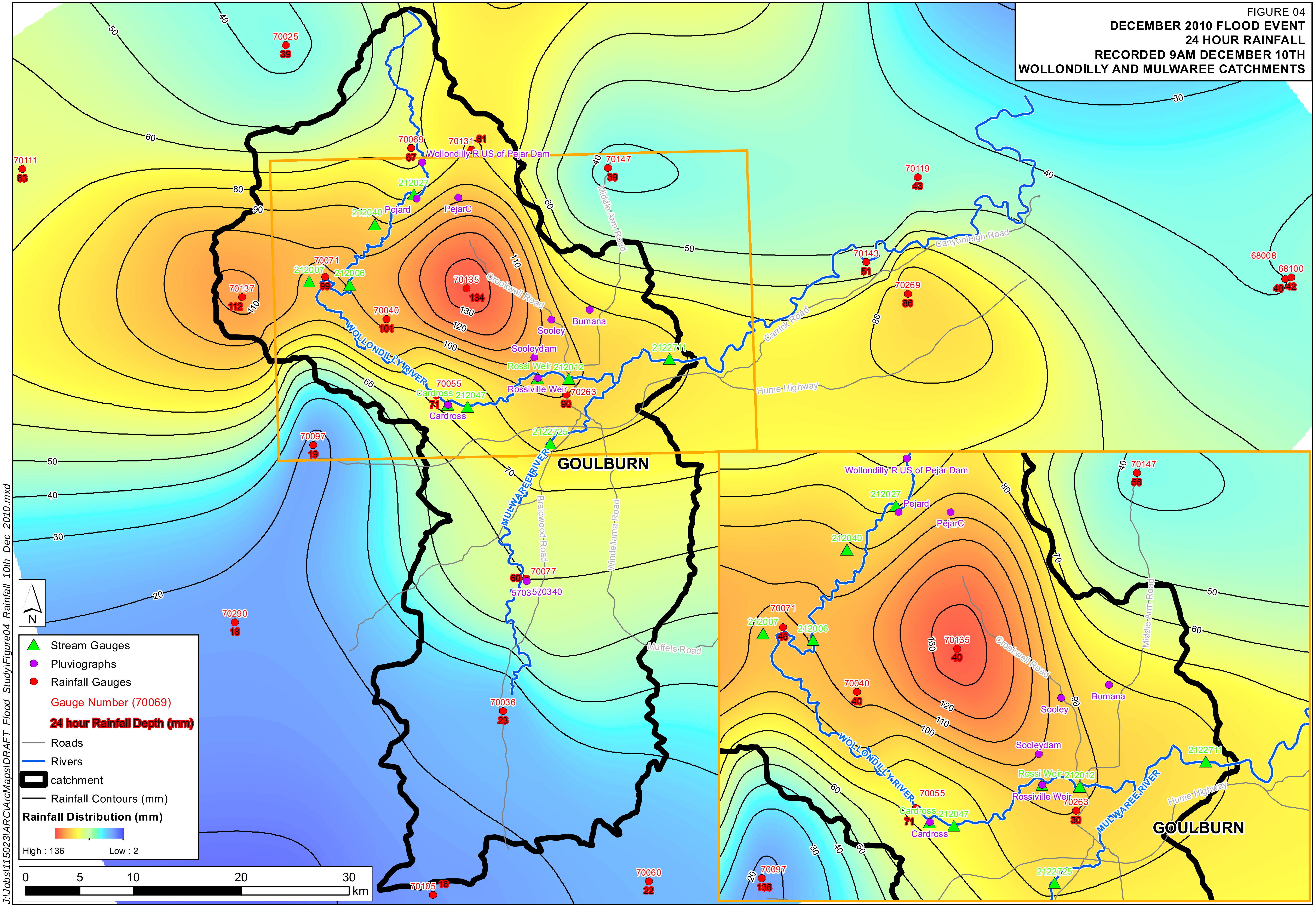

FIGURE 04
DECEMBER 2010 FLOOD EVENT
24 HOUR RAINFALL
RECORDED 9AM DECEMBER 10TH
WOLLONDILLY AND MULWAREE CATCHMENTS
GOULBURN
WOLLONDILLY RIVER
MULWAREE RIVER
Stream Gauges
Pluviographs
Rainfall Gauges
Gauge Number (70069)
24 hour Rainfall Depth (mm)
Roads
Rivers
catchment
Rainfall Contours (mm)
Rainfall Distribution (mm)
High : 136
Low : 2
0 5 10 20 30 km
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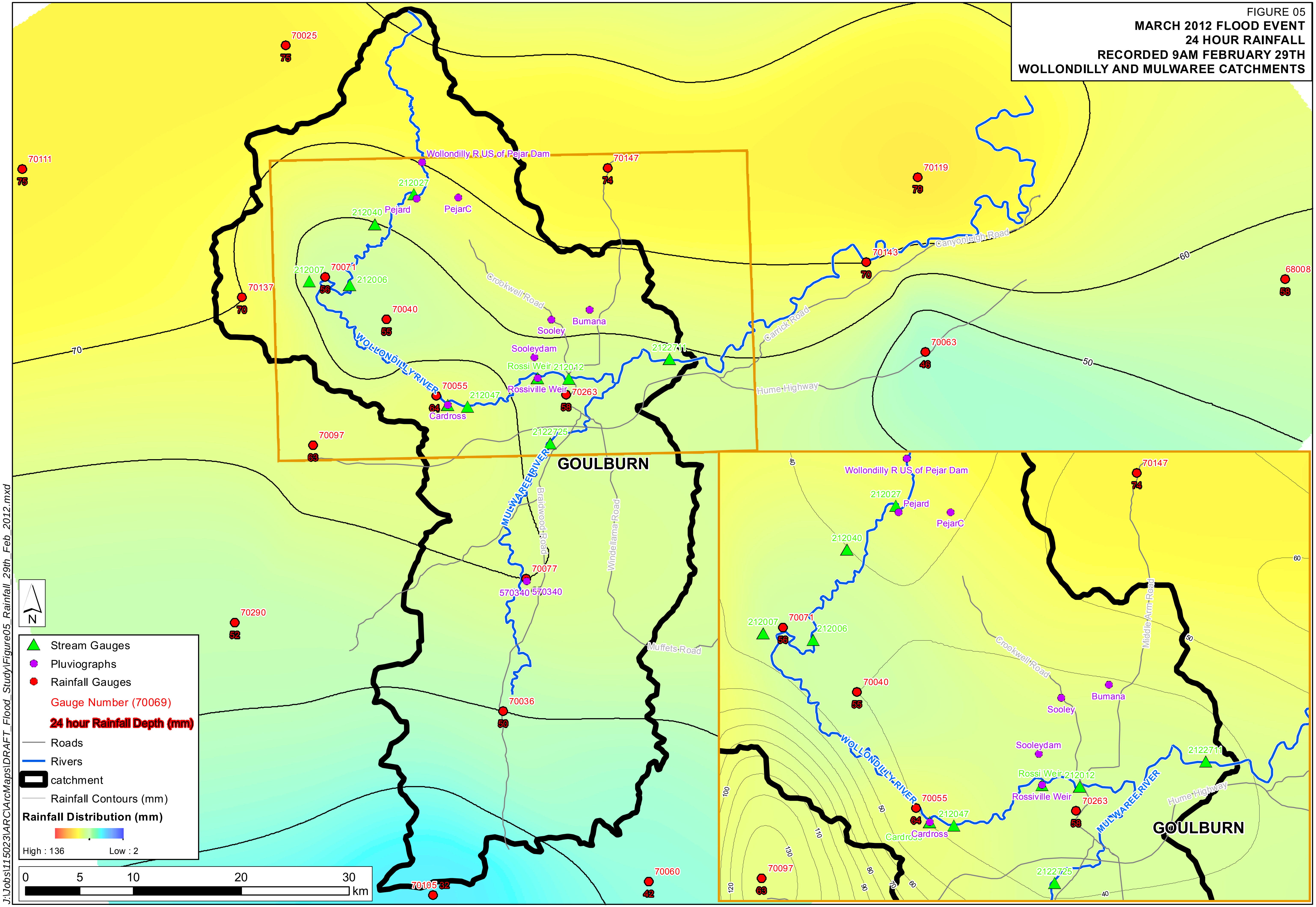

FIGURE 05
MARCH 2012 FLOOD EVENT
24 HOUR RAINFALL
RECORDED 9AM FEBRUARY 29TH
WOLLONDILLY AND MULWAREE CATCHMENTS

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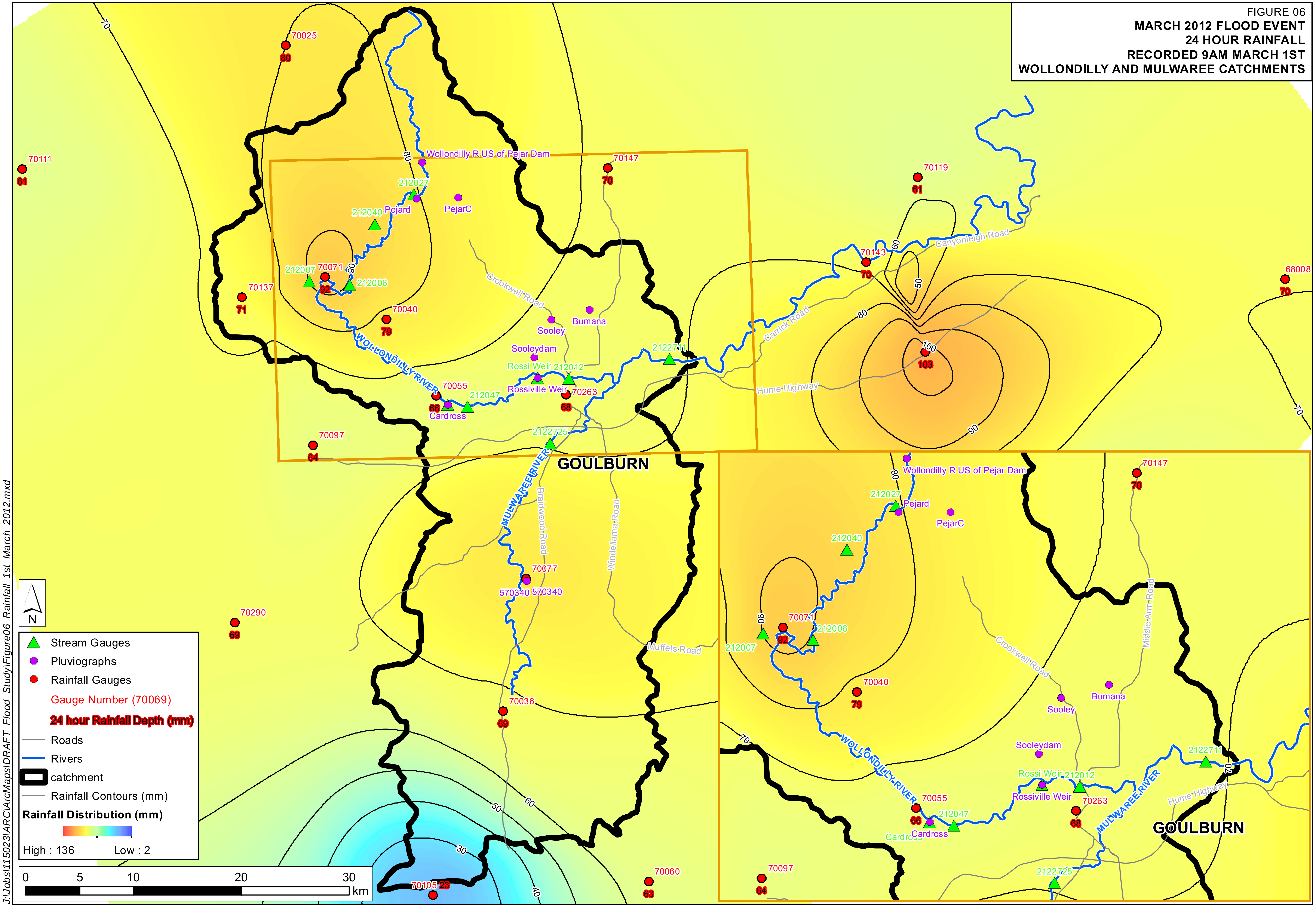

FIGURE 06
MARCH 2012 FLOOD EVENT
24 HOUR RAINFALL
RECORDED 9AM MARCH 1ST
WOLLONDILLY AND MULWAREE CATCHMENTS
GOULBURN
WOLLONDILLY RIVER
MULWAREE RIVER
Wollondilly R US of Pejar Dam
Pejard
PejarC
Bumana
Sooley
Sooleydam
Rossi Weir
Rossiville Weir
Cardross
Crookwell Road
Carrick Road
Canyonleigh Road
Hume Highway
Braidwood Road
Windellama Road
Muffets Road
Middle Arm Road
Stream Gauges
Pluviographs
Rainfall Gauges
Gauge Number (70069)
24 hour Rainfall Depth (mm)
Roads
Rivers
catchment
Rainfall Contours (mm)
Rainfall Distribution (mm)
High : 136
Low : 2
0 5 10 20 30 km
N
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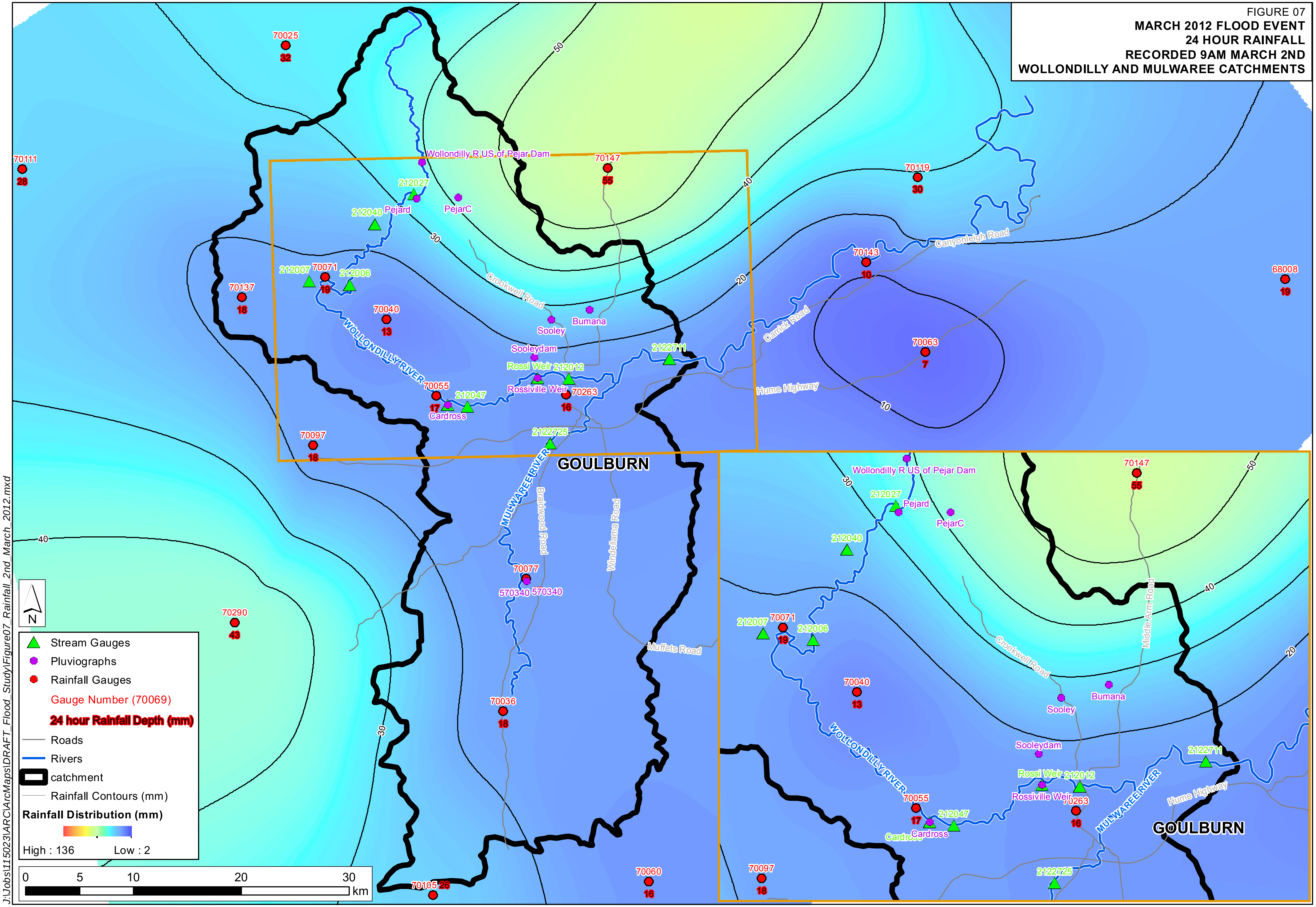
FIGURE 07
MARCH 2012 FLOOD EVENT
24 HOUR RAINFALL
RECORDED 9AM MARCH 2ND
WOLLONDILLY AND MULWAREE CATCHMENTS
Stream Gauges
Pluviographs
Rainfall Gauges
Gauge Number (70069)
24 hour Rainfall Depth (mm)
Roads
Rivers
catchment
Rainfall Contours (mm)
Rainfall Distribution (mm)
High : 136
Low : 2
0 5 10 20 30 km
GOULBURN
WOLLONDILLY RIVER
MULWAREE RIVER
Wollondilly R US of Pejar Dam
Crookwell Road
Hume Highway
Carrick Road
Canyonleigh Road
Braidwood Road
Windellama Road
Muffets Road
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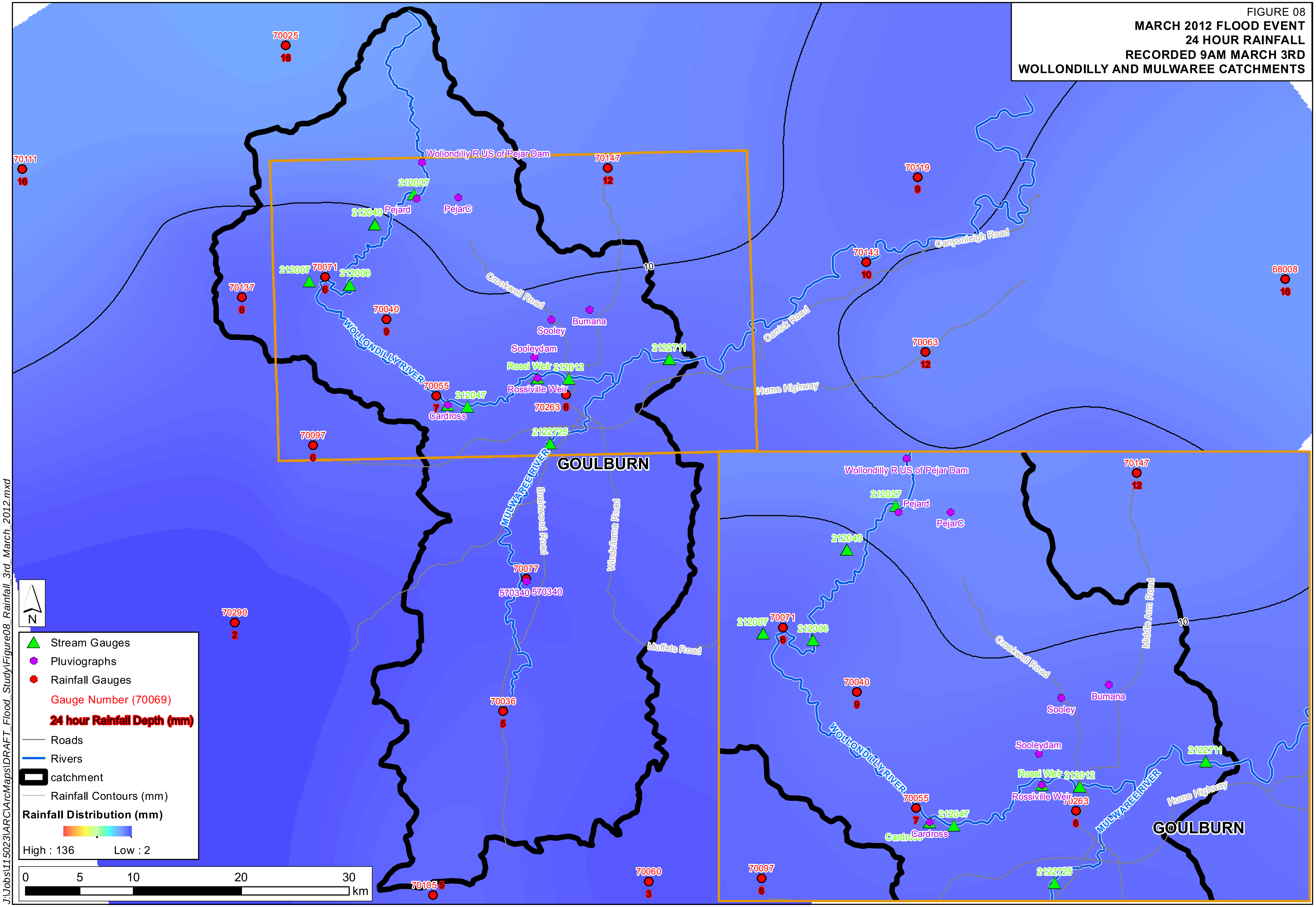

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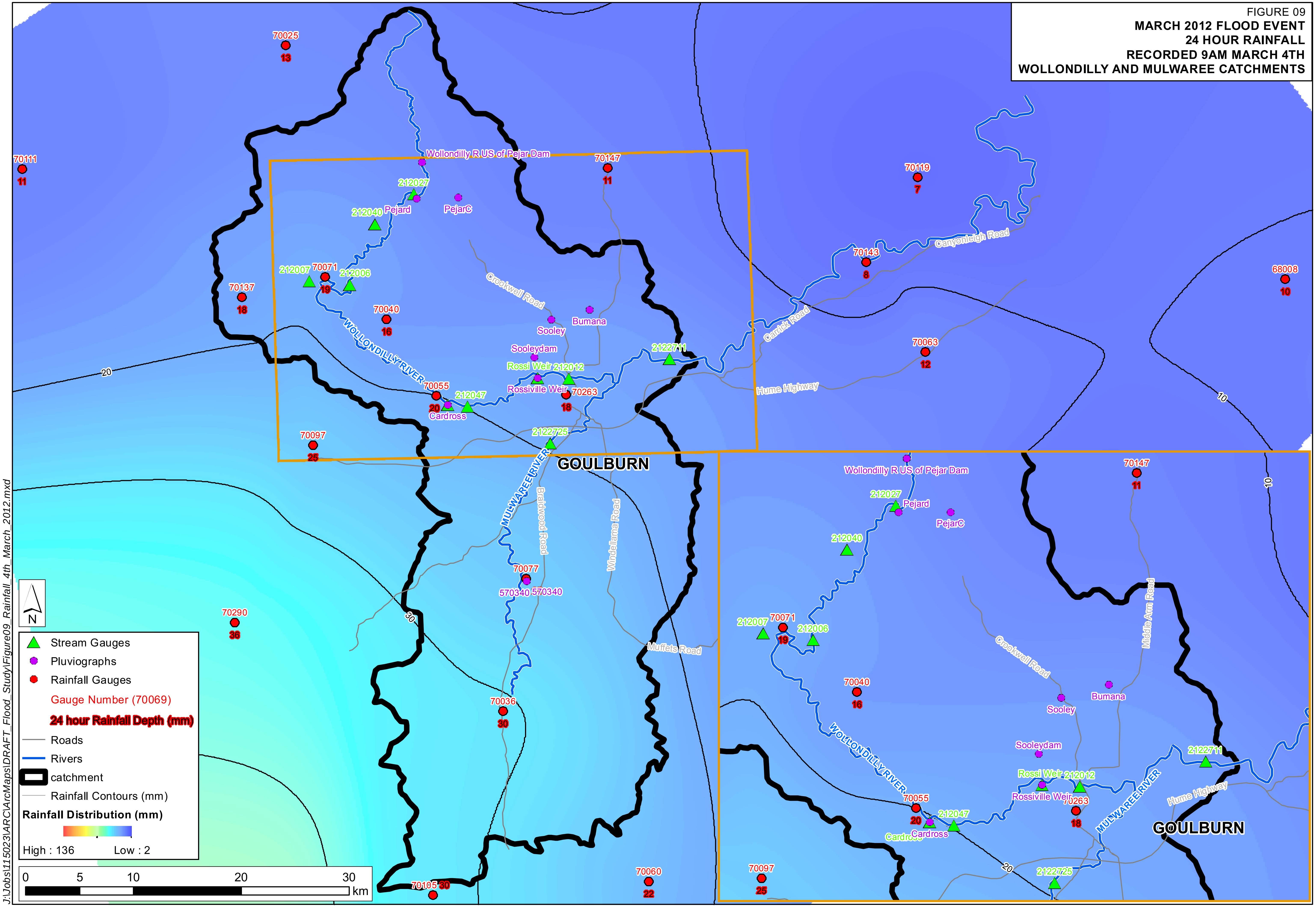

FIGURE 09
MARCH 2012 FLOOD EVENT
24 HOUR RAINFALL
RECORDED 9AM MARCH 4TH
WOLLONDILLY AND MULWAREE CATCHMENTS

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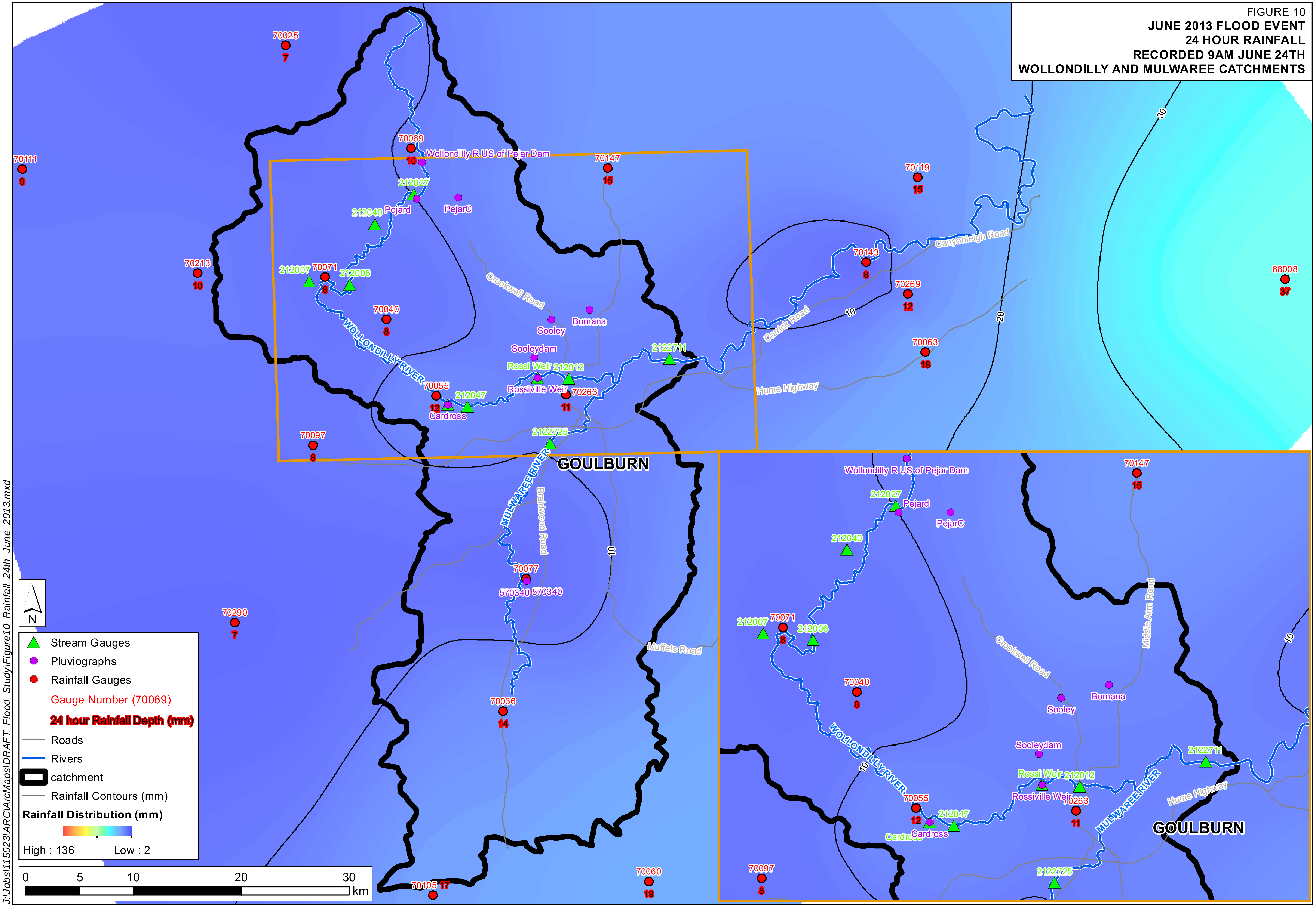
FIGURE 10
JUNE 2013 FLOOD EVENT
24 HOUR RAINFALL
RECORDED 9AM JUNE 24TH
WOLLONDILLY AND MULWAREE CATCHMENTS
GOULBURN
WOLLONDILLY RIVER
MULWAREE RIVER
Crookwell Road
Canyonleigh Road
Carrick Road
Hume Highway
Braidwood Road
Muffets Road
Middle Arm Road
Wollondilly R US of Pejar Dam
Pejard
PejarC
Sooley
Bumana
Sooleydam
Rossi Weir
Rossiville Weir
Cardross
Stream Gauges
Pluviographs
Rainfall Gauges
Gauge Number (70069)
24 hour Rainfall Depth (mm)
Roads
Rivers
catchment
Rainfall Contours (mm)
Rainfall Distribution (mm)
High : 136
Low : 2
0 5 10 20 30 km
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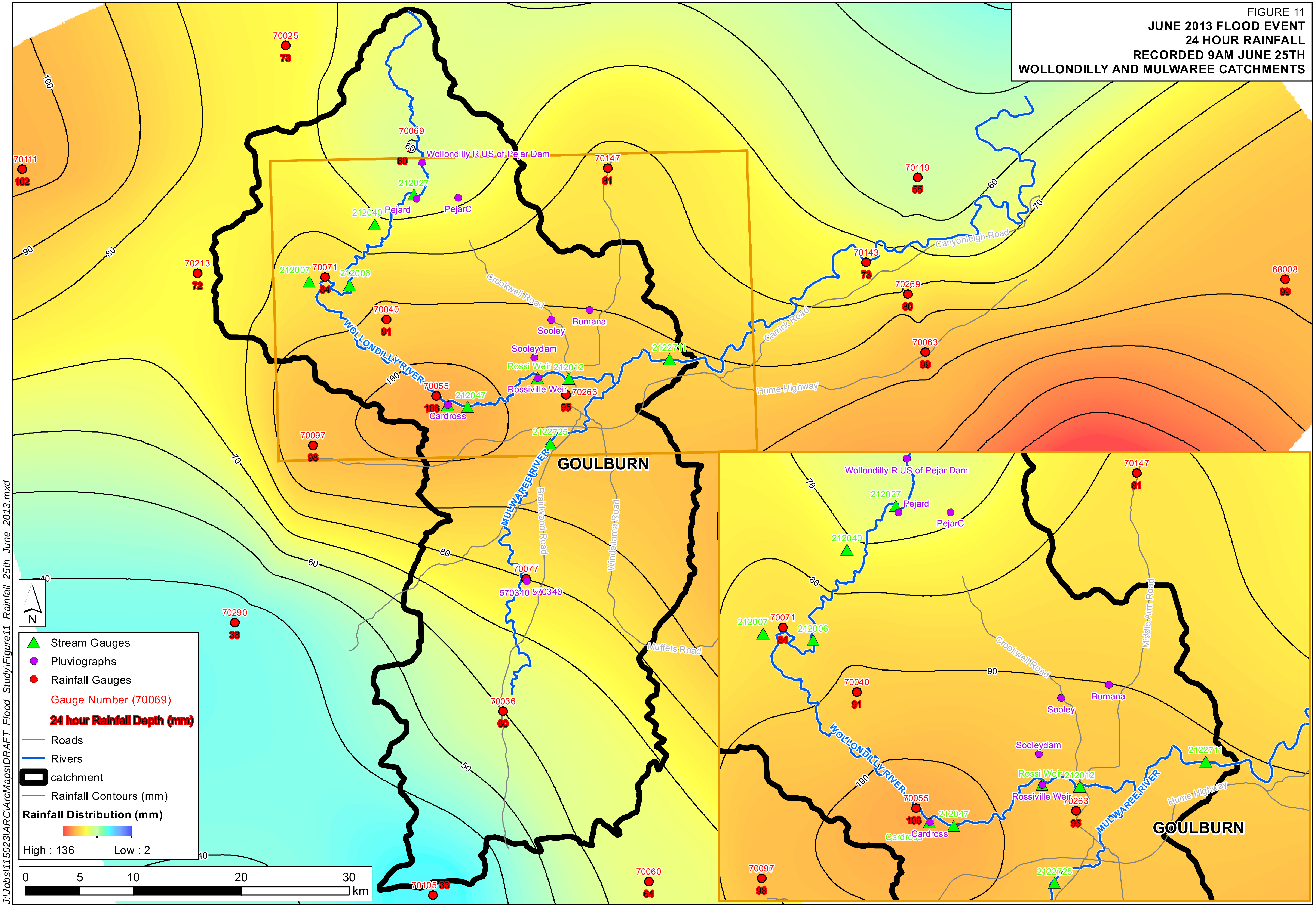


FIGURE 11
JUNE 2013 FLOOD EVENT
24 HOUR RAINFALL
RECORDED 9AM JUNE 25TH
WOLLONDILLY AND MULWAREE CATCHMENTS
GOULBURN
WOLLONDILLY RIVER
MULWAREE RIVER
Wollondilly R US of Pejar Dam
Pejard
PejarC
Sooley
Bumana
Sooleydam
Rossi Weir
Rossiville Weir
Cardross
Crookwell Road
Canyonleigh Road
Carrick Road
Hume Highway
Braidwood Road
Windellama Road
Muffets Road
Middle Arm Road
Stream Gauges
Pluviographs
Rainfall Gauges
Gauge Number (70069)
24 hour Rainfall Depth (mm)
Roads
Rivers
catchment
Rainfall Contours (mm)
Rainfall Distribution (mm)
High : 136
Low : 2
0 5 10 20 30 km
N
J:\Jobs\115023\ARC\ArcMaps\DRAFT_Flood_Study\Figure11_Rainfall_25th_June_2013.mxd

FIGURE 12
JUNE 2013 FLOOD EVENT
24 HOUR RAINFALL
RECORDED 9AM JUNE 26TH
WOLLONDILLY AND MULWAREE CATCHMENTS

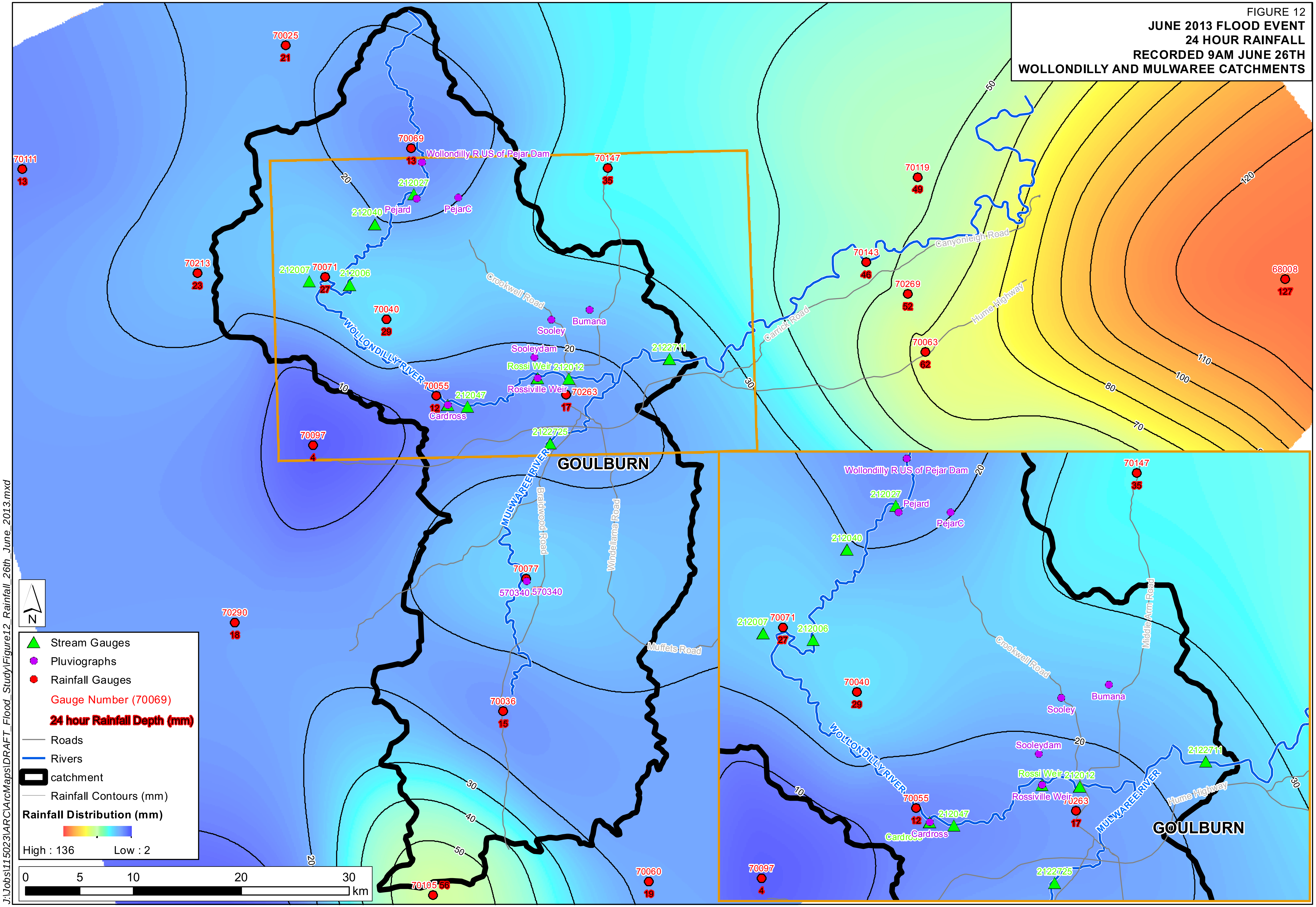
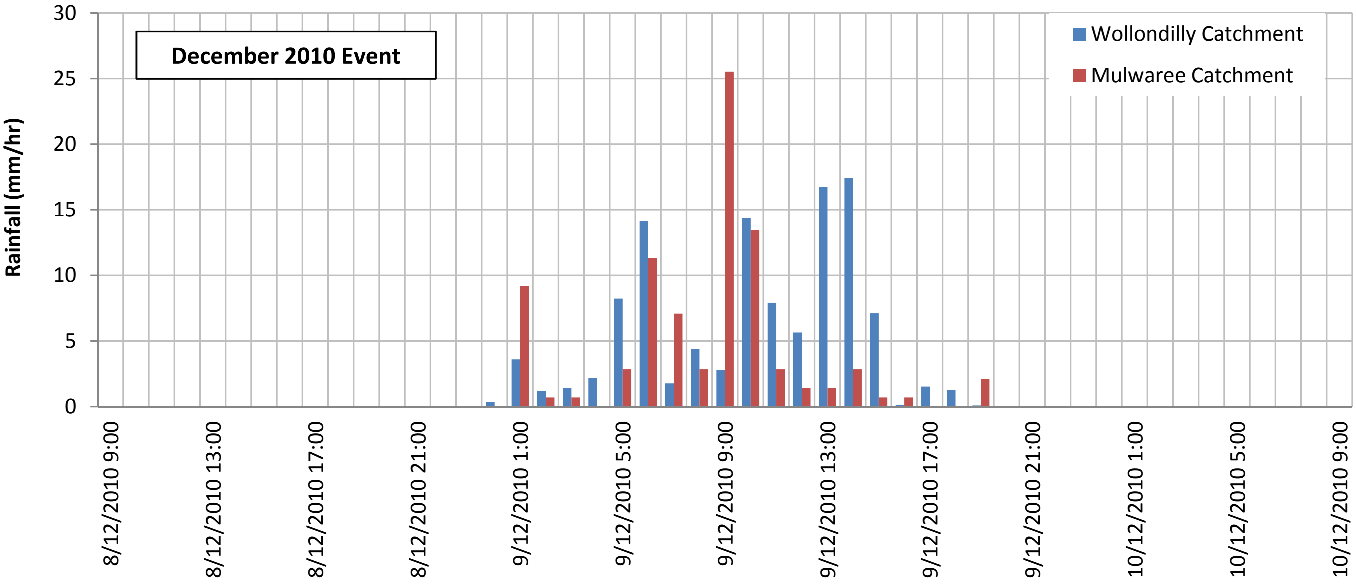
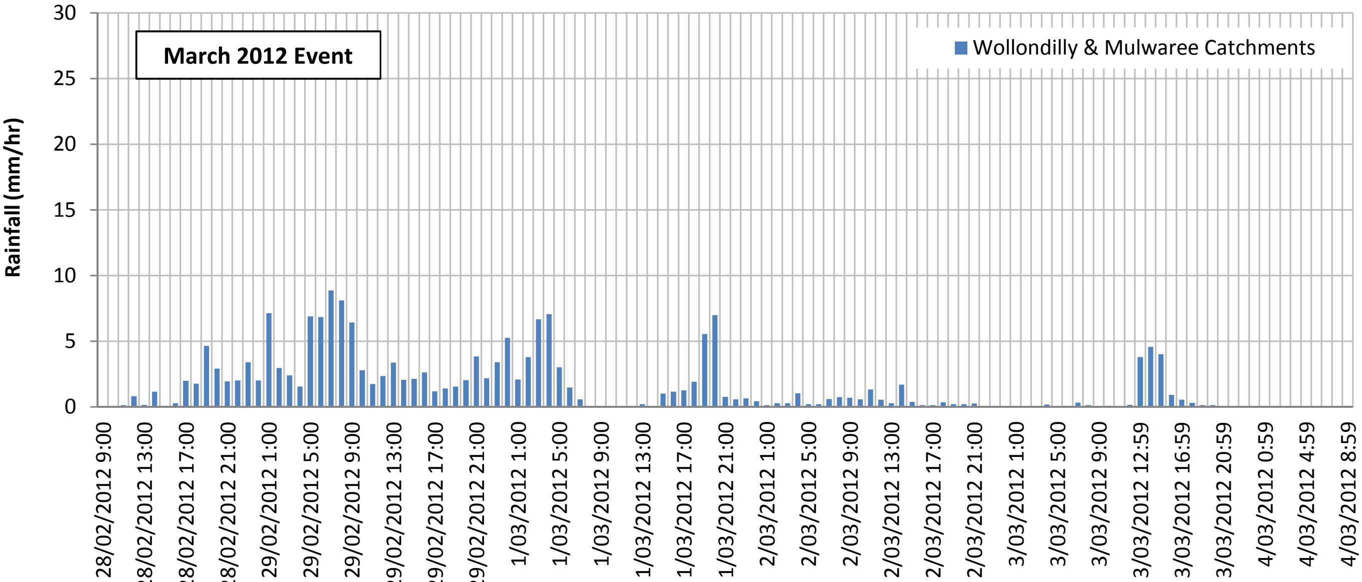


FIGURE 13
HISTORIC EVENTS RAINFALL HYETOGRAPHS





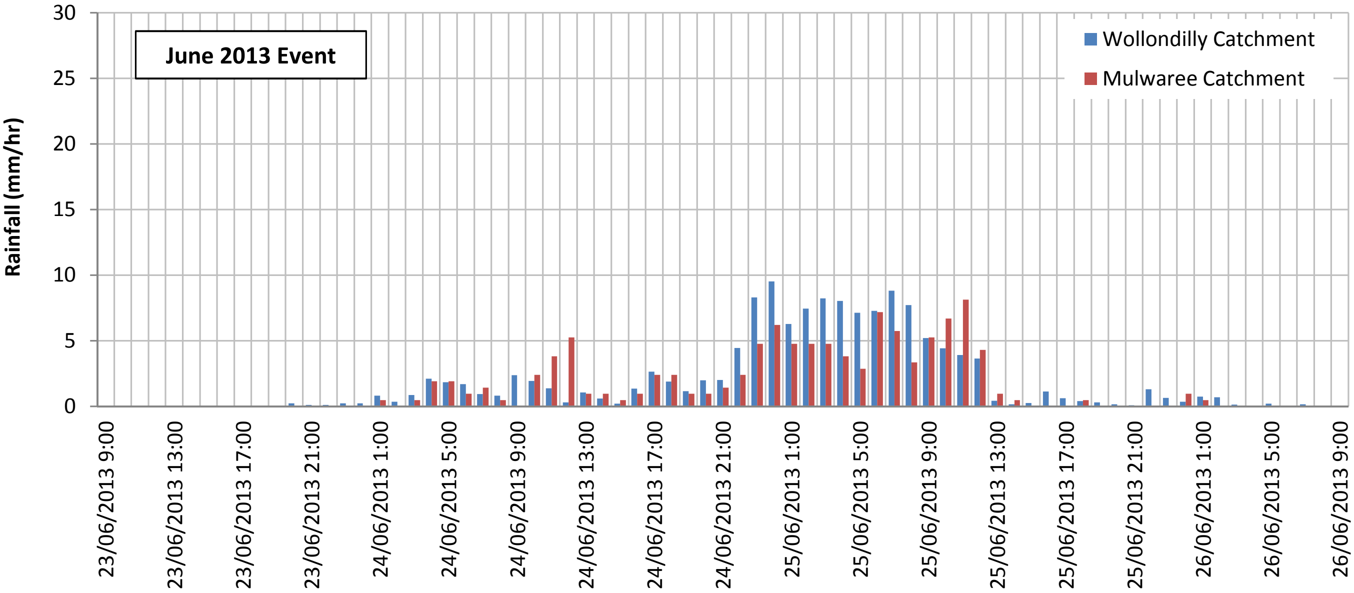


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FIGURE 14
WOLLONDILLY RIVER CATCHMENT PMP GSDM 6 HR CRITICAL DURATION

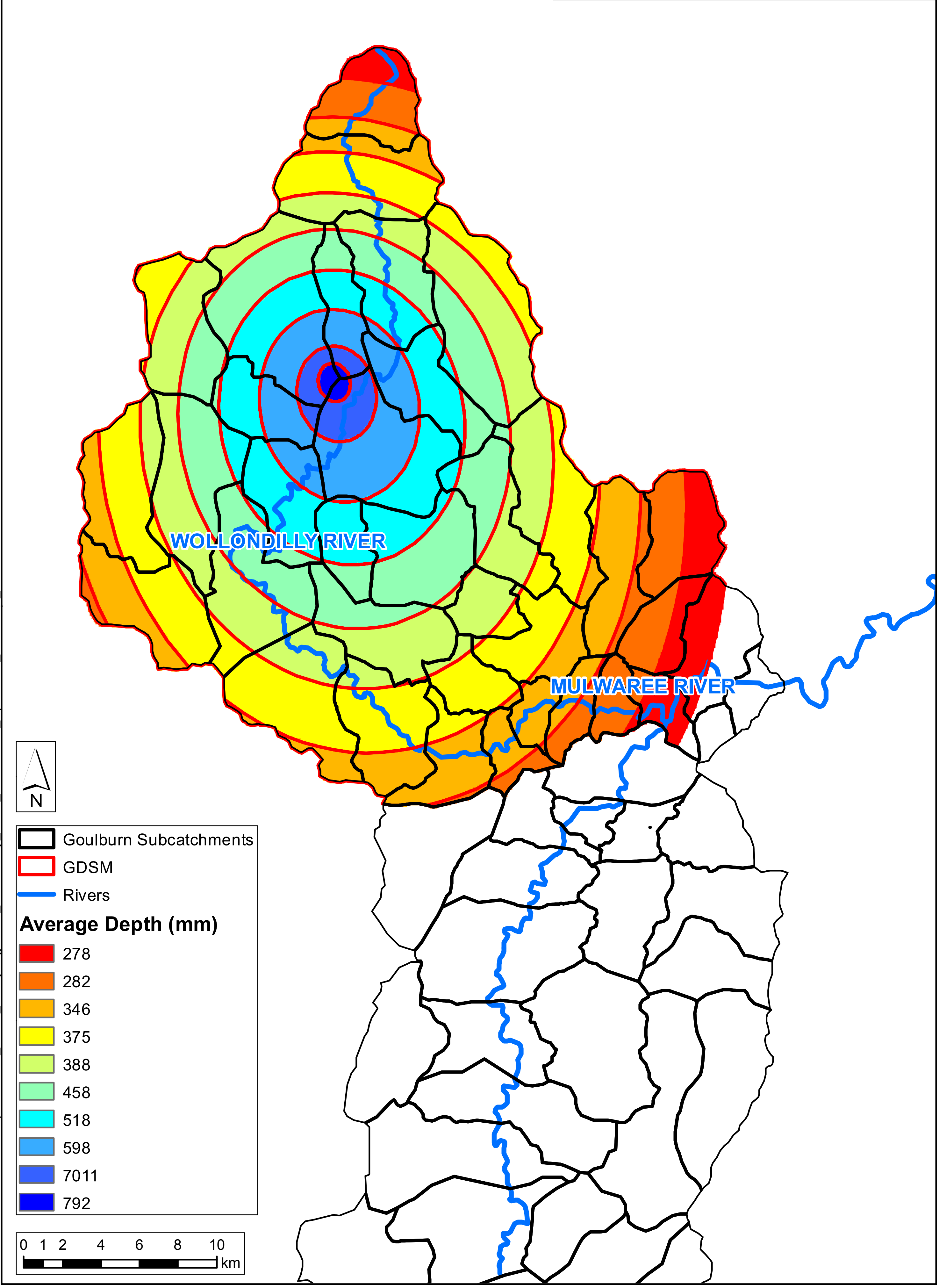

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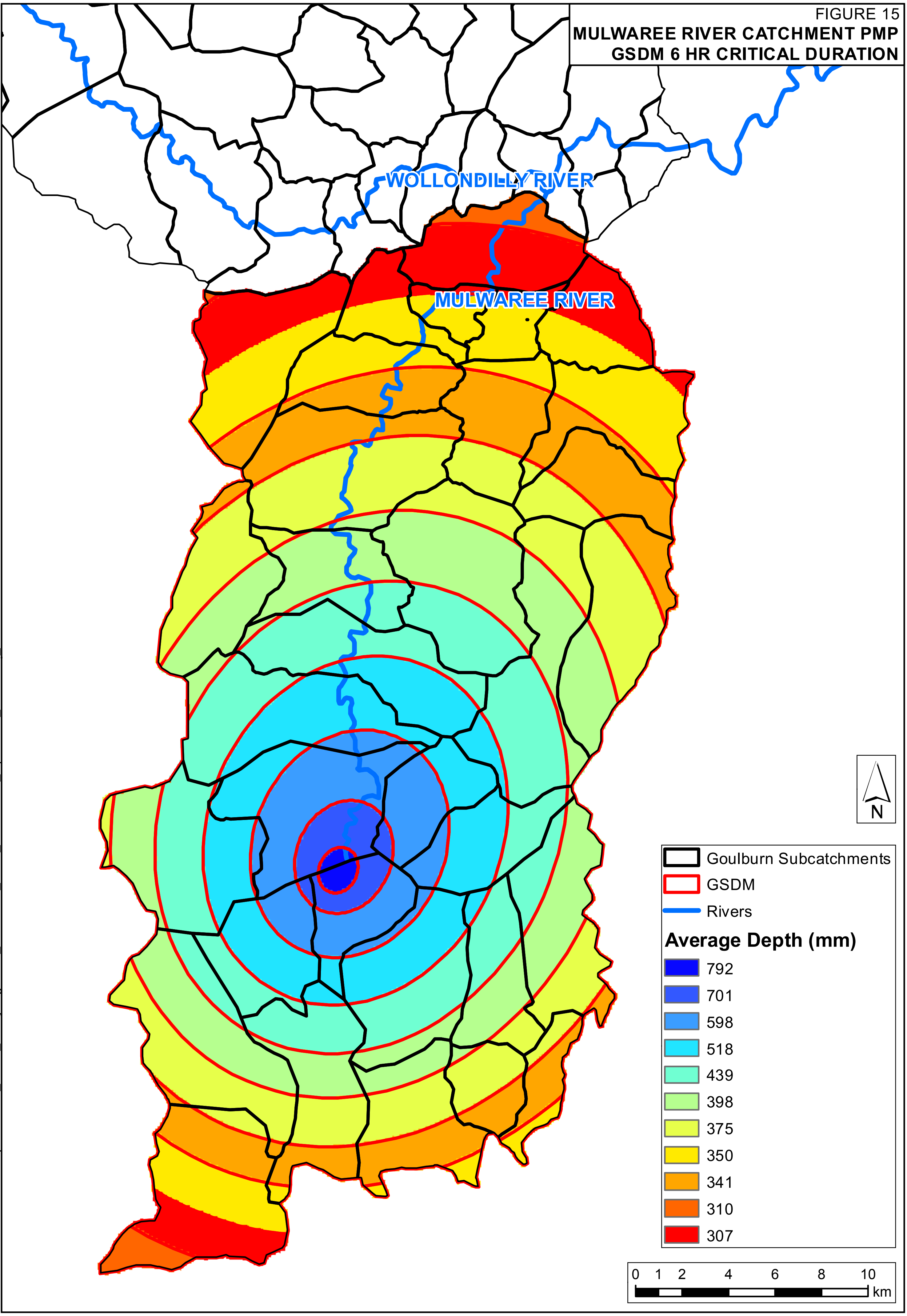
FIGURE 15
MULWAREE RIVER CATCHMENT PMP
GSDM 6 HR CRITICAL DURATION



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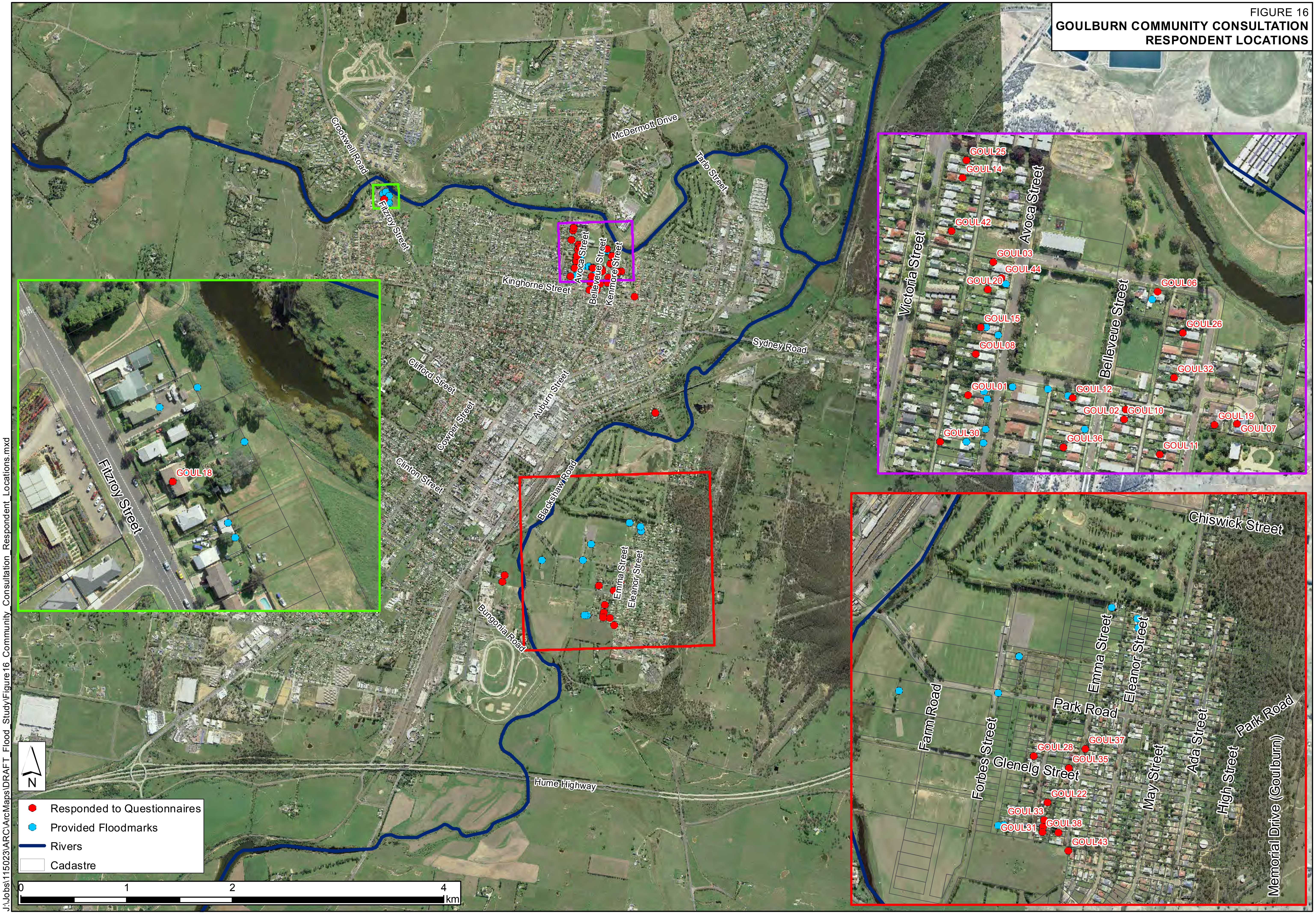
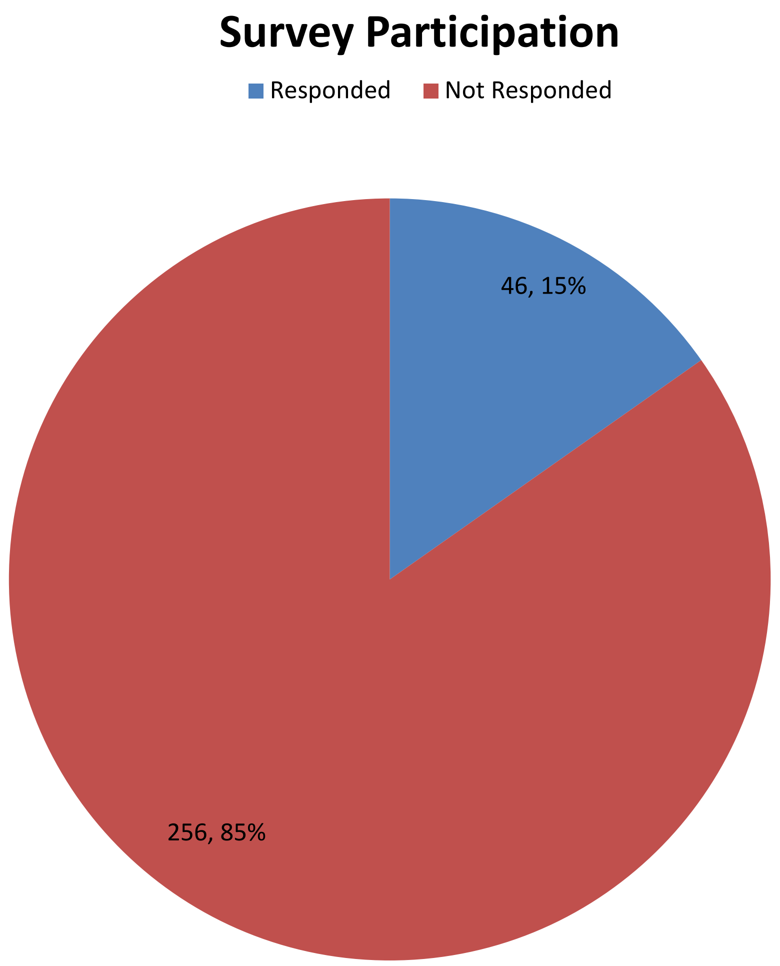


FIGURE 16
GOULBURN COMMUNITY CONSULTATION
RESPONDENT LOCATIONS

FIGURE 17a
GOULBURN
SURVEY SUMMARY



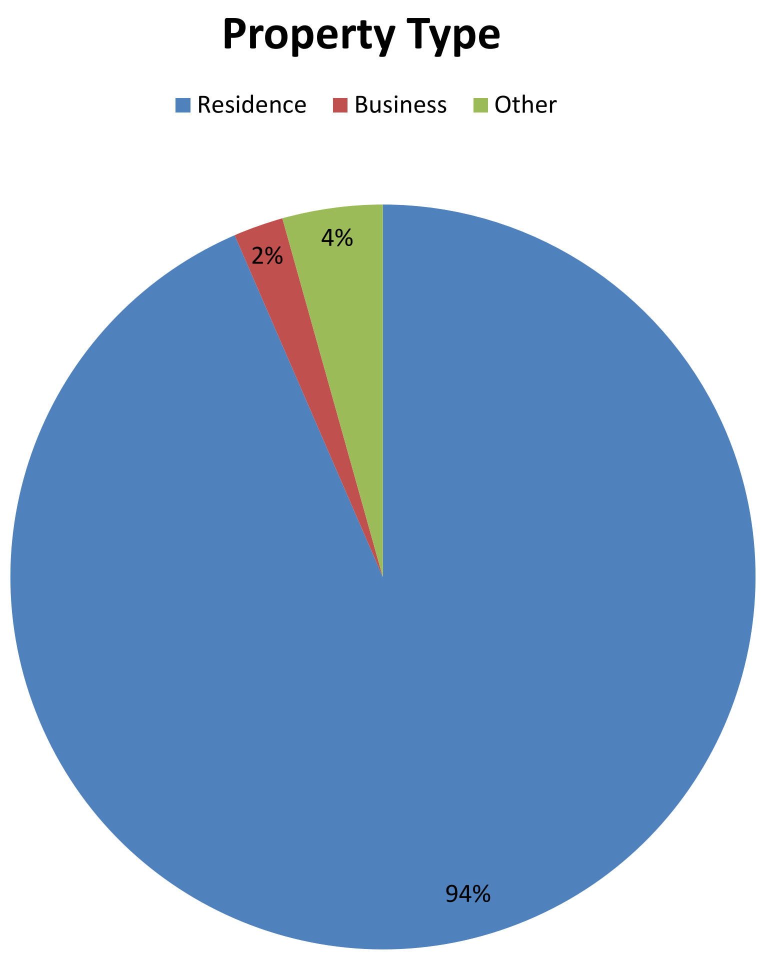


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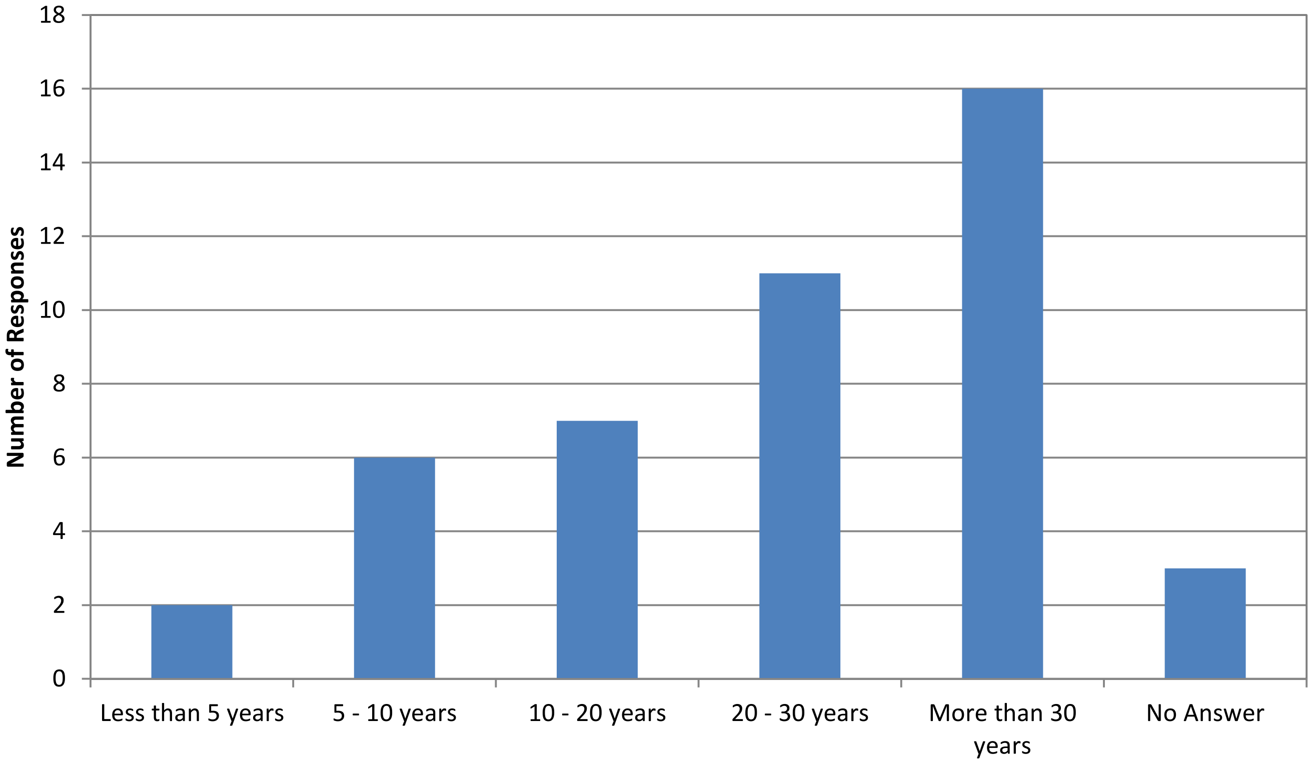
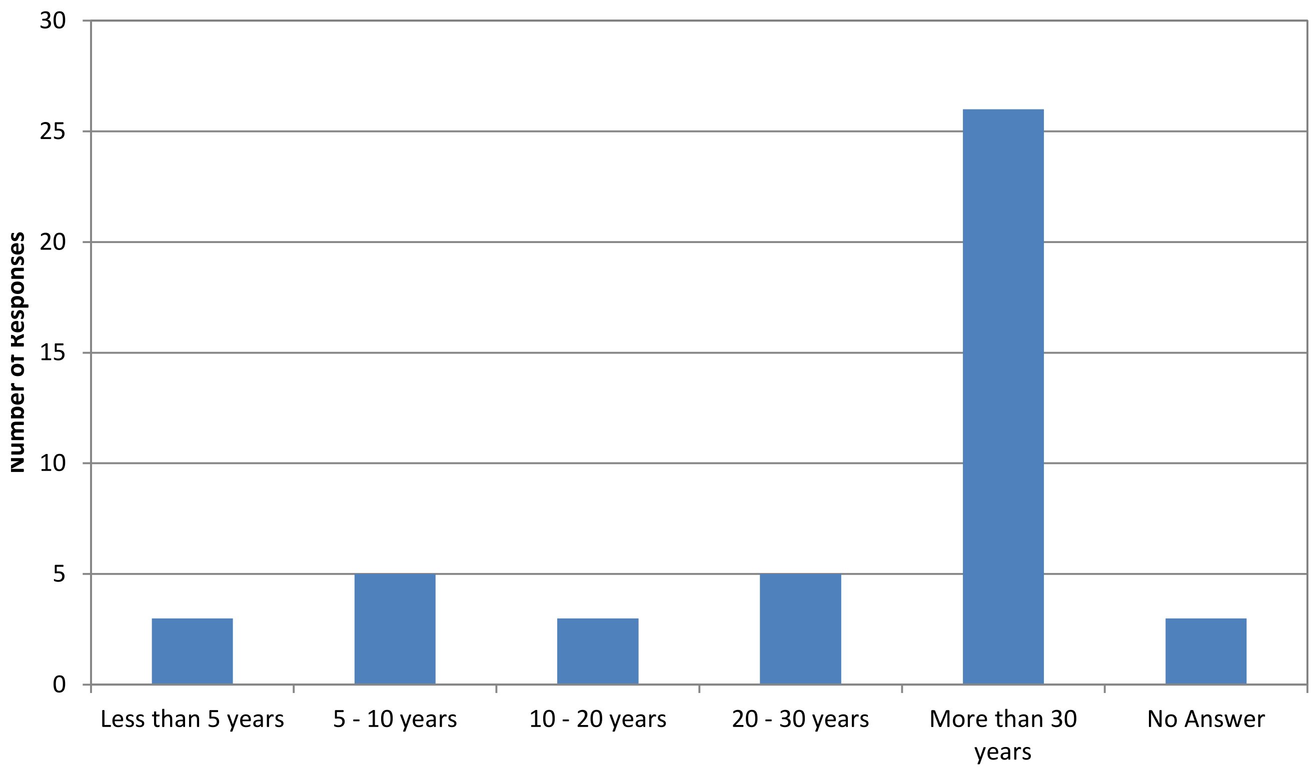
FIGURE 17b
GOULBURN
SURVEY SUMMARY

How long have you lived or worked at this address?



How long have you lived in the area?



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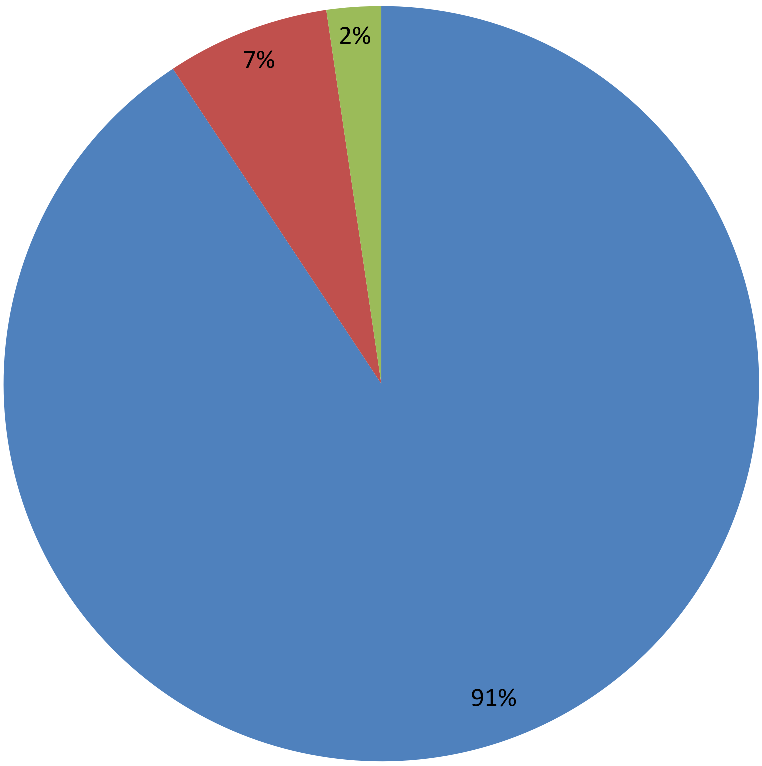
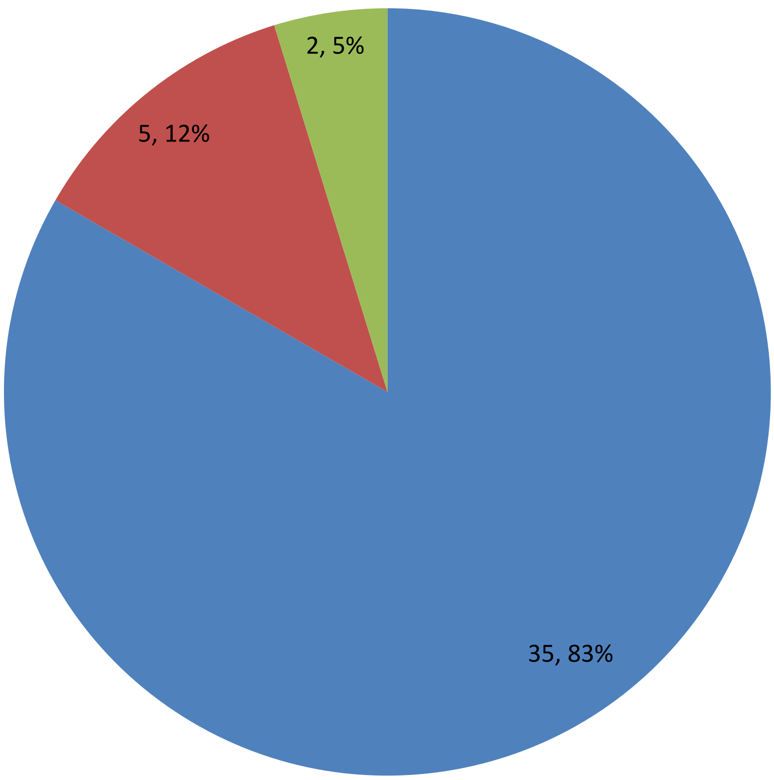
Are you aware of the Wollondilly River flooding at Goulburn?

Very Aware | Some awareness | Not aware at all

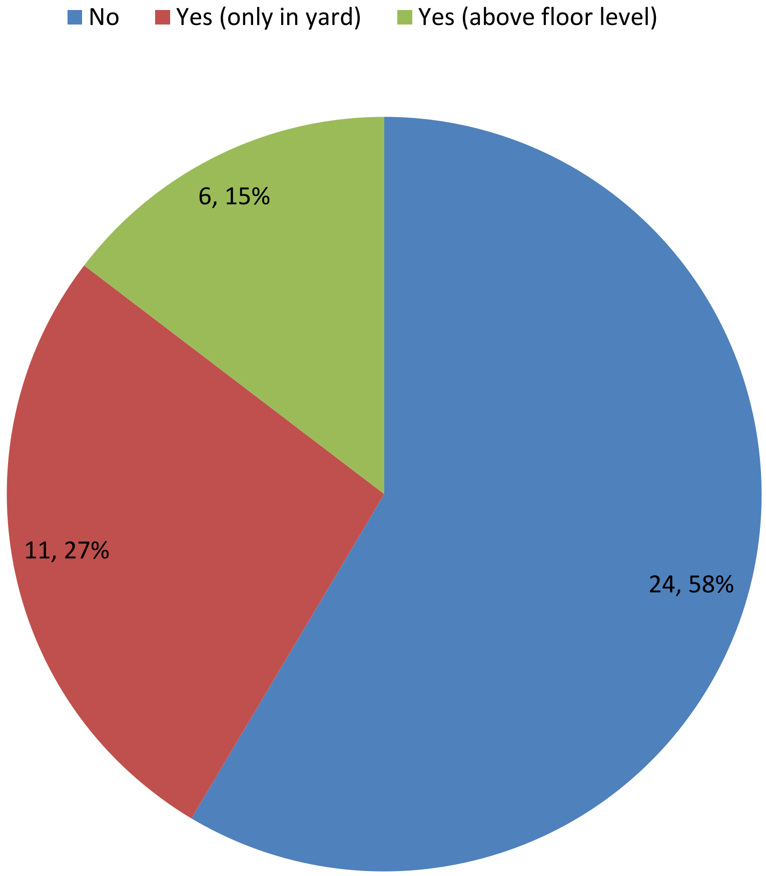


Are you aware of any flooding due to the Mulwaree River at Goulburn?

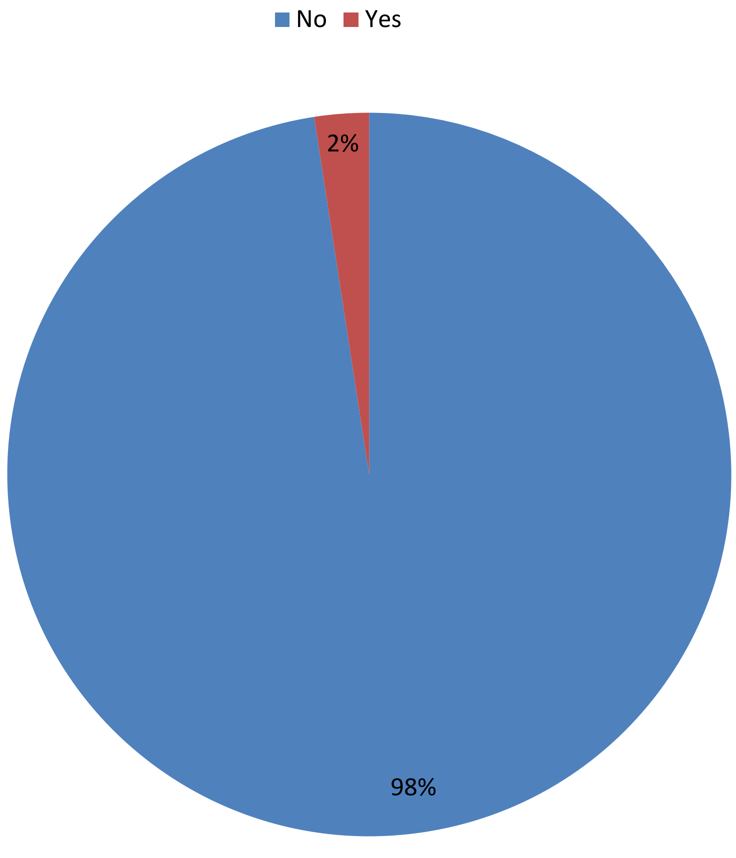
Very Aware | Some awareness | Not aware at all



Has your property ever been affected by flooding?



Do you have daily rainfall records for periods of flooding?



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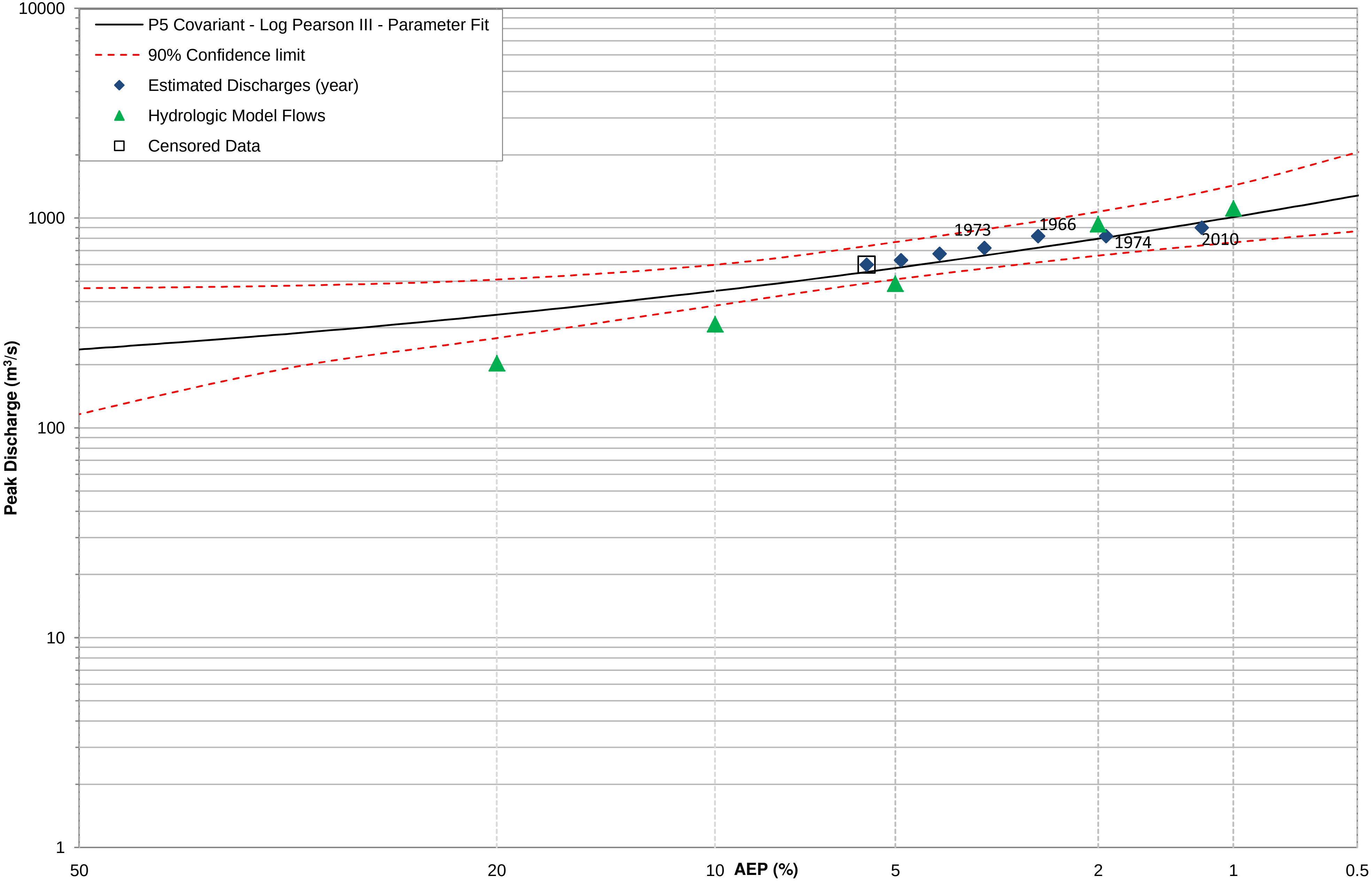

FIGURE 18
MARSDEN WEIR
CENSORED DATA BELOW 600 m³/S WITH RFFE PARAMETERS
LP3 ANALYSIS - BAYESIAN

J:\Jobs\115023\Hydrology\FFA\FFA_Probability_Plots_TheTowers.xlsx

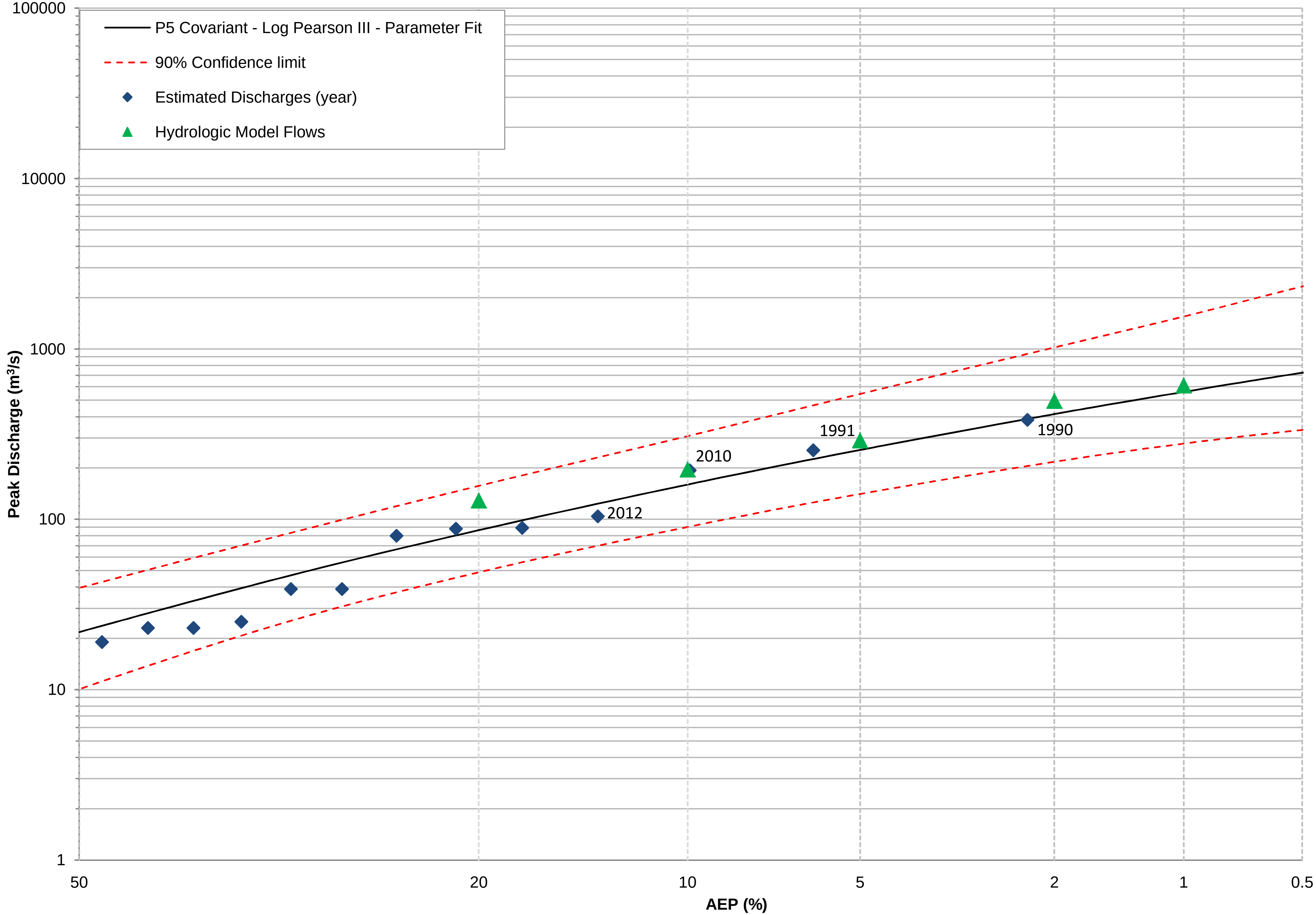

FIGURE 19
THE TOWERS
RFFE PARAMETERS
LP3 ANALYSIS - BAYESIAN

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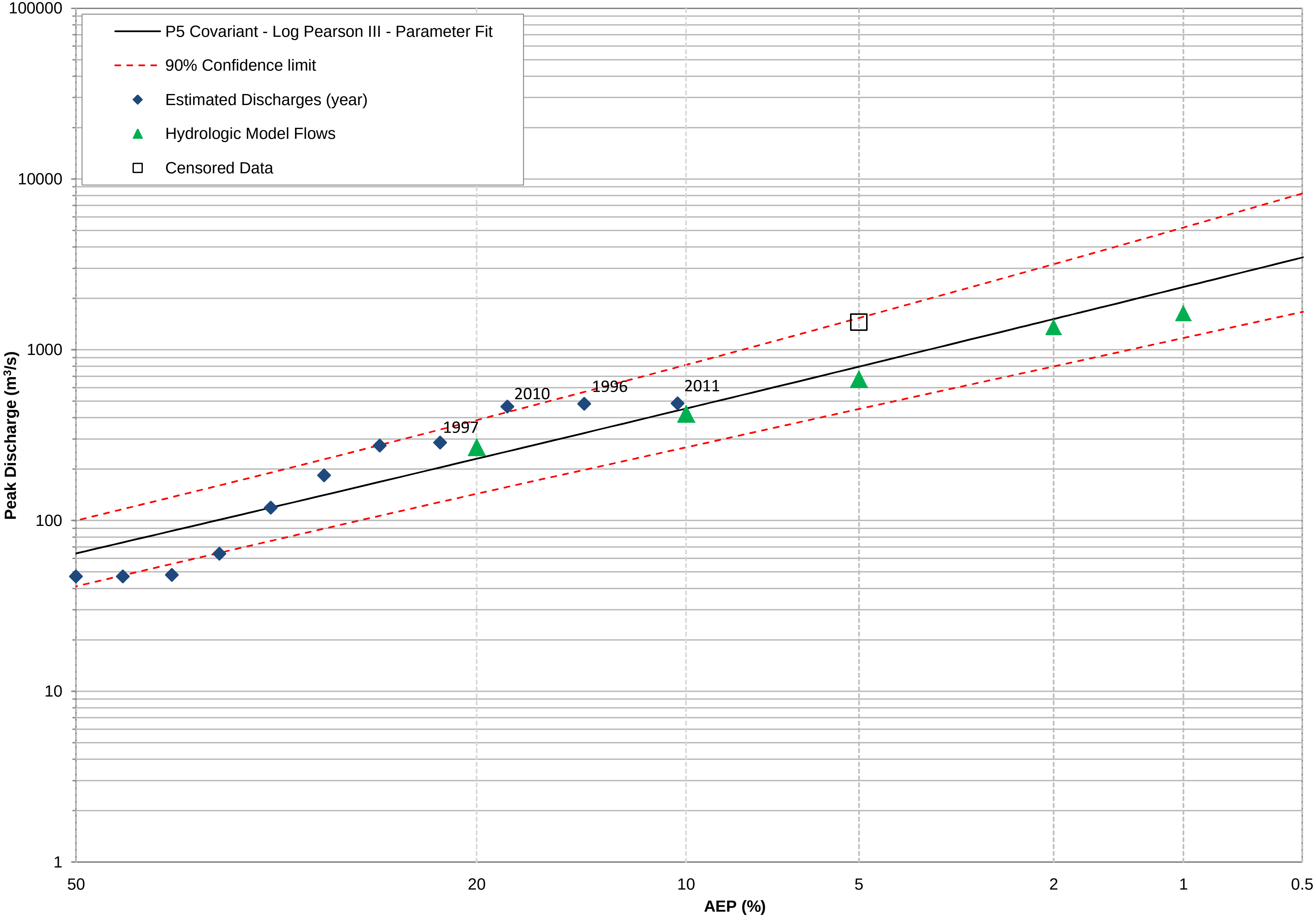
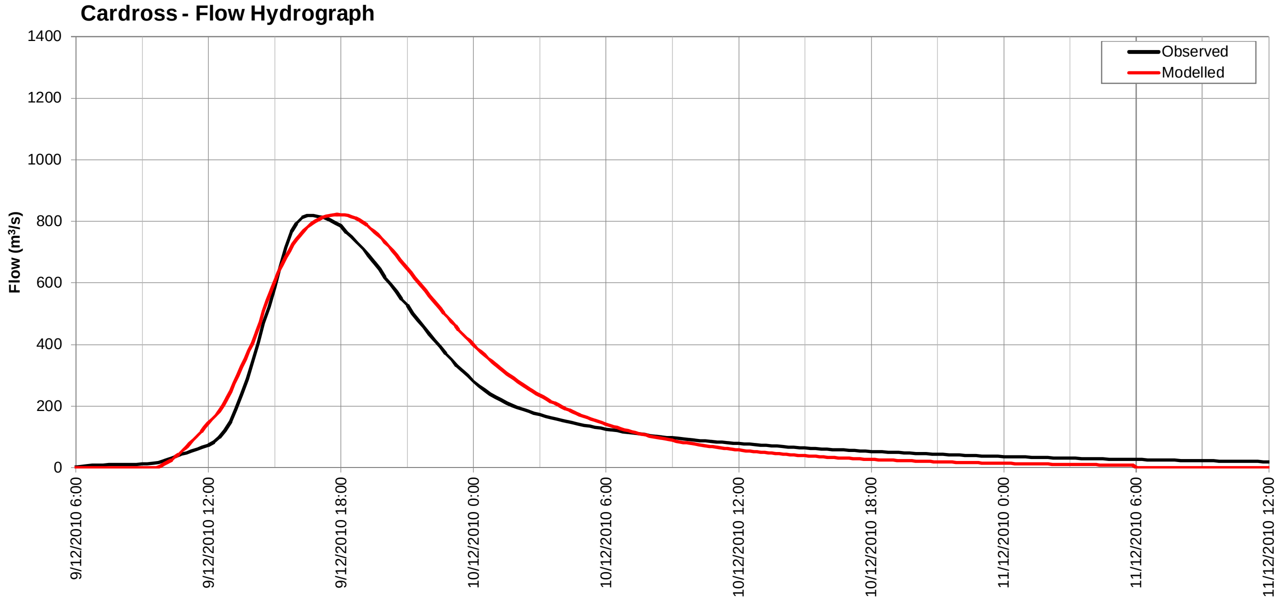


FIGURE 20
MURRAYS FLAT
LP3 ANALYSIS - BAYESIAN
2010 AND 1990 EVENTS INCLUDED AS CENSORED DATA WITH RFFE PARAMETERS


Cardross - Flow Hydrograph
Flow (m³/s)
1400
1200
1000
800
600
400
200
0
Observed
Modelled
9/12/2010 6:00
9/12/2010 12:00
9/12/2010 18:00
10/12/2010 0:00
10/12/2010 6:00
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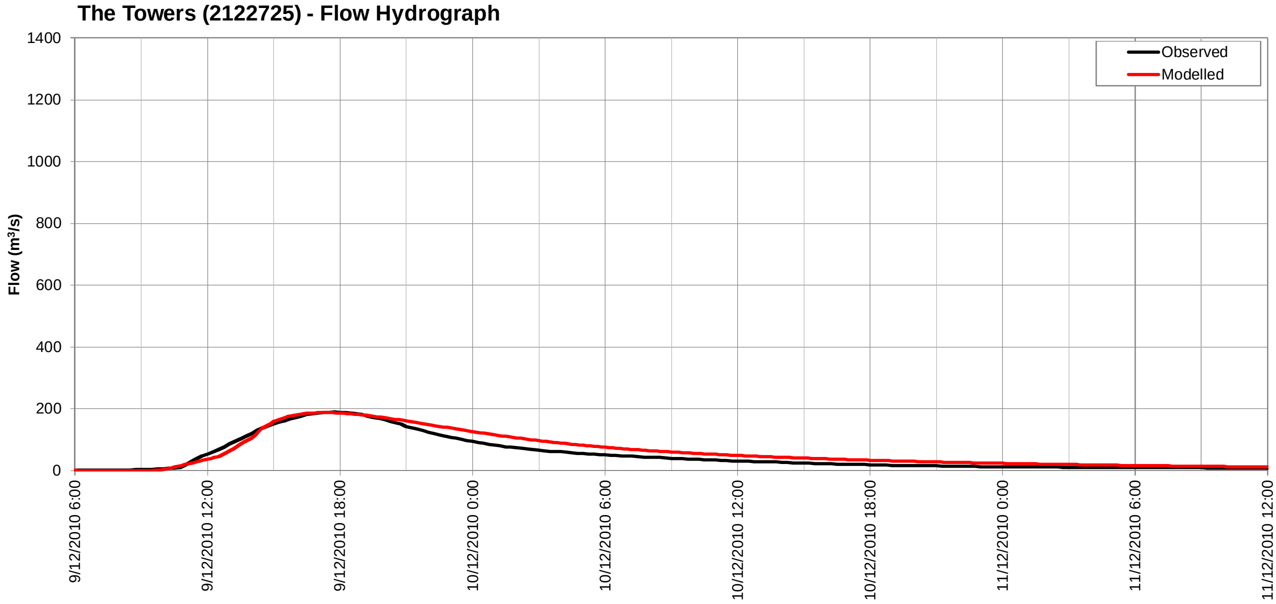
11/12/2010 12:00


The Towers (2122725) - Flow Hydrograph
Flow (m³/s)
1400
1200
1000
800
600
400
200
0
Observed
Modelled
9/12/2010 6:00
9/12/2010 12:00
9/12/2010 18:00
10/12/2010 0:00
10/12/2010 6:00
10/12/2010 12:00
10/12/2010 18:00
11/12/2010 0:00
11/12/2010 6:00
11/12/2010 12:00

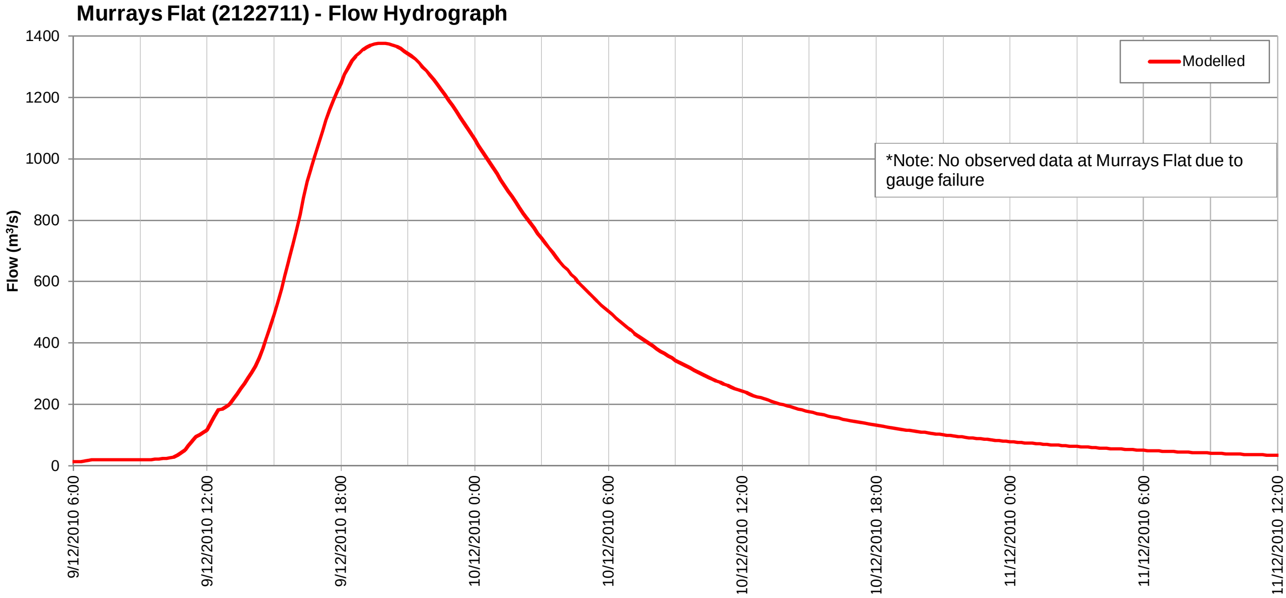
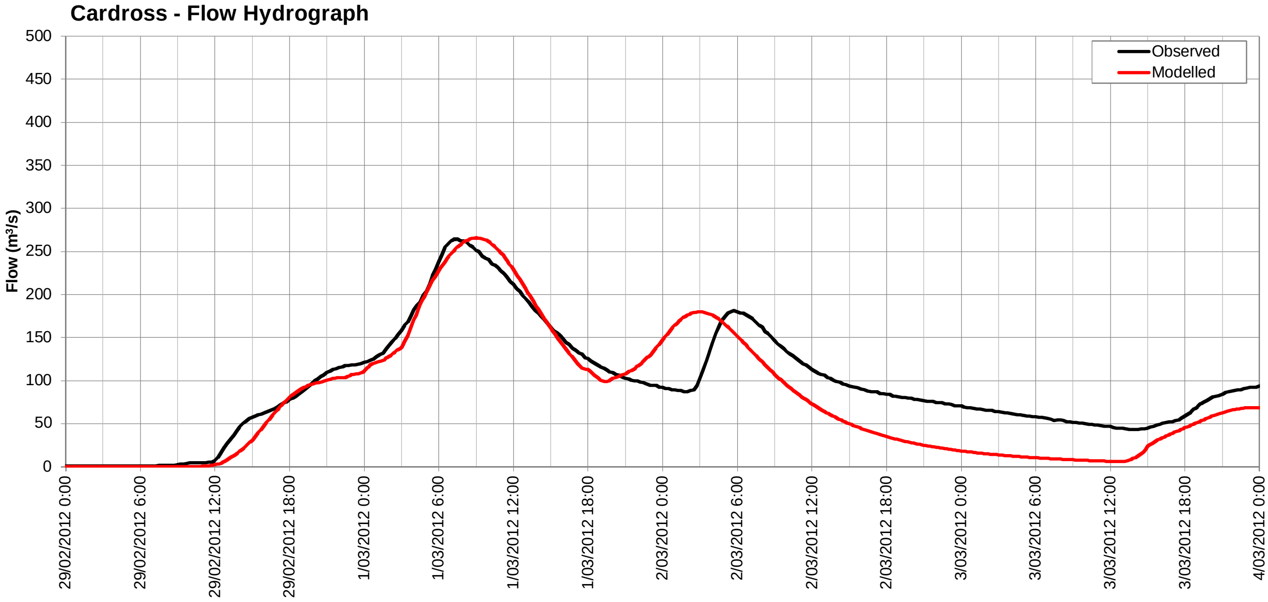
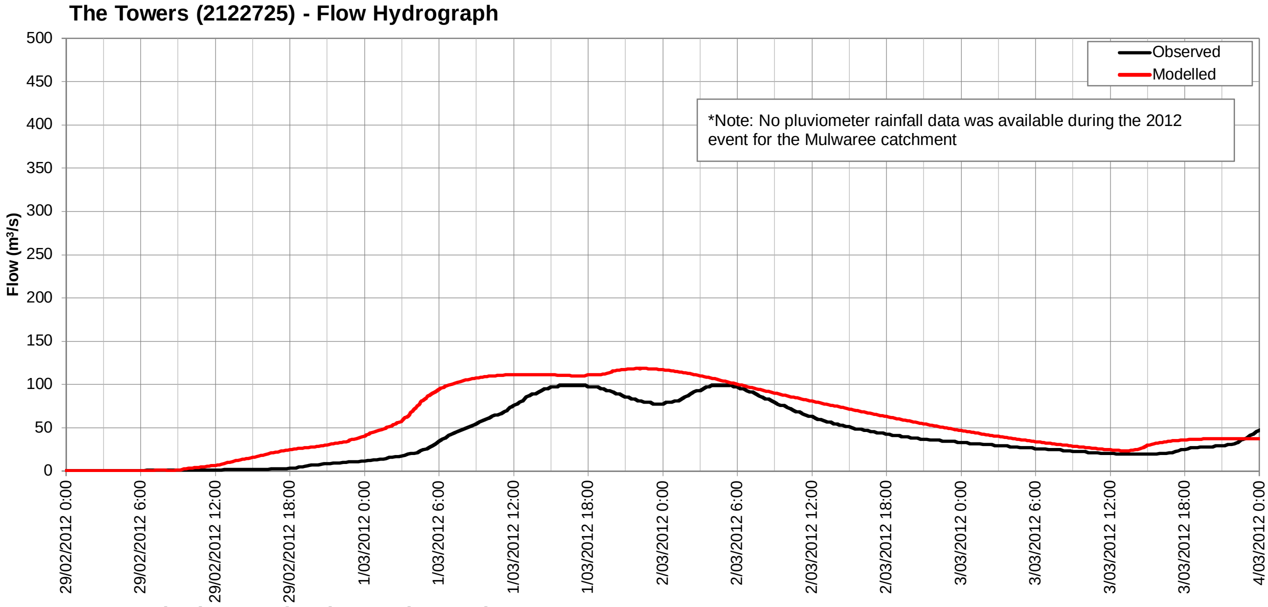

Murrays Flat (2122711) - Flow Hydrograph
Flow (m³/s)
1400
1200
1000
800
600
400
200
0
Modelled
*Note: No observed data at Murrays Flat due to gauge failure
9/12/2010 6:00
9/12/2010 12:00
9/12/2010 18:00
10/12/2010 0:00
10/12/2010 6:00
10/12/2010 12:00
10/12/2010 18:00
11/12/2010 0:00
11/12/2010 6:00
11/12/2010 12:00

FIGURE 22
HYDROLOGIC MODEL VALIDATION
MARCH 2012 EVENT





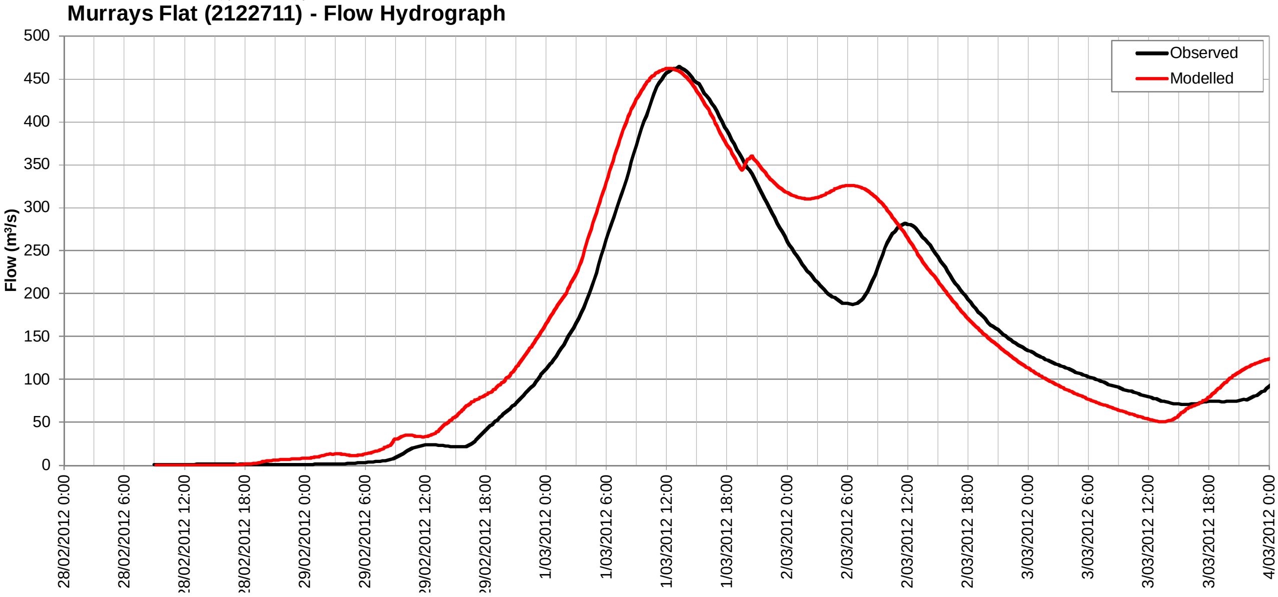


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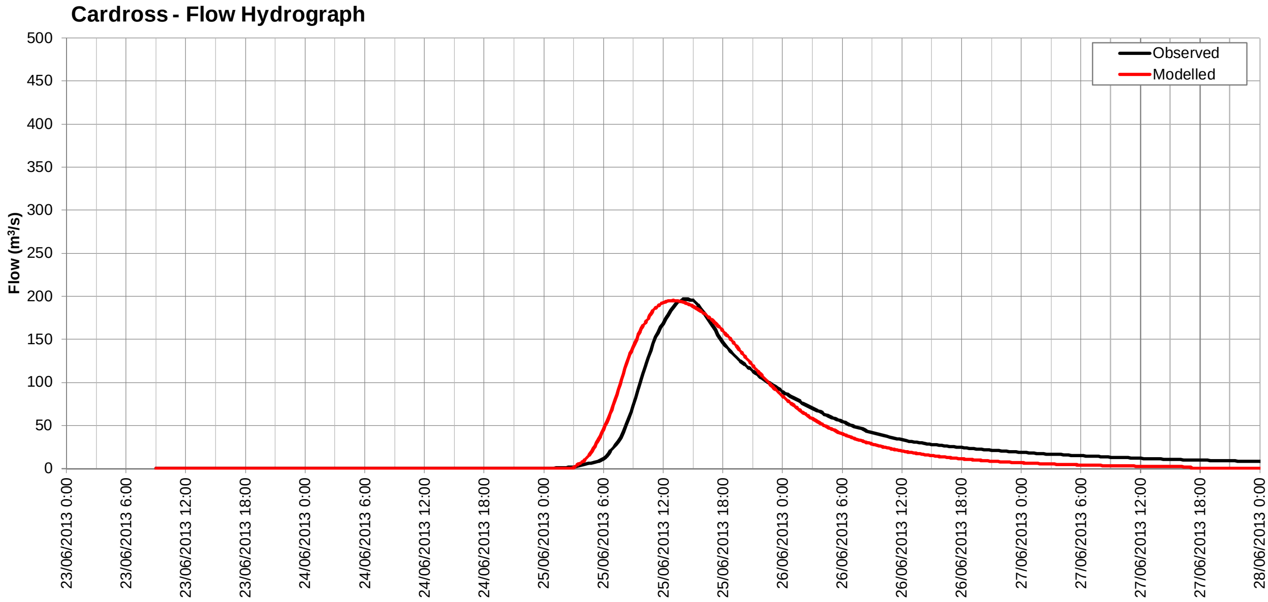
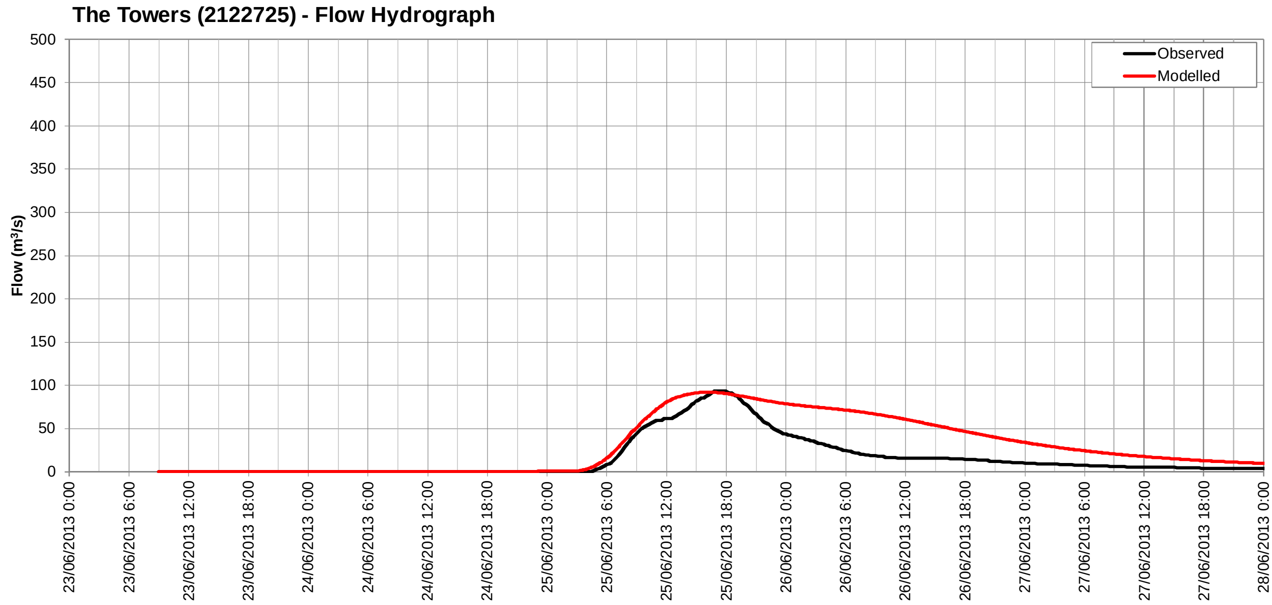
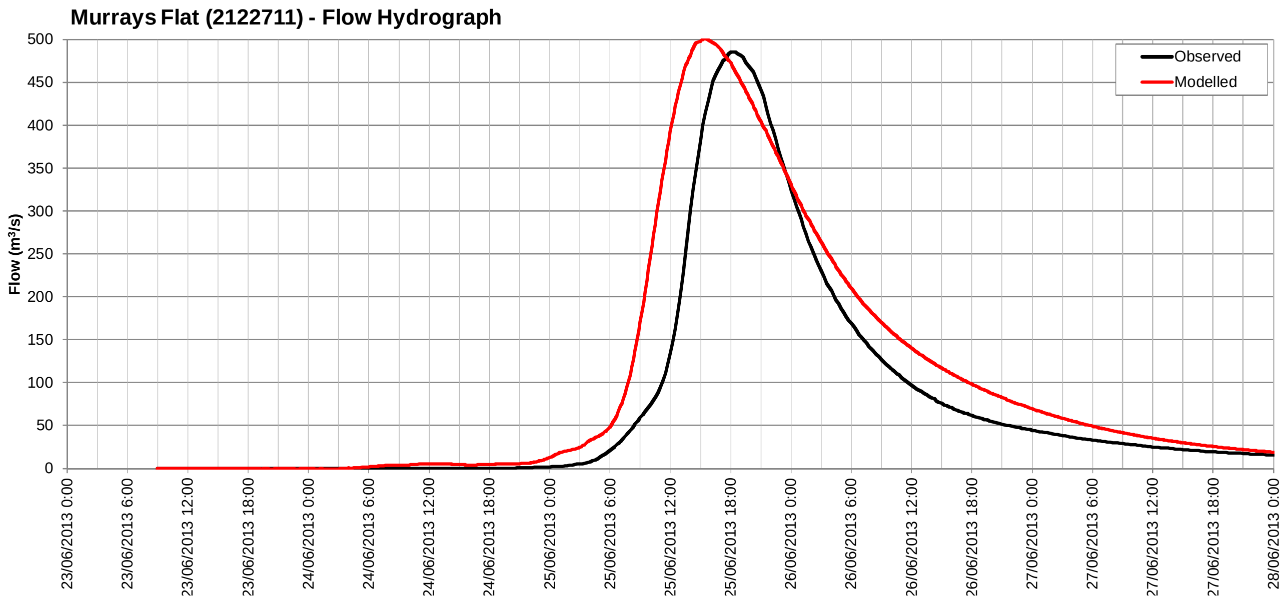
FIGURE 23
HYDROLOGIC MODEL VALIDATION
JUNE 2013 EVENT

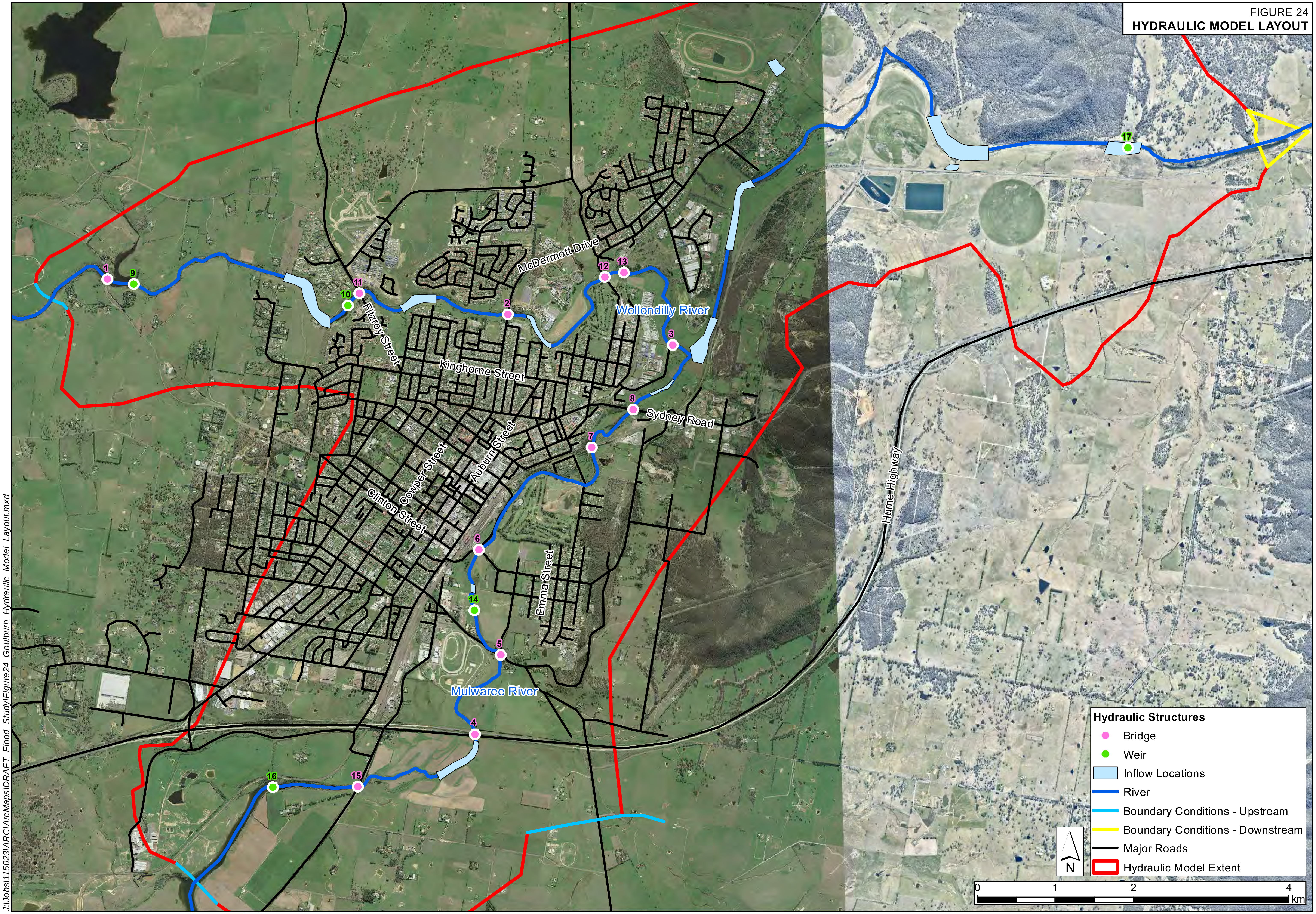






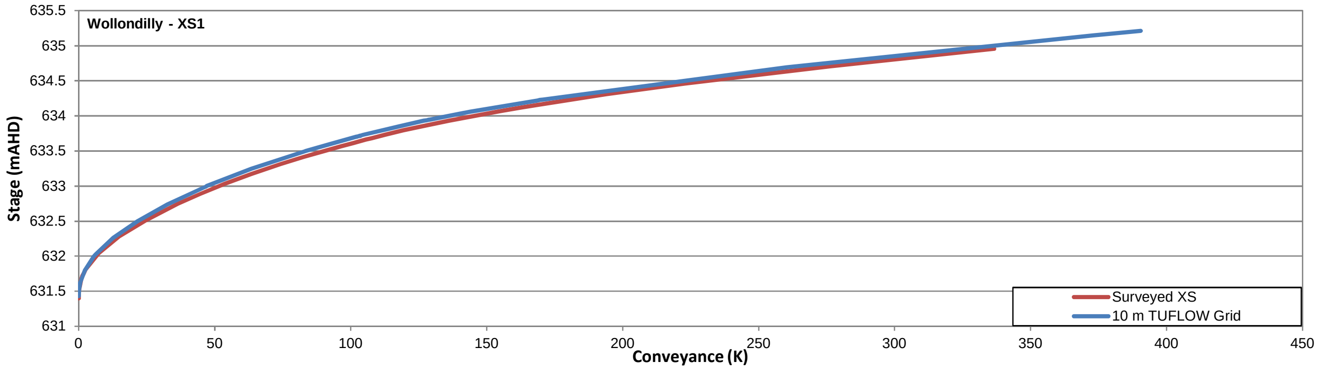
J:\Jobs\115023\Hydrology\WBNM\Goulburn_Flow_Hydrograph_ForDRAFTFS.xlsx

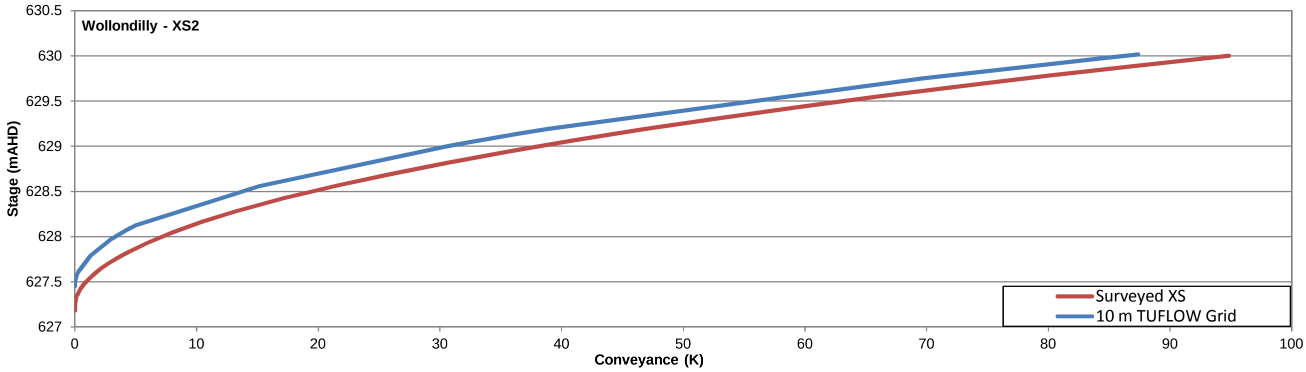
FIGURE 24
HYDRAULIC MODEL LAYOUT

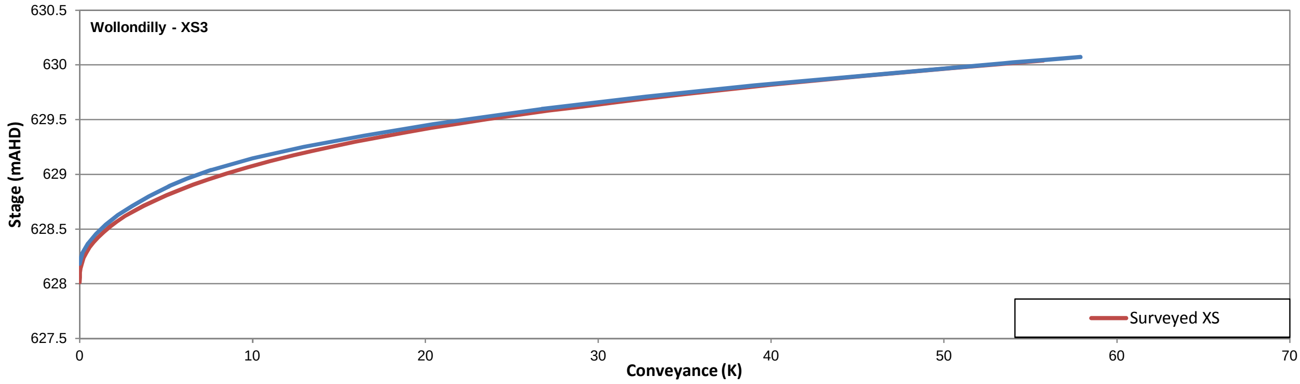


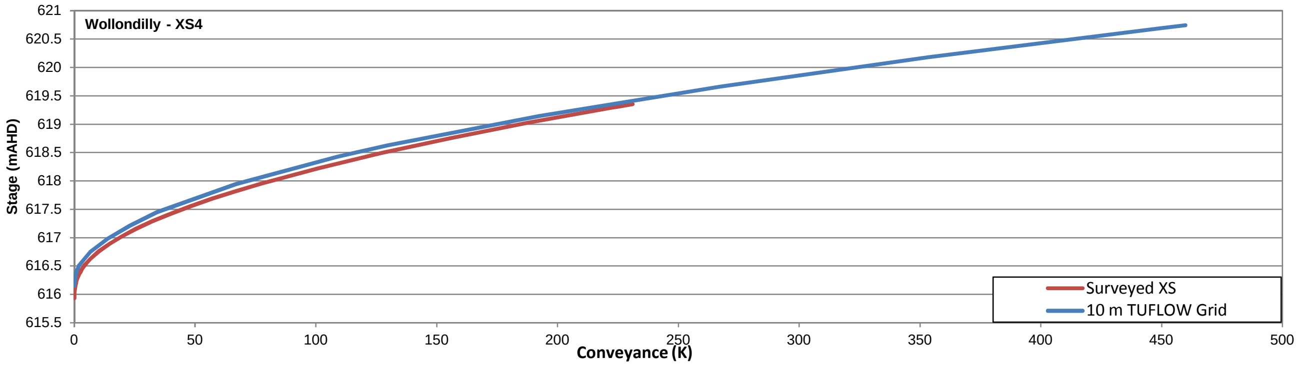
J:\Jobs\115023\ARC\ArcMaps\DRAFT_Flood_Study\Figure24_Goulburn_Hydraulic_Model_Layout.mxd

FIGURE 25
WOLLONDILLY RIVER CONVEYANCE COMPARISON
10 M TUFLOW GRID VS. SURVEY DATA



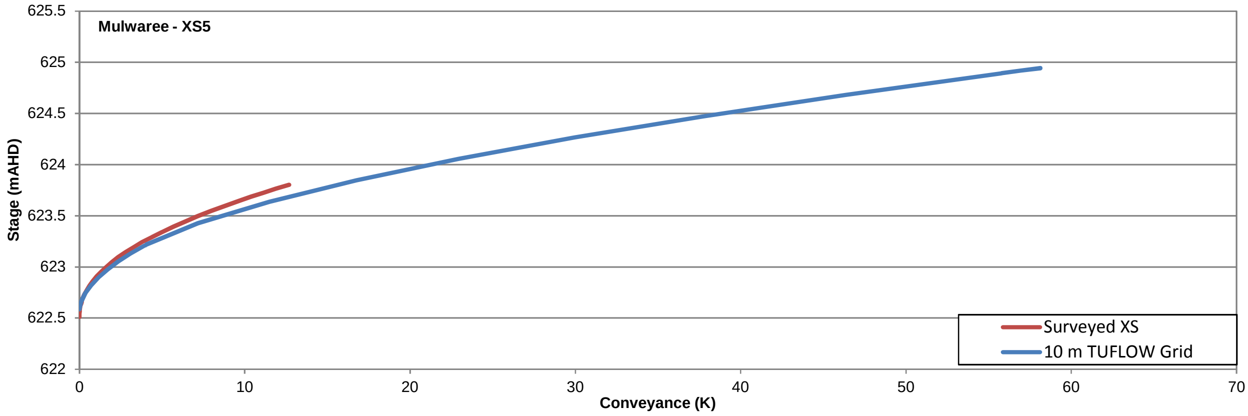


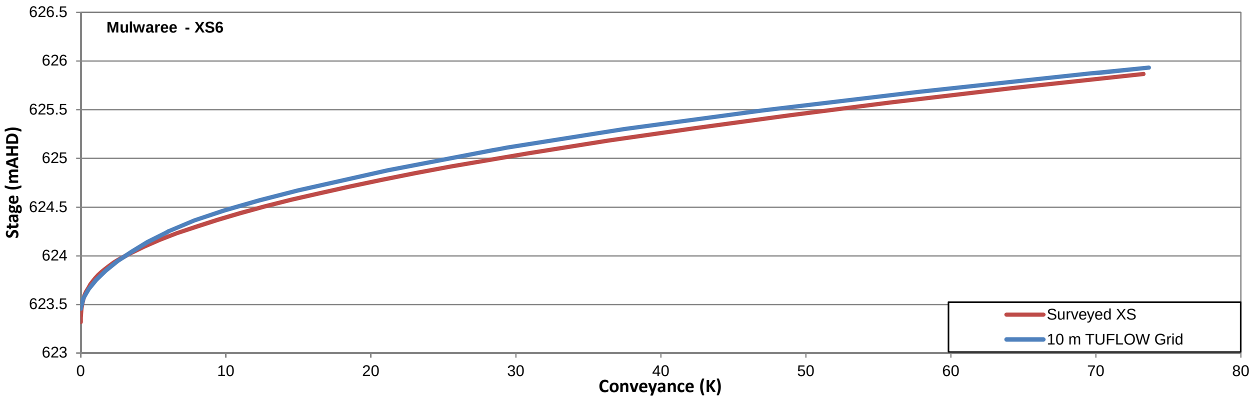


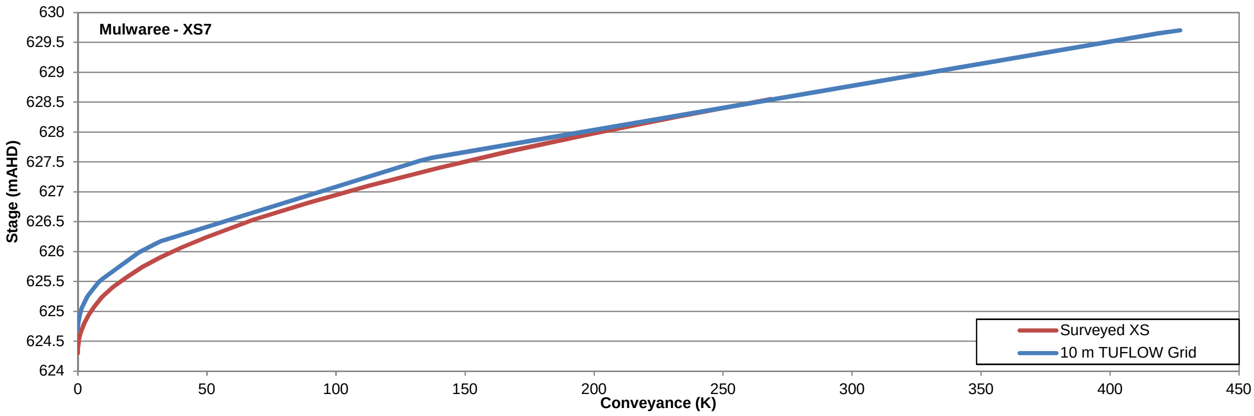


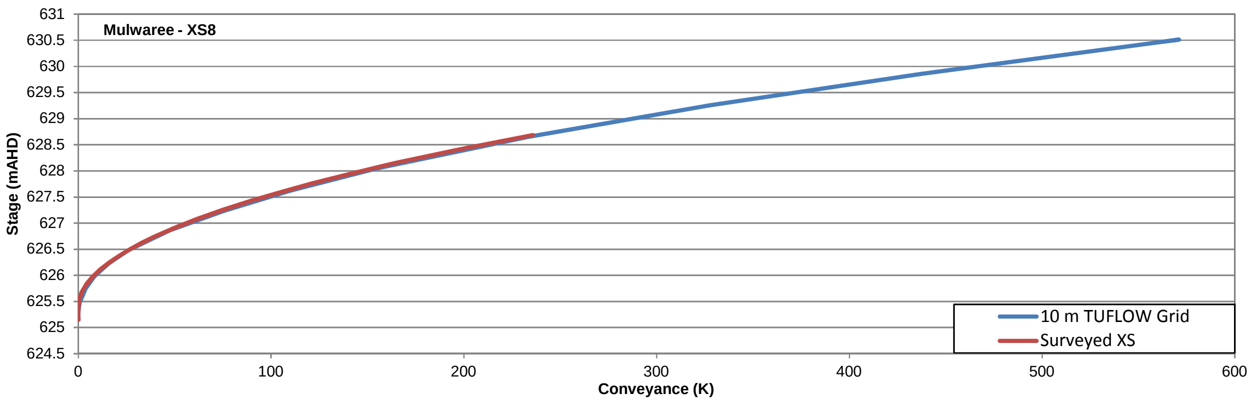
J:\Jobs\115023\Mapinfo\ConveyanceComparison\Conveyance_Comparison_forDRAFTFS.xlsx

FIGURE 26
MULWAREE RIVER CONVEYANCE COMPARISON
0 M TUFLOW GRID VS. SURVEY DATA



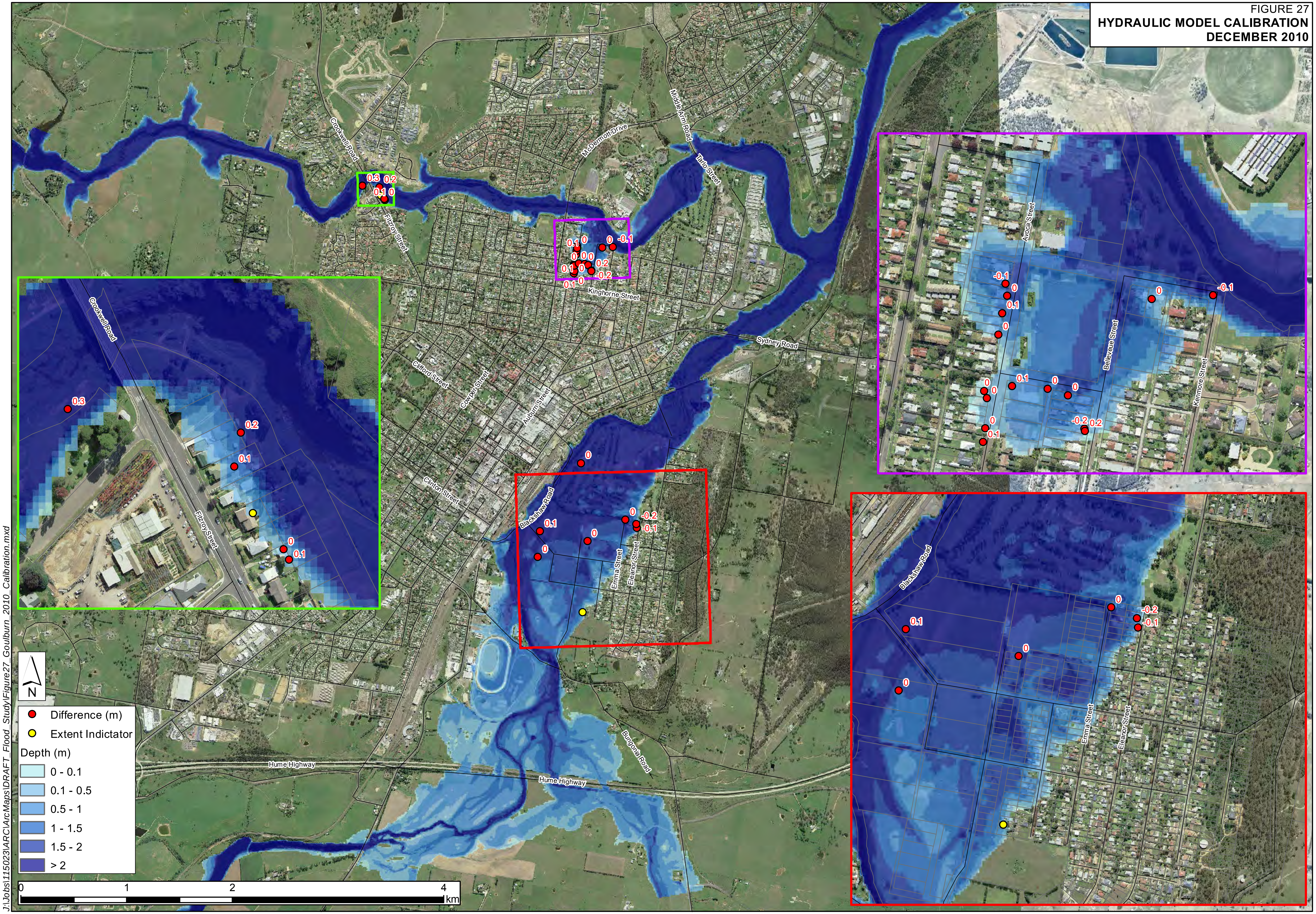






J:\Jobs\115023\Mapinfo\ConveyanceComparison\Conveyance_Comparison_forDRAFTFS.xlsx

FIGURE 27
HYDRAULIC MODEL CALIBRATION DECEMBER 2010



J:\Jobs\115023\ARC\ArcMaps\DRAFT_Flood_Study\Figure27_Goulburn_2010_Calibration.mxd

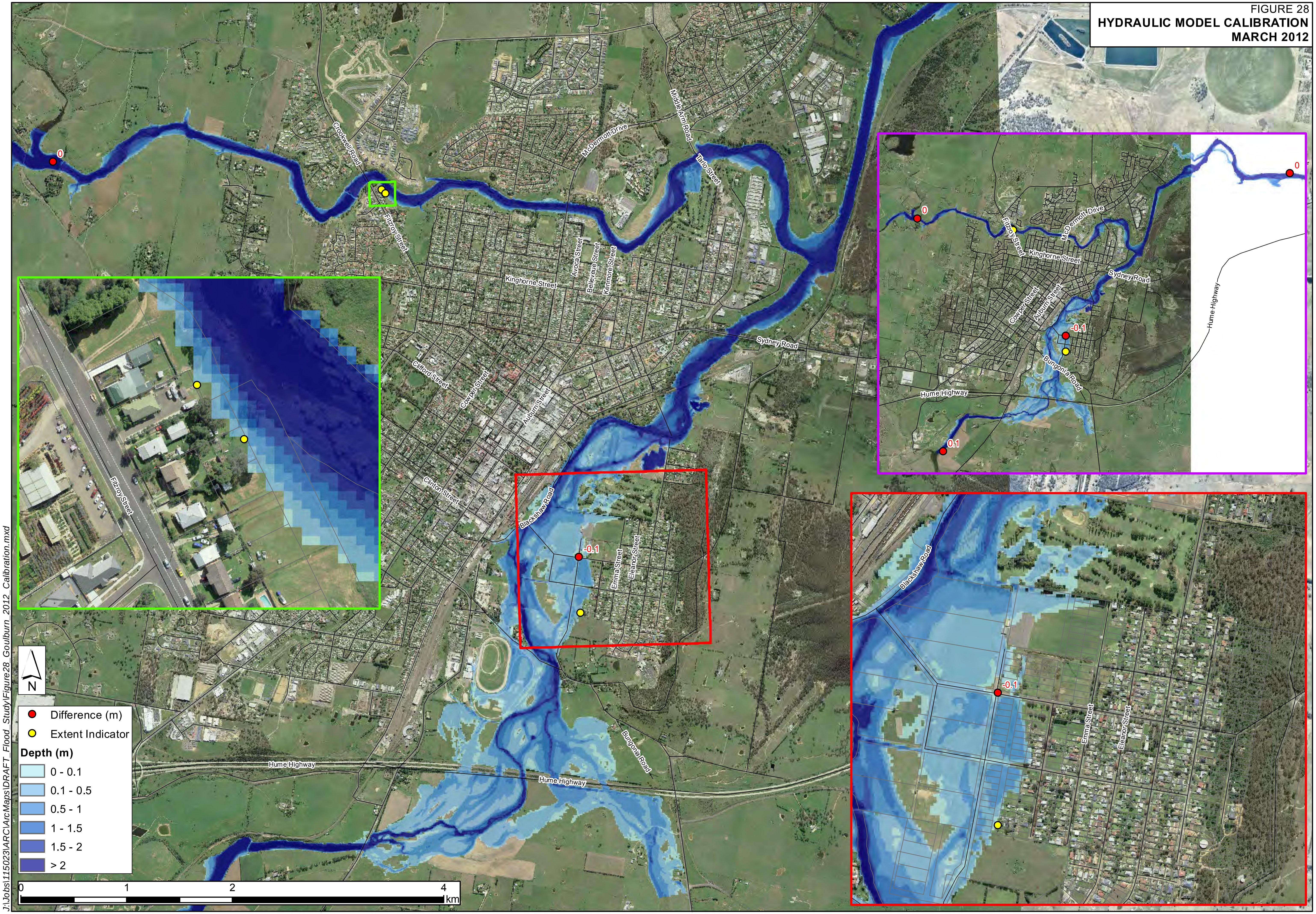
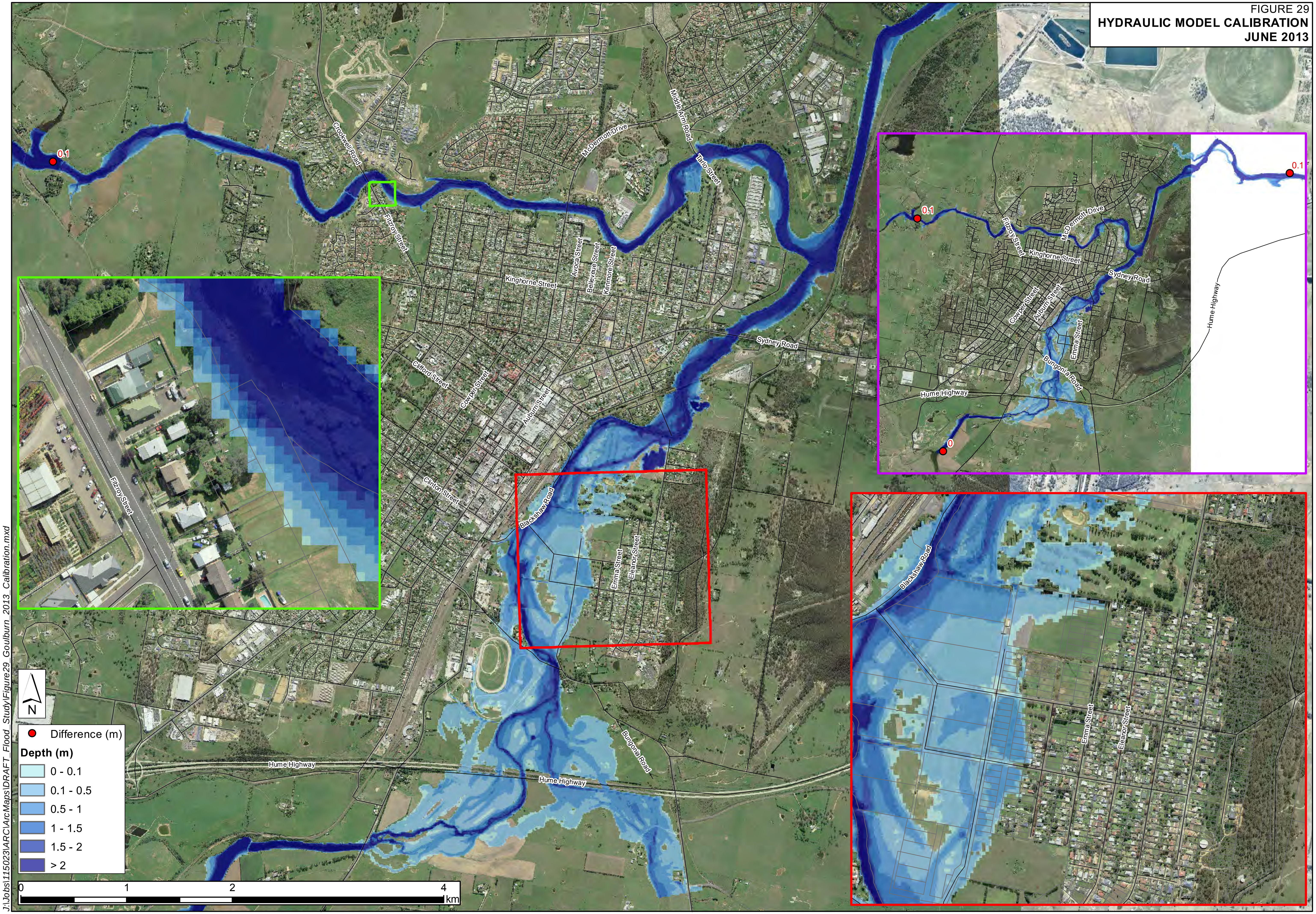


FIGURE 28
HYDRAULIC MODEL CALIBRATION
MARCH 2012
Difference (m)
Extent Indicator
Depth (m)
0 - 0.1
0.1 - 0.5
0.5 - 1
1 - 1.5
1.5 - 2
> 2
N
0
1
2
4
km
Crookwell Road
Fitzroy Street
McDermott Drive
Middle Arm Road
Tarlo Street
Kinghorne Street
Avoca Street
Bellevue Street
Kenmore Street
Sydney Road
Clifford Street
Cowper Street
Auburn Street
Clinton Street
Blackshaw Road
Emma Street
Eleanor Street
Bungonia Road
Hume Highway
0
-0.1
0.1
J:\Jobs\115023\ARC\ArcMaps\DRAFT_Flood_Study\Figure28_Goulburn_2012_Calibration.mxd

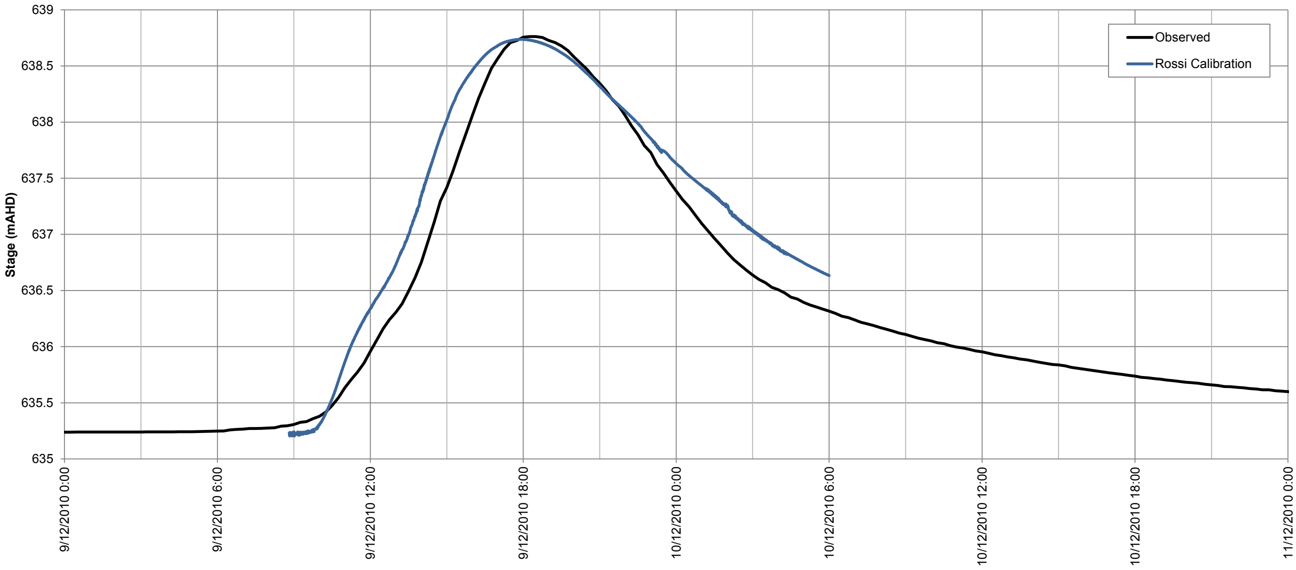
FIGURE 29
HYDRAULIC MODEL CALIBRATION
JUNE 2013



J:\Jobs\115023\ARC\ArcMaps\DRAFT_Flood_Study\Figure29_Goulburn_2013_Calibration.mxd

FIGURE 30
HYDRAULIC MODEL CALIBRATION
STAGE HYDROGRAPHS
DECEMBER 2010 EVENT

Rossi - Stage Hydrograph



The Towers (2122725) - Stage Hydrograph

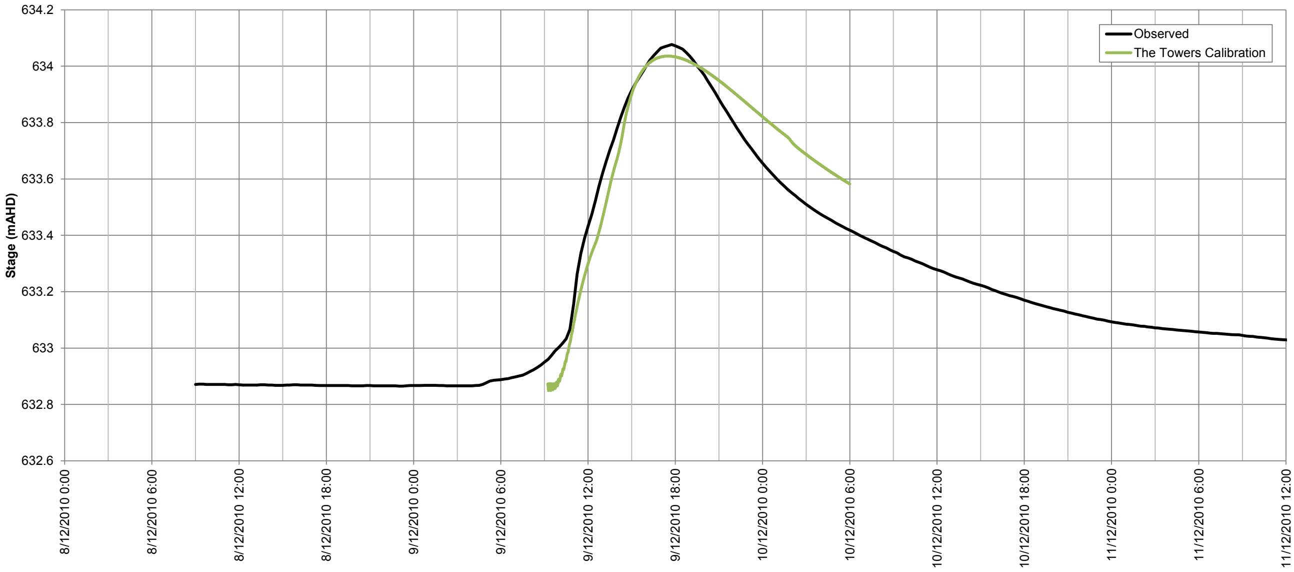
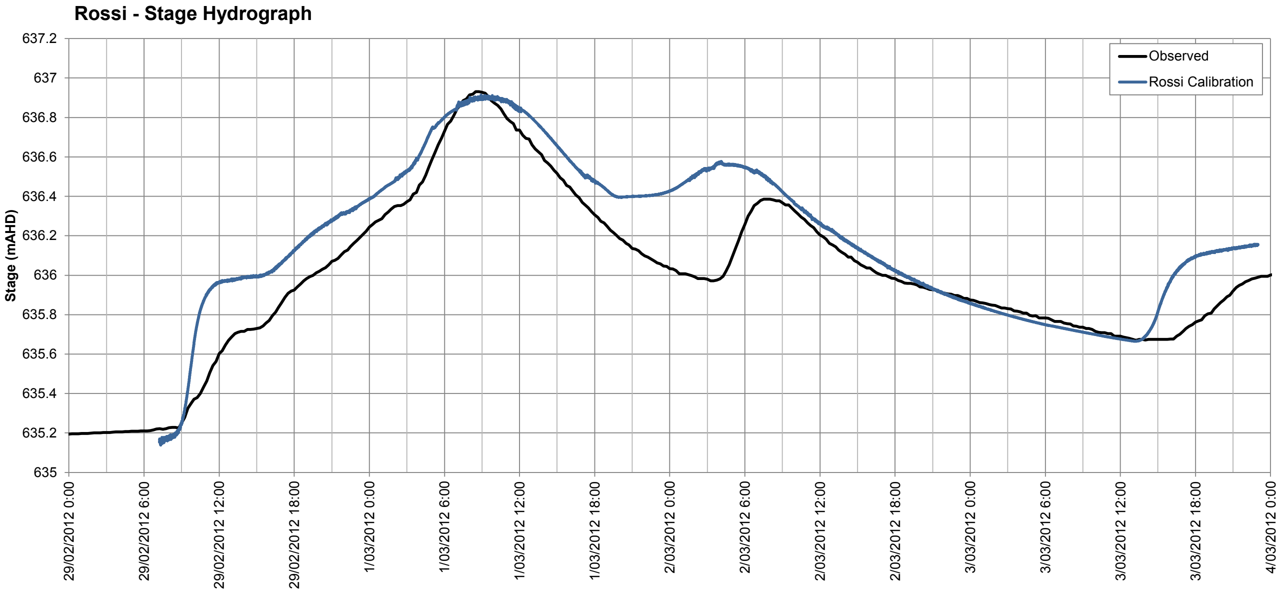
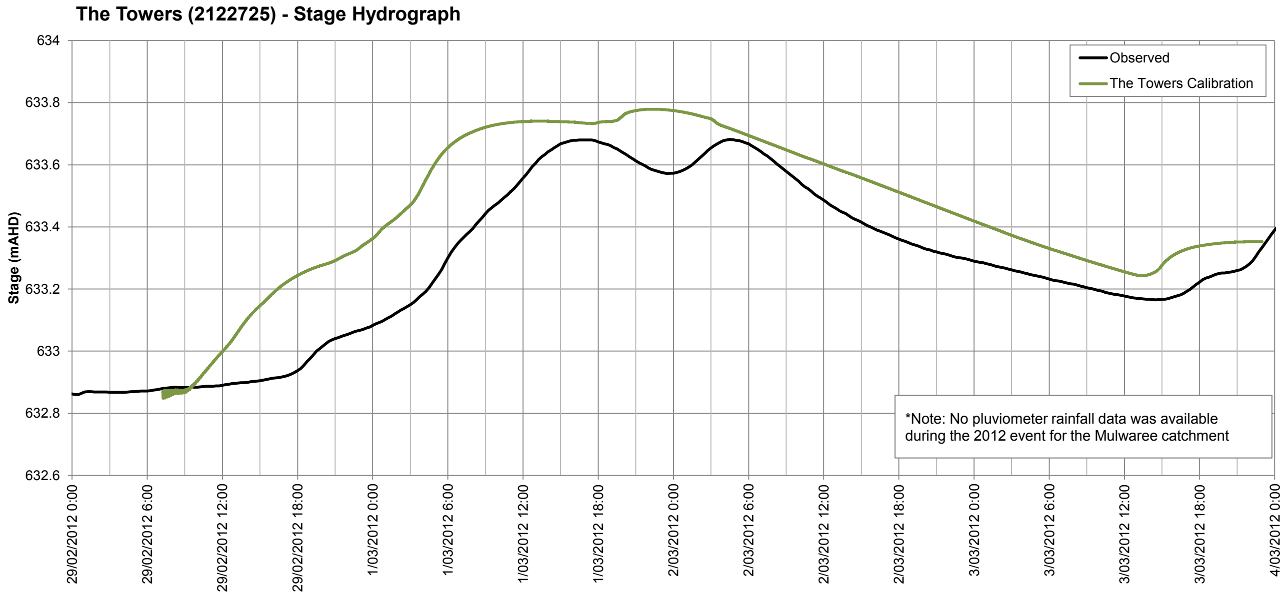
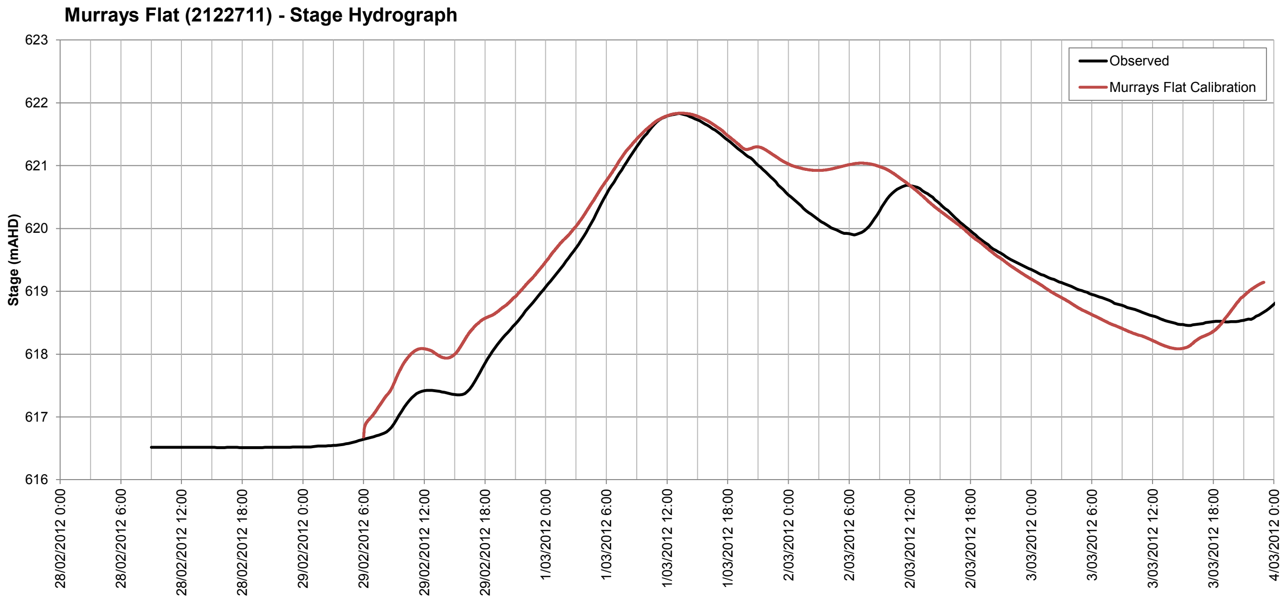


FIGURE 31
HYDRAULIC MODEL CALIBRATION
STAGE HYDROGRAPHS
MARCH 2012 EVENT



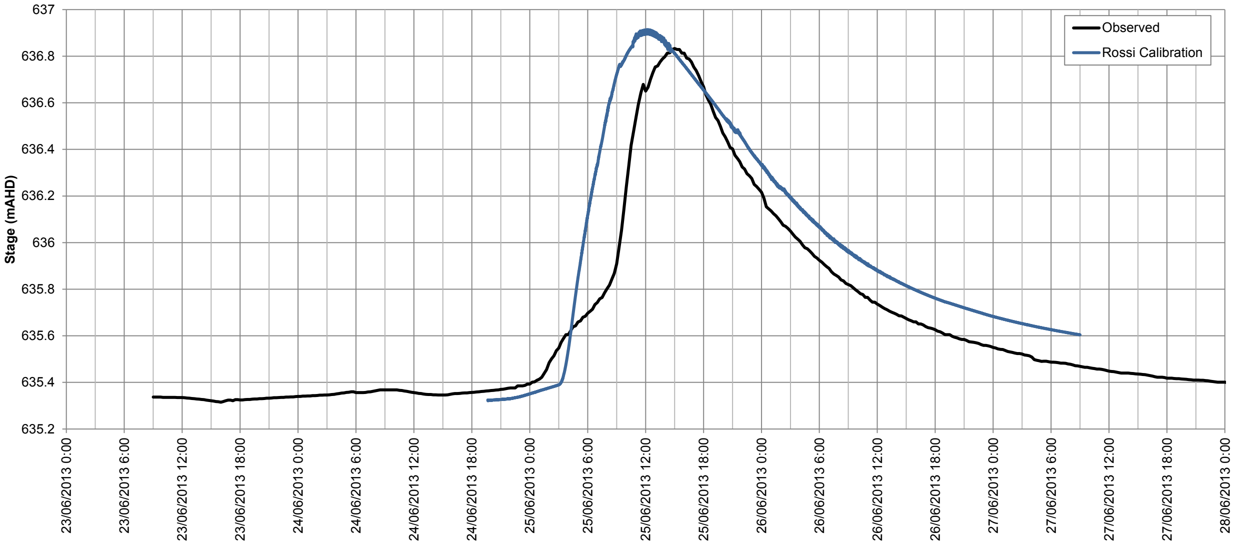




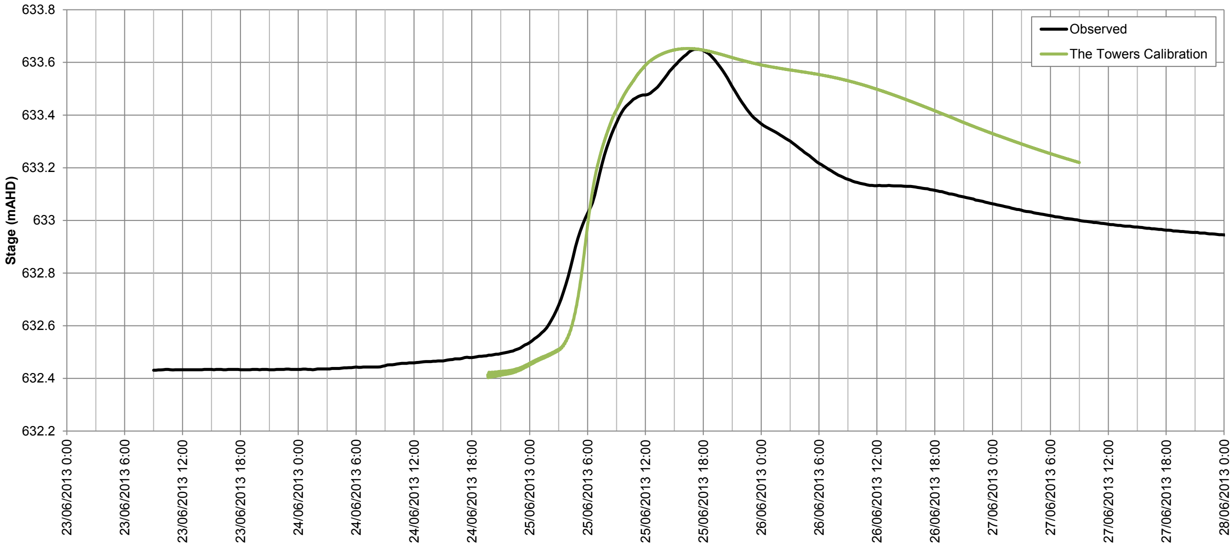
J:\Jobs\115023\TUFLOW\Goulburn\Calibration_Stage_Hydrographs_forDRAFTFS.xlsx

FIGURE 32
HYDRAULIC MODEL CALIBRATION
STAGE HYDROGRAPHS
JUNE 2013 EVENT

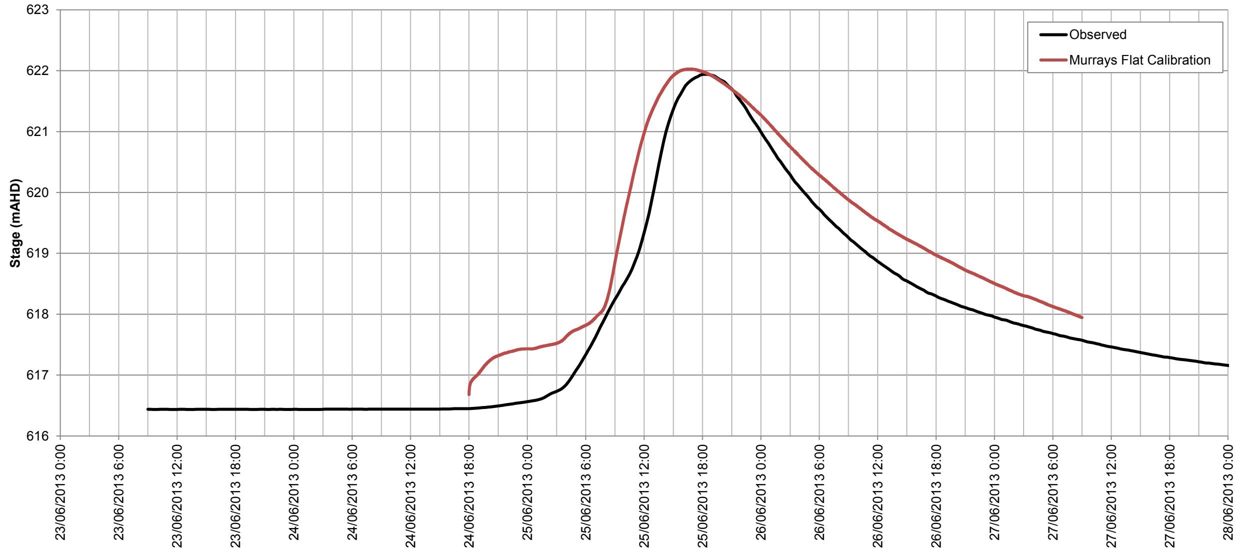
Rossi - Stage Hydrograph



The Towers (2122725) - Stage Hydrograph

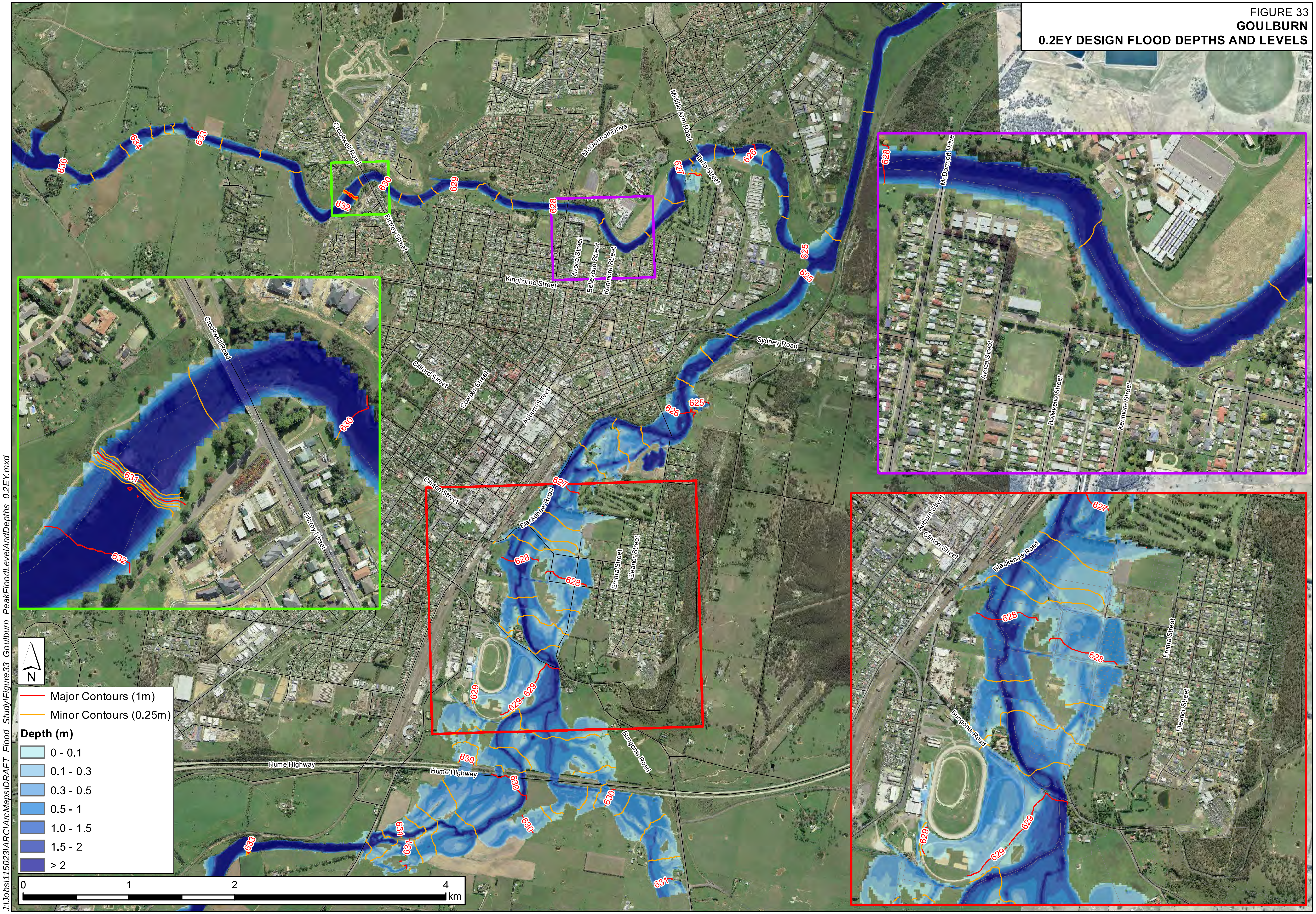


Murrays Flat (2122711) - Stage Hydrograph



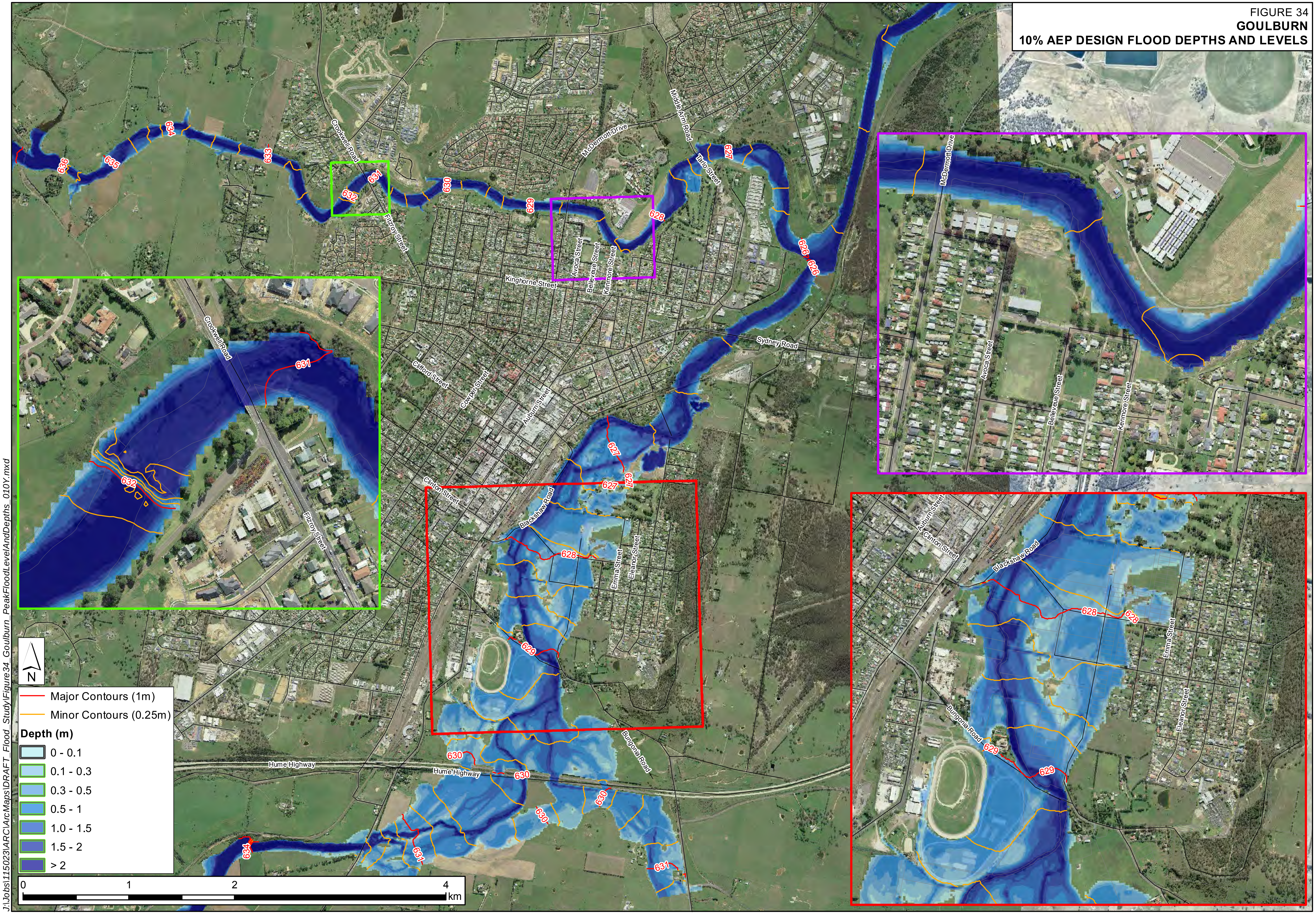
J:\Jobs\115023\TUFLOW\Goulburn\Calibration_Stage_Hydrographs_forDRAFTFS.xlsx

FIGURE 33
GOULBURN
0.2EY DESIGN FLOOD DEPTHS AND LEVELS



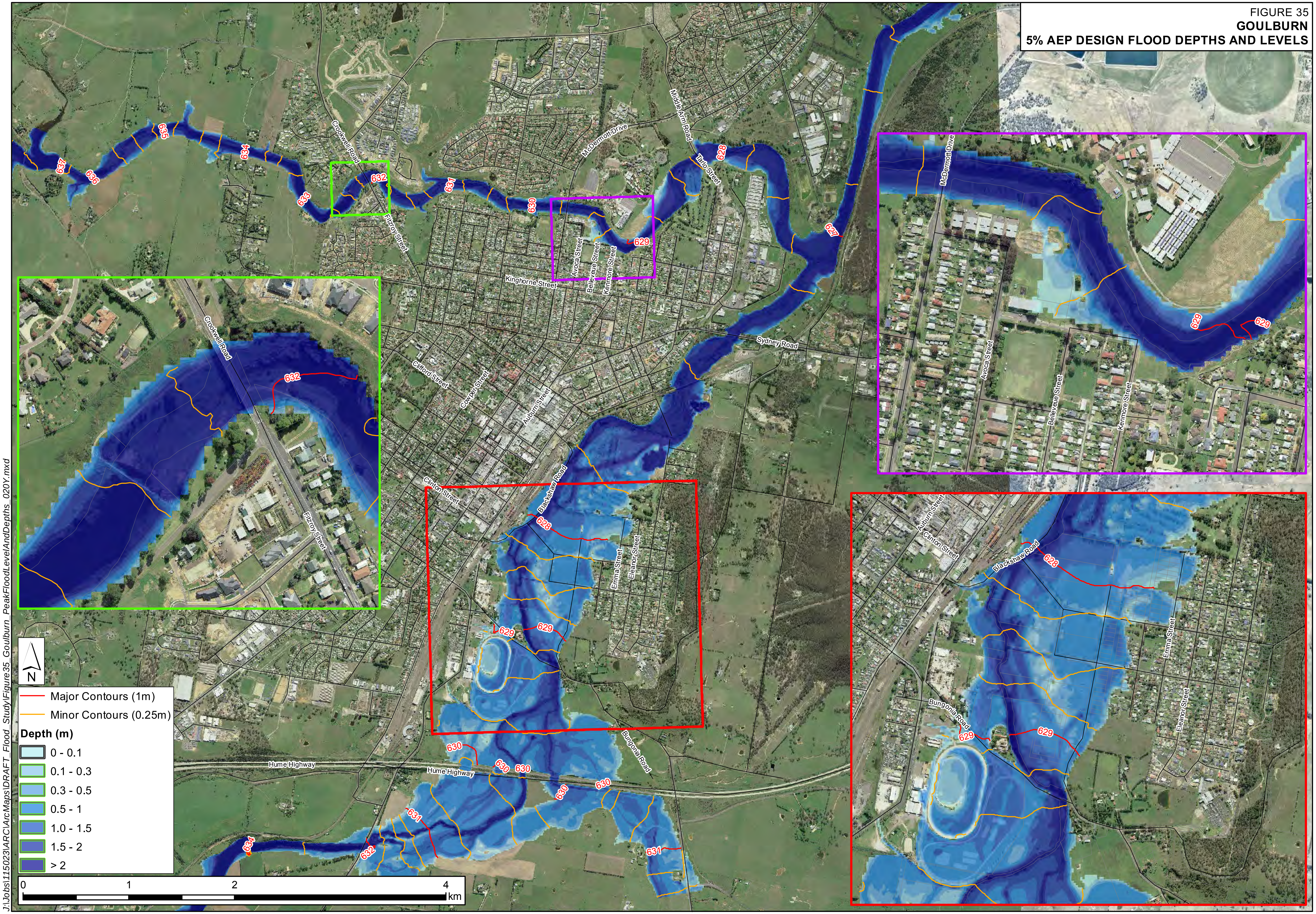
J:\Jobs\115023\ARC\ArcMaps\DRAFT_Flood_Study\Figure33_Goulburn_PeakFloodLevelAndDepths_0.2EY.mxd

FIGURE 34
GOULBURN
10% AEP DESIGN FLOOD DEPTHS AND LEVELS



J:\Jobs\115023\ARC\ArcMaps\DRAFT_Flood_Study\Figure34_Goulburn_PeakFloodLevelAndDepths_010Y.mxd

FIGURE 35
GOULBURN
5% AEP DESIGN FLOOD DEPTHS AND LEVELS



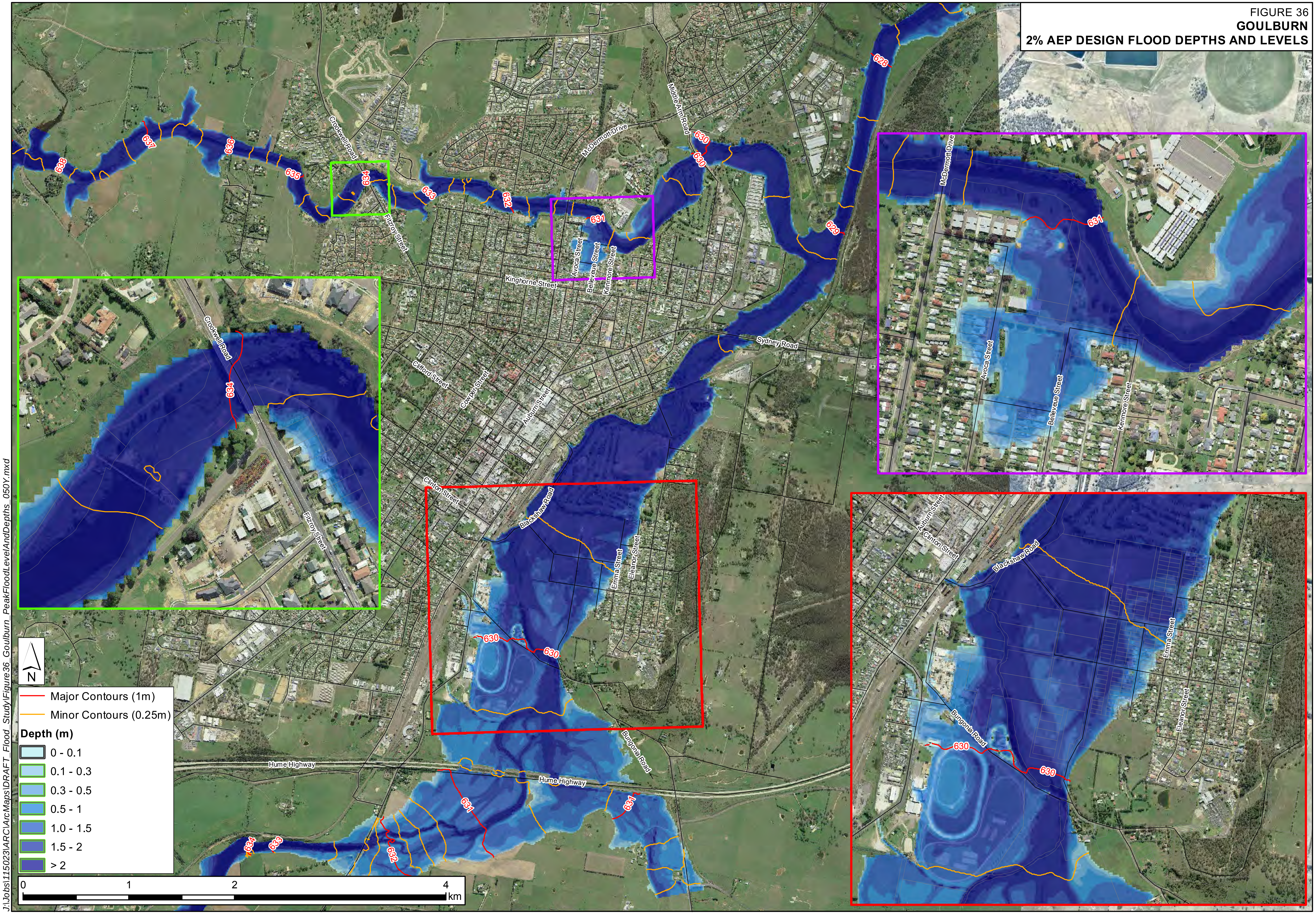
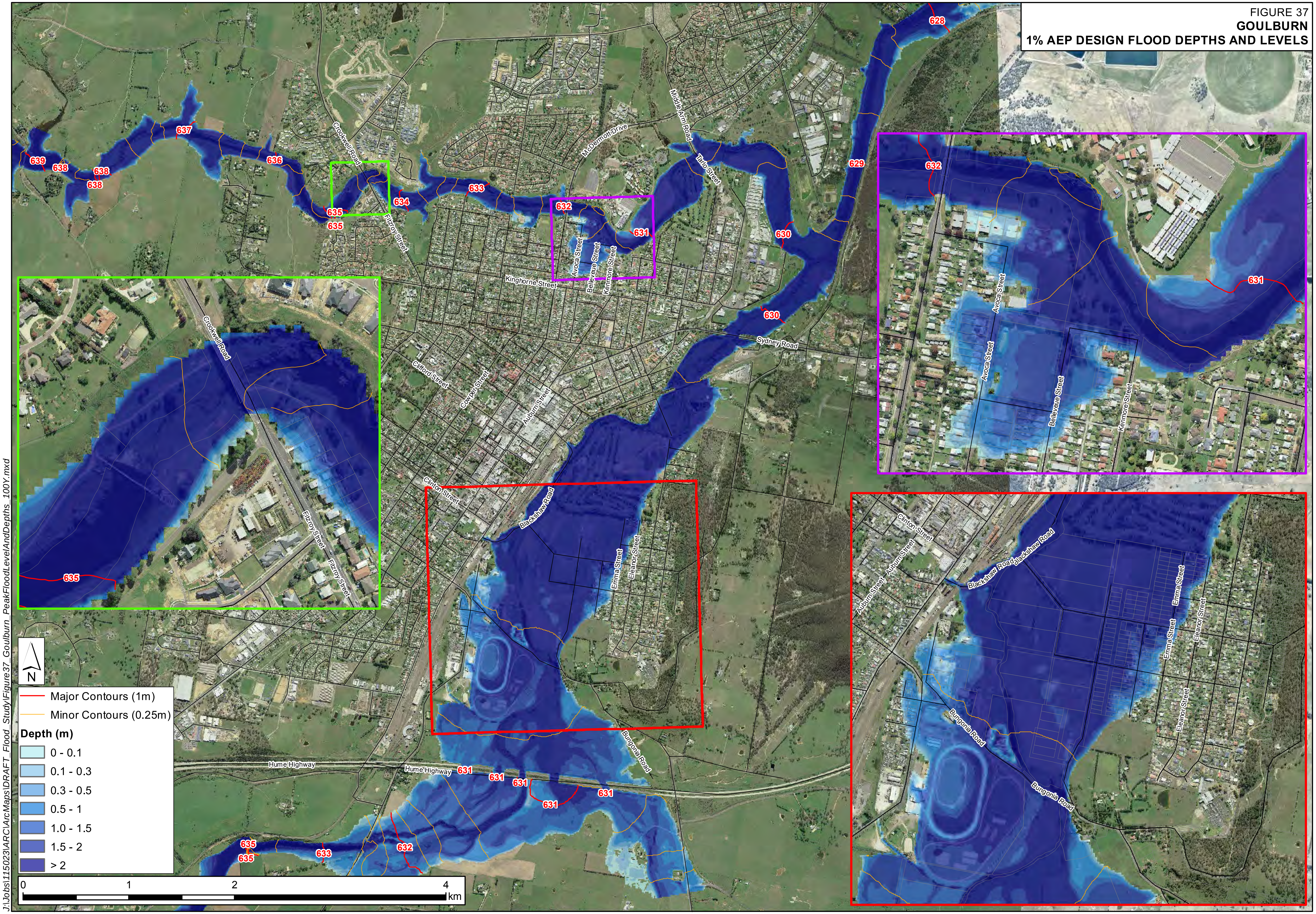


FIGURE 36
GOULBURN
2% AEP DESIGN FLOOD DEPTHS AND LEVELS
Major Contours (1m)
Minor Contours (0.25m)
Depth (m)
0 - 0.1
0.1 - 0.3
0.3 - 0.5
0.5 - 1
1.0 - 1.5
1.5 - 2
> 2
0
1
2
4
km
N
Crookwell Road
Fitzroy Street
McDermott Drive
Middle Arm Road
Avoca Street
Bellevue Street
Kenmore Street
Kinghorne Street
Sydney Road
Clifford Street
Cowper Street
Auburn Street
Clinton Street
Blackshaw Road
Emma Street
Eleanor Street
Bungonia Road
Hume Highway
J:\Jobs\115023\ARC\ArcMaps\DRAFT_Flood_Study\Figure36_Goulburn_PeakFloodLevelAndDepths_050Y.mxd

FIGURE 37
GOULBURN
1% AEP DESIGN FLOOD DEPTHS AND LEVELS



J:\Jobs\115023\ARC\ArcMaps\DRAFT_Flood_Study\Figure37_Goulburn_PeakFloodLevelAndDepths_100Y.mxd

FIGURE 38
GOULBURN
0.5% AEP DESIGN FLOOD DEPTHS AND LEVELS

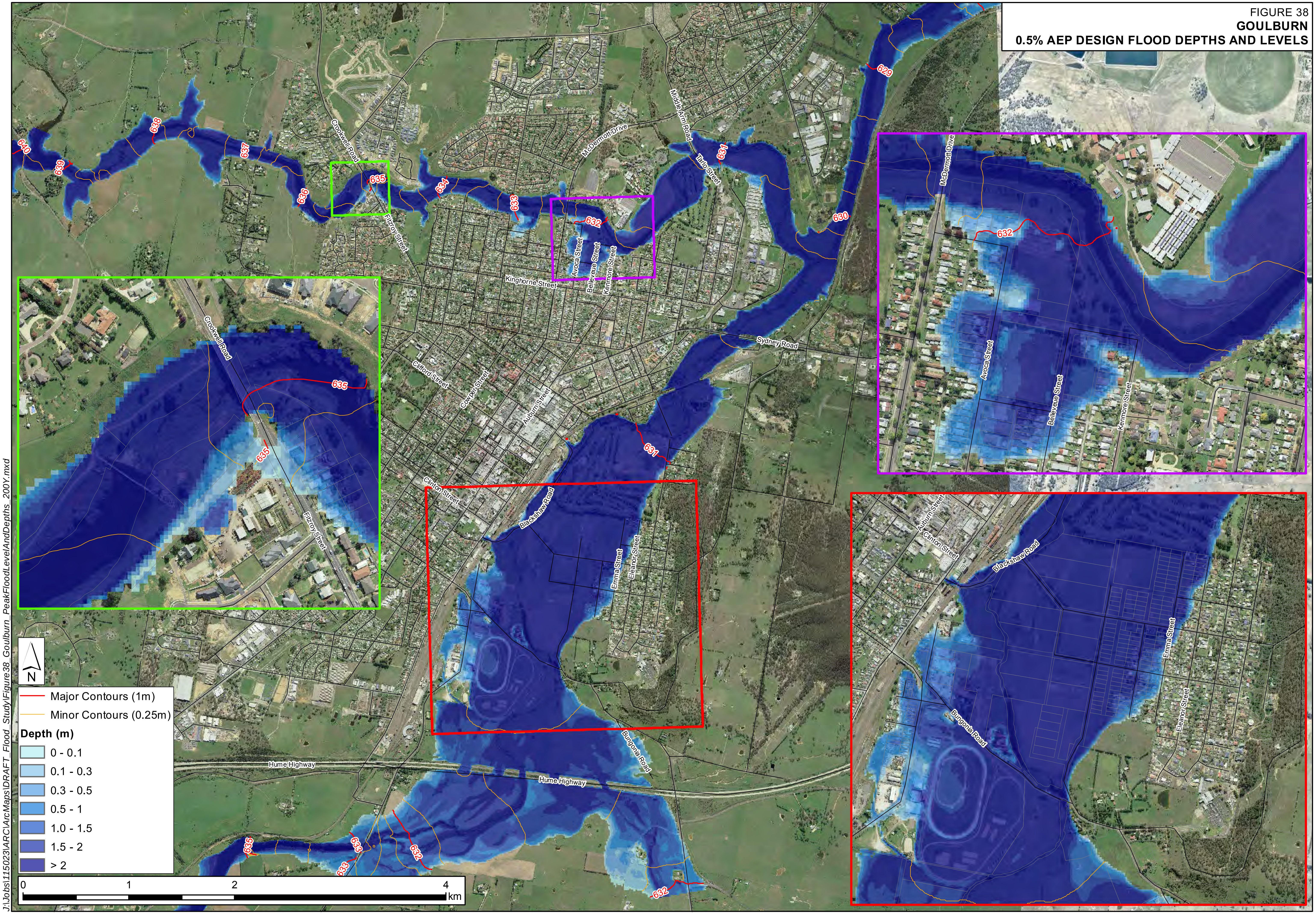
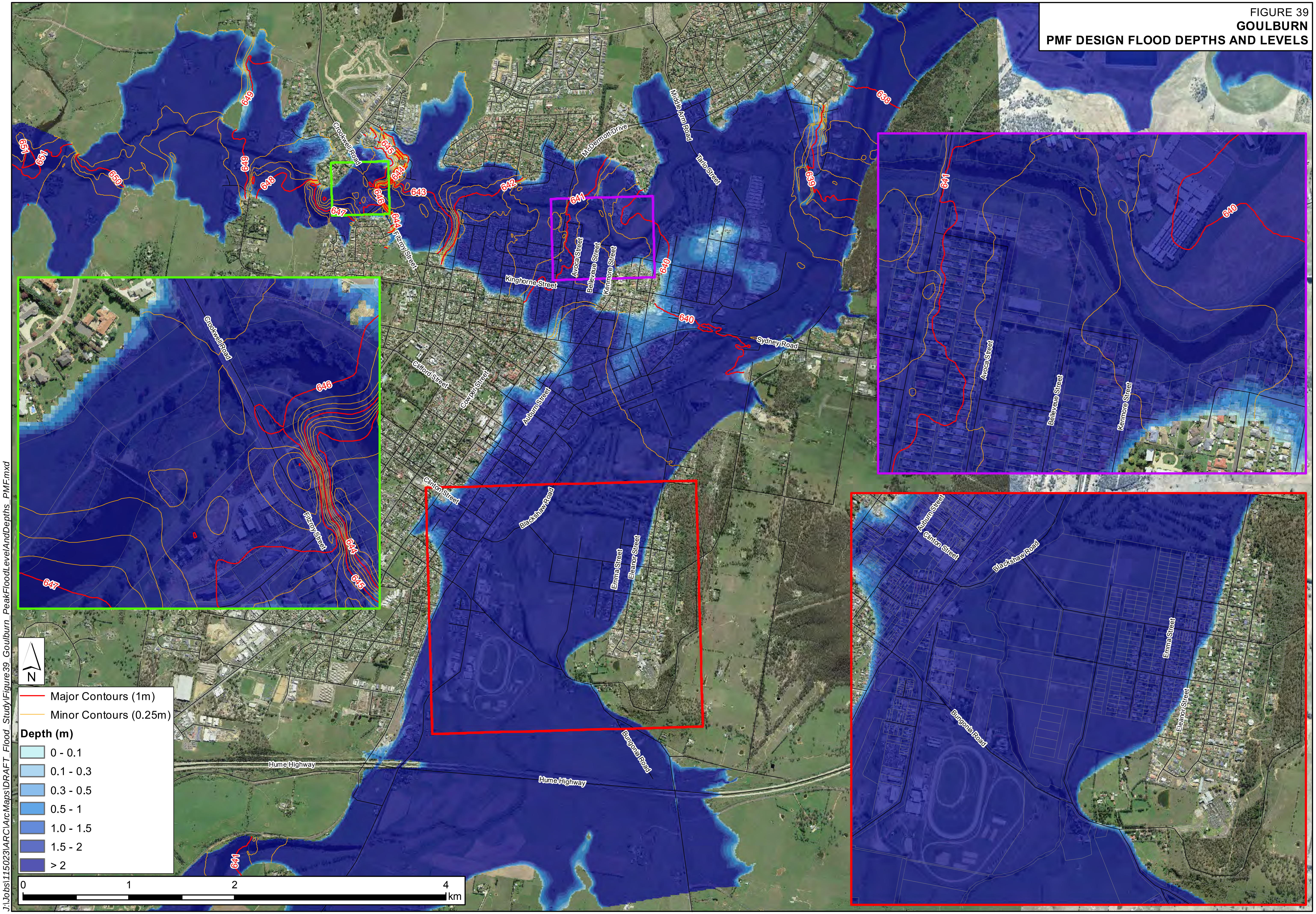
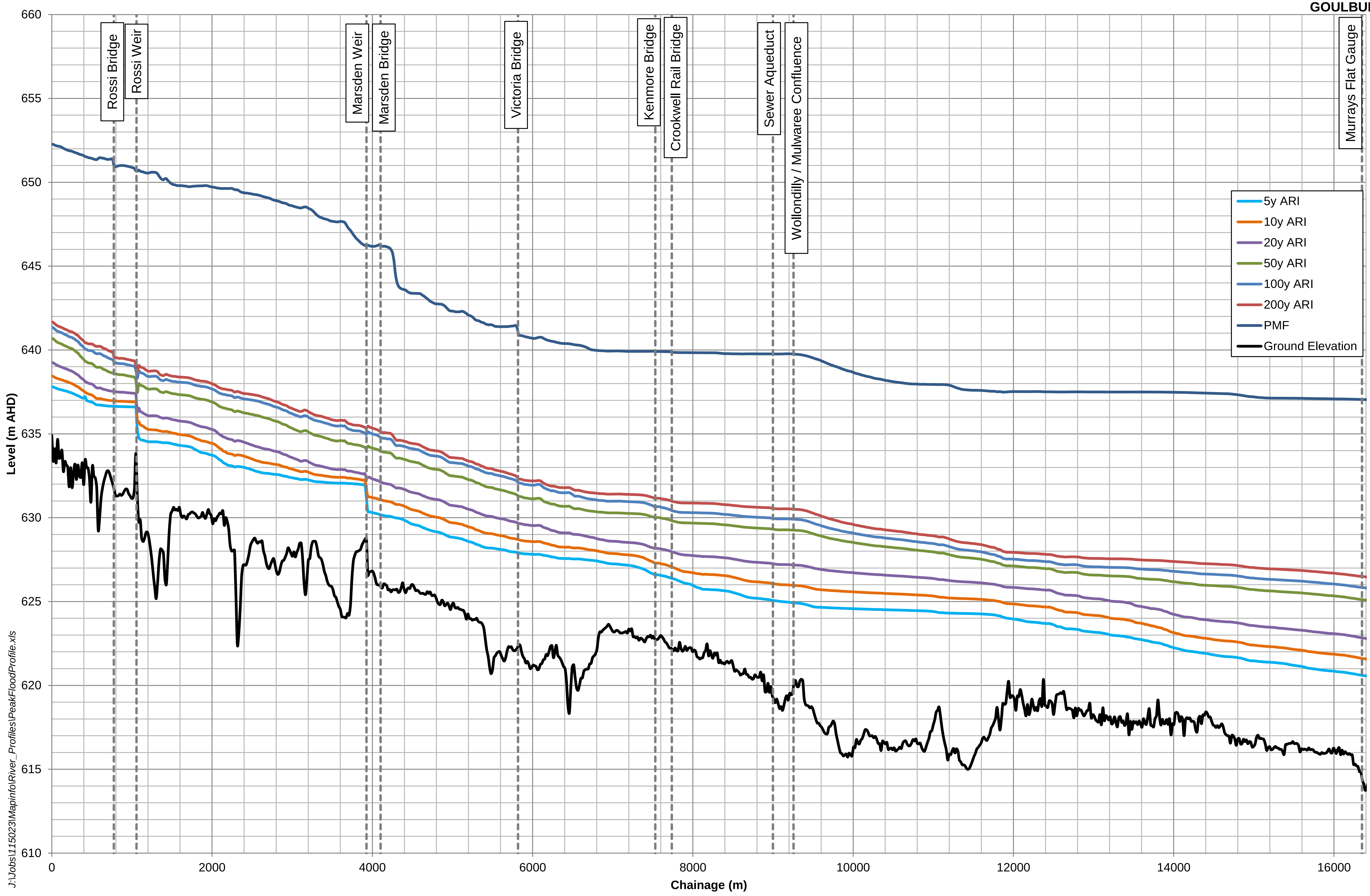


FIGURE 39
GOULBURN
PMF DESIGN FLOOD DEPTHS AND LEVELS



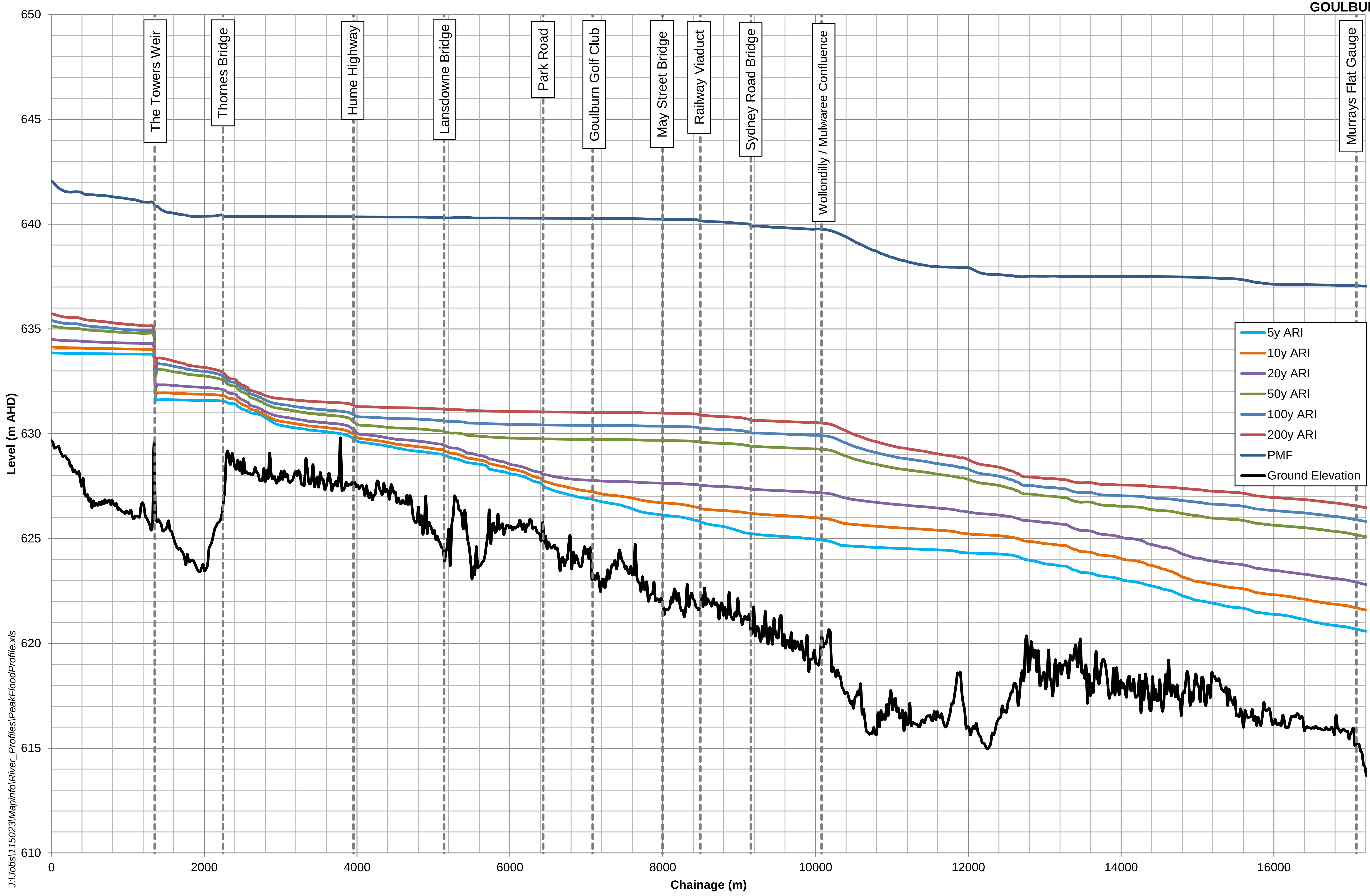
J:\Jobs\115023\ARC\ArcMaps\DRAFT_Flood_Study\Figure39_Goulburn_PeakFloodLevelAndDepths_PMF.mxd

FIGURE 40
WOLLONDILLY RIVER PEAK FLOOD PROFILE
DESIGN EVENTS
GOULBURN



J:\Jobs\115023\Mapinfo\River_Profiles\PeakFloodProfile.xls

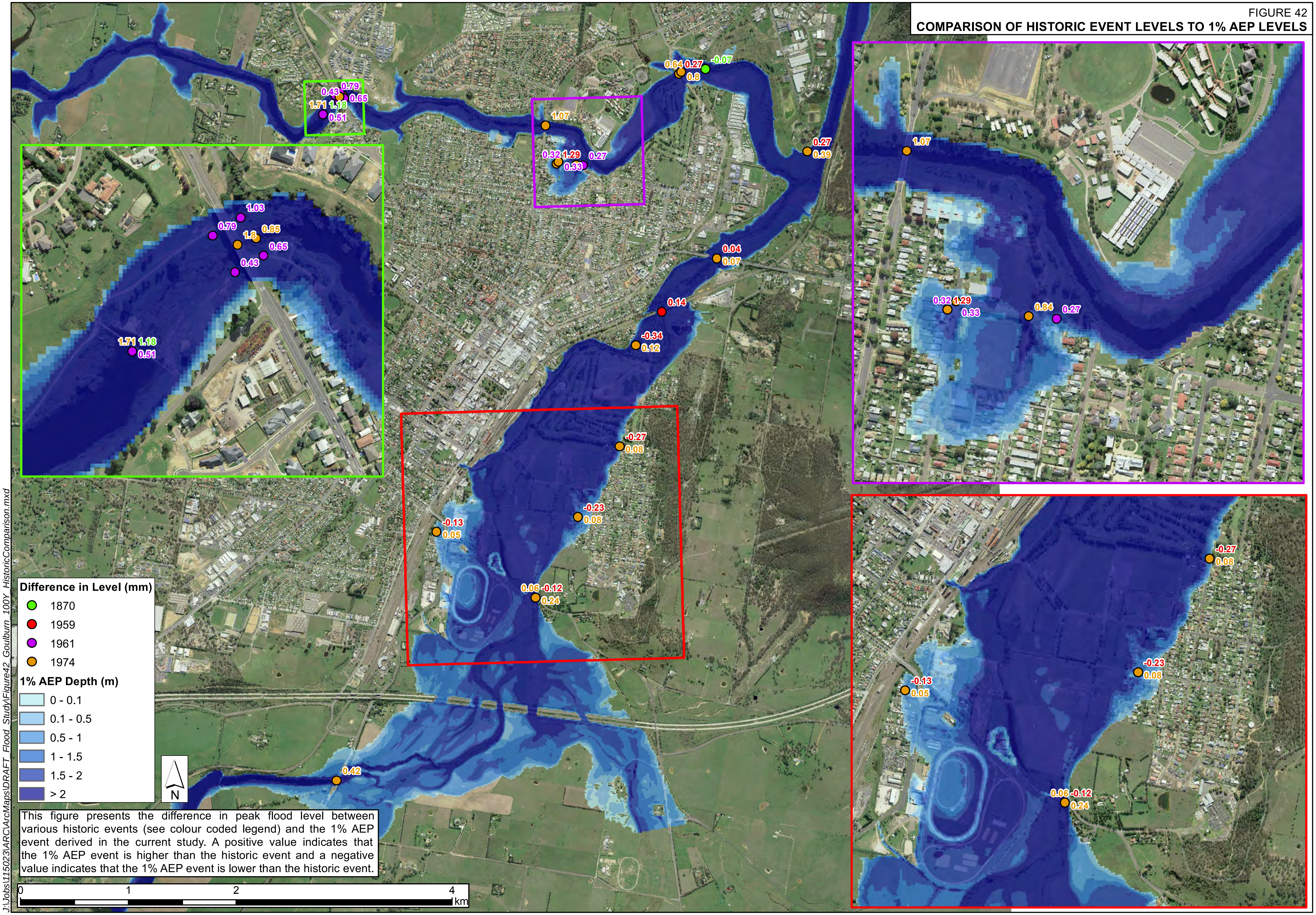
FIGURE 41
MULWAREE RIVER PEAK FLOOD PROFILE
DESIGN EVENTS
GOULBURN



J:\Jobs\115023\Mapinfo\River_Profiles\PeakFloodProfile.xls

FIGURE 42

COMPARISON OF HISTORIC EVENT LEVELS TO 1% AEP LEVELS



This figure presents the difference in peak flood level between various historic events (see colour coded legend) and the 1% AEP event derived in the current study. A positive value indicates that the 1% AEP event is higher than the historic event and a negative value indicates that the 1% AEP event is lower than the historic event.

J:\Jobs\115023\ARC\ArcMaps\DRAFT_Flood_Study\Figure42_Goulburn_100Y_HistoricComparison.mxd

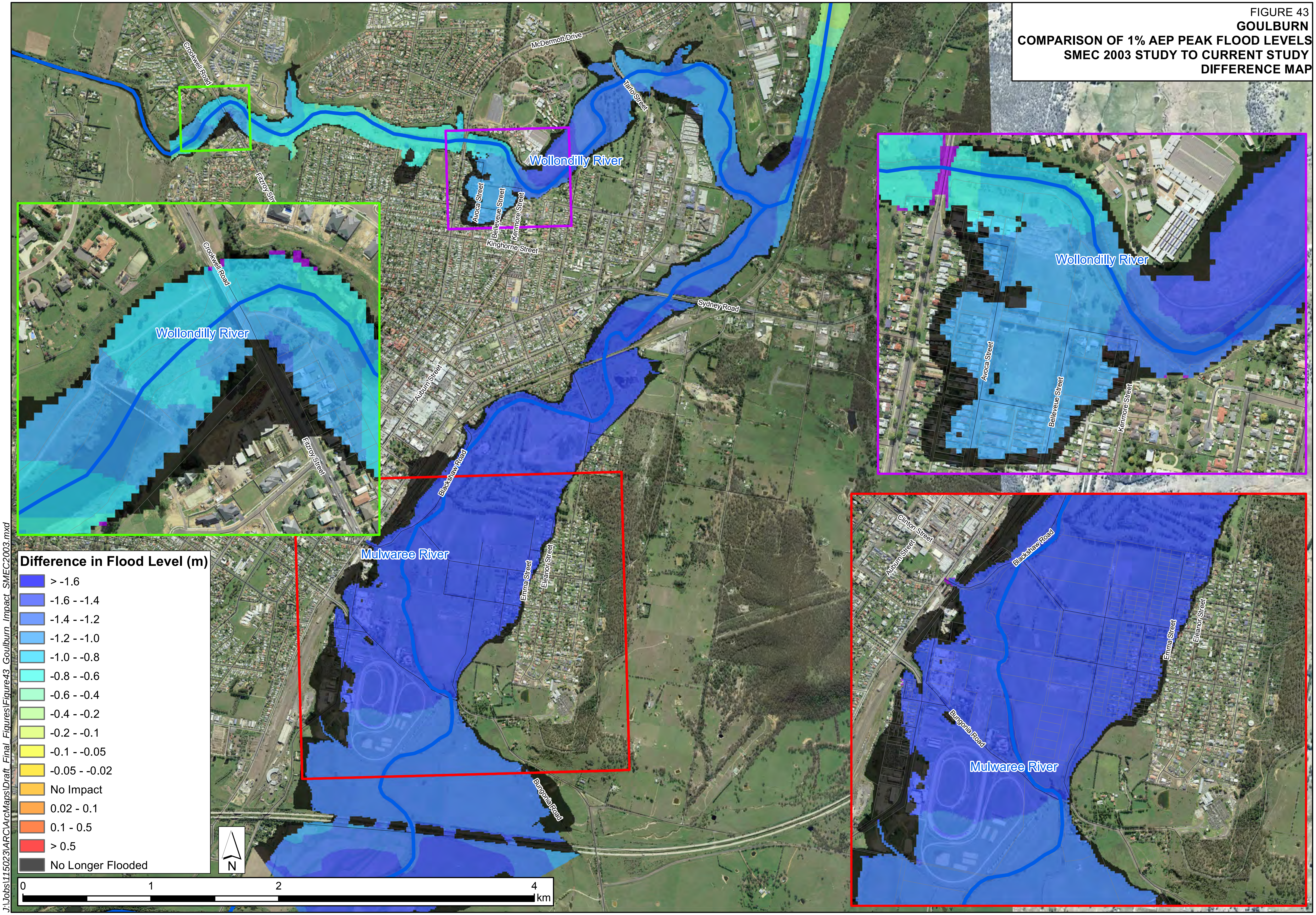
FIGURE 43

GOULBURN

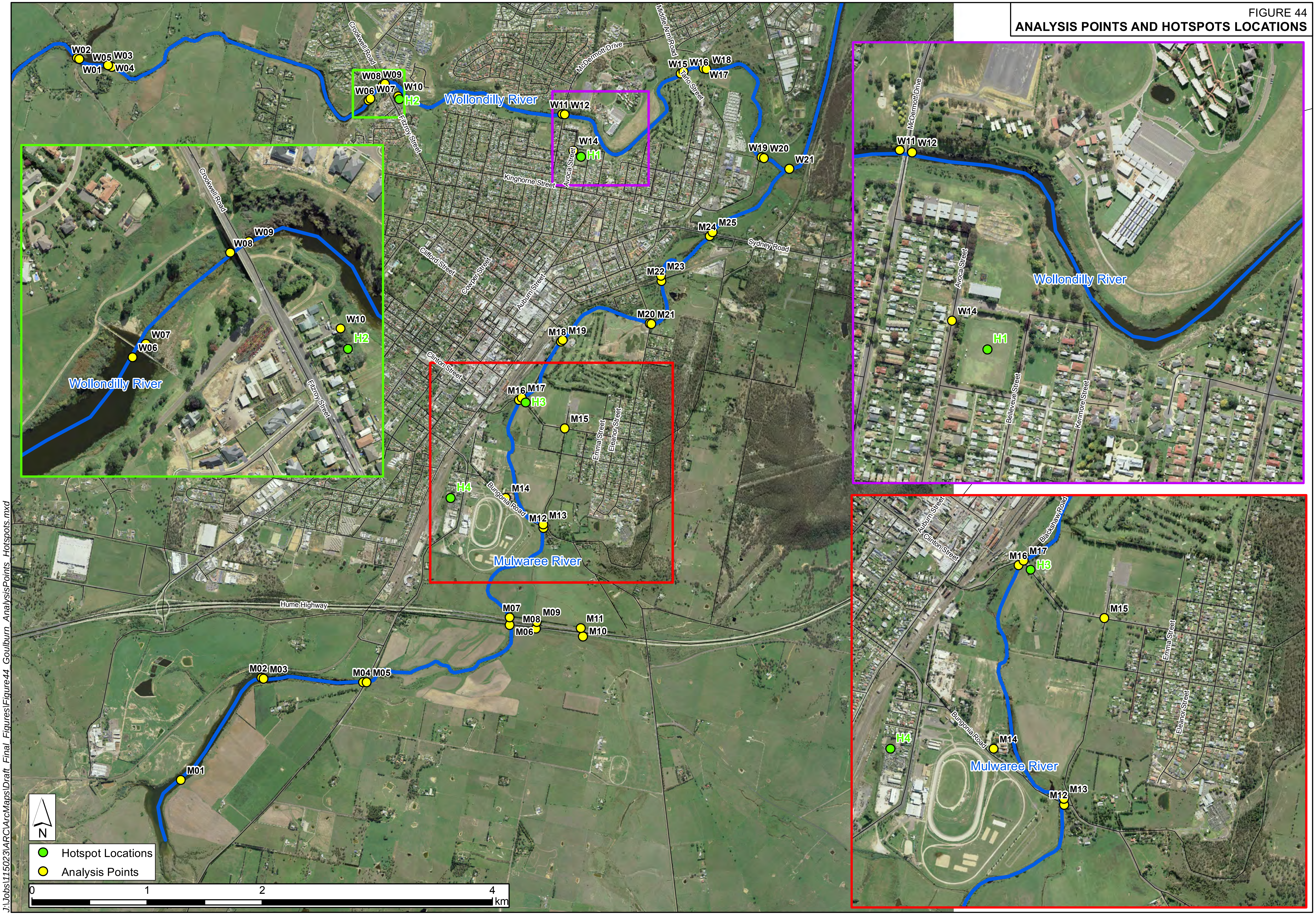
COMPARISON OF 1% AEP PEAK FLOOD LEVELS

SMEC 2003 STUDY TO CURRENT STUDY

DIFFERENCE MAP



J:\Jobs\115023\ARC\ArcMaps\Draft_Final_Figures\Figure43_Goulburn_Impact_SMEC2003.mxd



J:\Jobs\115023\ARC\ArcMaps\Draft_Final_Figures\Figure44_Goulburn_AnalysisPoints_Hotspots.mxd

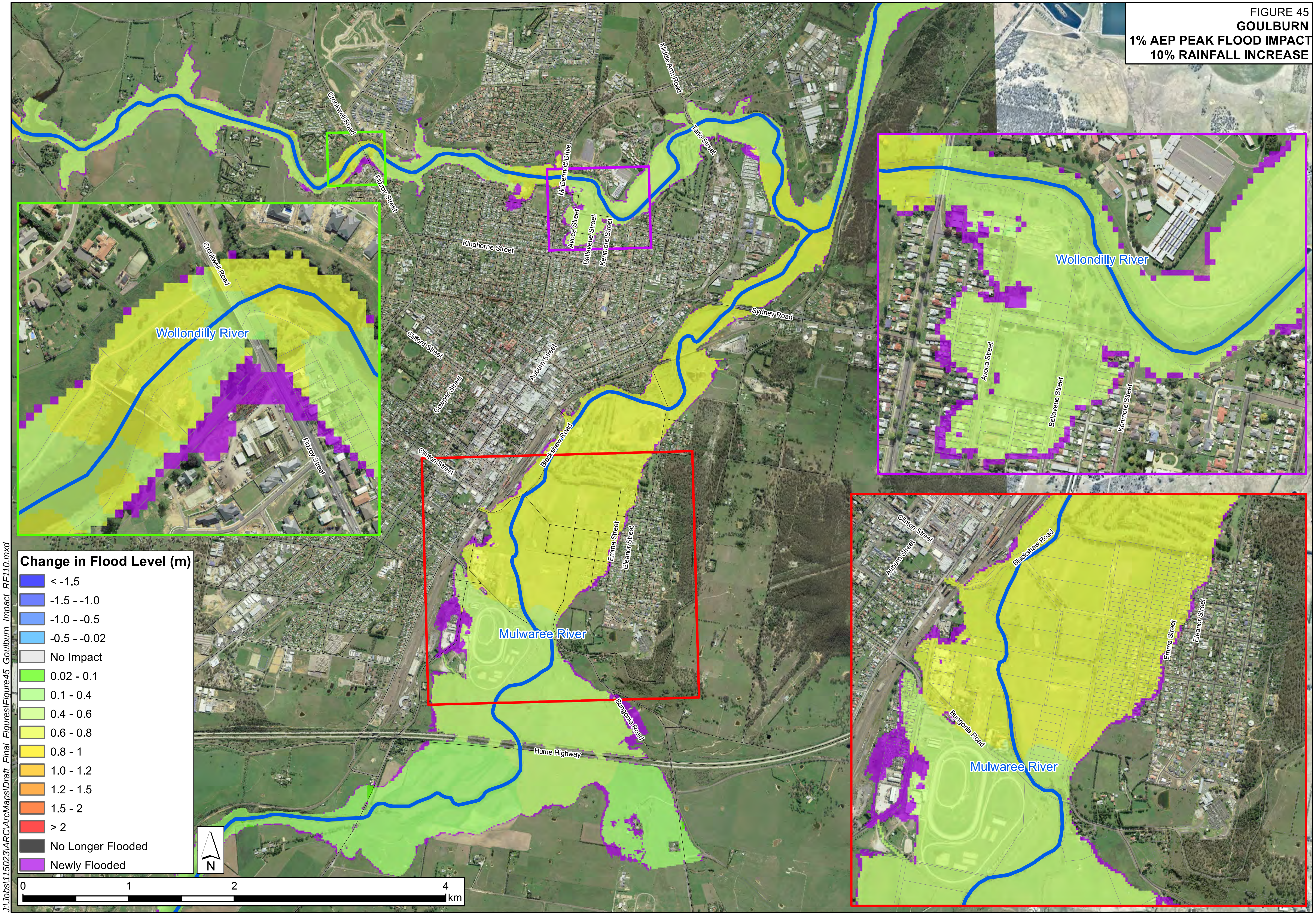
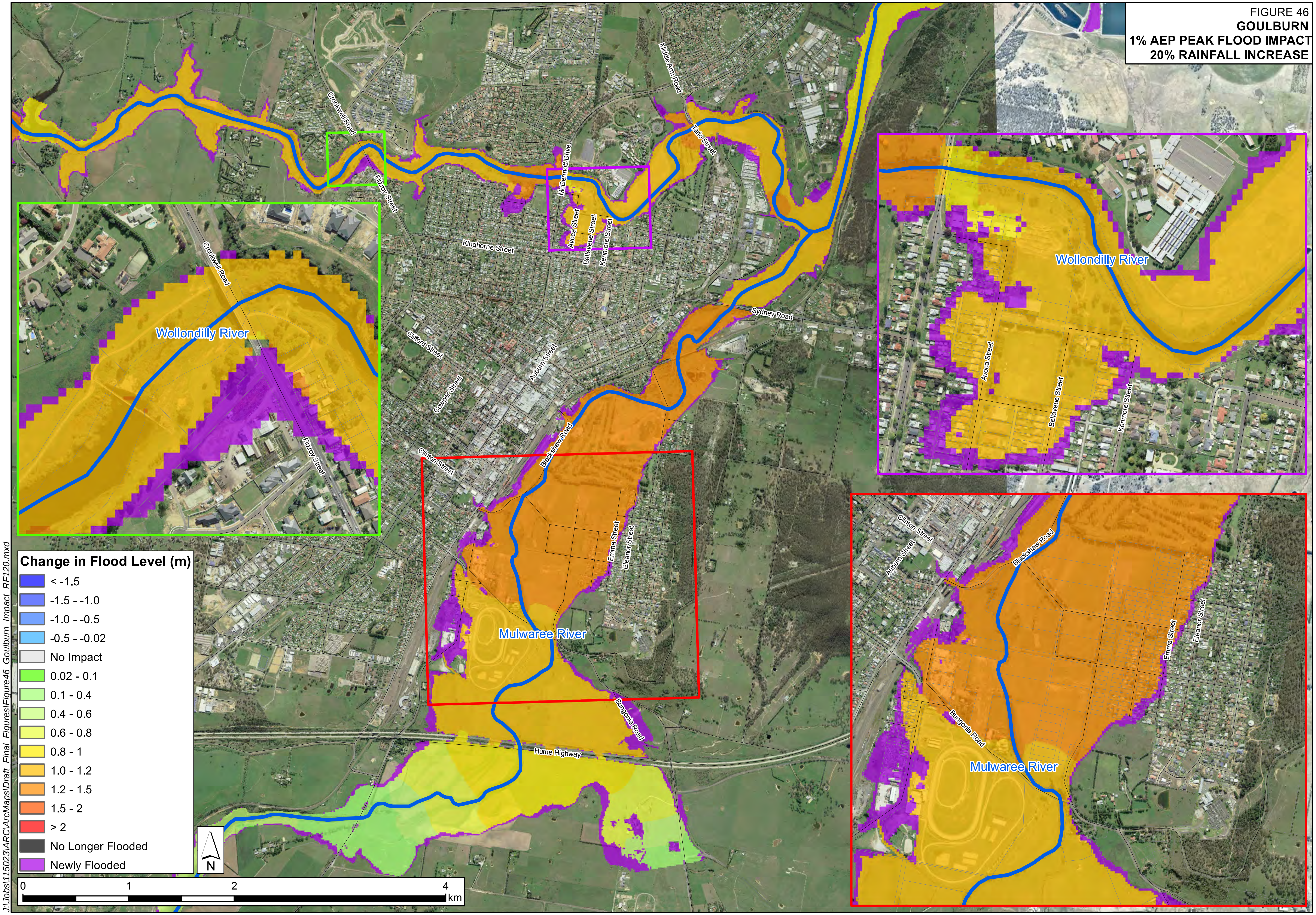


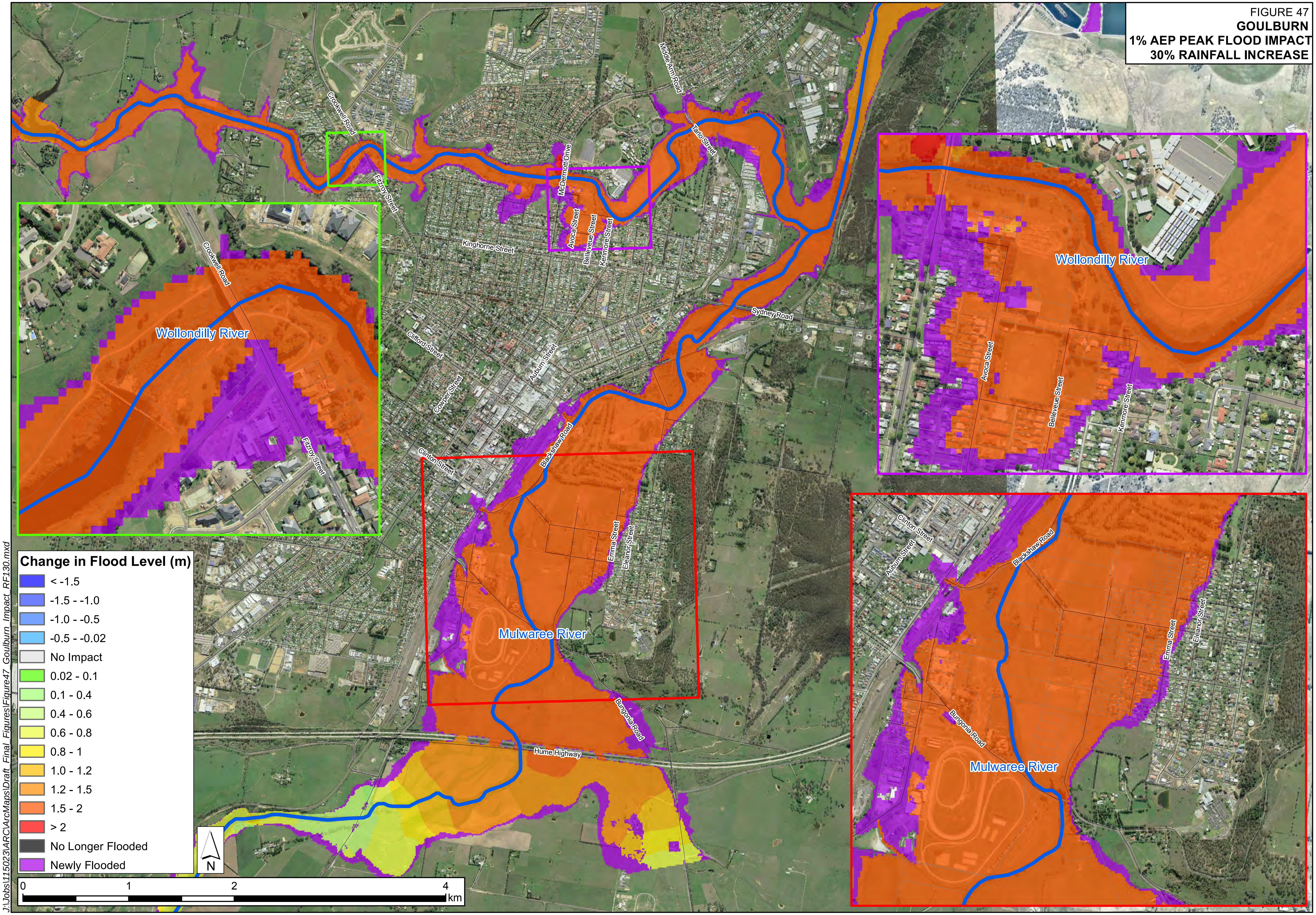
FIGURE 45
GOULBURN
1% AEP PEAK FLOOD IMPACT
10% RAINFALL INCREASE
Change in Flood Level (m)
< -1.5
-1.5 - -1.0
-1.0 - -0.5
-0.5 - -0.02
No Impact
0.02 - 0.1
0.1 - 0.4
0.4 - 0.6
0.6 - 0.8
0.8 - 1
1.0 - 1.2
1.2 - 1.5
1.5 - 2
> 2
No Longer Flooded
Newly Flooded
Wollondilly River
Mulwaree River
Crookwell Road
Fitzroy Street
McDermott Drive
Avoca Street
Bellevue Street
Kenmore Street
Middle Arm Road
Taralo Street
Kinghorne Street
Sydney Road
Clifford Street
Cowper Street
Auburn Street
Clinton Street
Blackshaw Road
Emma Street
Eleanor Street
Bungonia Road
Hume Highway
N
0 1 2 4 km
J:\Jobs\115023\ARC\ArcMaps\Draft_Final_Figures\Figure45_Goulburn_Impact_RF110.mxd

FIGURE 46
GOULBURN
1% AEP PEAK FLOOD IMPACT
20% RAINFALL INCREASE



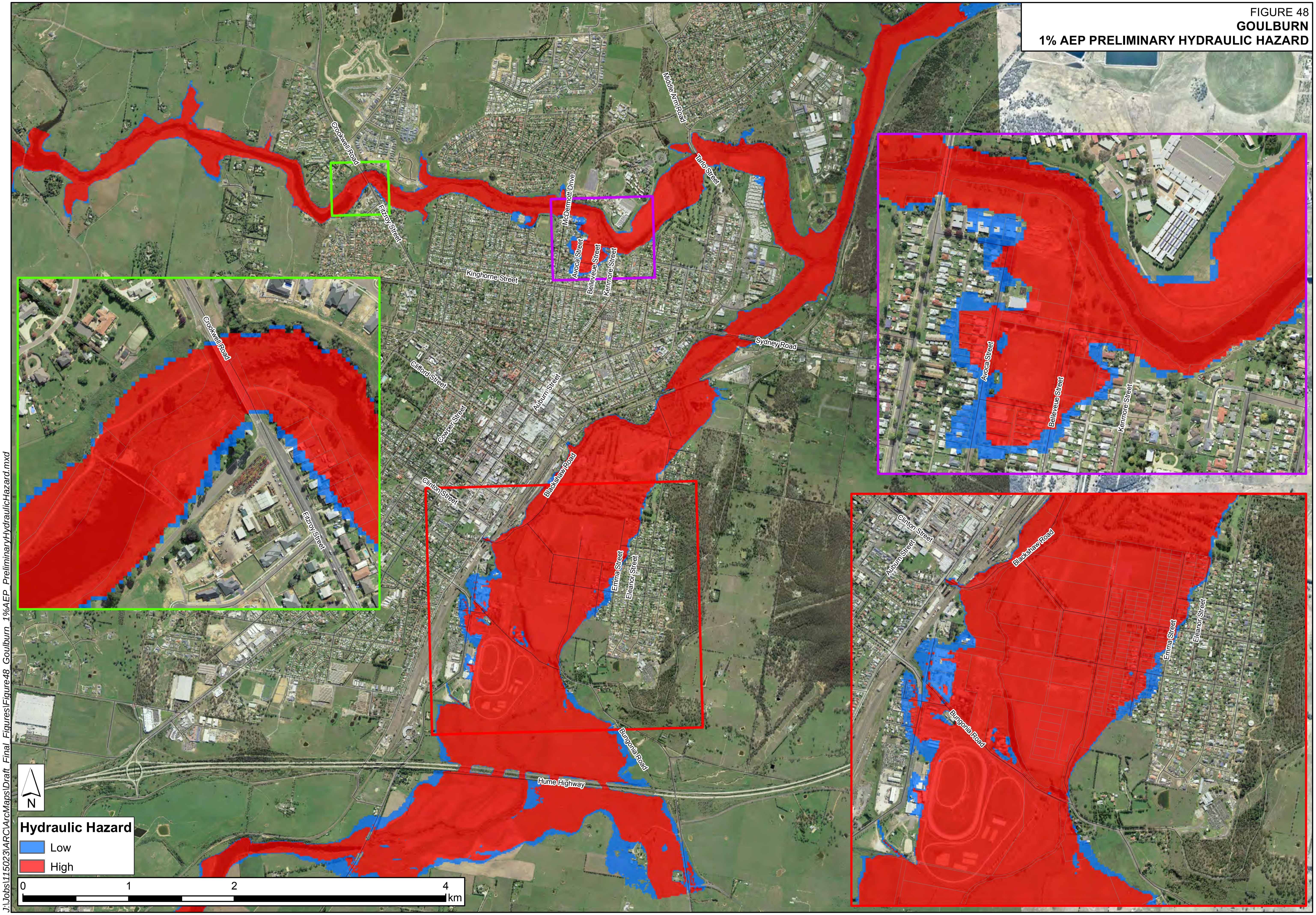
J:\Jobs\115023\ARC\ArcMaps\Draft_Final_Figures\Figure46_Goulburn_Impact_RF120.mxd

FIGURE 47
GOULBURN
1% AEP PEAK FLOOD IMPACT
30% RAINFALL INCREASE



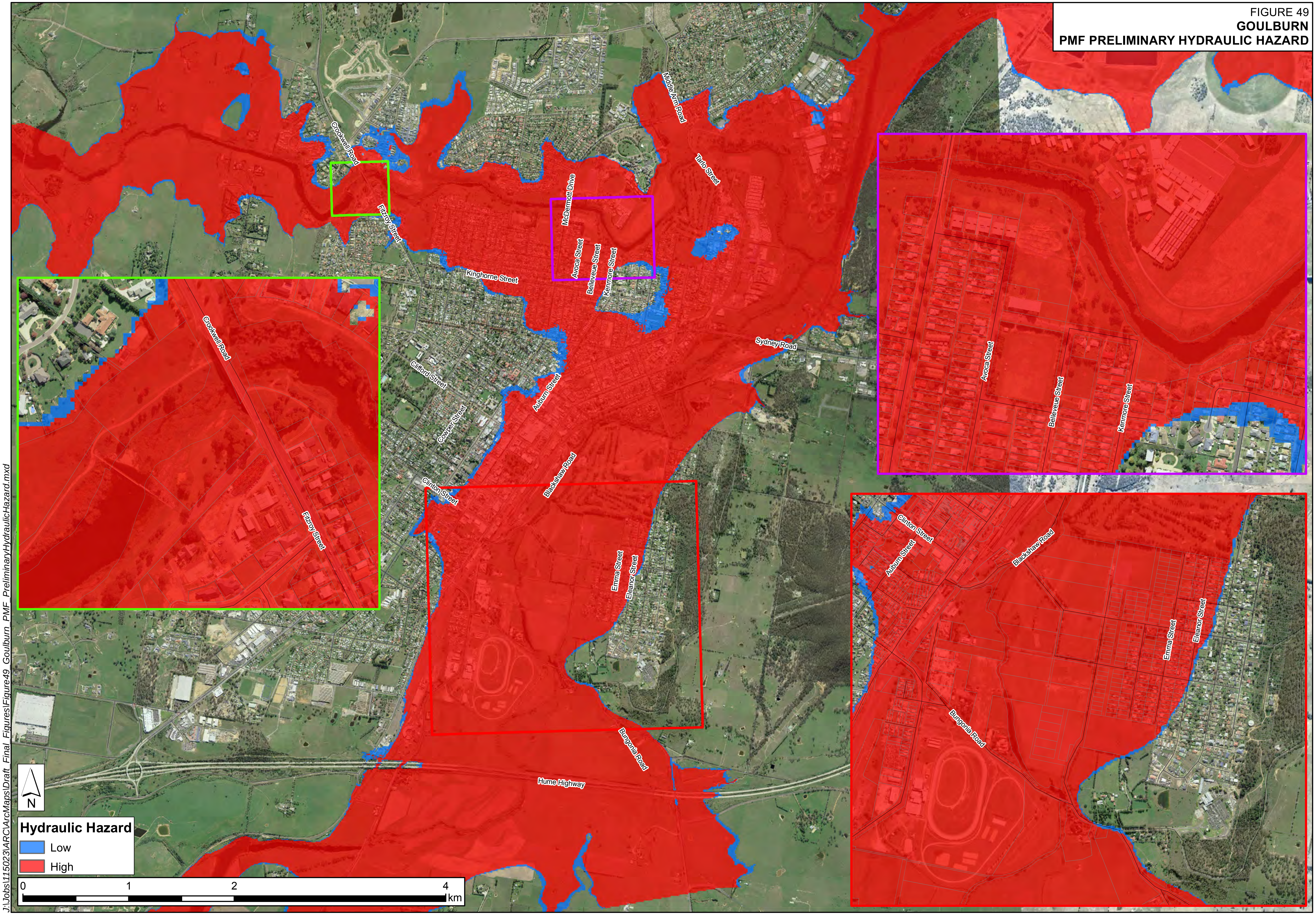
J:\Jobs\115023\ARC\ArcMaps\Draft_Final_Figures\Figure47_Goulburn_Impact_RF130.mxd

FIGURE 48
GOULBURN
1% AEP PRELIMINARY HYDRAULIC HAZARD



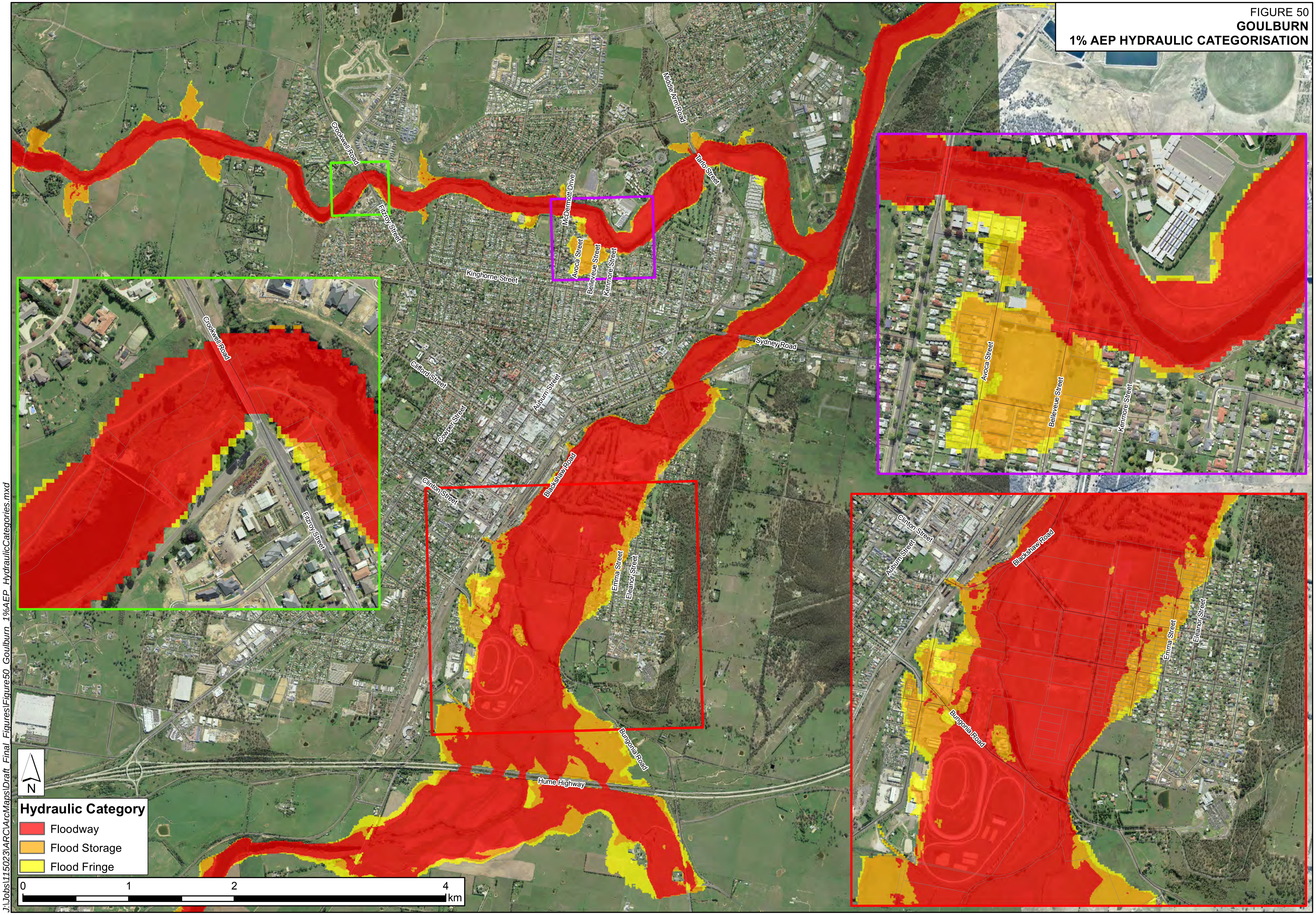
J:\Jobs\115023\ARC\ArcMaps\Draft_Final_Figures\Figure48_Goulburn_1%AEP_PreliminaryHydraulicHazard.mxd

FIGURE 49
GOULBURN
PMF PRELIMINARY HYDRAULIC HAZARD



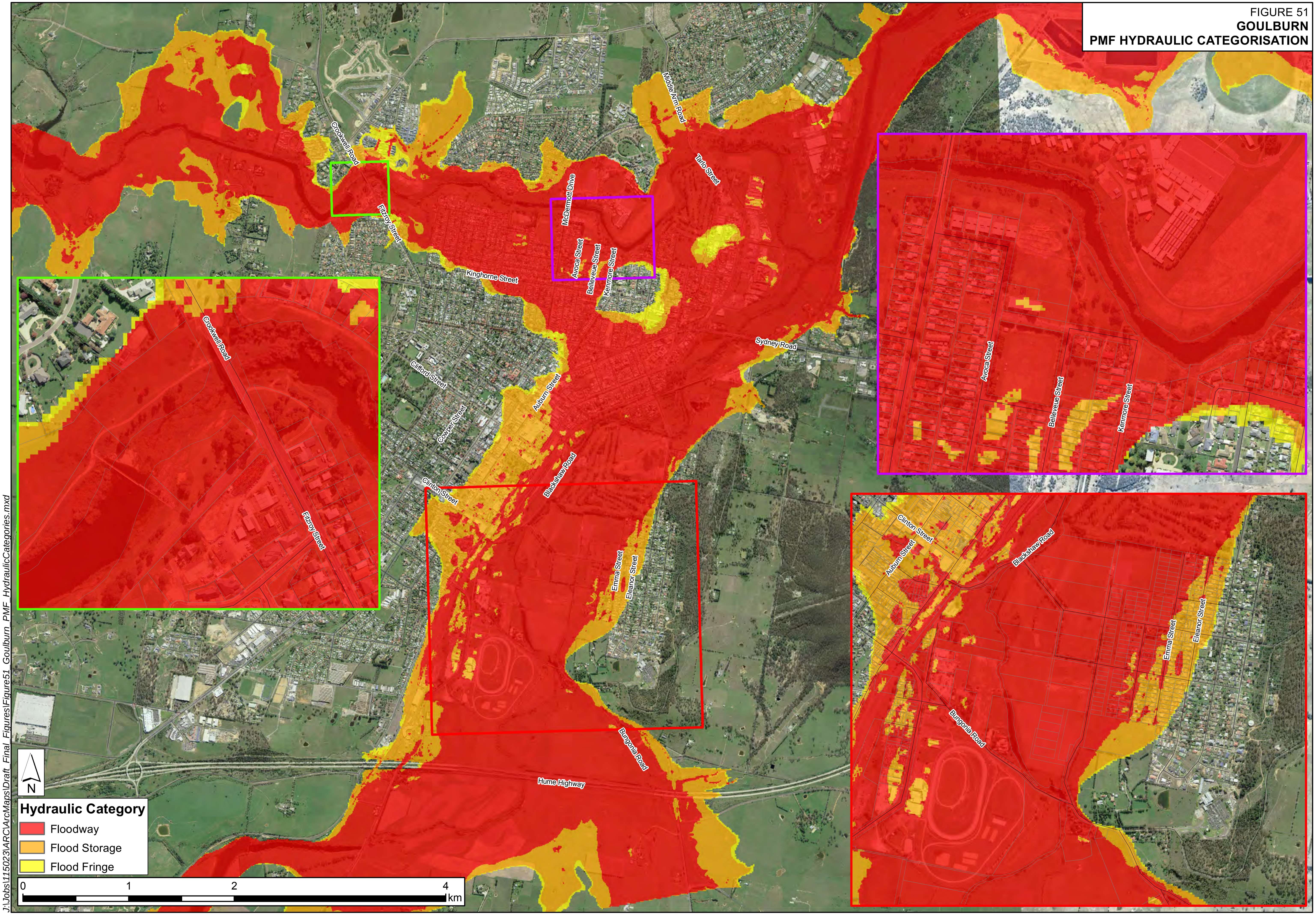
J:\Jobs\115023\ARC\ArcMaps\Draft_Final_Figures\Figure49_Goulburn_PMF_PreliminaryHydraulicHazard.mxd

FIGURE 50
GOULBURN
1% AEP HYDRAULIC CATEGORISATION

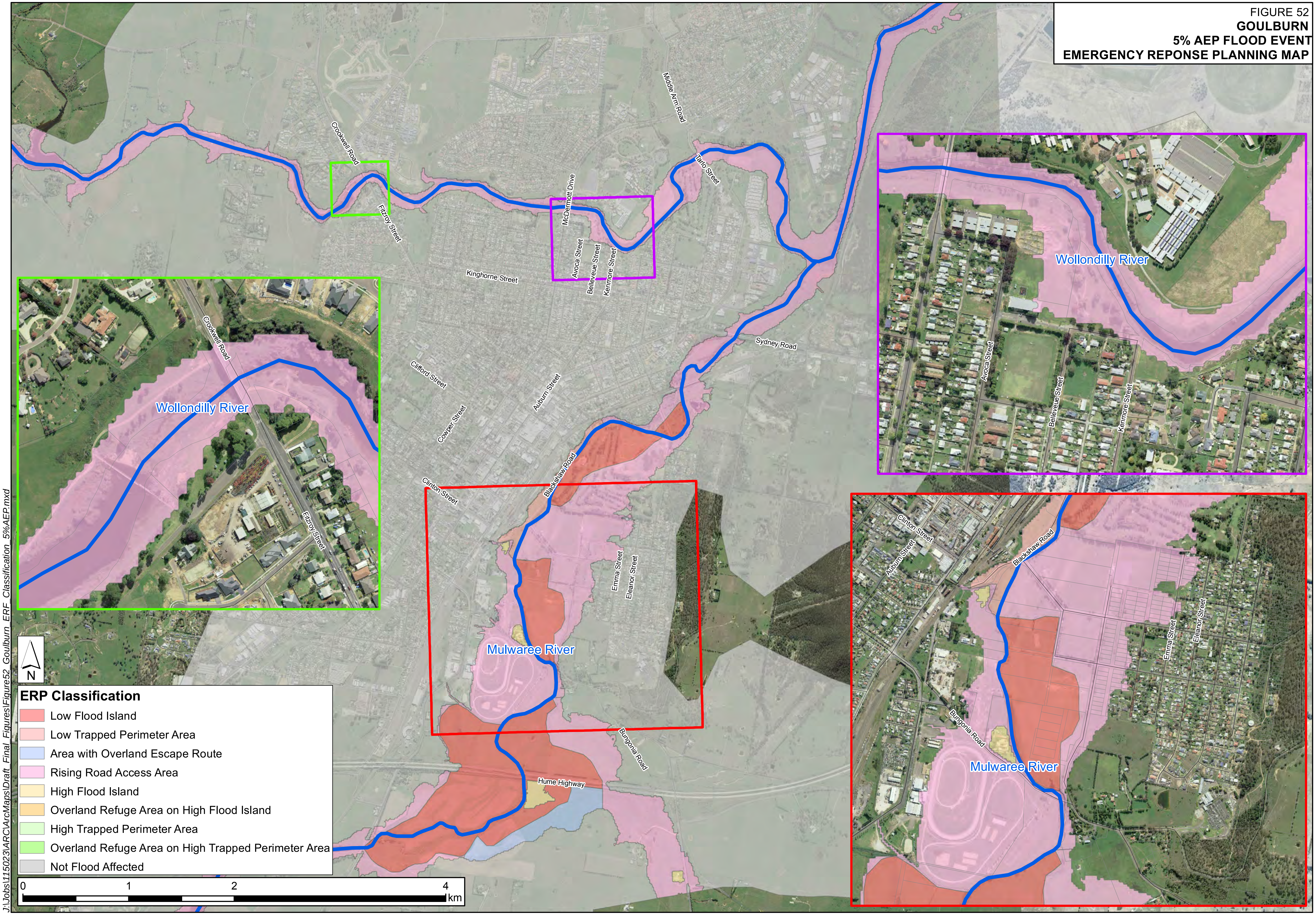


J:\Jobs\115023\ARC\ArcMaps\Draft_Final_Figures\Figure50_Goulburn_1%AEP_HydraulicCategories.mxd

FIGURE 51
GOULBURN
PMF HYDRAULIC CATEGORISATION

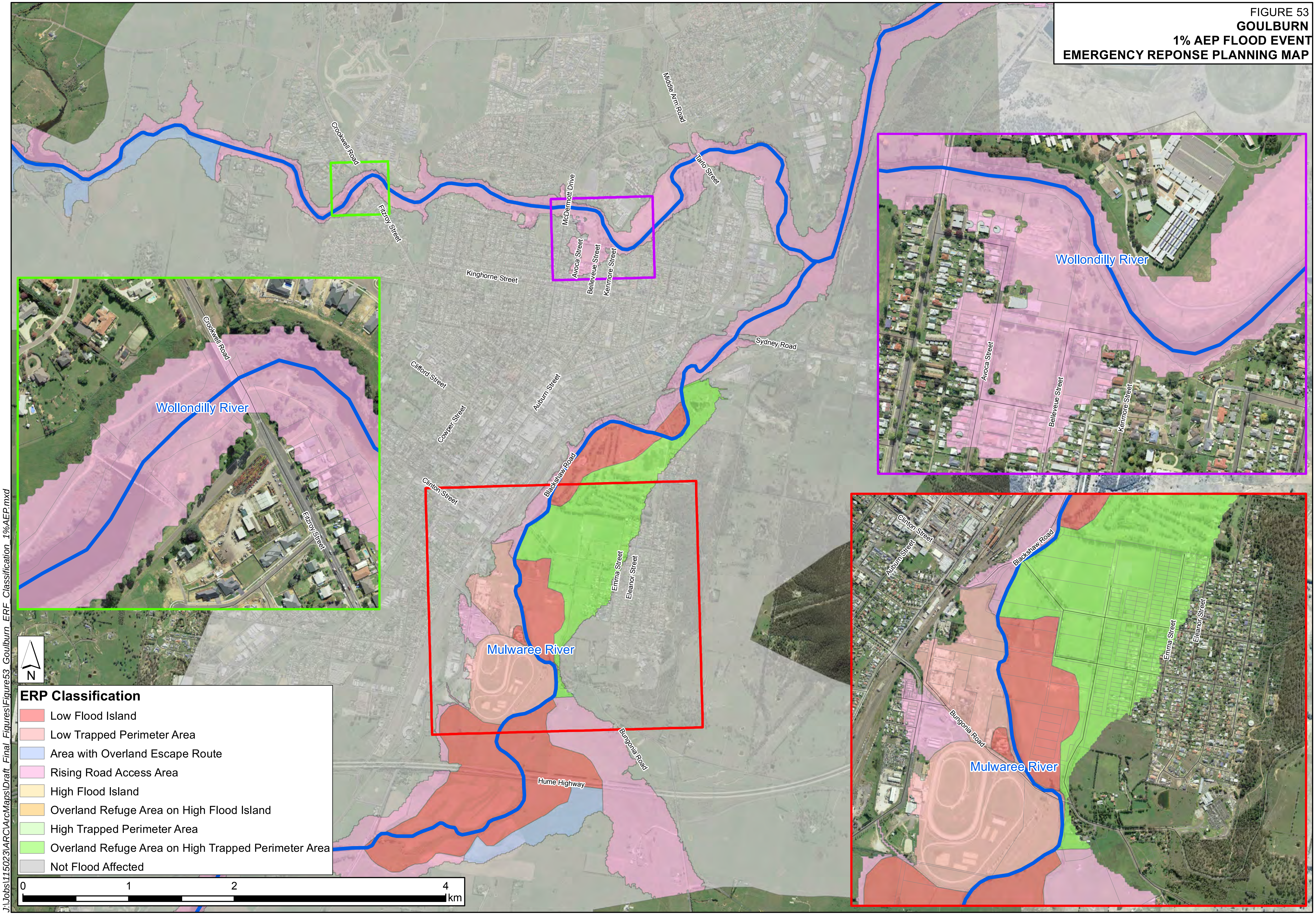


J:\Jobs\115023\ARC\ArcMaps\Draft_Final_Figures\Figure51_Goulburn_PMF_HydraulicCategories.mxd



J:\Jobs\115023\ARC\ArcMaps\Draft_Final_Figures\Figure52_Goulburn_ERF_Classification_5%AEP.mxd

FIGURE 53
GOULBURN
1% AEP FLOOD EVENT
EMERGENCY REPONSE PLANNING MAP



J:\Jobs\115023\ARC\ArcMaps\Draft_Final_Figures\Figure53_Goulburn_ERF_Classification_1%AEP.mxd

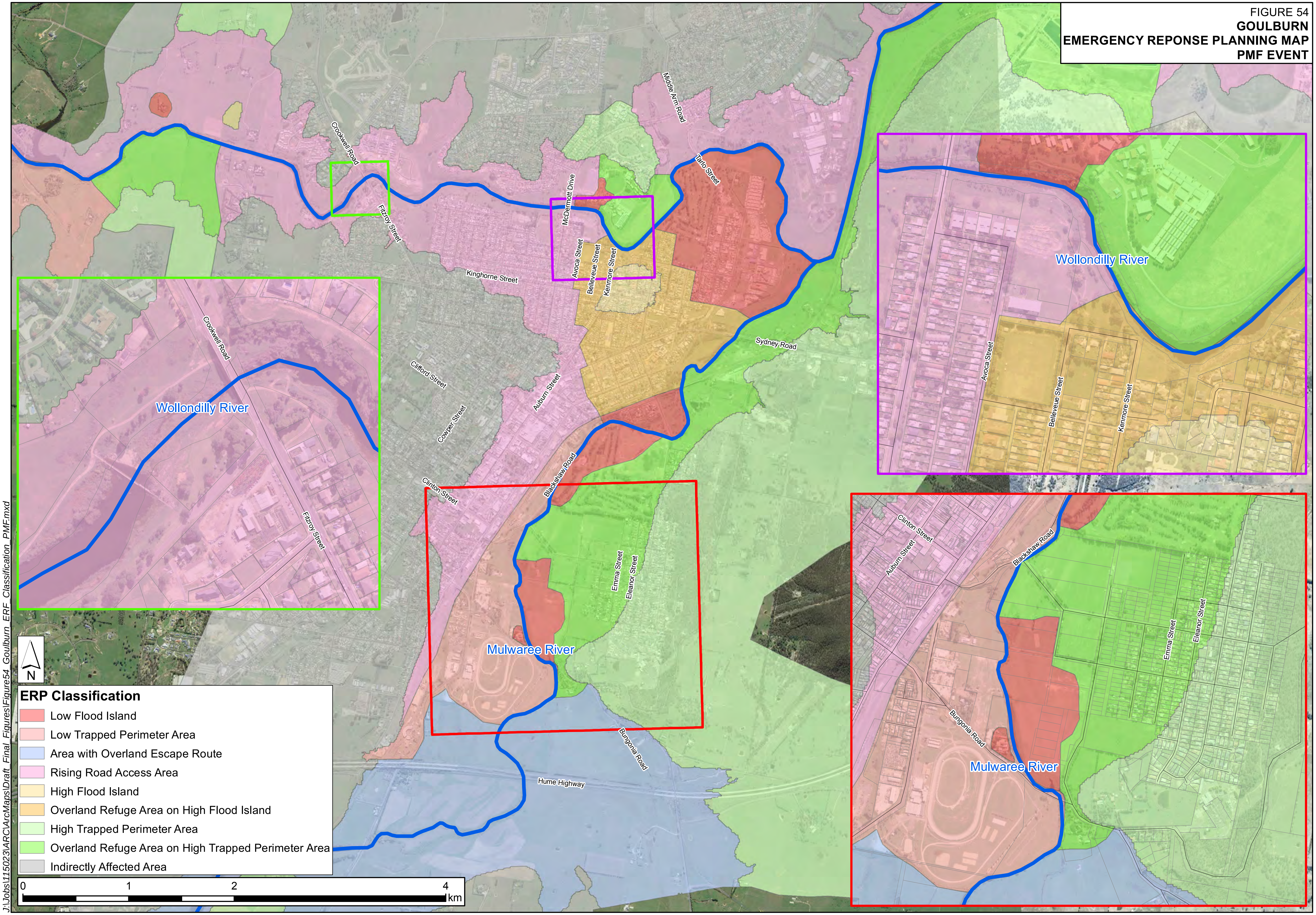


FIGURE 54
GOULBURN
EMERGENCY REPONSE PLANNING MAP
PMF EVENT
ERP Classification
Low Flood Island
Low Trapped Perimeter Area
Area with Overland Escape Route
Rising Road Access Area
High Flood Island
Overland Refuge Area on High Flood Island
High Trapped Perimeter Area
Overland Refuge Area on High Trapped Perimeter Area
Indirectly Affected Area
0
1
2
4
km
N
Wollondilly River
Mulwaree River
Crookwell Road
Fitzroy Street
McDermott Drive
Avoca Street
Belleveue Street
Kenmore Street
Tarlo Street
Middle Arm Road
Kinghorne Street
Sydney Road
Clifford Street
Cowper Street
Auburn Street
Clinton Street
Blackshaw Road
Emma Street
Eleanor Street
Bungonia Road
Hume Highway
J:\Jobs\115023\ARC\ArcMaps\Draft_Final_Figures\Figure54_Goulburn_ERP_Classification_PMF.mxd

Appendix A

APPENDIX A: GLOSSARY of TERMS

Taken from the Floodplain Development Manual (April 2005 edition)

acid sulfate soils	Are sediments which contain sulfidic mineral pyrite which may become extremely acid following disturbance or drainage as sulfur compounds react when exposed to oxygen to form sulfuric acid. More detailed explanation and definition can be found in the NSW Government Acid Sulfate Soil Manual published by Acid Sulfate Soil Management Advisory Committee.
Annual Exceedance Probability (AEP)	The chance of a flood of a given or larger size occurring in any one year, usually expressed as a percentage. For example, if a peak flood discharge of 500 m^3/s has an AEP of 5%, it means that there is a 5% chance (that is one-in-20 chance) of a 500 m^3/s or larger event occurring in any one year (see ARI).
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level.
Average Annual Damage (AAD)	Depending on its size (or severity), each flood will cause a different amount of flood damage to a flood prone area. AAD is the average damage per year that would occur in a nominated development situation from flooding over a very long period of time.
Average Recurrence Interval (ARI)	The long term average number of years between the occurrence of a flood as big as, or larger than, the selected event. For example, floods with a discharge as great as, or greater than, the 20 year ARI flood event will occur on average once every 20 years. ARI is another way of expressing the likelihood of occurrence of a flood event.
caravan and moveable home parks	Caravans and moveable dwellings are being increasingly used for long-term and permanent accommodation purposes. Standards relating to their siting, design, construction and management can be found in the Regulations under the LG Act.
catchment	The land area draining through the main stream, as well as tributary streams, to a particular site. It always relates to an area above a specific location.
consent authority	The Council, Government agency or person having the function to determine a development application for land use under the EP&A Act. The consent authority is most often the Council, however legislation or an EPI may specify a Minister or public authority (other than a Council), or the Director General of DIPNR, as having the function to determine an application.
development	Is defined in Part 4 of the Environmental Planning and Assessment Act (EP&A Act). **infill development:** refers to the development of vacant blocks of land that are generally surrounded by developed properties and is permissible under the current zoning of the land. Conditions such as minimum floor levels may be imposed on infill development. **new development:** refers to development of a completely different nature to that associated with the former land use. For example, the urban subdivision of an area previously used for rural purposes. New developments involve rezoning and typically require major extensions of existing urban services, such as roads, water supply, sewerage and electric power. **redevelopment:** refers to rebuilding in an area. For example, as urban areas age, it may become necessary to demolish and reconstruct buildings on a relatively large scale. Redevelopment generally does not require either rezoning or major extensions to urban services.
disaster plan (DISPLAN)	A step by step sequence of previously agreed roles, responsibilities, functions, actions and management arrangements for the conduct of a single or series of connected emergency operations, with the object of ensuring the coordinated

	response by all agencies having responsibilities and functions in emergencies.
discharge	The rate of flow of water measured in terms of volume per unit time, for example, cubic metres per second (m^3/s). Discharge is different from the speed or velocity of flow, which is a measure of how fast the water is moving for example, metres per second (m/s).
ecologically sustainable development (ESD)	Using, conserving and enhancing natural resources so that ecological processes, on which life depends, are maintained, and the total quality of life, now and in the future, can be maintained or increased. A more detailed definition is included in the Local Government Act 1993. The use of sustainability and sustainable in this manual relate to ESD.
effective warning time	The time available after receiving advice of an impending flood and before the floodwaters prevent appropriate flood response actions being undertaken. The effective warning time is typically used to move farm equipment, move stock, raise furniture, evacuate people and transport their possessions.
emergency management	A range of measures to manage risks to communities and the environment. In the flood context it may include measures to prevent, prepare for, respond to and recover from flooding.
flash flooding	Flooding which is sudden and unexpected. It is often caused by sudden local or nearby heavy rainfall. Often defined as flooding which peaks within six hours of the causative rain.
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 local overland flooding associated with major drainage before entering a watercourse, and/or coastal inundation resulting from super-elevated sea levels and/or waves overtopping coastline defences excluding tsunami.
flood awareness	Flood awareness is an appreciation of the likely effects of flooding and a knowledge of the relevant flood warning, response and evacuation procedures.
flood education	Flood education seeks to provide information to raise awareness of the flood problem so as to enable individuals to understand how to manage themselves an their property in response to flood warnings and in a flood event. It invokes a state of flood readiness.
flood fringe areas	The remaining area of flood prone land after floodway and flood storage areas have been defined.
flood liable land	Is synonymous with flood prone land (i.e. land susceptible to flooding by the probable maximum flood (PMF) event). Note that the term flood liable land covers the whole of the floodplain, not just that part below the flood planning level (see flood planning area).
flood mitigation standard	The average recurrence interval of the flood, selected as part of the floodplain risk management process that forms the basis for physical works to modify the impacts of flooding.
floodplain	Area of land which is subject to inundation by floods up to and including the probable maximum flood event, that is, flood prone land.
floodplain risk management options	The measures that might be feasible for the management of a particular area of the floodplain. Preparation of a floodplain risk management plan requires a detailed evaluation of floodplain risk management options.
floodplain risk management plan	A management plan developed in accordance with the principles and guidelines in this manual. Usually includes both written and diagrammatic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives.
flood plan (local)	A sub-plan of a disaster plan that deals specifically with flooding. They can exist at State, Division and local levels. Local flood plans are prepared under the leadership of the State Emergency Service.

flood planning area	The area of land below the flood planning level and thus subject to flood related development controls. The concept of flood planning area generally supersedes the Aflood liable land@ concept in the 1986 Manual.
Flood Planning Levels (FPLs)	FPL=s are the combinations of flood levels (derived from significant historical flood events or floods of specific AEPs) and freeboards selected for floodplain risk management purposes, as determined in management studies and incorporated in management plans. FPLs supersede the Astandard flood event@ in the 1986 manual.
flood proofing	A combination of measures incorporated in the design, construction and alteration of individual buildings or structures subject to flooding, to reduce or eliminate flood damages.
flood prone land	Is land susceptible to flooding by the Probable Maximum Flood (PMF) event. Flood prone land is synonymous with flood liable land.
flood readiness	Flood readiness is an ability to react within the effective warning time.
flood risk	Potential danger to personal safety and potential damage to property resulting from flooding. The degree of risk varies with circumstances across the full range of floods. Flood risk in this manual is divided into 3 types, existing, future and continuing risks. They are described below. **existing flood risk:** the risk a community is exposed to as a result of its location on the floodplain. **future flood risk:** the risk a community may be exposed to as a result of new development on the floodplain. **continuing flood risk:** the risk a community is exposed to after floodplain risk management measures have been implemented. For a town protected by levees, the continuing flood risk is the consequences of the levees being overtopped. For an area without any floodplain risk management measures, the continuing flood risk is simply the existence of its flood exposure.
flood storage areas	Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. The extent and behaviour of flood storage areas may change with flood severity, and loss of flood storage can increase the severity of flood impacts by reducing natural flood attenuation. Hence, it is necessary to investigate a range of flood sizes before defining flood storage areas.
floodway areas	Those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flows, or a significant increase in flood levels.
freeboard	Freeboard provides reasonable certainty that the risk exposure selected in deciding on a particular flood chosen as the basis for the FPL is actually provided. It is a factor of safety typically used in relation to the setting of floor levels, levee crest levels, etc. Freeboard is included in the flood planning level.
habitable room	**in a residential situation:** a living or working area, such as a lounge room, dining room, rumpus room, kitchen, bedroom or workroom. **in an industrial or commercial situation:** an area used for offices or to store valuable possessions susceptible to flood damage in the event of a flood.
hazard	A source of potential harm or a situation with a potential to cause loss. In relation to this manual the hazard is flooding which has the potential to cause damage to the community. Definitions of high and low hazard categories are provided in the Manual.
hydraulics	Term given to the study of water flow in waterways; in particular, the evaluation of flow parameters such as water level and velocity.

hydrograph	A graph which shows how the discharge or stage/flood level at any particular location varies with time during a flood.
hydrology	Term given to the study of the rainfall and runoff process; in particular, the evaluation of peak flows, flow volumes and the derivation of hydrographs for a range of floods.
local overland flooding	Inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.
local drainage	Are smaller scale problems in urban areas. They are outside the definition of major drainage in this glossary.
mainstream flooding	Inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary, lake or dam.
major drainage	Councils have discretion in determining whether urban drainage problems are associated with major or local drainage. For the purpose of this manual major drainage involves: • the floodplains of original watercourses (which may now be piped, channelised or diverted), or sloping areas where overland flows develop along alternative paths once system capacity is exceeded; and/or • water depths generally in excess of 0.3 m (in the major system design storm as defined in the current version of Australian Rainfall and Runoff). These conditions may result in danger to personal safety and property damage to both premises and vehicles; and/or • major overland flow paths through developed areas outside of defined drainage reserves; and/or • the potential to affect a number of buildings along the major flow path.
mathematical/computer models	The mathematical representation of the physical processes involved in runoff generation and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.
merit approach	The merit approach weighs social, economic, ecological and cultural impacts of land use options for different flood prone areas together with flood damage, hazard and behaviour implications, and environmental protection and well being of the State=s rivers and floodplains. The merit approach operates at two levels. At the strategic level it allows for the consideration of social, economic, ecological, cultural and flooding issues to determine strategies for the management of future flood risk which are formulated into Council plans, policy and EPIs. At a site specific level, it involves consideration of the best way of conditioning development allowable under the floodplain risk management plan, local floodplain risk management policy and EPIs.
minor, moderate and major flooding	Both the State Emergency Service and the Bureau of Meteorology use the following definitions in flood warnings to give a general indication of the types of problems expected with a flood: **minor flooding:** causes inconvenience such as closing of minor roads and the submergence of low level bridges. The lower limit of this class of flooding on the reference gauge is the initial flood level at which landholders and townspeople begin to be flooded. **moderate flooding:** low-lying areas are inundated requiring removal of stock and/or evacuation of some houses. Main traffic routes may be covered. **major flooding:** appreciable urban areas are flooded and/or extensive rural areas are flooded. Properties, villages and towns can be isolated.
modification measures	Measures that modify either the flood, the property or the response to flooding. Examples are indicated in Table 2.1 with further discussion in the Manual.

peak discharge	The maximum discharge occurring during a flood event.
Probable Maximum Flood (PMF)	The PMF is the largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation, and where applicable, snow melt, coupled with the worst flood producing catchment conditions. Generally, it is not physically or economically possible to provide complete protection against this event. The PMF defines the extent of flood prone land, that is, the floodplain. The extent, nature and potential consequences of flooding associated with a range of events rarer than the flood used for designing mitigation works and controlling development, up to and including the PMF event should be addressed in a floodplain risk management study.
Probable Maximum Precipitation (PMP)	The PMP is the greatest depth of precipitation for a given duration meteorologically possible over a given size storm area at a particular location at a particular time of the year, with no allowance made for long-term climatic trends (World Meteorological Organisation, 1986). It is the primary input to PMF estimation.
probability	A statistical measure of the expected chance of flooding (see AEP).
risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. In the context of the manual it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
runoff	The amount of rainfall which actually ends up as streamflow, also known as rainfall excess.
stage	Equivalent to Awater level@. Both are measured with reference to a specified datum.
stage hydrograph	A graph that shows how the water level at a particular location changes with time during a flood. It must be referenced to a particular datum.
survey plan	A plan prepared by a registered surveyor.
water surface profile	A graph showing the flood stage at any given location along a watercourse at a particular time.
wind fetch	The horizontal distance in the direction of wind over which wind waves are generated.

Appendix B

Newsletter - August 2015

Goulburn Flood Study

A Flood Study is currently being prepared for Goulburn Mulwaree Council examining flooding caused by the Wollondilly and Mulwaree Rivers in Goulburn.

The Floodplain Management Process

Goulburn Mulwaree Council is carrying out a Flood Study under the NSW Government's Flood Prone Land Policy. The primary objective of the Policy is to reduce the impact of flooding and flood liability on owners and occupants of flood prone land and to reduce losses from flooding. The Policy provides for technical and financial support by the State Government through four sequential stages:

1. **Flood Study**
   Determine the nature and extent of the flood problem
2. **Floodplain Risk Management**
   Evaluates management options for the floodplain in respect of existing and proposed development.
3. **Floodplain Risk Management Plan**
   Formal adoption by Council of a plan of management for the floodplain
4. **Implementation of the Plan**
   Construction of flood mitigation works to protect existing development and use of Council Planning Policies to ensure new development is compatible with the flood hazard.

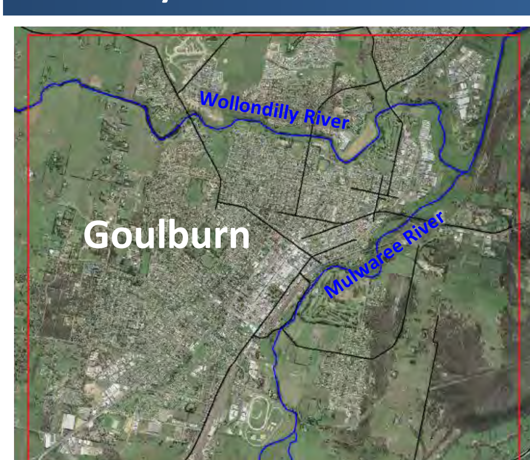
The Flood Study is phase one of the four step process listed above. The Study will define flood behaviour over a range of floods of varying magnitudes.

What's Happening Now?

This Flood Study aims to understand and determine the nature and extent of potential flooding due to the Wollondilly River and Mulwaree River at Goulburn. The first stage of the Flood Study will be to collect, compile and review all available information, including valuable community knowledge and experiences.

A computer model will determine the extent and nature of flooding in the Study Area (see below) and collected historical data, such as photos and observations of flooding behaviour, will be used to ensure model accuracy. In particular, information on observed peak flood levels is most important. This is where we need your help.

The Study Area



There is a long history of flooding in Goulburn due to the Wollondilly and Mulwaree Rivers and although infrequent, the cost of flooding to the community can be significant. The Wollondilly River has a catchment area of 720 km² at Goulburn and the Mulwaree River a catchment area of 760 km². Flooding of the Wollondilly and Mulwaree Rivers can occur simultaneously or independently, although flooding in the Wollondilly River often causes the Mulwaree to flood due to backwater.

Recently Goulburn has experienced flooding in low lying areas along these two rivers. Flood events in December 2010, March 2012 and June 2013 resulted in over floor flooding and the evacuation of residents in flood risk areas. In response to these floods, Council has undertaken a number of steps to reduce flood risk. This Flood Study is a part of this process.

Goulburn Flood Study

Community involvement in this Study is important. The Goulburn Floodplain Management Committee will oversee this Study and includes members from Council, Office of Environment and Heritage, Department of Planning, the State Emergency Services and local residents. A questionnaire is enclosed with this newsletter so that your views can also be included. You are also invited to contact WMAwater or Council personnel to discuss your flood experience and details of the Flood Study. Contact details are provided below.

How can I have my say?

Please complete the enclosed questionnaire and return to the FREEPOST address in the envelope provided before 1st October 2015.

If you have additional information or further comments, please attach these to your questionnaire response or email to the contacts below.

Alternatively you can or fill out the online survey at:

https://www.surveymonkey.com/r/Goulburn_FS

This newsletter and questionnaire forms part of our community consultation to collect information about previous floods and flood behaviour. The local knowledge and personal experiences of residents and business operators are an important source of information. We are specifically interested in historical records of flooding such as photographs, flood marks or observations that you may have.

Feedback from the community will be analysed and used to establish an accurate flood model of the study area. After data collection, the preliminary results will be produced and a draft study placed on public exhibition. You will be invited to view and comment on the Study and public forums will be held to present and discuss the Study results.

Contacts

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WMAwater
Level 2, 160 Clarence Street
Sydney, NSW 2000

Tel: 02 9299 2855

Marina Hollands
Manager Water Operations
Marina.hollands@goulburn.nsw.gov.au

Goulburn Mulwaree Council
Locked Bag 22, Goulburn, NSW 2580

Tel: 02 4823 4451

Goulburn Flood Study

Please complete this questionnaire and return to the FREEPOST address in the envelope provided before 1st October 2015.

1. Your details

(Please note your contact details are optional , will be held confidential and only used to contact you for more information regarding this study)

Name:
Address:
Telephone:
Email:
Can we contact you directly for more information? ☐ Yes ☐ No

2. Is this property a residence, business, other?

☐ Residence ☐ Business ☐ Other

If business or other please provide details – e.g. Joe's Fish Shop:

3. How long have you lived or worked at this address?

☐ Years ☐ Months

4. How long have you lived in this area?

☐ Years ☐ Months

5. Are you aware of the Wollondilly River flooding at Goulburn?

☐ Very aware ☐ Some awareness ☐ Not aware at all

If "Very aware" or "Some awareness" do you have any information we could use such as photographs of flooding or flood damages, recorded observations of flood depths or other information? Please provide details below or attach information. **Please include dates when known.**

Goulburn Flood Study

GOULBURN MULWAREE COUNCIL

6. Are you aware of any flooding due to the Mulwaree River at Goulburn?

☐ Very aware ☐ Some awareness ☐ Not aware at all

Please provide details below or attach information. **Please include dates when known.**

7. Has your property ever been affected by flooding?

☐ No ☐ Yes, but only the yard ☐ Yes, above the floor level

If yes, please provide details below. **Please include dates when known.**

8. Do you have daily rainfall records for periods of flooding?

☐ No ☐ Yes

If yes, please provide details below or attach information. **Please include dates when known**.

Please attach any additional information or comments to this questionnaire or email goulburn@wmawater.com.au

Appendix C

APPENDIX C: Design Event - Peak Flood Flows, Levels and Velocities

The Appendix presents peak flood flows, levels and velocities for the various design events at key locations on both the Wollondilly and Mulwaree Rivers. The locations where these flood characteristics were recorded are presented as 'Analysis Points' in Figure 44. Flow measurements were recorded for a cross section of the river, perpendicular to flow direction at the location of each Analysis Point.

Table C1: Wollondilly River – Design Event Flow at Various Locations

ID	Location	Wollondilly River – EVENT FLOW (m³/s)						
		0.2 EY	10%	5%	2%	1%	0.5%	PMF
W01	Rossi Bridge Upstream	200	310	490	930	1,110	1,300	11,020
W02	Rossi Bridge Downstream	200	310	490	930	1,110	1,300	11,020
W03	Rossi Gauge	200	320	490	930	1,110	1,290	11,020
W04	Rossi Weir Upstream	200	320	490	930	1,110	1,290	11,020
W05	Rossi Weir Downstream	200	320	490	930	1,110	1,290	11,020
W06	Marsden Weir Upstream	200	310	490	940	1,110	1,300	10,800
W07	Marsden Weir Downstream	200	310	490	940	1,110	1,300	10,800
W08	Marsden Bridge Upstream	200	310	490	940	1,110	1,300	10,800
W09	Marsden Bridge Downstream	200	310	490	940	1,110	1,300	10,800
W10	Behind Properties on Fitzroy Street	200	310	490	940	1,110	1,300	10,800
W11	Victoria Bridge Upstream	200	310	490	940	1,110	1,300	10,720
W12	Victoria Bridge Downstream	200	310	490	940	1,110	1,300	10,720
W13	Avoca St Mid	200	310	490	940	1,110	1,300	10,720
W14	Kenmore Bridge Upstream	200	310	490	940	1,110	1,290	9,490
W15	Kenmore Bridge Downstream	200	310	490	940	1,110	1,290	9,490
W16	Crookwell Rail Bridge Upstream	200	310	490	930	1,100	1,290	9,490
W17	Crookwell Rail Bridge Downstream	200	310	490	930	1,100	1,290	9,490
W18	Sewer Aqueduct Upstream	200	310	490	930	1,090	1,290	8,720
W19	Sewer Aqueduct Downstream	200	310	490	930	1,090	1,290	8,720
W20	Wollondilly/Mulwaree Confluence	270	420	660	1,290	1,540	1,790	5,460
W21	Murrays Flat Gauge	270	420	660	1,290	1,540	1,780	3,720

Table C2: Mulwaree River – Design Event Flow at Various Locations

ID	Location	Mulwaree River – EVENT FLOW (m³/s)						
		0.2 EY	10%	5%	2%	1%	0.5%	PMF
M01	The Towers Gauge	130	190	290	500	610	730	5,160
M02	The Towers Weir Upstream	130	190	290	520	620	730	5,160
M03	The Towers Weir Downstream	130	190	290	520	620	730	5,160
M04	Thornes Bridge Upstream	120	190	290	500	610	730	5,160
M05	Thornes Bridge Downstream	120	190	290	500	610	730	5,160
M06	Hume Highway Bridge 2 Upstream	70	90	120	160	170	200	5,960
M07	Hume Highway Bridge 2 Downstream	70	90	120	160	170	200	5,960
M08	Hume Highway Bridge 3 Upstream	30	60	90	180	220	270	5,960
M09	Hume Highway Bridge 3 Downstream	30	60	100	180	220	270	5,960
M10	Hume Highway Bridge 4 Upstream	20	30	40	80	100	120	5,960
M11	Hume Highway Bridge 4 Downstream	20	30	40	80	100	120	5,960
M12	Lansdowne Bridge Upstream	150	230	350	620	770	920	5,780
M13	Lansdowne Bridge Downstream	150	230	350	620	770	920	5,780
M14	Goulburn Brewery	150	230	350	620	770	910	5,750
M15	Park Road Roundabout	150	230	350	620	770	910	5,670
M16	Park Road Upstream	150	230	350	620	780	910	5,670
M17	Park Road Downstream	150	230	350	620	780	910	5,670
M18	Goulburn Golf Club Upstream	150	230	350	620	780	910	5,610
M19	Goulburn Golf Club Downstream	150	230	350	620	780	910	5,610
M20	May Street Bridge Upstream	150	230	350	620	780	910	5,550
M21	May Street Bridge Downstream	150	230	350	620	780	910	5,550
M22	Railway Viaduct Upstream	150	230	350	630	800	930	5,400
M23	Railway Viaduct Downstream	150	230	350	630	800	930	5,580
M24	Sydney Road Bridge Upstream	150	230	350	630	800	940	5,290
M25	Sydney Road Bridge Downstream	150	230	350	640	800	940	5,290

Table C3: Wollondilly River – Design Event Levels at Various Locations

ID	Location	Wollondilly River – EVENT LEVEL (mAHD)						
		0.2 EY	10%	5%	2%	1%	0.5%	PMF
W01	Rossi Bridge Upstream	636.6	637.0	637.5	638.6	639.2	639.8	651.3
W02	Rossi Bridge Downstream	636.6	637.0	637.5	638.6	639.1	639.6	650.9
W03	Rossi Gauge	636.6	636.9	637.4	638.4	638.9	639.4	650.8
W04	Rossi Weir Upstream	636.5	636.8	637.3	638.1	638.5	639.0	650.7
W05	Rossi Weir Downstream	634.9	635.7	636.4	637.7	638.3	638.9	650.7
W06	Marsden Weir Upstream	632.0	632.3	632.7	634.2	634.8	635.4	646.2
W07	Marsden Weir Downstream	630.5	631.3	632.2	634.2	634.8	635.4	646.2
W08	Marsden Bridge Upstream	630.2	631.1	632.2	634.0	634.6	635.2	646.3
W09	Marsden Bridge Downstream	630.2	631.1	632.1	634.0	634.6	635.1	646.2
W10	Behind Properties on Fitzroy Street	-	-	631.8	633.6	634.1	634.6	643.7
W11	Victoria Bridge Upstream	627.9	628.7	629.7	631.4	632.0	632.4	641.4
W12	Victoria Bridge Downstream	627.9	628.7	629.7	631.3	631.9	632.3	640.9
W13	Avoca St Mid	-	-	-	630.9	631.5	632.0	640.6
W14	Kenmore Bridge Upstream	626.6	627.3	628.2	630.0	630.6	631.2	639.9
W15	Kenmore Bridge Downstream	626.6	627.3	628.2	630.0	630.6	631.2	639.9
W16	Crookwell Rail Bridge Upstream	626.4	627.1	628.0	629.9	630.5	631.0	639.9
W17	Crookwell Rail Bridge Downstream	626.4	627.0	628.0	629.8	630.4	631.0	639.9
W18	Sewer Aqueduct Upstream	625.1	626.1	627.3	629.3	630.0	630.6	639.8
W19	Sewer Aqueduct Downstream	625.1	626.1	627.3	629.3	630.0	630.6	639.8
W20	Wollondilly/Mulwaree Confluence	624.9	626.0	627.2	629.3	629.9	630.5	639.8
W21	Murrays Flat Gauge	620.6	621.6	622.8	625.1	625.8	626.5	637.1

Table C4: Mulwaree River – Design Event Levels at Various Locations

ID	Location	Mulwaree River – EVENT LEVEL (mAHD)						
		0.2 EY	10%	5%	2%	1%	0.5%	PMF
M01	The Towers Gauge	633.8	634.1	634.4	635.0	635.3	635.6	641.6
M02	The Towers Weir Upstream	633.8	634.1	634.3	634.9	635.0	635.2	641.0
M03	The Towers Weir Downstream	631.8	632.1	632.5	633.1	633.3	633.5	640.8
M04	Thornes Bridge Upstream	631.6	631.8	632.1	632.6	632.8	633.0	640.4
M05	Thornes Bridge Downstream	631.6	631.8	632.1	632.5	632.7	632.9	640.4
M06	Hume Highway Bridge 2 Upstream	629.8	630.0	630.3	630.7	631.0	631.4	640.4
M07	Hume Highway Bridge 2 Downstream	629.7	629.9	630.1	630.5	630.8	631.3	640.3
M08	Hume Highway Bridge 3 Upstream	629.3	629.6	629.9	630.5	630.9	631.4	640.4
M09	Hume Highway Bridge 3 Downstream	629.3	629.5	629.8	630.4	630.8	631.3	640.3
M10	Hume Highway Bridge 4 Upstream	629.8	630.0	630.2	630.6	631.0	631.4	640.4
M11	Hume Highway Bridge 4 Downstream	629.6	629.8	629.9	630.4	630.8	631.3	640.4
M12	Lansdowne Bridge Upstream	629.0	629.2	629.5	630.1	630.6	631.2	640.3
M13	Lansdowne Bridge Downstream	628.9	629.2	629.4	630.1	630.6	631.2	640.3
M14	Goulburn Brewery	-	-	-	629.9	630.5	631.1	640.3
M15	Park Road Roundabout	-	628.1	628.2	629.8	630.4	631.0	640.3
M16	Park Road Upstream	627.6	627.9	628.1	629.8	630.4	631.0	640.3
M17	Park Road Downstream	627.5	627.7	628.0	629.8	630.4	631.0	640.3
M18	Goulburn Golf Club Upstream	626.9	627.3	627.8	629.7	630.4	631.0	640.3
M19	Goulburn Golf Club Downstream	626.9	627.2	627.8	629.7	630.4	631.0	640.3
M20	May Street Bridge Upstream	626.1	626.7	627.6	629.7	630.4	631.0	640.2
M21	May Street Bridge Downstream	626.1	626.7	627.6	629.7	630.4	631.0	640.2
M22	Railway Viaduct Upstream	625.8	626.5	627.6	629.6	630.3	630.9	640.2
M23	Railway Viaduct Downstream	625.8	626.4	627.5	629.6	630.3	630.9	640.2
M24	Sydney Road Bridge Upstream	625.3	626.2	627.4	629.4	630.1	630.7	640.0
M25	Sydney Road Bridge Downstream	625.2	626.2	627.4	629.4	630.0	630.6	639.9

Table C5: Wollondilly River – Design Event Velocities at Various Locations

ID	Location	Wollondilly River – EVENT VELOCITIES (m/s)						
		0.2 EY	10%	5%	2%	1%	0.5%	PMF
W01	Rossi Bridge Upstream	0.8	1.1	1.5	2.1	2.3	2.4	2.9
W02	Rossi Bridge Downstream	0.8	1.1	1.4	2.1	2.2	2.3	3.0
W03	Rossi Gauge	0.6	0.9	1.2	1.9	2.1	2.2	3.3
W04	Rossi Weir Upstream	1.6	2.0	2.5	3.4	3.6	3.6	4.4
W05	Rossi Weir Downstream	2.7	3.1	3.2	3.3	3.5	3.5	5.3
W06	Marsden Weir Upstream	0.8	1.1	1.7	2.2	2.3	2.4	3.3
W07	Marsden Weir Downstream	2.6	2.8	3.2	3.3	3.3	3.3	3.6
W08	Marsden Bridge Upstream	1.1	1.2	1.4	1.8	1.9	2.0	2.4
W09	Marsden Bridge Downstream	1.1	1.2	1.5	1.8	2.0	2.1	2.5
W10	Behind Properties on Fitzroy Street	-	-	0.3	0.6	0.6	0.6	2.0
W11	Victoria Bridge Upstream	1.1	1.4	1.6	2.1	2.2	2.4	2.9
W12	Victoria Bridge Downstream	1.1	1.3	1.6	2.1	2.3	2.4	3.4
W13	Avoca St Mid	-	-	-	0.1	0.1	0.1	0.8
W14	Kenmore Bridge Upstream	1.6	1.6	1.7	1.9	1.9	2.1	2.6
W15	Kenmore Bridge Downstream	1.4	1.5	1.6	1.9	1.8	2.1	3.1
W16	Crookwell Rail Bridge Upstream	1.2	1.3	1.5	1.8	1.8	2.0	3.0
W17	Crookwell Rail Bridge Downstream	1.3	1.4	1.7	2.0	1.9	2.2	3.5
W18	Sewer Aqueduct Upstream	1.4	1.5	1.7	1.7	1.6	1.8	3.1
W19	Sewer Aqueduct Downstream	1.3	1.5	1.8	1.9	1.7	2.0	3.5
W20	Wollondilly/Mulwaree Confluence	0.8	0.9	0.9	1.0	0.9	1.0	2.1
W21	Murrays Flat Gauge	1.1	1.3	1.5	1.6	1.6	1.7	1.9

Table C6: Mulwaree River – Design Event Velocities at Various Locations

ID	Location	Mulwaree River – EVENT VELOCITIES (m/s)						
		0.2 EY	10%	5%	2%	1%	0.5%	PMF
M01	The Towers Gauge	1.0	1.0	1.0	1.3	1.5	1.7	4.1
M02	The Towers Weir Upstream	0.7	0.8	0.8	1.1	1.2	1.3	3.3
M03	The Towers Weir Downstream	1.2	1.3	1.4	1.7	1.8	1.8	4.5
M04	Thornes Bridge Upstream	0.6	0.8	1.1	1.6	1.7	1.9	2.6
M05	Thornes Bridge Downstream	1.3	1.3	1.3	1.6	1.8	2.0	2.5
M06	Hume Highway Bridge 2 Upstream	0.9	1.0	1.2	1.2	1.2	1.2	1.7
M07	Hume Highway Bridge 2 Downstream	1.0	1.3	1.5	1.7	1.6	1.7	2.4
M08	Hume Highway Bridge 3 Upstream	1.0	1.0	1.0	1.1	1.1	1.3	2.0
M09	Hume Highway Bridge 3 Downstream	0.6	0.7	0.9	1.3	1.4	1.6	2.5
M10	Hume Highway Bridge 4 Upstream	0.6	0.7	0.8	1.0	1.0	1.2	1.8
M11	Hume Highway Bridge 4 Downstream	0.7	0.8	1.0	1.2	1.3	1.4	2.8
M12	Lansdowne Bridge Upstream	0.7	0.7	0.7	0.8	0.8	0.8	1.0
M13	Lansdowne Bridge Downstream	0.8	0.8	0.8	0.9	0.9	0.9	1.0
M14	Goulburn Brewery	-	-	-	0.5	0.7	0.8	1.2
M15	Park Road Roundabout	-	0.0	0.3	1.2	1.1	1.3	1.6
M16	Park Road Upstream	1.0	1.0	1.0	1.0	1.1	1.1	1.1
M17	Park Road Downstream	2.0	2.0	2.1	2.0	2.0	2.1	2.1
M18	Goulburn Golf Club Upstream	0.8	0.8	0.8	0.8	0.8	0.8	0.9
M19	Goulburn Golf Club Downstream	0.8	0.9	0.9	0.9	0.9	0.8	1.3
M20	May Street Bridge Upstream	0.8	0.8	0.8	0.8	0.8	0.8	0.9
M21	May Street Bridge Downstream	0.9	0.9	1.0	1.0	0.9	1.0	1.1
M22	Railway Viaduct Upstream	0.8	0.8	0.8	0.9	0.8	0.9	1.9
M23	Railway Viaduct Downstream	0.7	0.7	0.8	0.9	0.8	0.9	1.7
M24	Sydney Road Bridge Upstream	1.0	1.0	1.2	1.5	1.3	1.7	2.5
M25	Sydney Road Bridge Downstream	1.4	1.4	1.5	1.8	1.7	2.1	3.6