WAGGA WAGGA CITY COUNCIL

City of Wagga Wagga

WAGGA WAGGA
DETAILED FLOOD MODEL REVISION

FINAL REPORT

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AUGUST 2014

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Project Wagga Wagga Detailed Flood Model Revision	**Project Number** 113032
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Date 11 August 2014	**Verified by**

Revision	Description	Date
4	Final Report	AUGUST 2014
3	Draft Final Report	MARCH 2014
2	Draft Final	FEBRUARY 2014
1	Draft Report	NOVEMBER 2013

WAGGA WAGGA DETAILED FLOOD MODEL REVISION

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EXECUTIVE SUMMARY

Wagga Wagga is situated in the Riverina Region of NSW on the Murrumbidgee River floodplain. The township straddles the Murrumbidgee River with the majority of development on the southern floodplain. Since early settlement, Wagga has experienced numerous large floods, with four events (1852, 1853, 1870 and 1891) in the 1800's equalling or exceeding 10.5 m at the Hampden bridge gauge. Following significant flooding in the 1950's a levee was constructed to provide flood protection to the township of Wagga. Since the start of the 20th Century, only the March 2012 and August 1974 floods have exceeded 10.5 m at Wagga with the levee affording adequate flood protection to stop inundation of the southern and main part of town for both flood events (and also for numerous other smaller events). The larger of these two floods, the 1974 event, recorded the second highest flood level on record at the Hampden Bridge gauge (the highest being the 1853 event).

The flood of March 2012 produced peak flood levels comparable to the August 1974 event in the region surrounding Wagga. Analysis post the event indicated that a flow significantly less than expected (based on the most current rating curve at the time and also the 1974 event flow) was responsible. The March 2012 flow was gauged to be approximately 3,600 m^3/s (311 GL/day) at Wagga compared to the 5,200 m^3/s (450 GL/day) estimated for the 1974 flood. Following analysis of the 2012 event it is clear that the Murrumbidgee River at Wagga (and also at a number of other locations both up and down the River) has become less efficient at conveying flow. An approximate estimate of the reduction in conveyance is 25% (for equivalent stage).

During the March 2012 event this change in River conveyance led to a predicted peak flood level estimate of 10.9 m, hence exceeding the design height of the Wagga levee (10.8 m) only hours prior to the 2012 flood peak. Accordingly, an evacuation order was issued by the NSW State Emergency Service which affected approximately 9,000 people in households and businesses situated on the southern floodplain of Wagga Wagga including the Wagga Central Business District.

Since the 2004 flood study Council has been involved in an ongoing project to upgrade the Wagga levees. The 2012 flood event and the apparent decline in the River's conveyance evinced by that event gave cause for the design protection afforded by the levee to be revised. A further motivator of this study is that the NSW Floodplain Development Manual (NSW, 2005) recommends that flood studies at a location be reviewed following a significant event.

As such WMAwater were commissioned to:

1. Build a 2D hydraulic model (extending from Eringoarrah down to past Malebo Gap);
2. Calibrate the model to the 2012 event and validate to the 2010;
3. Revise the Flood Frequency Analysis derived 2004 1% AEP estimate (WMAwater, 2004);
4. Use revised 1% and 5% AEP events to establish new "design" flood levels (inclusive of levee failure where design flood levels exceed failure levels for these structures;

5. Establish design heights required for proposed levee upgrades;
6. Assess impacts of revised levees on flood levels;
7. Sensitivity test the design model in order to better inform the levee freeboard calculation;
8. Liaise with the NSW Office of Water re: 1974 modelling and changes to River conveyance; and finally
9. Produce a report documenting work done, assumptions etc. and presenting results inclusive of mapping.

Model Build and Calibration/Validation

Bathymetric survey for 66 km's of River was obtained (cross sections surveyed at 100 m intervals). The model bathymetry was then constructed based on a merging of River and overbank levels to create one composite Digital Elevation Model.

Calibration of the model to the 2012 event was successful with the NSW Office of Water's flow gauging, flow and stage hydrographs at Hampden Bridge and a variety of floodplain peak flood levels (58) all being accurately matched. The model was validated against the 2010 event successfully with a good match to flow, stage and peak flood levels.

Change in Murrumbidgee River Rating

To investigate what may have precipitated the River rating change observed in the 2010 and 2012 events, the following work was carried out:

- Collected historical topographic/bathymetric data and compared it with current bathymetric survey data;
- Compared aerial photos of contemporary vegetation levels with photos of vegetation in the 1940's and 1970's; and
- Calibrated the model to the event. The match was achieved by altering model roughness settings in recognition of the conditions at the time of the 1974 event.

The match between the model and the available calibration data for the 1974 modelling is good and indicates that vegetation on the floodplain can plausibly bring about the rating change observed recently at Wagga. Work to date indicates that above a certain level of roughness, blockage is facilitated which leads to higher effective levels of roughness. This accords both with event observations (during both the event and the clean up post event) and common sense.

Based on work to date the following is apparent:

- Riparian vegetation is relatively dense now compared to the 1974 event. This might be attributed to changes in land use policy, particularly the exclusion of grazing animals from the River bank area.
- In the 2012 event tributaries contributed to the flood and debris loads were high. In the 1974 event the bulk of the flood came from Burrinjuck Dam and hence it might be presumed there was less debris.
- In the 2012 event higher riparian roughness exacerbated by debris meant a higher level of stage was achieved relative to previous events such as the 1974 flood.

Revising the 2004 Study Flood Frequency Analysis Based on Recent Findings

The current 1% AEP flow at Wagga was defined by the 2004 flood study. As per the NSW Floodplain Development Manual (NSW, 2005) review of the 2004 flood study is required following the occurrence of a significant flood. The need for the review is further reinforced by the change in River conveyance as described above. The 2004 study, using best practice techniques, found a 1% AEP flow of 6,900 m^3/s (6,700 m^3/s prior to adjustment for fit). This estimate utilised four 19th century events (1852, 1853, 1870 and 1891) and assumed each of them to be larger than the 1925 flow of 3,600[1] m^3/s. A key part of the logic used to determine that these four events exceeded the 1925 flow (not stage), was that the flow regime during the 19th century was similar to what it was in 1974 (this being the primary calibration event used to setup the 2004 hydraulic model with validation events also being from the 1970's).

Given the impact of vegetation on flood levels at Wagga (as demonstrated by the 2010 and 2012 events) flow estimates for 19th century events have been revised down relative to the 2004 report[2]. This leads to the magnitude of the 1852 and 1891 events being reconsidered relative to the 1925 event. Taking into account historical vegetation (and associated roughness) it seems likely that neither event exceeded the flow of the 1925 event, particularly if Burrinjuck Dam had been in place at the time. Accordingly, of the four major historical events, only the 1853 and 1870 events have been used explicitly in Flood Frequency Analysis to determine design flows with the assumption that both of these events are larger than the 1925 flood, even with the inclusion of upstream dams.

Design Hydrology

Flood frequency analysis was used to calculate design flood estimates. The analysis consisted of fitting a probability distribution to a truncated series (events greater than 1,000 m^3/s) of annual peak discharges composed of a continuous data series (1892 – 2012) and incomplete series prior to 1892 (1838 – 1891). The incomplete data series includes the 1853 and 1870 floods which, as mentioned above, have been determined to be larger (in terms of flow) than the 1925 event. As the exact magnitude of these flood events is unknown Bayesian methods have been employed to incorporate these events into the Flood Frequency Analysis.

A Bayesian maximum likelihood approach using the software program 'Flike' (file version 5) was used to fit a Log-Pearson III probability distribution to the truncated series mentioned above. The updated Flood Frequency Analysis at Wagga gives the following revised design flood estimate flows:

- 1% AEP = 5,100 m^3/s; and
- 5% AEP = 2,900 m^3/s.

Proposed Levee Alignments and Levels

Applying the above mentioned flows to the calibrated 2012 flood model, the new 1% AEP design height has been determined. The 1% AEP level at the Hampden Bridge gauge is calculated to be 11.31 m (181.36 mAHD). This revised 1% AEP estimate is 0.4 m higher than the highest

[1] Estimated using the 2004 hydraulic model.

[2] The assumption that vegetation in the mid 19th century was "thicker" than in 1974 inform these lower revised 19th century flow estimates.

ever recorded flood level (1853) of 10.9 m which relates to the inclusion of levees and increased vegetation density.

In addition to this the North Wagga levee 5% AEP design height has been calculated.

Impacts of proposed levee Upgrade

Flood level impacts associated with upgrading the Main City and North Wagga levees have been assessed. Impacts in the region of the Wagga Main City Levee are generally less than 0.15 m in the 1% AEP flood. A maximum impact of 0.1 m (for a relatively small area) is expected with the North Wagga levee upgrade for the 5% AEP event (note there is no impact associated with the proposed North Wagga levee upgrade for the 1% AEP event). The majority of the impacted region is sparsely populated and the relatively small impacts in relation to the freeboard (0.5 m) will minimise any additional over floor flooding.

Comparison to Reference 2 Peak Flood Levels

The current study 1% AEP peak flood levels were compared to 1% AEP levels calculated in the Reference 2 study. 1% AEP flood levels were found to be typically 0.2 – 0.3 m lower in the current study than that calculated for the Reference 2 study in the vicinity of the Main City and North Wagga Levees.

Model Sensitivity

Model sensitivity runs are carried out in order to determine how robust model results are. The less model results change for a given change in parameter, then the more confidence can be had in the model design flood level predictions.

The following sensitivity runs have been carried out:

- +/- 10% flow; and
- +/- 10% roughness.

The 10% value for roughness was chosen on the basis that the difference between the 1974 floodplain roughness and current conditions is in the order of 20%. As such 10% seemed more realistic for a further increase in roughness (on top of current conditions), given that a change from very low relative roughness to very high relative to roughness is 20% (with event debris blockage accounting for some portion of this change).

Modelled sensitivity to changes in model parameters was examined along the length of the Main City Levee. It was found that the model results are relatively (in regards to the applied freeboard) insensitive to the tested parameters. Peak flood level sensitivity to roughness was found on average to be 0.15 m and did not exceed 0.2 m in the vicinity of the levee. On average the increase in peak flood level due to a 10% increase in flow was found to be 0.22 m and the maximum increase in the vicinity of the levee did not exceed 0.24 m.

Estimated Model Accuracy

The Reference 1 study provides an estimate of the order of accuracy for design flood levels of ±0.5 m. This accuracy has been improved upon for the current study by utilisation of additional calibration events and data, as well as modern engineering techniques. Sensitivity analysis results indicate that the order of accuracy of design peak flood levels for the current study is ±0.25 m. This should be taken into account when determining freeboard within the Study Area.

Public Exhibition

The Wagga Wagga Detailed Flood Model Revision Draft Final Report was placed on exhibition for 28 days for public comment. As part of the public exhibition process the draft report was promoted via FloodFutures, the community engagement platform for Council's floodplain management activities and projects.

The 28 day exhibition period has concluded and Council received three submissions to the report. The submissions along with the submission responses, are attached in Appendix F, but have had names blacked out for privacy reasons.

1. INTRODUCTION

1.1. General

Wagga Wagga is located in the Riverina region of NSW. The study area (depicted in Figure 1) is subject to flooding from the Murrumbidgee River. The Murrumbidgee River traverses the floodplain from east to west and is a major tributary to the Murray System draining some 100,000 km^2. The catchment area of the Murrumbidgee River at Wagga Wagga is approximately 26,400 km^2.

The majority of the Murrumbidgee River floodplain in this area is used for agricultural purposes with most urban and industrial developments concentrated in Central Wagga Wagga and North Wagga. Other significant commercial/industrial areas are located on the southern floodplain and east of Wagga Wagga along the Sturt Highway (Hammond Avenue). Recent population growth has mainly been centred in the southern and elevated areas of Wagga Wagga. Other significant residential centres away from the floodplain comprise Kooringal, Lake Albert, Tatton, Turvey Park, Mt Austin, Glenfield, Tolland, Bourkelands and Lloyd.

Wagga Wagga is situated at the boundary of two very differing geographical regions. The sharp relief of the Great Dividing Range (in the upper catchment) flattens to form the Riverina Plain. A Digital Elevation Model (DEM) of the region shown in Figure 2 illustrates the contrast between the mountainous eastern end of the study area and the flatter regions to the west.

The model domain covers the Murrumbidgee River floodplain and this region is represented by the model extent shown in Figure 1. The modelled reach includes the area 5 km upstream of Oura which is located approximately 15 km east of Wagga Wagga (upstream) and runs downstream of the Malebo Gap some 9 km to the west (downstream) of Wagga Wagga. The total river length modelled is approximately 63 km.

1.2. Background

In March 2012 the Murrumbidgee River flooded. Homes, businesses and land were inundated from Jugiong to Darlington Point. On the 5th of March higher than expected flood level readings at Eringoarrah forced a revision of the 10.6 m flood expected to arrive at Hampden Bridge on March 6th. The revised estimate of 10.9 m (higher than the levee design height) meant that evacuation of the entire CBD was required. An estimate of the number of people evacuated from the Wagga region is approximately 9,000, the vast bulk of these, came from the southern floodplain.

North Wagga was also evacuated, however, given North Wagga levee's height is approximately at the 20Y ARI level (corresponding to a Hampden Bridge gauge level of ~ 9.95 m), water overtopped the levee and inundated approximately 190 homes.

A feature of the flood was that the peak flood level resulted from 311 GL/day whilst the previous rating (based on 1974 flood etc.) indicated that approximately 400 GL/day would be required to achieve such a stage height (see Section 3 for further details).

Following the March 2012 floods Council wishes to ensure that management of community flood risk remains best practice. As such Council wishes to update design flood levels as per the NSW Floodplain Development Manual (FDM) recommendations following a significant flood event. Further, following both the December 2010 and the March 2012 events, the NSW Office of Water (NOW) gauging's have led to a revision of the rating table for the Hampden Bridge gauge (amongst other Murrumbidgee River gauges). The revision of the rating is quite substantial with approximately 25% less flow required to achieve a similar level to that predicted by the previous stage-discharge rating relationship (see Section 3) and observed during past events. The revision of the Hampden Bridge gauge rating has a substantial impact on the flood protection afforded Wagga Wagga by the current levees. As such some review of the rating change is required and included in the project objectives described below.

1.3. Objectives

Wagga Wagga City Council (Council) has appointed WMAwater to provide hydrology services to assist in the design of levee augmentation. Building on previous studies (see Section 2.1) the following work is required:

- Extend the model domain of the Reference 2 model. Previous model extent included Braehour to upstream of Malebo Gap. The domain has now been extended to include Oura in the upstream and in the downstream the model boundary has been extended downstream of Malebo Gap which is likely to impact upstream levels for very large flood events. Total River length to be modelled is approximately 63 kilometres;
- Update model bathymetry utilising bathymetry survey (see Section 2.2.2);
- Calibrate the model to the March 2012 event,
- Validate the model to the December 2010 event;
- Carry out sensitivity testing to observe changes in results based on variable model boundary conditions, roughness settings etc;
- Update the Wagga Wagga Flood Frequency Analysis (FFA) to determine design flows;
- Re-define design flood layers (1% and 5% Annual Exceedance Probability (AEP)) including the Flood Planning Area (FPA) and Flood Planning Level (FPL);
- Utilise revised design information in ongoing work aiming to revise the design of the Main City and North Wagga levees;
- Produce an impact assessment for the proposed levee works including identification of impacts on adjoining private property;
- Compare modelled results with the event gauging taken by NOW as well as NOW's stage-discharge rating for Hampden Bridge and provide a discussion with respect to any discrepancies;
- Model the 1974 flood event using the revised model to determine the veracity of the Wagga rating; and
- Provide advice on the stage/discharge relationship at Wagga Wagga for stages exceeding the most recent event (March 2012).

1.4. Existing Levees

The main purpose of this study is to determine revised design heights for the Wagga Wagga Main City and North Wagga levees. The details of these levees are contained in the following sections.

1.4.1. Wagga Main City Levee

The existing Wagga Main City levee provides protection to the southern floodplain of Wagga Wagga. The levee follows the Murrumbidgee River from near Koringal Road in the east to the Olympic Highway in the west and has a length of approximately 9.6 km. The levee currently has a design height of 10.8 m (at the Hampden Bridge gauge) with a freeboard of 0.9 m. The revised design flood height for the Main City Levee (11.3 m) is presented in Figure 23 (this is the peak flood level of the 1% AEP flood along the levee alignment). Section 5.4.2.1 and Appendix E describe the levee failure spillway locations.

1.4.2. North Wagga Levee

The existing North Wagga levee provides limited protection to the township of North Wagga situated on the northern floodplain of the Murrumbidgee River. The levee surrounds North Wagga and has a length of approximately 4.3 km. The levee currently has a design height of 9.9 m (at the Hampden Bridge gauge) with a freeboard of 0.3 m and the spillway is located along Hopkirk Street (see Figure D3). The revised design flood height for the North Wagga Levee is presented in Figure 24. In addition to the North Wagga levee a smaller separate levee also provides protection to houses along Mill and East Streets. The locations of both levees are displayed in Figure 19 along with the 5% AEP flood extent.

1.5. Public Exhibition

The Wagga Wagga Detailed Flood Model Revision Draft Final Report was placed on exhibition for 28 days for public comment. As part of the public exhibition process the draft report was promoted via FloodFutures, the community engagement platform for Council's floodplain management activities and projects.

Two public meetings occurred to discuss the outcomes of the updated modelling with the community in North Wagga Wagga (21 May 2014) and Gumly Gumly (28 May 2014). Furthermore, a video of the presentation has been made available on the FloodFutures website.

During the public exhibition period (19 May – 16 June 2014) 1,226 people visited FloodFutures with 228 document downloads and 30 plays of the Revised Flood Model presentation.

The 28 day exhibition period has concluded and Council received three submissions to the report. These are attached in Appendix F, but have had names blacked out for privacy reasons.

WMAwater have assisted Council in providing input to the responses to the submissions and this advice has been reviewed by the Office of Environment and Heritage who are of the opinion that the responses adequately cover the issues raised in the submissions.

2. AVAILABLE DATA

Various items of data as well as reports salient to the study have been collected and reviewed. Most reports and datasets were sourced from Council and supplemented by additional survey where required. The key focus of the exercise was to collect data suitable for the model calibration and validation process.

This section provides a summary of the reports as well as a description of the various forms of data utilised in the study.

2.1. Previous Studies

For the purpose of this study data has been acquired from previous studies focusing on Murrumbidgee River flooding at Wagga Wagga. Details of the data acquired from these studies are outlined in the following sections.

2.1.1. Murrumbidgee River Wagga Wagga Flood Study, WMAwater, 2004 (Reference 1)

The Murrumbidgee River Wagga Wagga Flood Study was completed in 2004 and used a 1D RUBICON model to determine design flood extents and levels. Various details salient to the current study were obtained. In particular, the methods and data used in the Reference 1 study for FFA have been incorporated into the current study to help develop design flows. Furthermore, details on how floodplain structures have changed overtime (see Table 1) were obtained. 1974 Peak flood level marks around Wagga Wagga and Oura were also extracted and used to assess modelled levels so that the veracity of the Wagga rating curve pre ADCP gaugings could be determined (see Section 3).

Table 1: Historical changes to flood impacting structures in Wagga Wagga (Reference 1)

Date	Works on the Floodplain	Comment
Various	Narrung Street Sewage Treatment Ponds: • 1914 - The site was first developed as a sewage plant for the town of Wagga Wagga. • early 1950's - A formalised series of treatment ponds were constructed between the plant and the river. • 1967/1968 - The ponds were upgraded to a new configuration including construction of four ponds west of the Bomen rising main. • approx. 1977 - Three ponds west of the Bomen rising main were removed in order to reduce upstream flood levels. The bank around the emergency overflow pond (the remaining pond to the west) may have also been lowered at the same time. • mid 1990's - A floodway was partially constructed through the ponds. • 2007 to 2010 – Treatment Works reconstructured and use of ponds reduced substantially	Council is aware of the restriction caused by construction of the banks around the treatment ponds (Reference 4) and is currently addressing this issue including the associated environmental/public health issues.
1930s	Gobba weir and levee	(Upgrading to eastern end in late 1960's/early 70's)
1960	Main City levee constructed on southern floodplain.	Limited the width of floodplain.

1975	Raising of East Street and Mill Street levee to 179.3 mAHD.	Up to 1 m high and 200 m long. This prevents floodwaters up to 9.3 m on the gauge (179.35 mAHD) from entering the northern floodway and cutting the Junee Road.
1975	Eunony Bridge was completed. In the August 1974 flood the bridge was only partially constructed with the approaches constructed by the time of the October 1975 flood.	
1975	The Gumly Gumly levee was temporarily raised to its present level following the August 1974 flood.	
1978-1983	The Main City levee was upgraded to approximately 1 m above the 1974 flood level.	
1978	A private levee was constructed around the Allonville Motel and the access road to the Murray Cod Hatchery was raised.	
Late 1980's	The Sturt Highway was raised by up to 0.2 m.	
1990	Construction of the North Wagga Wagga levee to the 1 in 20 ARI +0.3m freeboard event	
1992	The Gumly Gumly levee was formalised to approximately the 1 in 10 ARI event.	
1995	Construction of Wiradjuri Bridge	Minor alterations to access road between Wiradjuri and Parken Pregan bridges
1997	Construction of Gobbagombalin Bridge	Changes to northern edge of floodplain from Gobba lagoon to Coolamon Road

2.1.2. Wagga Wagga Murrumbidgee River Model Conversion Project, WMAwater, 2010 (Reference 2)

The Wagga Wagga Murrumbidgee River Model Conversion Project revised the Reference 1 Study with the RUBICON model being converted into a 2D model (TUFLOW) and new design flood extents and levels were calculated. The following data was sourced from Reference 2:

- Wagga Wagga Main City levee alignment and heights;
- North Wagga levee alignment and heights;
- Bridge locations and details;
- Calibrated roughness values with spatial distribution (albeit for the more limited model extent); and
- The 1974 inflow hydrograph.

Note that elements of the above listed information were also available from Reference 1.

2.1.3. Wagga Wagga Whole of LGA Model, WMAwater, 2011 (Reference 3)

The Wagga Wagga LGA flood study (WMAwater 2011) defined design flood levels for the entire LGA (those areas impacted by Riverine flooding only). The whole of LGA model built on the Reference 2 model to extend the model boundaries. As such this study provided only additional data relative to the Reference 2 study. Importantly however, it did provided additional 1974 peak flood levels at the eastern and western extremities of the local government area to facilitate model verification.

2.1.4. Murrumbidgee River Flooding - Flood Intelligence Collection - March 2012 - Draft (Reference 4)

WMAwater were engaged by the SES in order to collect flood data associated with the March 2012 flood event with the brief being to collect flood intelligence associated with Murrumbidgee River flooding from Jugiong to Hay. Flood intelligence describes flood behaviour and the consequence flooding has for the community. Flood intelligence enables the SES to determine the likely impacts (or consequences) of flooding and what actions should be undertaken by response agencies.

In particular, this study provided 58 2012 peak flood level marks within the model domain. 50 of these marks were able to be used during model calibration (see Section 6.1.3) with eight discarded for reasons discussed in Section 2.4.3.1.

2.1.5. Murrumbidgee River Flooding - Flood Data Collection - December 2010 (Reference 5)

This study was similar to the Reference 4 study in that it aimed to obtain flood intelligence pertinent to the December 2010 Murrumbidgee River flood event. This study provided 25 peak flood level marks for the 2010 flood event. These marks were able to be used during model validation (see Section 6.2.3).

2.2. Model Build Data

Topographical and survey data is vital for model configuration and provides a basis for the hydraulic model build. Structures such as bridges, levees and culverts need to be realistically represented to reproduce hydraulic properties accurately. Structures details have been obtained from Reference 2 and from design drawings provided by Council.

All topographical and survey data used in this Study is outlined in the following sections.

2.2.1. Airborne Laser Survey

ALS data was recorded in 2008 by Fugro Spatial Systems Pty Ltd (Fugro) for the entire Murrumbidgee River floodplain from downstream of Burrinjuck Dam to the confluence of the River with the Murray. The data was collected on behalf of the then Department of Environment and Climate Change (now Office of Environment and Heritage) with the work managed by the Land and Property Management Authority (LPMA), who are also the custodians of the data.

The ALS provides ground level spot heights from which a Digital Elevation Model (DEM) has been constructed. This data has a vertical accuracy of +/- 0.15 m and a horizontal accuracy of +/- 0.5 m at the first confidence interval (68% of all data). When interpreting the above, it should be noted that the accuracy of the ground definition can be adversely affected by the nature and density of vegetation and/or the presence of steeply varying terrain. The DEM formed the basis of the 2D model build and is presented in Figure 2.

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, River survey was carried out. River survey was undertaken by a qualified hydrosurvey firm (Hydrographic & Cadastral Survey Pty Ltd) who produced a dataset of 668 cross-sections which defined the River bathymetry at the time of survey (displayed in Figure 3). It is noteworthy that the survey was undertaken at a time of relatively low water (levels in the order of 1 m at Hampden Gauge). The survey Brief and Quotation are displayed in Appendix B.

The cross sections were used to generate a DEM of the Rivers bathymetry (within the river banks). The bathymetry was then combined with the overbank DEM to create a DEM of the River and overbank combined. The combined DEM was then used for modelling purposes.

The survey brief called for specific cross-sections at locations where they could be compared to historic cross-sections (see Section 3.2) as well as photography of the overbank (on each bank for each surveyed cross section) so that model roughness values could be estimated (see Section 3.3).

2.2.3. Hydraulic Structures

Structures such as bridges, levee banks and road crossings can have a significant impact on flood behaviour.

A total of five bridges have been identified in the study area. The bridges are (from upstream to downstream), Eunony, Railway, Hampden, Wiradjuri and Gobbagombalin. Design drawings of these bridges have been used to define bridge details in the model.

The Wagga Wagga Main City levee and the North Wagga levee both have significant impacts on flows and have been incorporated into the model using design levee heights obtained from the Reference 2 study. In addition to these current structures there are a number of historical levees which fill low spots in the natural river bank in the vicinity of the urbanised regions of Wagga. These maximise the inbank flow, delaying breakouts. Primarily these levees run from near Kurrajong Lagoon upstream of Eunony Bridge down to Gobba weir.

A discussion of the model structure implementation method is contained in Section 5.4.

2.3. Gauge Data – Hampden Bridge

Flood heights, rating curves, cross-sections and other details for the Hampden Bridge gauge at Wagga Wagga (No. 410001) were obtained from PINNEENA. This data was not only used to inform flow inputs into the hydraulic model for calibration/validation (see Section 2.4.1) but also as a basis (use of peak gauge heights) for FFA used to determine design flows[3] (see Section 4.3).

Hampden Bridge was the first stream gauging station established on the Murrumbidgee River. Records are available from October 1868 but there are significant gaps in the data set up until 1885. Prior to 1972 the Hampden Bridge gauge was read manually and generally only a daily water level was available, however additional levels during flood events were also sometimes recorded. An automatic gauge recording complete and accurate definitions of the flood hydrograph has been installed since 1972.

Prior to 1886 there is only limited official height data available from PINNEENA. The Reference 1 study went to considerable effort to obtain additional information on flooding during this period. In total, 175 years (1838 – 2012) of record have been used in FFA (see Section 4.3) with the data set obtained from the Reference 1 study reviewed and then utilised in the current study. Additional data post the Reference 1 study has been added to the data set (2003 – 2012) to account for more recent flood events.

For the purpose of FFA the data has been separated into two periods (as per the Reference 1 study). Data post 1892 has been named the ‘continuous data period’ with the remaining data set named as ‘data prior to 1892’. The details of these data sets are discussed in the following sections.

2.3.1.1. Continuous Data Period

Data available for the period 1892 – 2012 is not homogeneous as there have been numerous changes in the catchment. The biggest has been the construction of Burrinjuck Dam, although land clearing is also likely to be a significant factor. Burrinjuck Dam has been modified twice (1956 and 1995) and took 16 years to build (1912 to 1928). There have also been other large dams built in the catchment (e.g. Blowering) all of which contribute to making it difficult to construct a homogeneous data set.

The Reference 1 study concluded that the period from 1892 to 2012 was reasonably homogeneous. This conclusion was based on the 1925 flood occurring just prior to Burrinjuck Dam being completed (hence being effectively attenuated by the Dam) and the fact that there were no other large event between 1892 and 1925 that might significantly influence the high flow record.

Table 2 displays the annual maximum series of peak flood levels and gauge heights for the continuous data period. 121 years of record have been used in FFA in combination with information describing the ‘data prior to 1892’ data set (see Section 2.3.1.2). Data prior to 1892 has been incorporated into the FFA using Bayesian techniques.

[3] Note that the flows used in FFA pre 1990 have been informed using the rating table created from the current studies 1974 model (see Section 6.4.5). Flows from 1991 – 2012 have been determined using the most recent NoW rating.

Table 2: Continuous Data Set

Year	Month	Gauge Height (m)	Flow (m³/s)	Year	Month	Gauge Height (m)	Flow (m³/s)	Year	Month	Gauge Height (m)	Flow (m³/s)
1892	Oct	8.357	980	1933	Sep	5.563	427	1974	Aug	10.741	5216
1893	Jun	7.264	661	1934	Oct	9.195	1820	1975	Oct	9.582	2370
1894	Apr	9.144	1736	1935	Oct	6.325	520	1976	Oct	9.38	2062
1895	Jun	5.283	395	1936	Jul	7.62	731	1977	Jul	4.159	278
1896	Jun	3.962	259	1937	Oct	3.581	226	1978	Sep	8.906	1442
1897	Jan	4.877	352	1938	Sep	2.362	127	1979	Oct	3.72	238
1898	Feb	5.182	384	1939	Aug	8.611	1168	1980	Jul	3.454	214
1899	Aug	7.239	656	1940	Sep	2.286	122	1981	Jul	6.304	515
1900	Jul	9.957	3262	1941	Jan	3.759	242	1982	Aug	3.016	179
1901	Nov	6.807	587	1942	Jul	6.325	520	1983	Aug	8.851	1385
1902	Dec	2.438	133	1943	Oct	5.969	476	1984	Aug	8.961	1502
1903	Sep	6.096	491	1944	Jul	1.702	79	1985	Sep	5.922	470
1904	Jul	3.81	247	1945	Nov	3.353	206	1986	Nov	7.058	626
1905	Jul	8.382	990	1946	Jul	4.877	352	1987	Jun	4.679	330
1906	Oct	8.687	1232	1947	Dec	5.893	465	1988	Dec	5.408	410
1907	Dec	3.124	188	1948	May	4.928	356	1989	Apr	9.382	2062
1908	Sep	4.801	343	1949	Oct	6.706	572	1990	Jul	7.654	628
1909	Aug	7.239	656	1950	Mar	10.058	3500	1991	Jul	9.612	1583
1910	Sep	4.572	318	1951	Sep	7.772	764	1992	Oct	7.927	679
1911	Jul	4.572	318	1952	Jun	9.703	2630	1993	Oct	8.847	969
1912	Sep	6.833	589	1953	Nov	7.772	764	1994	Feb	3.914	232
1913	Jul	6.02	481	1954	Feb	3.404	211	1995	Jul	7.623	624
1914	Mar	3.505	220	1955	Aug	8.433	1027	1996	Oct	7.535	611
1915	Sep	6.858	594	1956	Jul	9.601	2400	1997	Jan	4.109	248
1916	Oct	8.738	1280	1957	Jul	2.235	118	1998	Sep	5.229	348
1917	Oct	8.636	1192	1958	Oct	7.137	640	1999	Jan	4.208	256
1918	Aug	7.925	799	1959	Oct	9.068	1641	2000	Sep	6.692	506
1919	Oct	2.743	157	1960	Sep	8.915	1454	2001	Oct	4.682	296
1920	Aug	6.655	564	1961	Dec	7.163	644	2002	Jan	3.328	186
1921	Sep	7.442	696	1962	Sep	6.782	582	2003	Aug	4.577	288
1922	Aug	9.169	1778	1963	Aug	4.42	303	2004	Sep	3.514	200
1923	Oct	7.442	696	1964	Oct	7.747	759	2005	Sep	5.398	365
1924	Aug	7.772	764	1965	Aug	2.591	145	2006	Jan	3.505	200
1925	May	10.109	3676	1966	Nov	7.188	648	2007	Dec	2.392	121
1926	Jun	6.198	503	1967	Mar	2.591	145	2008	Oct	1.702	77
1927	Oct	3.81	247	1968	Aug	6.147	497	2009	Dec	1.864	87
1928	Jul	3.962	259	1969	Jun	7.645	738	2010	Dec	9.702	1686
1929	Oct	2.743	157	1970	Sep	8.865	1403	2011	Aug	5.6	386
1930	Oct	4.597	322	1971	Feb	8.458	1054	2012	Mar	10.602	3625
1931	Jun	9.601	2400	1972	Sep	4.257	287				
1932	Sep	7.645	738	1973	Aug	5.789	454				

2.3.1.2. Data Prior to 1892

As mentioned, the Reference 1 study made considerable effort to extend the total record period and include several large flood events that occurred in the 1800's prior to the continuous data period mentioned above. Various sources were analysed and it was concluded that from 1838 to 1892 four large flood events occurred which could potentially affect the high flood record.

Flood events in 1852, 1853, 1870 and 1891 all exceeded 10 m on the Hampden Bridge gauge and caused significant flooding in the region.

It is difficult to calculate these flows for present day conditions due to the influence of the dams, changes in vegetation extent/density and the presence of the Wagga and North Wagga levees, however their peak heights at Wagga Wagga were generally 0.5 m or more higher than the 1925 flood. It was therefore assumed in the Reference 1 study that these events exceeded the 1925 flood event discharge inclusive of a hypothetical Burrinjuck Dam. This assumption is reviewed in the current study, see Section 4.3.

Table 3: Four Large Floods Prior to 1892

Year	Month	Gauge Height (m)
1852	June	10.67
1853	July	10.9
1870	April	10.67
1891	June	10.46

Note: the 1925 peak flood level is 10.1 m[4]

2.4. Model Calibration/Validation Data

Generally calibration/validation is a process whereby historical events are used to test a models ability to accurately replicate observed behaviour (i.e. match historical flood levels). A feature of this study is the excellent data available for the March 2012 and December 2010 flood events. Available calibration data (for both events) is listed below and detailed in the ensuing sections:

- ADCP measurement of flow upstream of Gobbagombalin Bridge (at close to peak flow);
- Recorded water level at Hampden Bridge Gauge (410001);
- Dozens of peak flood level marks for both the 2010 and 2012 events; and
- Aerial photos of the flood extent at or near the peak of flooding (provided by Nearmap).

Most data has been collected via the SES flood intelligence reviews (References 4 & 5) for the December 2010 and March 2012 events and from NoW.

Similar data sets were available for the 1974 event and were used to determine if changes on the floodplain could influence the stage/discharge relationship at Wagga Wagga (see Section 3).

2.4.1. Stream Gauge Data

Flow hydrographs and stage hydrographs were available for the Hampden Bridge gauge at Wagga Wagga (see Section 2.3) for the 2012, 2010 and 1974 floods. Flow input into the model is based on the Hampden Bridge hydrograph with some minor adjustments to peak flow to account for specific event discrepancies that are explained in the following sections. Table 4 displays the observed and model input flow for the 2012, 2010 and 1974 events.

[4] Whilst the 1925 peak flood level is lower than levels achieved by 19th century events, vegetation levels are lower in 1925 relative to 19th century conditions.

Table 4: Comparsion of Observered and Modelled Input Flows

Event	NoW Flow (m^3/s)	Model Input Flow (m^3/s)	Comment
2012	3,630	3,800	5% increase in flow for model input (see Section 2.4.1.1)
2010	1,680	1,680	No adjustment to observed flow
1974	5,710	5,425	5% reduction in flow for model input (see Section 2.4.1.2)

An explanation for the changes made to the model input hydrographs is presented in the following sections.

In addition, observed and modelled stage hydrographs were compared for all events. Results are discussed in Sections 6.1.2, 6.2.2 and 6.4.2 for the 2012 calibration, 2010 validation and 1974 events respectively.

2.4.1.1. 2012 Flow Adjustment

The 2012 event was gauged upstream of Gobbagombalin Bridge at the flood peak (see Section 2.4.2). The gauging was undertaken in two sections, one that captured the flow in the main channel and another for flow on the northern floodplain. Image 1 displays the gauging cross sections for the 2012 event. The pink line indicates the northern cross section (Cross Section 1, Image 2) and the purple lines indicate the two routes taken for the main channel (Cross Sections 2 and 3, Image 2). The cross sections do not completely align and consequently a portion of flow was not gauged and hence was not included in flow calculations (see Image 2). Missed flow was estimated (via hydraulic modelling) to be approximately 100 m^3/s (3% of the gauged flow). As such, 3% of the total flow was added to the observed hydrograph and an additional 2% was added to account for upstream attenuation. Accordingly the modelled input peak flow is 5% greater than the peak event flow estimated by NoW.

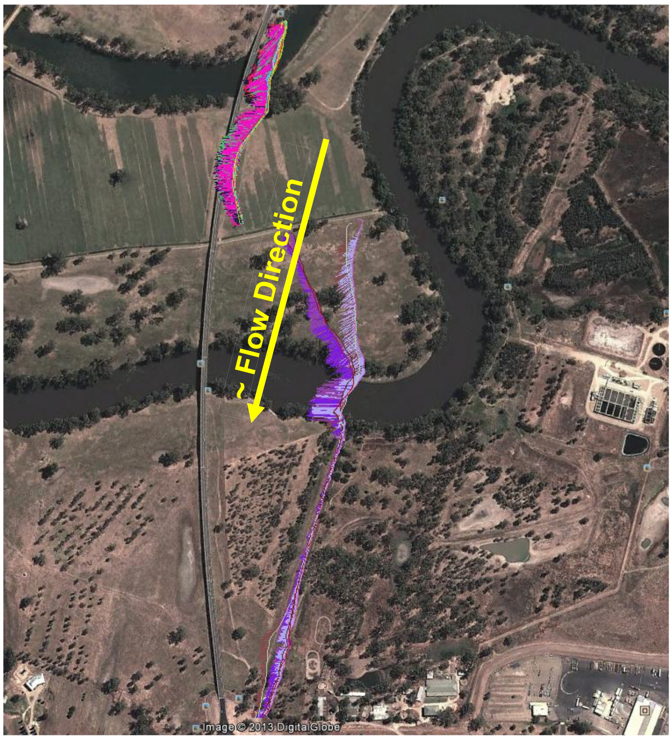


Image 1: 2012 ADCP gauging cross sections

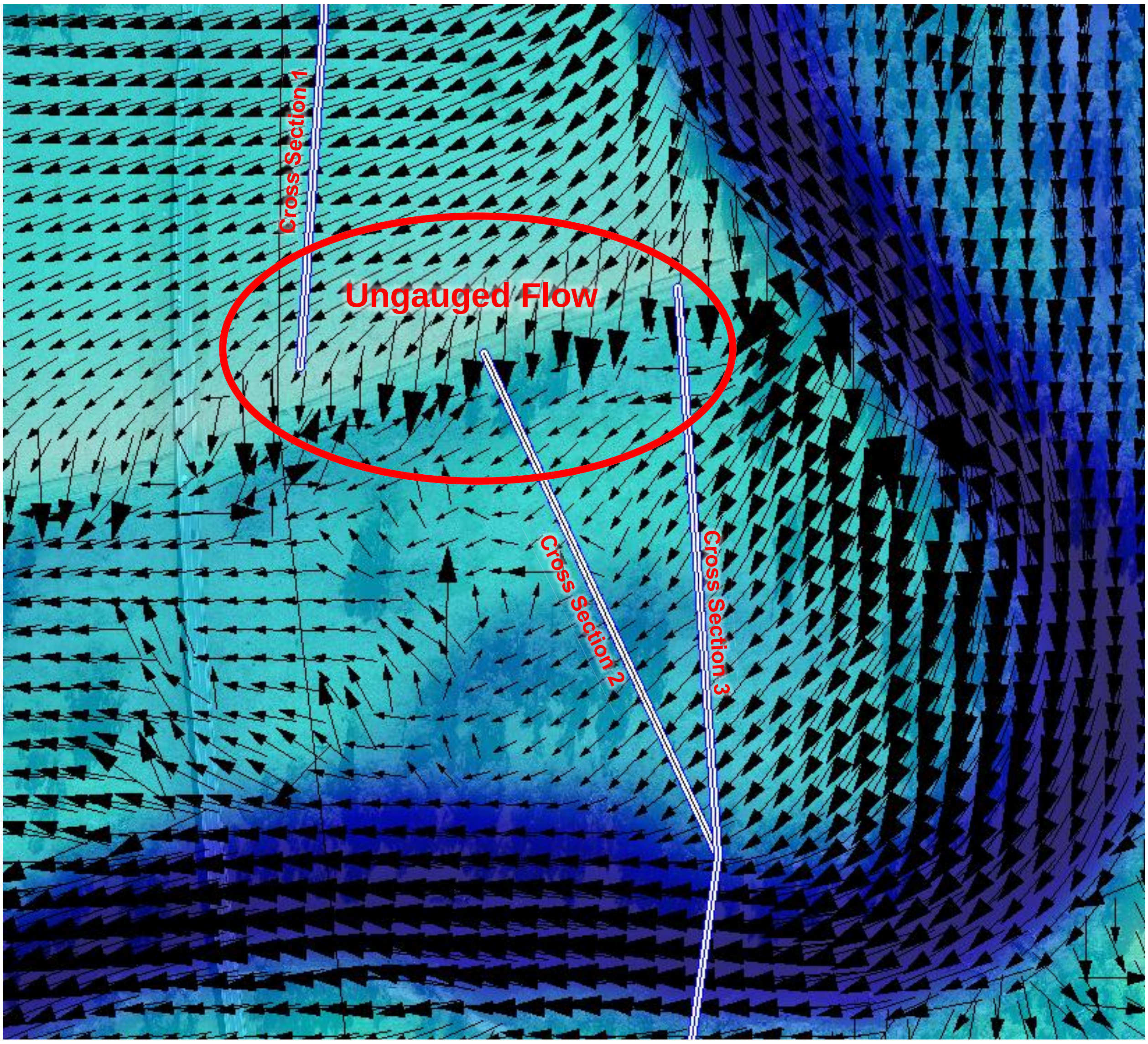


Image 2: 2012 modelled flow direction (ungauged flow)

The same cross section locations were used during gauging of the 2010 event however, the ungauged flow between the cross sections was negligible for the smaller 2010 event.

2.4.1.2. 1974 Flow Adjustment

The 1974 event was gauged upstream of the Railway Bridge (towards the eastern end of town) post the flood peak. The event was gauged at 10.357 m (see Section 2.4.2) and was determined to have a flow of 4,172 m^3/s at this stage. The 1974 peak flow estimate (NoW) of 5,711 m^3/s was determined via extrapolation. In previous studies (References 1, 2 and 3) the peak flow has been reduced by 5% to improve model calibration. This has also been done for the current study to achieve a better model fit for this event and is considered reasonable due to possible inaccuracies associated with extrapolating flow significantly above the maximum gauged flow.

2.4.2. Gauged Flow Records

Flood Flows for the 1974, 2010 and 2012 events were gauged by NOW. The 1974 gauging occurred post the peak upstream of the Railway Bridge using impeller techniques prevalent at the time, whilst the 2010 and 2012 events were gauged upstream of Gobbagombalin Bridge at approximately the peak of the respective events using the latest ADCP techniques.

As such for each of these events, highly accurate, location and time specific, point measurements exist of flow. These are the most accurate data the study has in regard to flood flow magnitude and as such this data source has been prioritised for calibration purposes (see Section 5.7).

2.4.3. Peak Flood Levels

The Reference 4 and 5 studies provide numerous peak flood levels for the 2012 and 2010 flood events suitable for model calibration/validation. 58 peak flood marks were obtained for the 2012 flood in the vicinity of Wagga Wagga and 25 for the 2010 event. For the 1974 flood 90 flood marks were obtained from the Reference 3 study. These levels were compared to modelled peak flood levels to determine if overall model behaviour was accurate.

2.4.3.1. Peak Flood Level Survey Errors

Whilst attempting to calibrate/validate the hydraulic model, peak model results were compared to surveyed peak flood levels (see Section 6.1 and 6.2). However it was noted that some of the flood marks obtained from the Reference 4 and 5 studies were significantly different to surrounding flood marks. This led to some work to remove erroneous flood marks.

Six recorded flood marks from the 2010 flood and eight from the 2012 flood have been removed from the model calibration/validation data set.

2010 Peak Flood Levels
Flood marks 57, 63, 64, 65, 66 and 67 from the survey brief contained in Appendix B of the Reference 5 study have been investigated and shown to be unlikely to provide an accurate reflection of peak flood levels.

Flood mark 57 was eliminated by examining the approximate level of the adjacent peak flood extent (achieved by examining an aerial image of the peak flood extent and the DEM, displayed as green points on Image 3) and then comparing this to the surveyed flood level (displayed as a red point on Image 3). It was found that the surveyed peak flood level was approximately 1 m lower than the expected flood level.

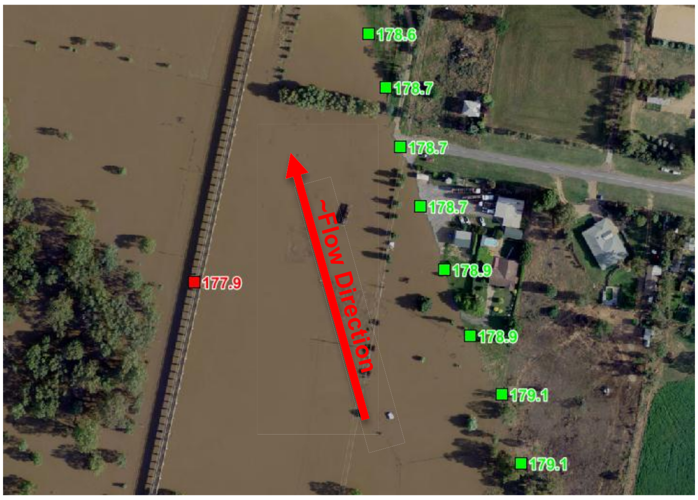


Image 3: Floormark 57

Flood mark 66 was recorded as not overtopping Hampden Avenue at this location (see Reference 4). However Image 5 displays that water clearly did overtop the road at this location and flowed with significant depth in the area. As flood mark 67 has been recorded at the same level as 66 and is in close proximity it can be assumed that this flood mark is incorrect also. Interestingly, these flood marks have the same recorded level as flood mark 57 which shows that there was a large body of still or very slow flowing water sometime after the flood peak which is probably why the recorded marks were particularly obvious and recorded incorrectly.

Image 4: Flood mark 66, brief photograph

Image 5: Flood marks 66 and 67

Similarly flood marks 64 and 65 (point displayed in red, Image 6) are all likely caused by post peak still/slow flowing water as mentioned for flood marks 66 and 67. Again examination of peak flood level imagery in combination with the DEM (point displayed in green, Image 6) revealed that the recorded flood marks were lower than the peak flood level. Both flood marks 66 and 67 are estimated to be recorded as 0.7 m lower than the actual peak flood level.

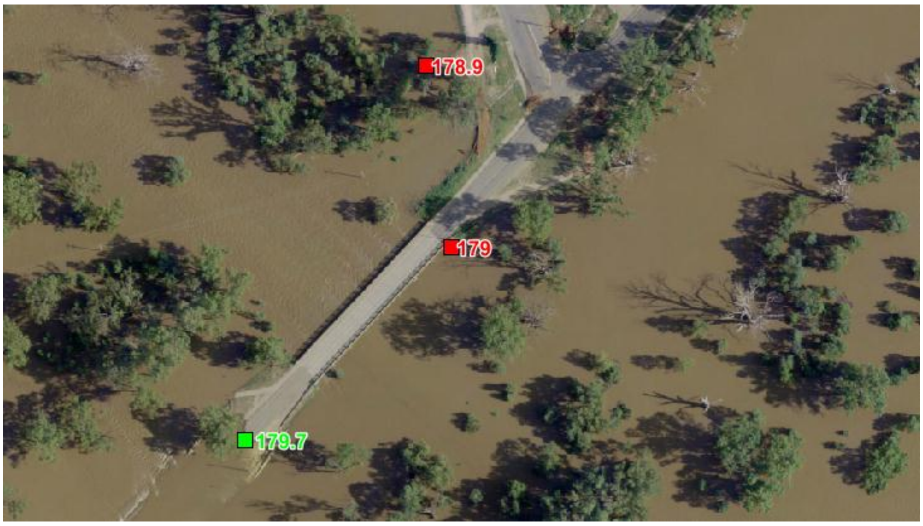


Image 6: Flood marks 64 and 65 (displayed in red)

Flood mark 63 was recorded to have the same peak flood level as flood mark 64 even though it is situated approximately 1 km downstream (and therefore must be lower in level). Accordingly flood mark 63 was removed from the data set.

2012 Peak Flood Levels

Flood marks 39, 40 and 41 have been removed from the calibration data set as they display the peak flood level created by Kyeamba Creek flooding. Tributary flooding has not been modelled as part of the current study.

Flood marks 99 and 97 have been removed due to the high discrepancy (-0.6 m and +0.4 m) from flood mark 98. Flood mark 98 has a high degree of confidence associated with it as a local resident watched the 2012 flood and marked the high water level with a nail in a tree. Furthermore flood marks 99 and 97 are two marks that were particularly difficult to identify and had an associated low degree of confidence.

Flood mark 85 is likely incorrect with an approximate 0.5 m underestimate of peak flood level. Investigation of the flood mark photographs in in the Reference 4 Survey Brief display the recorded flood mark as being well above the height of the person who recorded it and was possibly missed due to difficulties in seeing the true flood mark. Image 7 shows the observer displaying the false flood mark well above head height and the true peak flood level is estimated to be higher still.

Flood marks 95 and 87 are recommended for depth comparisons due to error in the surveying work. The surveyor was required to survey the ground height at the location of each peak flood level which can be used to compare to model depth.

Image 7: Flood mark 85, 2012 flood

2.4.4. Peak Flood Extents

An estimate of peak flood extent informed by aerial flood imagery for the 1974, 2010 and 2012 floods was created for comparison to modelled peak flood extents. It should be noted that the imagery was recorded around the time of the flood peak at Wagga Wagga and as such flood extents upstream and downstream of Wagga are post and pre peak respectively. This makes comparison of flood extents misleading for these regions. The flood extents are displayed on the calibration/validation and 1974 model figures (Figure 15 - Figure 17).

3. CHANGE IN STAGE/DISCHARGE RELATIONSHIP AT WAGGA

3.1. Introduction

The March 2012 event, given the relatively low peak flow recorded, achieved remarkably high flood levels at Wagga Wagga. At Wagga, a peak flow of approximately 311 GL/day was gauged during this event, however previous rating relationships derived from gauging of earlier events would suggest a flow of approximately 400 GL/day. Due to this discrepancy the rating table at Wagga has been revised by NoW, however no mechanism has yet been identified as the cause of this shift in rating.

The change in stage/discharge relationship is possibly due to changes in River and/or floodplain characteristics, which have altered flow efficiency for the worse. Potentially, one or a combination of things may have occurred:

- Conveyance capacity has been reduced – this means that the flow capacity, at a given river height, has been reduced. This could occur due to a change in cross-section shape or area within the River in-bank or on the floodplain. Potential sources of this include the construction of roads and levees on the floodplain or the siltation of the River in-bank due to the construction of upstream dams; and/or
- An increase in the cross-sections roughness. An increase in roughness may be caused by an increase in the density of vegetation situated in the River channel/bank or floodplain and/or the construction of man-made structures on the floodplain. Debris lodged in vegetation could also significantly increase channel roughness.

Alternatively, a change in recording techniques and technologies may have led to discrepancies in recorded flow values. Previously, flow recordings were undertaken with a current meter however in the last decade Acoustic Doppler Current Profiler (ADCP) technology has been introduced to record flows.

As part of this study WMAwater has been asked to investigate the shift in rating via computer modelling and a review of available data. Details are presented in the following sections.

3.2. Comparison of 2012 Bathymetry to Historic Cross Sections

A cross section from the 1974 gauging and also numerous cross sections obtained from the RTA (survey undertaken in 2001) and from the Reference 1 Rubicon model were compared to the bathymetry survey mentioned in Section 2.2.2. Ideally this comparison provides an indication of changes to the in-bank morphology that have occurred over time.

The 1974 gauging cross section (located at the Railway Bridge) shows that the current River thalweg at this location is significantly lower than what it was in 1974 (see Figure 4). However as there is only one cross section available for this time it is hard to draw any conclusion as to what the general change in bathymetry over this period may be. Specifically, it is not reasonable to extrapolate this difference in in-bank characteristics as this may be localised.

Also, in regards to the change in rating it would appear that this decrease in thalweg depth would increase channel conveyance and as such is not the cause for the changes in stage/discharge relationship mentioned above.

The RTA cross sections compare very well to the 2013 hydrosurvey with the general in-bank shape and depth remaining relatively stationary for the majority of cross sections (see Figure 5 to Figure 8). This means that it is likely that there have been no significant changes since the RTA survey undertaken in 2001. Some minor changes between cross sections have occurred but these changes are neither positively nor negatively biased indicating that there has been no general trend in changes to bathymetry during this period.

The Rubicon cross sections were not able to be properly aligned to the ALS or 2013 cross sections which made comparison infeasible.

Generally it cannot be concluded that there has been a significant trend in river morphology in recent history. Accordingly the same model bathymetry has been used for modelling of the 2012, 2010 and 1974 floods and hence the assumption has been made that in-bank conveyance has remained relatively stationary.

3.3. Variance in Vegetation Density and Land Use

Variations in land use and vegetation density over time can have a significant impact on flood characteristics due their effect on channel roughness. Roughness, represented by the Manning's 'n' coefficient, is an influential parameter in hydraulic modelling. As part of the calibration process roughness values are adjusted within ranges defined in the literature so that the model may match observed peak flood levels at a variety of locations.

Chow (1959) provides the definitive reference work in regard to the setting of roughness values for hydraulic calculations. Chow presents a series of channel "scenarios" with varying characteristics and the derived roughness values for each. Chow also proposes a custom roughness calculation implementing the following equation (equation 5-12 from Chow 1959):

$$n = (n0 + n1 + n2 + n3 + n4).m5$$

Via this equation a representative 'n' is aggregated from addition of different elements. Value ranges are defined in Table 5-5 (Chow, 1959) and for the case of the Murrumbidgee River the following value ranges are obtained:

- Earth channel hence n0 = 0.02 (only value appropriate for a natural channel);
- Irregularity is moderate to severe ("badly sloughed banks of natural stream") n1 = 0.01;
- Variation of channel cross-section is "alternating occasionally" (large and small sections alternate occasionally) n2 = 0.005 (mid value);
- Relative effect of obstructions is minor (second best category) (refers to debris deposits, stumps, exposed roots, boulders and fallen and lodged logs) n3 = 0.01-0.015;
- Vegetation is high (high is for conditions comparable to the following; trees with brush and some weeds and so n4 = 0025-0.05 (mid value); and

- Degree of meandering is appreciable (mid value) and so m5 = 1.15.

Use of these values generates a Manning's n value ranging from 0.08 (lower end estimate) – 0.12 (upper end estimate). Henderson (1966) also provides roughness values for various land use and flow conditions. Table 4-2 of Henderson (1966) states that for a natural channel, roughness may vary between 0.025 - 0.03 for a clean and straight channel, between 0.033 - 0.04 for a winding channel with pools and shoals and between 0.075 - 0.15 for a very weedy, winding and overgrown channel.

The 1-D approach adopted by Chow and Henderson in deriving the above values includes turbulent losses as part of the overall roughness and therefore is ideal for hydraulic calculations. However the 2-D TUFLOW model implicitly contains some of these energy losses and as such recommended values can be on the conservative side. For the purpose of this study, both references are considered in the selection of Manning's 'n' coefficients however the Murrumbidgee River and surrounding banks are discretised along roughness zones rather than using a general value for the area.

The relatively large width of the Murrumbidgee river (approximately 60 m for the area under consideration) results in an average of three grid cells representing the deep channel area (20 m x 20 m grid) permitting a separate roughness definition for the river banks. As part of the bathymetry survey, photographs of the overbank were obtained providing an indication of roughness values. Image 8 - Image 13 show sample bank photos taken approximately every 10 km (locations displayed on Figure 3) and it can be seen that flow resistance from the bank is likely to be high therefore meriting a different value to that selected for the main channel. Table 5 summarises the roughness values used and for the bank area a Manning's 'n' of 0.1 is used for current conditions (namely the 2010, 2012 events and all design runs). Conversely riparian vegetation was significantly lower at the time of the 1974 event as illustrated in Image 14 which results in significantly lower roughness values (see Table 5).

Figure 9 and Figure 10 take a more detailed look at the changes between current and past land use and riparian vegetation. Comparison between April 1971 aerials taken by the Central Mapping Authority and the recent July 2012 aerial survey undertaken by Council provides a striking illustration of the increase in flow resistance for current times. Particularly in terms of riparian vegetation where a policy of protection towards River Red Gum and Yellow Box trees has resulted in a remarkable increase in extent and density. The effect the policy has had is impressive when observing the change between 1971 and 2012 (Image 14) and the relatively minor change in the similar period between 1944 and 1971.

Image 8: Section 069*

Image 9: Section 169

Image 10: Section 269

Image 11: Section 369

Image 12: Section 469

Image 13: Section 569

**Please refer to Figure 3 for section locations.*

Image 14: Aerial of the North Wagga Flats Area

Source: City of Wagga Wagga.
Date: 1st of July 2012

Source: Central Mapping Authority, Department of Lands.
Date: 16th of April 1971

Source: Survey Flight, Royal Australian Airforce.
Date: 5th of March 1944

Aerial photography analysis provides the impetus for the reduction in the ‘channel bank’ roughness from a Manning’s ‘n’ of 0.1 to 0.04 and a reduction of 0.1 to 0.06 for the riparian vegetation roughness. A minor increase in general floodplain roughness from 0.034 for the 1974 event to 0.04 for the 2012 event was implemented in order to optimise results and the difference might speculatively be put down to differences in land use and the historical tendency to remove vegetation. Conversely, the increased roughness of the floodplain between Wagga Wagga City and North Wagga Wagga as well as between Olympic Highway and Travers Street is self-evident from Figure 10. The increased roughness around Parken Pregan Lagoon is easily identifiable in Image 14. A map illustrating the land use applied in the model is shown in Figure 11 for recent/design events and Figure 12 for the 1974 event.

Table 5: Event Specific Land Type Classification and Manning's Coefficient

Material Type	Manning's n	
	2010 & 2012 Events	1974 Event
General, low level vegetation	**0.04**	**0.034**
Deep channel area delimiting hydrosurvey extent	0.03	0.03
Channel banks	**0.1**	**0.04**
Riparian vegetation	**0.1**	**0.06**
Urban	0.05	0.05
Rural properties	0.066	0.066
Cropping areas	0.072	0.072
Industrial areas	0.054	0.054
Parks	0.06	0.06
Golf courses	0.072	0.072
Low density trees	0.06	0.06
Medium density trees	0.078	0.078
Wagga Wagga Hills Open Forest	0.072	0.072
Floodplain	0.04	0.04
Floodplain between North Wagga and Cartwrights Hill	0.06	0.06
Area between Olympic Highway and Travers Street	**0.072**	**0.06**
Parken Pregan Lagoon	**0.09**	**0.06**
Floodplain between Wagga Wagga city and North Wagga	**0.095**	**0.06**

The effective roughness[5] of both models has been calculated and it was found that an increase of approximately 20% in effective roughness is apparent between the 1974 and 2012 models. As conveyance is inversely proportional to roughness this increase in roughness has led to an approximate 20% decrease in channel/floodplain conveyance. This effect is marked by the change in stage/discharge relationship mentioned previously in this report (see Sections 1.2 and 3.1).

The properties located on the floodplain east of Wagga Wagga are not considered part of the effective flow path due to the presence of flow retarding fences and buildings. In the model this was achieved by nulling grid cells based on digitised building outlines. This effectively constricted the available flow area. The "loss" of temporary floodplain storage by nulling the building outlines is a slightly conservative assumption as in reality some floodwaters may enter these buildings under some flooding scenarios. However this approach was adopted as it was considered that the impact of such an assumption would be negligible relative to the overall flood runoff volume. Note that in adopting this strategy it was ensured that buildings did not form unrealistic water tight seals to downstream or laterally available inundation areas.

3.4. Hydraulic Structures

Various changes to roads, bridges and other manmade structures that influence flow characteristics have been made on the floodplain surrounding Wagga Wagga between the 1974 and 2012 floods. These changes can influence flood levels and thus affect the stage/discharge relationship at the Hampden Bridge gauge. When modelling the 1974 event, infrastructure

[5] Note that the effective roughness weighted by Depth, Velocity and Velocity/Depth product has also been calculated with all variations to effective roughness calculated to be approximately 20%.

elements not present at the time were removed from the model. Specific changes to roads, bridges and levees between 1974 and 2012 are discussed in Section 5.

3.5. Event Variation of Debris Load

The debris load of a flood can significantly affect the conveyance capacity of a channel. Depending on the level of existing vegetation, debris can accumulate increasing effective roughness, thus increasing peak flood levels.

On the Murrumbidgee River, events that have the majority of flow contributed from regions upstream of Burrinjuck Dam will have lower debris loads than events with significant contributions downstream of the Dam. For example, 1974 flood waters are described as having less debris than the 2012 event. This makes sense in that many tributaries downstream of the Dam experienced floods of record during the 2012 event which contributed greatly not only to flow but also debris load in the form of trees and logs etc. Greater overbank vegetation during the 2012 event, combined with the larger debris load, meant that roughness during the 2012 event was much higher than it was during the 1974 flood which had lower debris loads and occurred in a period with less overbank vegetation.

3.6. Conclusions

The Murrumbidgee River model at Wagga Wagga was able to match 1974 observations successfully (see Section 6.4) by adjusting infrastructure to 1974 conditions and by modifying vegetation as per 1971 aerial photography (as per Section 3.3). This leads to the conclusion that the change in stage/discharge relationship at Wagga is substantially due to vegetation changes on the floodplain that have occurred over time. A change in effective roughness of approximately 20% has led to the stage/discharge relationship changing such that a given flow now produce relatively higher flood levels.

Accordingly confidence in the approximate magnitude of the 1974 event is confirmed and the change in flow recording techniques (see Section 3) is dismissed as a significant influencing factor. Changes in floodplain usage/vegetation and variation in debris load are the likely culprits for the change in stage/discharge relationship.

As such, 2012 and 2010 gauged flows have been used in model calibration and the most recent NoW rating has been used to determine flows for the period of 1991 – 2012 for use in FFA. FFA flows from the period from 1892 - 1990 utilise the rating curve produced from the 1974 model (see Section 6.4) with the assumption that vegetation density has remained relatively constant during this period.

4. HYDROLOGY

4.1. Background

The key purpose of this study is to define design flood behaviour for the Study Area described in Section 1.1 (See Figure 1). To achieve this goal the development of design flows via FFA (described in the ensuring sections) for input into a 1D/2D hydraulic model (see Section 5) was required.

The FFA undertaken as part of the current study builds on best practise engineering techniques used in the 2004 study (Reference 1) and also incorporates recent data from post the 2004 study. In addition to this, the current study incorporates the idea that vegetation changes have significantly altered the Wagga stage-discharge relationship (see Section 3), something not recognised at the time of the Reference 1 study.

4.2. Introduction

There are two basic approaches to undertaking design flood analysis:

- The rainfall runoff routing approach; 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 flood frequency approach.

The flood frequency approach is generally preferred over the rainfall/runoff routing approach where the length and quality of the observed record and accuracy of the rating curve are considered adequate. In addition, large complex upstream catchments will lead to less reliable design flow estimates when using rainfall/runoff routing methods.

Accordingly, this section describes the FFA undertaken as part of the current study which is based on work undertaken in the 2004 study (Reference 1). The analysis constitutes the hydrological analysis component of the study and aims to describe the probability of a given discharge occurring on the Murrumbidgee River at Wagga Wagga. Calculated design flows (as time varying hydrographs) are then input into the hydraulic model so that design levels can be determined.

4.3. Flood Frequency Analysis

4.3.1. 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 (See Section 2.3).

The FFA undertaken as part of this study uses the data set described in Section 2.3. Using this data set the analysis follows methods prescribed by Australian Rainfall & Runoff (AR&R) and builds on the method used in Reference 1. Where applicable data (annual maximum peak flood levels) from the Reference 1 study in conjunction with new years of record (2002 – 2012) have been incorporated into this analysis.

Generally speaking, the analysis consisted of fitting a probability distribution to a truncated series (events greater than 1,000 m³/s) 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 the Wagga Wagga 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 2.3 and 4.3.2) and the second stage involves fitting a probability distribution to the data set to determine design flows (see Section 4.3.4).

4.3.2. Background to Design Flood Estimation at Wagga Wagga

The 2004 Wagga Wagga flood study (WMAwater, 2004) derived a 1% AEP flow estimate based on FFA. Key features of this work included:

- The use of Bayesian methods to include four 19th century events which were adjudged to exceed the 1925 flow peak at Wagga; and
- The use of RUBICON derived rating curves, rather than NOW ratings, to estimate the flow of historical events.

The 2004 work used best practise engineering techniques and found a 1% AEP peak flow of 6,700 m^3/s[6]. Subsequent to the completion of the 2004 work, significant floods occurred in 2010 and 2012. These events highlighted a significant change in the rating at Wagga (approximate 25% reduction in conveyance for a given stage). Consideration of a number of factors tends to indicate vegetation and land-use changes as a plausible source of the rating change (see Section 3).

Given vegetation changes (likely in tandem with vegetation blockage) can significantly alter the stage-discharge relationship at Wagga, this new paradigm tends to call into question the work done in the 2004 report which adjudged that four 19th century events exceed the 1925 flow. This follows as the 2004 RUBICON model was calibrated to the 1974 event and given 1974 vegetation conditions constitute a low end vegetation scenario, the 2004 work likely overestimated the discharge of 19th century events.

As 19th century event peak discharges are likely to be revised downwards due to vegetation

[6] The expected probabilities were adjusted to account for sample bias yielding a 1% AEP flow of 6,900 m^3/s.

density at the time (as per Section 4.3.2.1), the issue of Burrinjuck Dam and the attenuating impact it may have had on the events in question, requires some investigation (see Section 4.3.2.3).

The following sections review the suitability of the inclusion of 19^{th} century events in Wagga FFA. Essentially, given higher vegetation levels and hence lower peak discharges for 19^{th} century events, inclusion of all four historical events as larger than the 1925 event (as per the 2004 flood study) requires revision.

4.3.2.1. Historical Vegetation near Wagga

A brief and non-exhaustive literature review was performed to obtain a better understanding of floodplain vegetation near Wagga in the 1800's. Details are generally vague but some insight into the condition of the Murrumbidgee River valley at the time was obtained.

Extracts from 'Two expeditions into the interior of southern Australia' by Charles Sturt, 1828, (Reference 6) indicate that much of the Murrumbidgee Floodplain upstream of Wagga, nearer to Jugiong and Gundagai, was particularly free from dense vegetation and suitable for pastoral grazing without significant land development. On the other hand the same document also indicates that the regions surrounding Wagga were more densely vegetated then other regions upstream. In Pondebadgery (now Wantabadgery), Sturt noted:

> *'To the west, a high line of flooded-gum trees extending from the river to the base of the hills'*

This suggests that the floodplain in the direction of Wagga Wagga was well vegetated. Furthermore, closer to Wagga in a region suspected to be Oura, Sturt noted that:

> 'There was an evident change in the river; the banks were reedy, the channel deep and muddy, and the neighbourhood bore more the appearance of being subject to overflow than it had done in any one place we had passed over'.

In the same region he refers to the area to the west (i.e. in the direction of Wagga Wagga) as *'wooded country'*.

'An Historical Analysis of Cattle Grazing Practices on the Flood Plain of the Murrumbidgee River', by Troy Whitford (Reference 7), indicates that the vegetation in the regions surrounding Wagga Wagga probably remained close to its 'original state' up until 1860 after which significant clearing occurred:

> *"The years after 1860 were dominated by the clearing of land. The ring barking of trees and removal of scrub was a primary activity of landholders, along with attempts to develop alternative stock watering points such as wells and bores. Early settlement schemes initiated during this period were heavily in favour of the established squatter. Many of the earliest selectors were either related to squatters or were "dummy selectors": That is, people with whom the squatters had made some arrangement to protect their land or*

improvements."

"The Wagga Wagga districts community's relationships with the Murrumbidgee River and Wetlands Over Time' paper (Reference 8) notes that changes in the second half of the 19th century that came in with the Crown Land Acts from 1861 and subsequent legislative alterations (motivated by NSW government policy that sought to create denser settlement) meant that over time lands were improved (from an agricultural perspective). This led to:

> *"from the 1860s to well into the 20th Century, closer settlement and the impact of grazing and agriculture on the river, appear to have increased the pressure on the river considerably."*

In addition, historical letters mentioned in Reference 8 noted that:

> *"with regard to selection, every available acre on both sides of the river has been taken up…"*
>
> *"But with it all, the miles of rung trees and deserted humpies tell their own tale of disappointed and unsuccessful selectors."*

The above tends to indicate that the floodplain near Wagga Wagga was more densely vegetated in the 1800's than it was in 1974 for example. In particular floodplain roughness is likely to have been significantly higher in the 19th Century than in 1974 (and perhaps currently) however the riparian zone in the 19th century is likely to have been less densely vegetated than now (2012 conditions) due to livestock being free to access the river. Post 1860 it is thought that vegetation began to decline until it reached conditions similar to those in 1974 in the early 20th Century[7].

As such it is expected that overall roughness in the 1800's was higher in the first part of the century than in the later part of the century. Further it is anticipated that for all of the 1800's roughness exceeded 1974 conditions.

4.3.2.2. Magnitude of Historical Events

Flood events in 1852, 1853, 1870 and 1891 all exceeded 10.5 m at Wagga and caused major flooding. Including these events in the 2004 FFA work (using Bayesian techniques) increased the 1% AEP estimate substantially. The events were included in the 2004 FFA work on the basis that their peak flow at Wagga exceeded the 1925 flow (estimated at ~ 3,600 m^3/s by 2004 study).

It is difficult to accurately estimate flows for the 19th century events due to unknowns related to vegetation density (see Section 4.3.2.1), the influence of upstream dams (particularly Burrinjuck) and possible changes in the river/floodplain.

Nevertheless an attempt has been made to estimate these flows and to then compare them with the current study 1925 flow estimate of 3,675 m^3/s.

[7] For example aerial photography from the 1940's shows a landscape very similar to the 1971 aerial photography (although this analysis has not been exhaustive).

Table 6 displays flow estimates for the four historic events at Wagga with the lower limiting flows being determined by the 2012 model (with levees), the upper limiting by the 1974 model (sans levees) and the best estimate by an 1800's model[8].

Table 6: Four Large Floods Prior to 1892

Year	Level	Flow (m³/s)		
		Lower Limit (2012 model)	Upper Limit (1974 model)	**Best Estimate (1800's model)**
1852	10.67	3,700	5,200	**4,000**
1853	10.90	4,200	5,800	**4,500**
1870	10.67	3,700	5,200	**4,000**
1891	10.46	3,300	4,600	**3,500**

The 2012 model has relatively high channel and floodplain roughness and incorporates the North Wagga and Wagga Wagga levees and it is reasonable to assume that 19th century event peak flows at Wagga are unlikely to be lower than these estimates.

The upper limiting flows have been determined using the 1974 roughness with 1800's conditions (i.e. levee, bridges etc. removed). Investigation of historic vegetation (see Section 4.3.2.1) indicates that channel and floodplain roughness is unlikely to be lower than during the 1974 flood and it is not expected that flows for these events would exceed the upper limit displayed below.

The 1800's model has the levees and bridges removed and uses the 2012 roughness layer with an increase in general roughness from 0.04 to 0.055 to account for higher levels of vegetation on the floodplain, however riparian roughness has been decreased to account for livestock having access to the river. Table 7 displays the applied roughness for the various models.

Table 7: Applied Model Roughness Values

Material Type	**Manning's n**			
	Current	1974	Pre-Levee	19th Century
General, low level vegetation	**0.04**	**0.034**	**0.034**	**0.055**
Deep channel area delimiting hydrosurvey extent	0.03	0.03	0.03	0.03
Channel banks	**0.1**	**0.04**	**0.04**	**0.04**
Riparian vegetation	**0.1**	**0.06**	**0.06**	**0.07**
Urban	0.05	0.05	0.05	**-**
Rural properties	0.066	0.066	0.066	0.066
Cropping areas	0.072	0.072	0.072	0.072
Industrial areas	**0.054**	**-**	**-**	**-**
Parks	0.06	0.06	0.06	0.06
Golf courses	0.072	0.072	0.072	0.072
Low density trees	0.06	0.06	0.06	0.06
Medium density trees	0.078	0.078	0.078	0.078
Floodplain	0.04	0.04	0.04	0.04
Floodplain between North Wagga and Cartwrights Hill	0.06	0.06	0.06	0.06
Area between Olympic Highway and Travers Street	**0.072**	**0.06**	**0.06**	**0.06**
Parken Pregan Lagoon	**0.09**	**0.06**	**0.06**	**0.06**
Floodplain between Wagga Wagga city and North Wagga	**0.095**	**0.06**	**0.06**	**0.06**

A result of interest from runs carried out herein is that the levee makes much less difference to

[8] 1800's model is based on the 1974 model however levee is removed, 2012 roughness map used and roughness values as per Table 7.

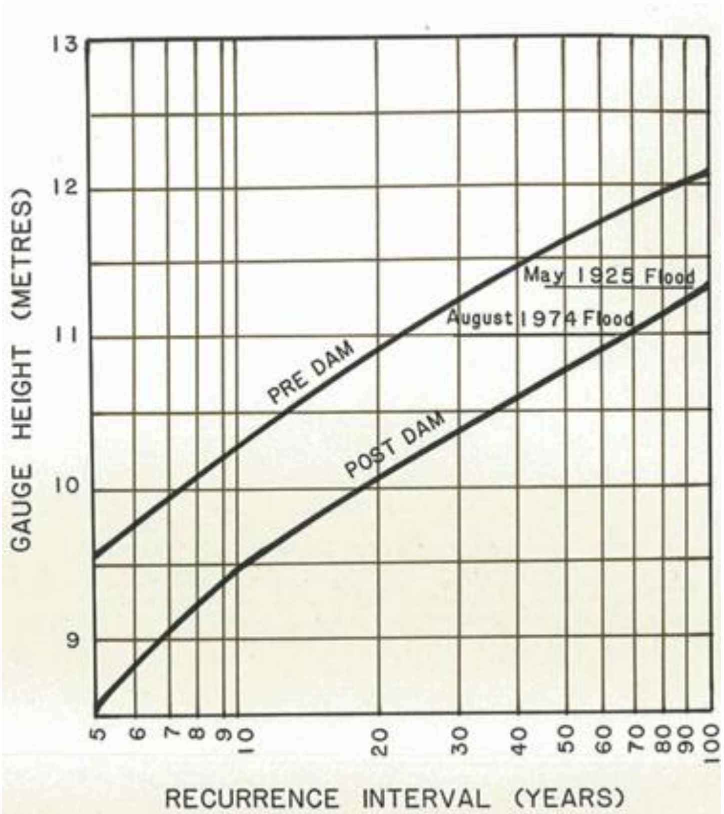
flood levels (for flows for which it has been tested) than might have been initially guessed. High ground on the southern floodplain (natural) tends to limit the amount of flow that can be conveyed via the southern floodplain.

4.3.2.3. The Effect of Burrinjuck Dam

A key issue with FFA is that the data series used must be homogeneous, that is, based on the same conditions over time. In using FFA for Wagga, a clear challenge is that conditions have not stayed the same over time. Instead dams have been built, changes in land use have occurred etc. Of the changes that have occurred, the most obvious one is the construction of significant upstream storages such as Burrinjuck (completed ~ 1930) and Blowering (completed in 1960's).

Previous work carried out by Public Works (Reference 9) indicates that Blowering has little impact on Murrumbidgee River flood flows. On the other hand the analysis, which was carried out for Gundagai, indicated a relatively significant effect at Gundagai when comparing pre and post Burrinjuck Dam 1% AEP levels (see Chart 1).

Chart 1: Gauge Height Versus Recurrence Interval at Gundagai - Pre and Post Dam



Herein reconsideration of the effect of Burrinjuck Dam on downstream flows is carried out using two basic methods. Firstly historic events are examined and secondly, pre and post dam FFA has been carried out. Note that the technique used to estimate the impact of Burrinjuck Dam is constrained by the non-availability of 19th century hydrologic data.

Historical events provide an estimate of Burrinjuck Dams potential impact. Table 8 below presents the dam inflows and discharges for numerous large historical flood events (criteria for inclusion was that event inflow exceeded 2,000 m³/s).

Table 8: Burrinjuck Dam Events Attenuation

Year	Dam Inflow (m³/s)	Discharge (m³/s)	Attenuation (%)
1916	2,963	1,949	34
1922	4,729	2,560	46
1925	10,000	5,166	48
1934	2,834	2,557	10
1935	2,101	489	77
1974	5,380	4,530	16
2012	3,310	2,859	14[9]
Average	**4,870**	**3,270**	**28**
SD.	**2,477**	**1,163**	**15**

It can be seen that the mitigating effects of Burrinjuck Dam for these events range from 10 to 77% with an average attenuation of 28%. The largest event to enter the Dam is the 1925 flood with a peak inflow of approximately 10,000 m³/s. Significant attenuation was observed in that particular event where approximately 50% of the flow was stored in the Dam. The largest of these events at Wagga was the 1974 event which was attenuated by 16%, still significant in regards to a flow exceeding 5,000 m³/s at Wagga. Median attenuation of the events is 34%.

As previously mentioned, several methods exist for investigating the likely attenuation provided by upstream dams on Murrumbidgee floods. The above analysis is limited to a number of historic events and sample size isn't large enough to provide a high degree of confidence. Also, the dam is expected to attenuate larger events less than smaller events. Furthermore the impact of the dam is heavily dependent on seasonality. Burrinjuck Dam's primary purpose is downstream irrigation and therefore events occurring when the dam is full are going to be significantly less attenuated than those occurring at the end of irrigation season when the dam is likely to not be full.

The second method of estimating the influence of upstream dams on flood flows was to undertake pre and post dam FFA analysis. This method did not provide any insight into the difference in design flows pre and post dams due to the insignificant difference between the data sets. Figure C1 displays the exceedance probabilities for both the pre and post data sets and further details are contained in Appendix C.

Overall then, it is not possible to describe individual event attenuation for the four historical events. However, based on the analyses carried out above, a conclusion can be reached that for the 1% AEP event, there is likely some attenuation. An estimate of the attenuation of the 1% AEP event could certainly, at the lower end, range from 10% - 20%[10]. As such 15% seems a reasonably conservative number to use in the ensuing analysis.

[9] 14% is an underestimate in that the event had two peaks of similar magnitude and the dam managed to capture the entire part of the first hydrograph peak. Really this figure could be as high as 100% for the first peak flow of the 2012 event.

[10] 10% is the least attenuation shown for seven historic events looked at. 28% is mean attenuation for seven events examined. 1974 is largest event on record (barring 1853 perhaps) and experienced attenuation of 16%. All of these values have been considered when deciding on a 15% estimate of attenuation.

4.3.2.4. Best Estimate of Historic Event Flows Assuming Existing Conditions

The estimated 1925 peak flow is 3,675 m^3/s based on the current 1974 model rating curve. Peak flow estimates for the four 19th century events taken from Table 1 are as follows:

- 1852 – 4,000 m^3/s. Assuming 15% attenuation this flow becomes 3,400 m^3/s which is less than the 1925 estimate;
- 1853 – 4,500 m^3/s. Assuming 15% attenuation this flow becomes 3,825 m^3/s and as such it exceeds the 1925 flow;
- 1870 – 4,000 m^3/s. As per 1852 although given clearing work done between the 1850's and the 1870's there is doubt as to whether or not this event can be concluded to be smaller than 1925; and
- 1891 – 3,500 m^3/s. Even prior to attenuation one might exclude the 1891 event although it is likely that the flow estimate is on the low side given this event occurred late in the 19th century following substantial land clearing. Assuming 15% attenuation against the flow estimate of 3,500 m^3/s gives a flow of ~ 3,000 m^3/s and as such it seems reasonable to state that this event is not larger than the 1925 event.

As such it seems that at least both the 1852 and 1891 events are smaller than the 1925 event.

4.3.2.5. Conclusions

In summary:

- the 2004 report currently defines the 1% AEP flow at Wagga as 6,900[11] m^3/s;
- the 2004 work included four 19th century events as greater than 1925 discharge. Flow estimates used to establish this ranking came from the 2004 RUBICON model;
- recent 2010/2012 events highlighted that the stage-discharge relationship in the River has changed to become less efficient;
- this study identified that the most likely cause of the change in rating was changes to vegetation in the overbank and floodplain (see Section 3);
- when including likely 19th century conditions (with respect to vegetation) in hydraulic modelling, flow estimates for 19th century events are lowered relative to 2004 work;
- when comparing revised 19th century flow estimates with the 1925 flow estimate at Wagga, and assuming low end Burrinjuck Dam attenuation, both the 1852 and 1891 events can be removed as events exceeding the 1925 event peak flow; and
- the 1853 and 1870 events have been assumed to be larger than the 1925 event under existing conditions for the purpose of FFA.

Wagga Wagga FFA has been updated on this basis in order to revise the 1% AEP flow estimate with further details in the ensuing sections.

[11] The expected probabilities were adjusted from 6,700 m^3/s to account for sample bias yielding a 1% AEP flow of 6,900 m^3/s.

4.3.3. Adopted Data Set

FFA has been performed on the highest recorded value of discharge for each year of record at the Hampden gauge at Wagga Wagga (see Table 2). Using a series of annual maximums lowers the risk of two successive peaks being dependent, and is recommended by ARR (2012). This data can be separated into two periods, the continuous data period (1892 – 2012) and the period prior to 1892 (1838 – 1891). The details of these two sets are described in Section 2.3 and a review of the Reference 1 assumptions on the magnitude of the historical events in relation to the 1925 flood has been examined in the previous sections. It has been determined that two of the four major events that occurred prior to the continuous record were larger than the 1925 flood.

4.3.4. Probability Distribution

A Bayesian maximum likelihood approach was used to fit a specified probability distribution to each of the scenarios. 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. It was found that the LP3 distribution fitted the data better than the GEV distribution and as such was used in preference (GEV distribution results are displayed in Appendix D). Flike (file version 5) was used to apply the Bayesian maximum likelihood approach.

The auxiliary events (1853 and 1870) were included in the software as ‘censored flows’ as the exact flow of each of these events is unknown. This approach entails setting a threshold and stating the number of pre-record flows above and below the threshold, where the threshold is the lowest of whichever flows are considered. It should be noted that the plotting position (displayed in Figure 13) of known events over a threshold are estimates only as the true rank (in relation to the historic events) of these events are not known. Furthermore, the data series has been truncated to remove more frequent events and all events less than 1000 m^3/s have been incorporated as censored data.

4.3.5. Design Flow Results

The update to the FFA has been carried out as follows:

- Flow estimates for the period of 1892 – 1990 have been derived from rating tables from the 1974 model from the current project rather than the 2004 work (note the two models produced similar ratings and so this change will have only a minor impact on flow estimates and hence the FFA work, see Section 6.4.5);
- Flow estimates for the period 1991 – 2012 have been determined using the most current NoW rating; and
- The 1853 and 1870 events have been assumed to be larger than the 1925 event and have been incorporated into the analysis using Bayesian techniques.

Following this methodology it was found that the continuous record, truncated to exclude frequent events (events less than 1000 m³/s), but including two significant events prior to the continuous record (1853 and 1870), formed the best representation of the record. Fitting a probability distribution to this record produced the revised 1% AEP estimate of 5,100 m³/s, just slightly less than the 1974 peak flow of ~ 5,200[12] m³/s. The frequency plot at Wagga is displayed in Figure 13. The frequency plot displays both the expected probability and the bias adjusted expected probability which accounts for sample bias. The bias adjusted probability distribution is preferred for determining flows for design events. However, it should be noted that the two probability distributions are approximately equal for exceedance probabilities between 2% and 0.5%.

The approximate exceedance probabilities of the 2012, 2010 and 1974 floods as determined by the selected distribution are displayed in Table 9.

Table 9: Calibration/Validation Event AEP

Event	**2012**	**2010**	**1974**
Flow (m³/s)	3,800	1,680	5,200
AEP (%)	~ 3%	< 5%	~ 1%

[12] Note the NOW estimate of 5,711 m³/s for peak stage of 10.74 m was based on extrapolation of a gauging taken post the peak at a stage of 10.36 m and is likely an over estimate. Attenuation is responsible for the reduction in input flow (to the model) from 5,425 m³/s (see Section 2.4) to 5,200 m³/s at the gauge.

5. HYDRAULIC MODELLING

5.1. Introduction

The current study makes use of the 1D/2D hydraulic model TUFLOW. The TUFLOW modelling package includes a finite difference numerical model for the solution of the depth averaged shallow water flow equations in two dimensions. The TUFLOW software is produced by BMT WBM (Reference 10) and has been widely used for a range of similar projects. The model is capable of dynamically simulating complex overland flow regimes. It is especially applicable to the hydraulic analysis of flooding in rural areas which is typically characterised by long duration events and a combination of supercritical and subcritical flow behaviour.

For the hydraulic analysis of overland flow paths, a two-dimensional (2D) model such as TUFLOW provides several key advantages when compared to a traditional one-dimensional (1D) model. For example, in comparison to a 1D approach, a 2D model can:

- provide localised detail of any topographic and/or structural features that may influence flood behaviour;
- better facilitate the identification of the potential overland flow paths and flood problem areas; and
- inherently represent the available floodplain storage within the 2D model geometry.

Importantly, a 2D hydraulic model can better define the 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 the outcomes to be incorporated into Council's planning activities.

5.2. Model Domain

The hydraulic model extent stretches from approximately 5 km upstream of Oura to 9 km downstream of Malebo Gap giving a total river reach of approximately 63 km. The model extent (shown in Figure 14) covers an area of approximately 220 km^2.

5.3. Model Grid Size

The model grid size utilised in the model build process is a finite difference grid of 20 m by 20 m. The model grid size was adopted following consideration of the extent of the modelling area, the required time step to satisfy the Courant criterion (relates to model stability), adequate resolution of the in-bank capacity and the resulting model run times involved. It was found that this resolution adequately represents in-bank hydraulic properties of the Murrumbidgee River. This issue is further addressed in Section 5.4. Model sensitivity to the selected grid size has also been investigated (see Section 5.8).

5.4. Structures

Structures such as bridges, levee banks and road crossings can have a significant impact on flood behaviour. Consequentially, proper discretisation and correct representation in the model is important. Further details are presented in the following sections.

5.4.1. Bridges

A total of six bridges have been identified in the study area. Five of these traverse the Murrumbidgee River namely (from upstream to downstream) Eunony, Railway, Hampden, Wiradjuri and Gobbagombalin Bridges. A bridge crossing Parken Pregan lagoon is also within the model extent and therefore included. The obstruction resulting from the bridge piers and deck have been modelled in three layers allowing for flow under and over the bridge deck. A fourth layer assumes the flow is unimpeded (i.e. flow over the top of the bridge rails). Each layer was assigned its own percentage blockage and form loss coefficient which were adjusted for each of the six bridges:

- Beneath the bridge deck: Blockage varied from 3% - 12% depending on the dimension, shape and spacing of the bridge piers. A small form loss of 0.05 (dimensionless) is used for the energy losses associated with the piers;
- The bridge deck: 100% blocked and the form loss coefficient is increased to 1 to account for the additional energy losses associated with flow surcharging the deck;
- The bridge rails: Blockage for the rails varies from 5% for the negligible steel rails on the railway bridge to 25% for the concrete rails across Parken Pragen Lagoon. A form loss of 1 was assumed for the rails; and
- Flow over the top of the rails: Flow assumed to be unimpeded.

Locations of modelled bridges are shown in Figure 14.

5.4.2. Levee Embankments

Figure 14 also shows the location of levees included in the model. The Wagga Main City levee constructed on the Southern floodplain, the North Wagga Wagga levee and the Gumly Gumly levee are discretised as impenetrable barriers to flow up to their crest elevation at which point they are overtopped making the zone behind them inundation prone. Temporary levees built around a number of quarries prior to flood events to prevent inundation are also included in the model. An additional temporary levee was constructed along Hampton Avenue prior to the December 2010 flood with the location of this levee presented in Figure 14.

For the 1974[13] event, the North Wagga and Gumly Gumly levees are removed from the model (as they were formalised to current levels in 1990 and 1992 respectively) and the elevation of Hampden Avenue was reduced by 0.1 m in order to represent the 1974 floodplain conditions. For further details please refer to Table 1.

[13] Note that many of the levees were partially present during this event but were overtopped and as such did not function as effective inundation controls (e.g. Gumly Gumly, North Wagga etc.).

Note that in all runs it is presumed that sand bags are used at the Sturt Highway near Marshall's Creek. Modelled runs assume that the sand bags are placed at a crest elevation of 182.4 mAHD which corresponds to the levee elevation at that point.

5.4.2.1. Levee Failure for Design Events

For the design runs a number of assumptions regarding levee behaviour have been made. The Main City levee has been assumed to fail at the locations specified in Appendix E, Figure E1 as follows:

- Failure at each location will start when the flow reaches a gauge height of 11.06 m or a level of 181.11 mAHD (as per Reference 2). The levee breach at each location is approximately 400 m wide;
- Failure will occur during a period of 5 minutes (that is the time when the levee starts to fail until reaching final cross section elevation);
- Final levee height would be half the height of the 1974 flood level, taking as base, a ground point on the city side of the levee; and
- Side slope of failure will be 1 in 2 (1 unit of rise per 2 units of run). See Appendix E, Figure E2 for a detailed drawing of the levee failure cross-section.

The North Wagga Wagga levee was represented in the model using the same methods as described above. The failure point was situated on the levee along Hopkirk Street (see Figure E3) with the following features:

- Length of failure of approximately 145 m;
- Trigger height of 180.15 mAHD; and
- Final height of 178.3 mAHD.

5.5. Roughness Values

Table 5 presents model roughness values for calibration and design runs and the 1974 run as discussed in Section 3.3.

5.6. Boundary Conditions

Upstream boundary conditions: Inflows were applied at the upstream end of the model (5 km upstream of Oura). Hydrographs for each respective event are detailed in Section 2.4.1.

Downstream boundary conditions: The downstream end of the model is located 9 km downstream of Malebo Gap as the constriction in flow area at the Malebo Gap may be an important control on water levels upstream (for larger events). Furthermore, a fixed water level boundary located 10 km downstream of the model extent is specified so that the resulting backwater profile does not impact upstream water levels and velocities.

5.7. Hydraulic Model Calibration/Validation

Model calibration was performed on the March 2012 flood and model validation on the

December 2010 event. Further to this the model was adjusted for the 1974 event to determine the validity of previous ratings at the Hampden Bridge gauge (see Section 3). As discussed in Section 2.4 a variety of data was available for the calibration exercise. Calibration data was prioritised in the following order:

1. Matching gauged flows at NoW gauging cross sections (see Section 2.4.2);
2. Matching the stage hydrograph level recorded at the Hampden Bridge gauge (see Section 2.4.1);
3. Matching peak flood levels mentioned in Section 2.4.3; and
4. Matching observed flood extents with modelled extents.

Model calibration/validation results for the 2012 and 2010 events are contained in Sections 6.1 and 6.2. Model results for the 1974 run are contained in Section 6.4.

5.8. Sensitivity Analysis

Sensitivity analysis was carried out in order to assess the effect that adjusting model parameters has on model results. Comparisons were carried out using peak flood levels and flows for the 2012 calibration event. The following scenarios were modelled in the hydraulic model:

- An increase in flow of 10%;
- A decrease in flow of 10%;
- An increase in Manning's n roughness of 10%;
- A decrease in Manning's n roughness of 10%; and
- A model grid size of 15 m (compared to 20 m as used in the other models).

A 10% difference in roughness was informed by the fact that a range of roughnesses for Wagga seems to be represented by 1970's conditions (lower end) and current conditions (upper end), the difference between which is 20% (0.042 to 0.05 for "effective" roughness, calculated as the average model roughness of wet cells over the domain). However it seems that a further 20% increase relative to current conditions is unlikely. Instead an increase of 10% relative to current (2012) conditions seems appropriate.

Obviously FFA work carried out shows that given various assumptions for 19th century events, there is a sensitivity in 1% AEP flow estimates. However the 10% run here is simply to observe the relative effect of 10% more flow (compared to roughness sensitivity).

All sensitivity analysis results are contained in Section 6.8.

6. HYDRAULIC MODEL RESULTS

A summary of the hydraulic model results are contained in the following sections. Hydraulic results provide peak flood surface levels, depths and extents for the calibration (see Section 6.1) and validation (see Section 6.2) events as well as design floods (see Section 6.6). In addition to this results for the 1974 flood are displayed in Section 6.4. Calibration/validation peak flood levels have been compared to surveyed flood levels (Section 2.4.3) and stage hydrographs (Section 2.4.1) where available and flows have been compared to gaugings where available (Section 2.4.2). In addition, peak flood extents have been compared to model flood extents where available.

6.1. Calibration Results – March 2012 Flood

6.1.1. Flow Comparison at Gauged Location

Comparison of modelled flows to gauged flows near Gobbagombalin Bridge (see Section 2.4.2 and Figure 15) found good agreement with the difference in modelled and observed flows less than 1%. The flow distribution (i.e. main channel to northern channel) was also accurately matched. Table 10 displays the flow comparisons for the 2012 event.

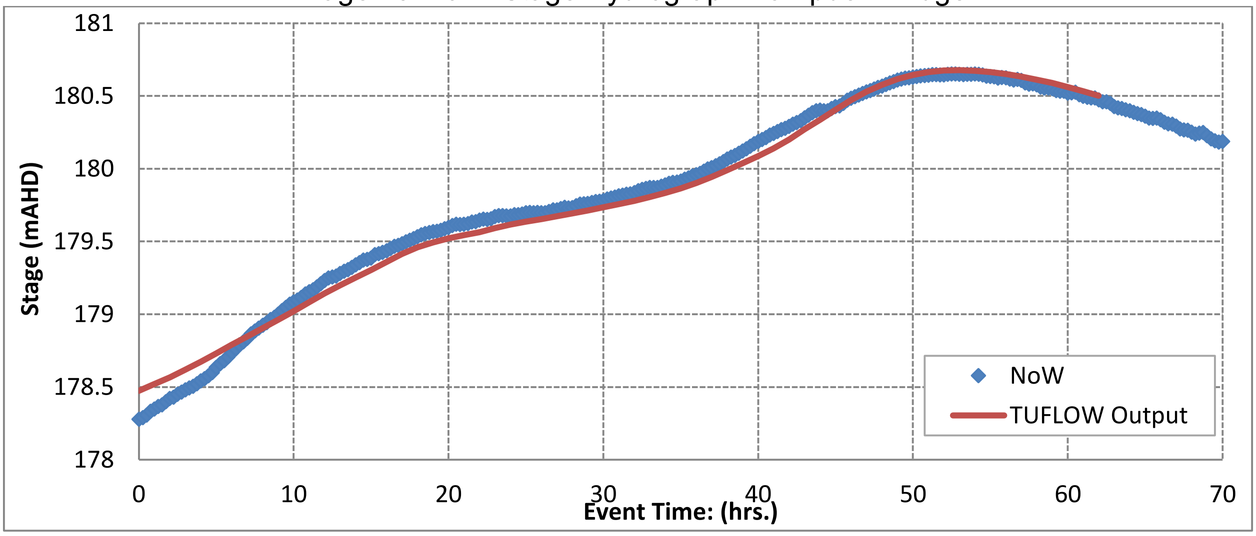
Table 10: Flow comparison at gauging locations – 2012 event

Gauging	Observed Flow (m^3/s)	Modelled Flow (m^3/s)	% Difference
Main Channel	1,680	1,689	0.5%
North Floodplain	1,814	1,833	1%
Total Flow	3,494	3,522	0.8%

6.1.2. Stage Hydrograph Comparison

The observed stage hydrograph at the Hampden Bridge gauge (see Section 2.4.1.1) was compared to modelled flood levels (see Image 15). The modelled flood level and timing was found to accurately represent observed conditions with a difference of 0.03 m at the peak.

Image 15: 2012 Stage Hydrograph Hampden Bridge

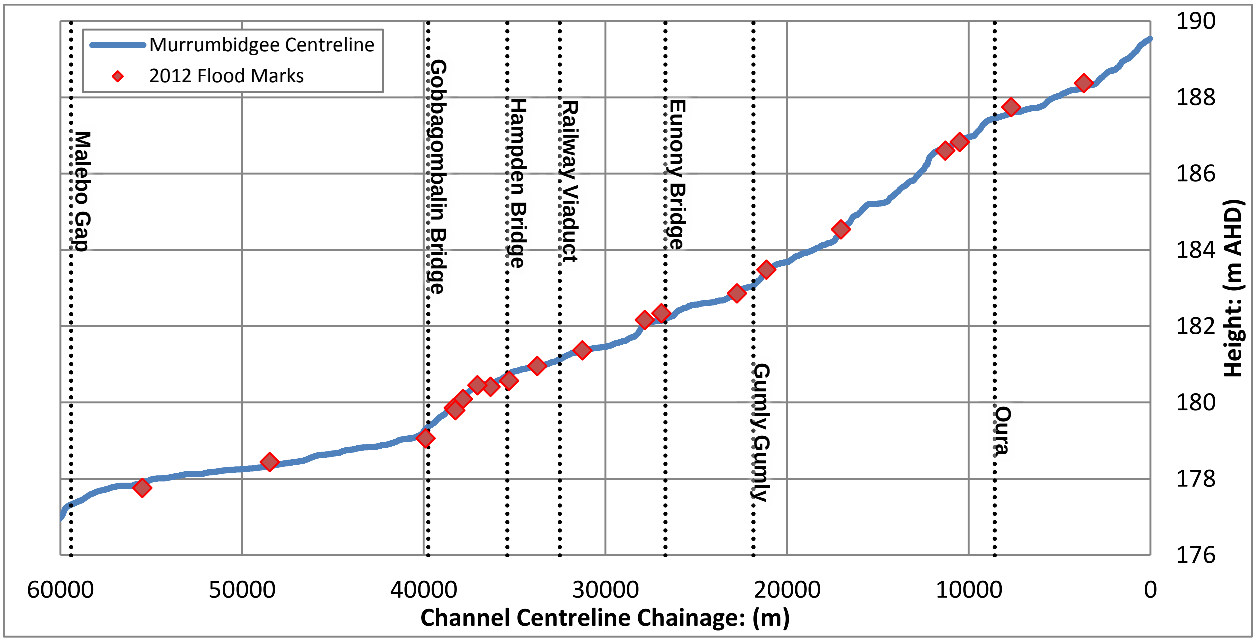


6.1.3. Peak Flood Level Comparison

Figure 15 shows the modelled March 2012 flood event depths and extent (raster) as well as a comparison of observed peak flood levels to modelled levels (displayed as red points) over the model domain. The maximum difference in peak flood level is an under estimate of 0.2 m at one point and an over estimate of 0.2 m at another (i.e. the modelled level is 0.2 m lower and 0.2 m higher than that observed), however a mean absolute error of approximately 0.07 m was achieved. This calibration is based on comparison of modelled and surveyed peak flood levels at 50 locations (a total of 58 peak flood levels were surveyed however eight of these were not used in calibration for reasons discussed in Section 2.4.3). Variation between observed and modelled levels was not positively or negatively biased, i.e. variance was due to minor localised effects, not overall model behaviour.

Chart 2 displays the peak modelled flood level of the 2012 flood event with a selection of recorded peak flood marks (Reference 4) that are situated proximate to the river. It can be seen that the modelled peak flood level aligns well with that observed.

Chart 2: 2012 Flood Profile and Flood Marks



6.1.3.1. 2012 Spatial Error Variance

A review of the spatial variance in the difference between observed peak flood levels to modelled levels revealed that for the 2012 event the model on average accurately reproduces observed flood behaviour throughout the model domain. Flood marks with large differences between modelled and observed levels tend to be scattered and are often surrounded by flood marks which have calibrated accurately.

The model is most accurate in the region surrounding the Main City and North Wagga Levees with a high density and percentage of flood marks calibrating well indicating the model results are similar to true flood behaviour.

Upstream, in the region surrounding Oura, the 2012 calibration also performed well with the exception of one flood mark. This may indicate the presence of a steep flood gradient in the region which was unable to be reproduced by the model. A possible cause of this is dynamic blockage which was unable to confirmed by observation.

Downstream of Gobbagombalin Bridge, flood marks are poorly distributed and calibration accuracy is in the order of 0.1 – 0.2 m. However no spatial bias is present indicating that the model may still be functioning well in this downstream area. Importantly, the flood mark furthest downstream at the Malebo Gap calibrated with a high degree of accuracy indicating the flood behaviour at the downstream end of the model is likely good. This is a particularly good result as the Malebo Gap is constrained and therefore variation in level is high compared to other regions with wider floodplain.

6.1.4. Peak Flood Extent Comparison

Figure 15 displays a comparison of observed and modelled peak flood extent. As mentioned in Section 2.4.4 the aerial photography on which the flood extent has been based was taken at the time of the peak at Wagga Wagga. Accordingly the peak flood extent fits well in the areas surrounding Wagga Wagga, however the match upstream and downstream of Wagga is not as good as the represented flood extent is post and pre peak respectively.

6.2. Validation Results

6.2.1. Flow Comparison at Gauged Location

Comparison of modelled flows to gauged flows near Gobbagombalin Bridge (see Section 2.4.2) found accurate representation of the peak event flow with only 2% difference between modelled and observed. The flow distribution was found to have too much flow in the northern floodplain and not enough in the main channel. This is likely to be because the 2010 flood was only slightly out of bank and therefore more likely to be affected by localised blockages that make it difficult to get breakout flow paths correct. Table 11 displays the flow comparisons for the 2010 event.

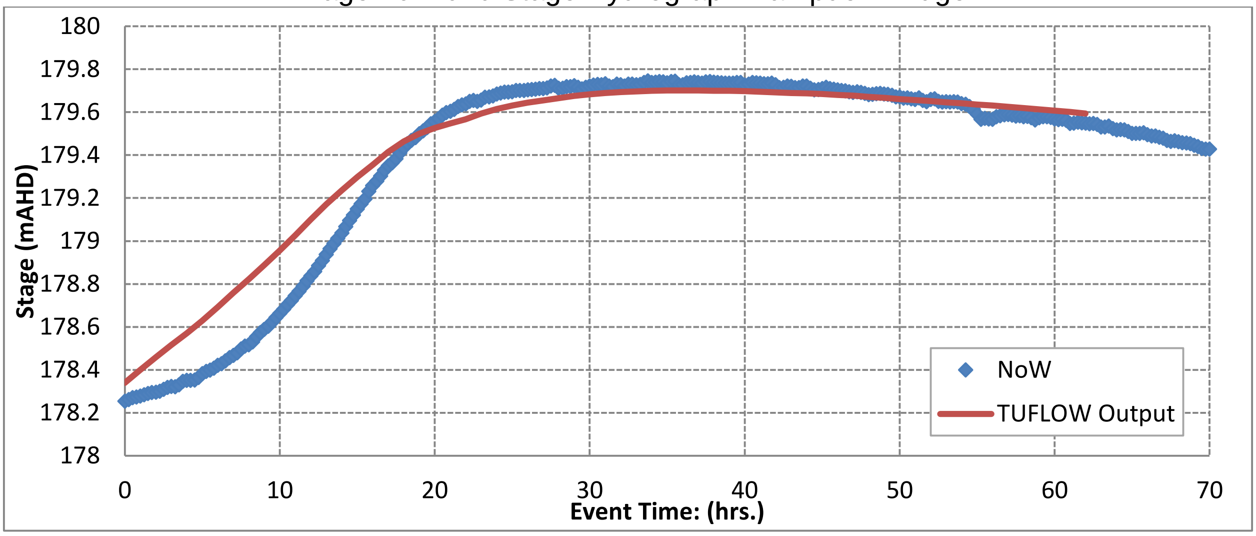
Table 11: Flow comparison at gauging locations – 2010 event

Gauging	Observed Flow (m^3/s)	Modelled Flow (m^3/s)	% Difference
Main Channel	992	858	-14%
North Floodplain	615	789	28%
Total Flow	1,607	1,647	2%

6.2.2. Stage Hydrograph Comparison

The observed stage hydrograph at the Hampden Bridge gauge (see Section 2.4.1.1) was compared to modelled flood levels (see Image 16). The modelled flood level and timing was found to accurately represent observed conditions with a difference of 0.04 m at the peak.

Image 16: 2010 Stage Hydrograph Hampden Bridge

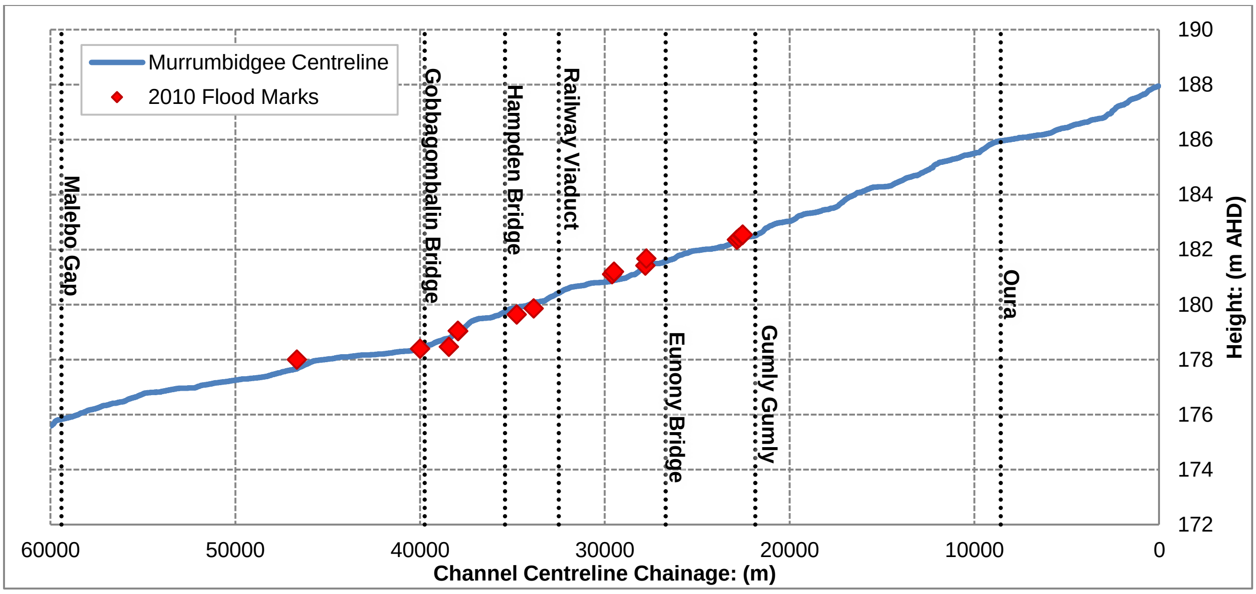


6.2.3. Peak Flood Level Comparison

Figure 16 shows 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) over the model domain. The maximum difference in peak flood level is an under estimate of 0.3 m at one point and an over estimate of 0.3 m at another (i.e. the modelled level is 0.3 m lower and 0.3 m higher than that observed), however a mean absolute error of approximately 0.15 m was achieved. This validation is based on comparison of modelled and surveyed peak flood levels at 19 locations (see Section 2.4.3). Variation between observed and modelled levels was not noticed to be positively or negatively biased, i.e. variance was due to minor localised effects, not overall model behaviour.

Chart 3 displays the peak modelled flood level of the 2010 flood event with a selection of recorded peak flood marks (Reference 5) that are situated proximate to the river. It can be seen that the modelled peak flood level aligns well with observed levels.

Chart 3: 2010 Flood Profile and Flood Marks



6.2.3.1. 2010 Spatial Error Variance

Flood mark density is relatively sparse for the 2010 event, however as with the 2012 model results no spatial bias is present indicating that generally the model accurately reproduces observed flood behaviour. Flood marks with large differences between modelled and observed levels are often beside flood marks which have calibrated accurately. In addition, as speculated previously, localised dynamic blockage which is unable to be reproduced during modelling probably impacts on peak flood levels particularly for events which are only slightly out of bank (such as the 2010 flood).

As with the 2012 model results, the highest degree of accuracy is in the region surrounding the Main City and North Wagga Levees with all flood marks in the vicinity calibrating well indicating the model results are similar to true flood behaviour in this region.

Available flood marks for this event only extend as far upstream (in the Wagga region) as Gumly Gumly, with the majority of marks in this area calibrating well. Upstream of here the accuracy of the model for this size event is speculative as no calibration data is available. However as global model parameters have been used it is likely that flood behaviour is reproduced relatively well. Good calibration of the 2012 event also tends to imply reasonable model performance for the 2010 event is likely.

Downstream of Gobbagombalin Bridge, there is only one flood mark available for comparison. This flood mark has not calibrated with a high degree of accuracy (-0.3 m difference between modelled and observed). Due to upstream flood marks calibrating well it is assumed that this is due to either localised flood mark error or perhaps error associated with the flood mark itself. Further downstream, in the vicinity of the Malebo Gap the accuracy of modelling is unknown as no flood marks exist in this region. Accuracy of the model in this region for events of size similar to the 2010 event is unknown. Again, given good 2012 calibration one might assume performance is reasonable.

6.2.4. Peak Flood Extent Comparison

Figure 16 displays a comparison of observed and modelled peak flood extent. As mentioned in Section 2.4.4 the aerial photography on which the flood extent has been based was taken at the time of the peak at Wagga Wagga. Accordingly the peak flood extent match fits well in the areas surrounding Wagga Wagga, however the match upstream and downstream of Wagga is not as good as the represented flood extent is post and pre peak.

For the 2010 event it seems likely that the water exiting at the oxbow adjacent to Mill Street is overestimated possibly explaining the reason for the overestimated proportion of flow going through the northern channel at Gobbagombalin Bridge. Extents around Eunony Lagoon are again marginally overestimated. Extents proximate to Wagga Wagga and North Wagga Wagga are in good agreement with aerial images.

6.3. Discussion of Calibration/Validation Results

The overall calibration/validation results are considered to be good to excellent in regards to the four calibration data sets (see Section 5.7). The results show that the model accurately reproduces peak flood levels and total flows for events of varying sizes. Furthermore the flow distribution is shown to be accurate for larger events meaning that for both the 1% and 5% AEP events the model will produce reliable results.

The results from the calibration/validation runs imply that a high degree of confidence can be had in the Wagga Wagga design flood level estimates, particularly at the 1% AEP level.

6.4. 1974 Model Results

6.4.1. Flow Comparison at Gauged Location

Comparison of modelled flows upstream of the Railway Bridge (see Section 2.4.2) were found to accurately represent the gauged flow (at a gauge height of 10.357 m) with only 3% difference between modelled and observed. Table 12 displays the flow comparisons for the 1974 event.

Table 12: Flow comparison at gauging locations – 1974 event

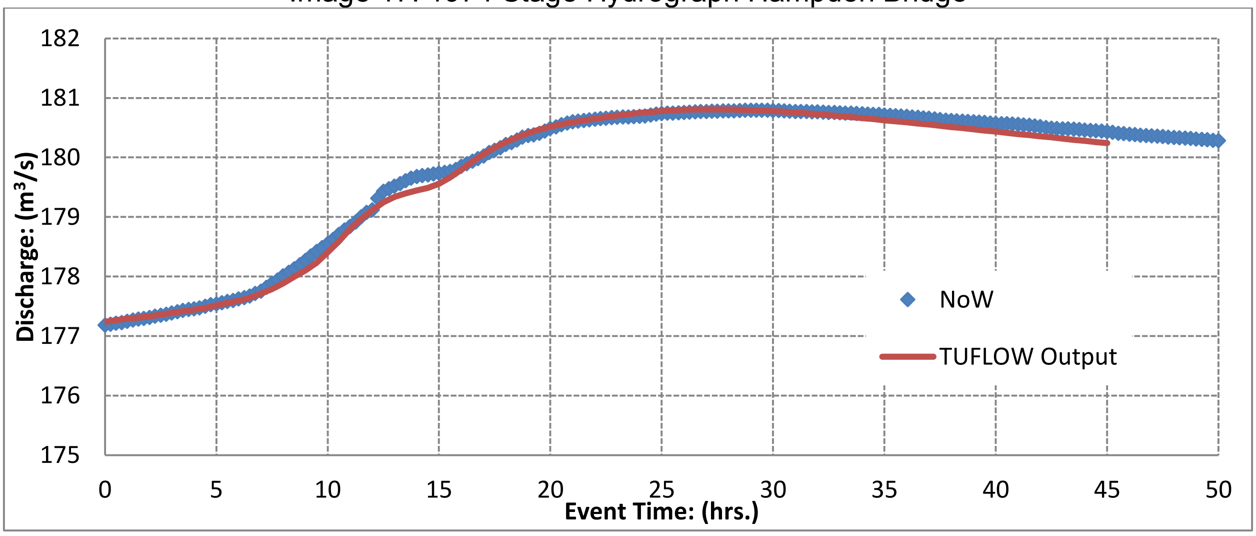
Gauging	Observed Flow (m^3/s)	Modelled Flow (m^3/s)	% Difference
Total Flow	4,172	4,087	2%

Note: The observed flow correlates to a gauge height of 10.357 m, lower than the peak flood height of 10.74 m

6.4.2. Stage Hydrograph Comparison

The observed stage hydrographs at the Hampden Bridge gauge (see Section 2.4.1.1) were compared to modelled flood levels (see Image 17). The modelled flood level and timing was found to accurately represent observed conditions with a difference of 0.03 m at the peak.

Image 17: 1974 Stage Hydrograph Hampden Bridge

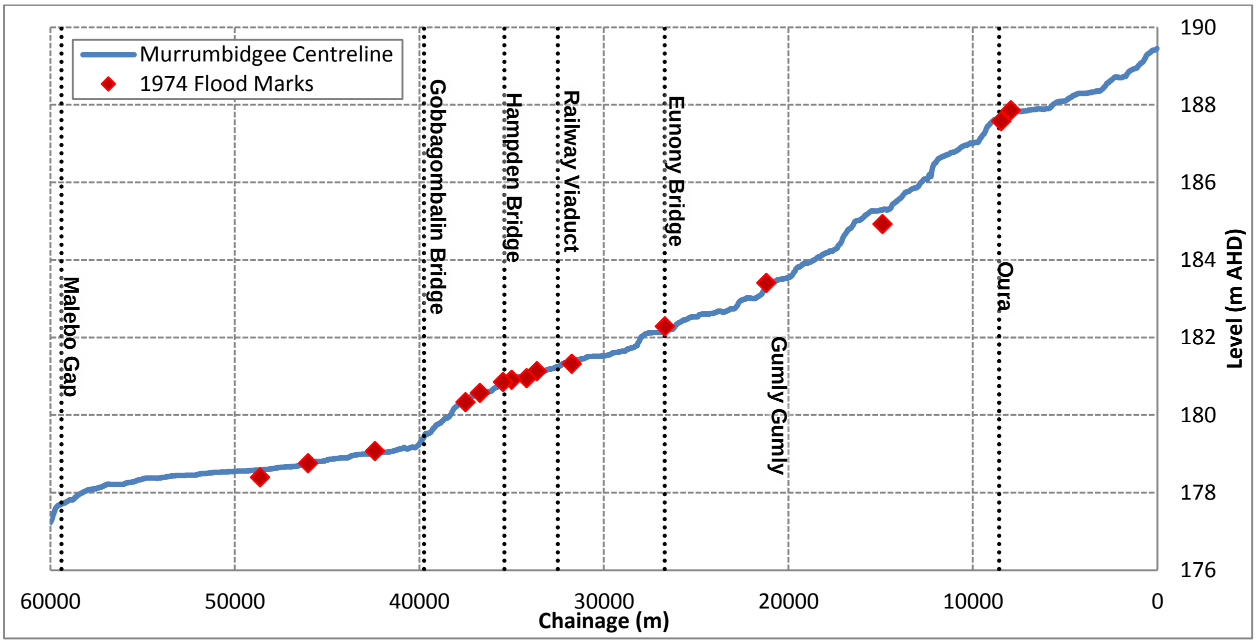


6.4.3. Peak Flood Level Comparison

Figure 17 displays the modelled August 1974 flood event depths and extent (raster) as well as a comparison of observed peak flood levels to modelled levels (displayed as red points) over the model domain. A comparison of modelled and surveyed peak flood levels at 90 locations (see Section 2.4.3) was performed. A mean absolute error of approximately 0.13 m was achieved. Variation between observed and modelled levels was not positively or negatively biased, i.e. variance was due to minor localised effects, not overall model behaviour.

Chart 4 displays a profile of peak modelled flood level for the 1974 flood event with a selection of recorded peak flood marks (Reference 2) that are situated proximate to the river. It can be seen that the modelled peak flood level aligns well with that observed particularly in the area for which the old aerial imagery was available stretching from Kyeamba Creek to San Isidore (approximately 17,000 – 45,000 m chainage on Chart 4).

Chart 4: 1974 Flood Profile and Flood Marks



6.4.3.1. 1974 Spatial Error Variance

Flood marks are densely distributed in the region surrounding Wagga for the 1974 flood however upstream and downstream of Wagga the density of flood marks reduces significantly. Again model accuracy is not spatially biased with flood marks not displaying large differences between modelled and observed levels and instead tending to be scattered and often surrounded by flood marks which have calibrated accurately.

As with the 2010/2012 model results, the model is most accurate in the region surrounding the Main City and North Wagga Levees with a high density and percentage of flood marks calibrating well indicating the model results are similar to true flood behaviour.

In the upstream a number of flood marks were available at Oura which as in the 2012 flood displayed a significant flood gradient. The gradient between measured calibration points at Oura (0.33% compared to an average water surface slope of 0.02%) was not replicated by the 1974 model. Careful consideration of Image 18 does not provide any justification for the steep gradient between calibration points relative to present day conditions, and investigation into the validity of these points and communication with Council indicates that the two downstream points are likely inaccurate. The paucity of data in the region for the 1974 event means that no conclusive adjustment to the local roughness could be made and again localised dynamic blockage is suspected as the culprit.

Downstream of Gobbagombalin Bridge, there are a number of flood marks available for comparison however they extend approximately 3km downstream of the Bridge only. The flood marks in this region have generally calibrated well without bias. Further downstream, in the vicinity of the Malebo Gap the accuracy of modelling is unknown as no flood marks exist in this region, however it is assumed that the model is producing relatively accurate results as global model parameters have been used.

Image 18: Oura in flood (looking north west) – September 1970

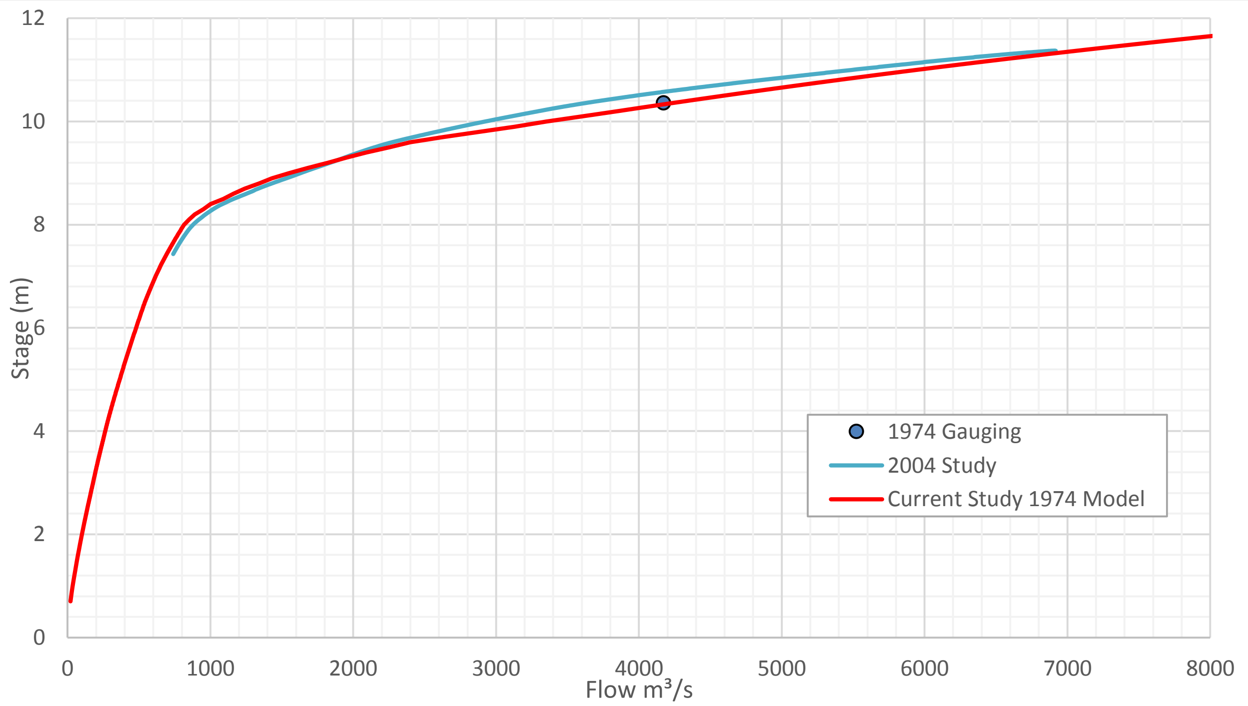
6.4.4. Peak Flood Extent Comparison

Figure 17 displays a comparison of observed and modelled peak flood extent. The modelled 1974 follows closely that digitised from the flood aerial. The Kyemba Creek catchment area appears to be overestimated however the area is in flood for the smaller 2012 event suggesting that the flood extent had receded at the time of the aerial being taken. Conversely the Northern Murrumbidgee floodplain seems to be marginally underestimated.

6.4.5. Comparison of Stage/Discharge Relationship to the 2004 Study

Chart 5 displays the model derived rating curve for the current 1974 event model (in red) as well as the 2004 study rating (in blue). It can be seen that the two rating curves are similar with generally no more than 0.2 m difference in peak flood level for a given flow between the two ratings. It should also be noted that the current model rating curve does provide a better match to the 1974 event gauging (displayed as a blue dot).

Chart 5: 2004/Current Study - 1974 Event Model Rating Comparison



6.4.6. Discussion of 1974 Results

The Wagga Murrumbidgee River model matched 1974 observations successfully (see Section 6.4) by adjusting infrastructure to 1974 conditions and by modifying vegetation as per 1971 aerial photography (as per Section 3.3). The results show that the model accurately reproduces peak flood levels and total flows. Comparison of the rating curve to the 2004 study rating curve further indicates the robustness of the model.

This has implications for the change in stage/discharge relationship mentioned in Section 3 and tends to confirm the validity of the significant change in stage/discharge relationship at the Hampden Bridge gauge (see Section 3 for further details).

6.5. High Flow Stage/Discharge Relationships

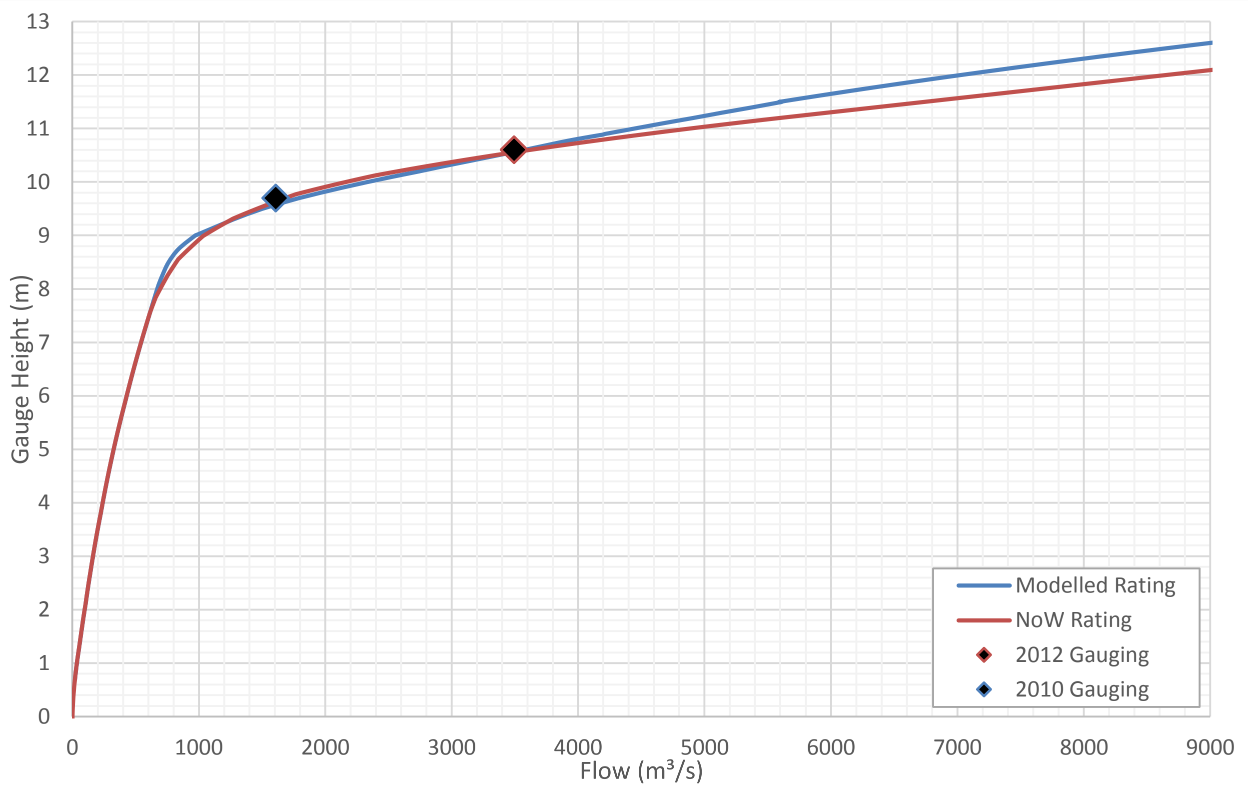
A topic of interest for Council as they investigate levee re-design is the stage-discharge relationship for flows exceeding those gauged by NOW. NOW currently extrapolate the rating curve to a stage of approximately 11 m (181 mAHD), significantly higher than the maximum gauged level.

At Wagga Wagga the use of the NOW rating (above gauged flows) is less desirable than it might be for other locations for two main reasons, these are:

1. Wagga has severe flood consequences for flow heights above the maximum gauged height (March 2012), as the design height of the levee, as it stands, is in the order of 10.8 m at Hampden Gauge (0.2 m higher than March 2012 event). As such high levels of confidence in the rating are required; and
2. Wagga has manmade structures on the floodplain which are intended to impact on flooding extent. These structures will complicate the high flow rating versus a natural system.

For this reason WMAwater has defined the high flow rating up to 12 m at the Hampden Bridge Gauge using the model established during this project. The Stage/Discharge relationship is displayed in Chart 6 along with the revised NoW rating.

Chart 6: Model Derived Rating and NoW Rating for Current 2012 Conditions



Note: The above derived rating has been determined from numerous runs at varying flows to avoid skewing of the rating curve by the effects of hysteresis. The relationship is based on current conditions and will change with levee augmentation.

6.6. Design Results

Figure 18 and Figure 19 display the 1% and 5% AEP design flood results for the Study Area. These figures display the design flood depths and flood level contours for the region.

It should be noted that inundation patterns and/or peak flood levels shown for these design events are based on best available estimates of flood behaviour within the catchment. Inundation from local creek and local overland flow have not been modelled and as such flood extents and depths may vary depending on the actual rainfall event, relative timing of flows and local influences.

The 1% AEP level at the Hampden Bridge gauge is 181.36 mAHD with a flow of 5,100 m^3/s. This correlates to a gauge height of 11.31 m. It should be noted that even with the significant reduction in 1% AEP flows compared to the 2004 Study (6,900 m^3/s) the 1% AEP peak flood level at the Hampden Bridge gauge is only 0.05 m lower than the 2004 estimate (11.36 m). This is due to changes in floodplain roughness and the effect on conveyance discussed in Section 3.

The peak flood level profiles for the Study Area (following the Murrumbidgee River centre line) are displayed in Figure 20. This includes levels for both design floods (1% and 5% AEP events) and the 2012, 2010 and 1974 events.

Examining Figure 20, it is interesting to note the variation between the 1974 and 2012 flood profiles. Towards the Malebo Gap the 1974 flood is noticeably higher than the 2012 event. This is due to the 1974 flood being larger and more voluminous causing greater backwatering at the Malebo Gap which is a high flow restriction. Other regions have more localised effects for example at Gumly Gumly where the influence of changes in vegetation (see Section 3.3) and construction of quarries on the riverbank since the 1974 flood caused the 2012 flood to be higher in this region (a finding that was independently confirmed during field work performed as part of the Reference 4 study). Further upstream at Oura the 1974 flood was markedly higher that the 2012 flood due to the larger flow and the constrained floodplain. Calibration of the 1974 and 2012 models to observed flood marks confirm these findings.

The Hampden Bridge stage exceedance probabilities for both the 2012 and 1974 ratings are displayed in Figure 21.

6.6.1. Design Result Comparison to the WMAwater 2010 Study

The 1% AEP design results for the current study were compared to the design results of the Reference 2 study (assuming no levee failure). Figure 22 displays the difference in peak flood level between the 2010 Study and the current study. A negative difference indicates that the current study level is lower than the 2010 study.

Some differences were apparent with the current study generally producing slightly lower flood levels than the 2010 study. A maximum difference in peak flood level between the two models was approximately 0.3 m with the current model producing levels 0.3 m to 0.05 m lower along the levee alignment than the 2010 model results. Significant changes to model roughness, bathymetry and 1% AEP flow are responsible for these differences.

6.7. Proposed Levee Alignments

The design flood heights for the Main City and North Wagga levees are displayed in Figure 23 and Figure 24. These levels indicate the peak flood level for the specified design flood along the levee alignment. The sensitivity results are also displayed for the Main City Levee with a further discussion of this contained in Section 6.8.

Note that the North Wagga Levee is a ring levee and the displayed peak flood level at the start and end of each of the alignments are the same points.

6.8. 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. Figure 23 displays a comparison of peak flood profiles along the proposed Main City Levee alignment for the various sensitivity runs.

Roughness values presented in Table 5 were increased and decreased by 10%. An increase in roughness led to a maximum increase in peak flood level of 0.18 m along the length of the levee, although on average the increase in peak flood level was less than 0.15 m. A decrease in roughness was found to reduce peak flood levels by a similar magnitude with an average decrease of 0.15 m experienced.

In addition the models sensitivity was tested by increasing and decreasing the 1% AEP input flow (see Section 2.4.1) by 10%. With a 10% increase in flow an average increase in peak flood level of 0.21 m was experienced with a maximum increase of 0.24 m in the vicinity of the levee. A decrease of 10% flow created on average a decrease of 0.22 m in peak flood level.

Model results were shown to be insensitive to grid size when comparing the current study 20 m grid to a 15 m grid. Impacts of less than 0.05 m were noted with the majority of regions experiencing only 0.02 m difference.

It should be noted that the model was most sensitive to the above mentioned variables in regions upstream of the major flow constrictions such as Gobbagombalin Bridge, Malebo Gap and Oura.

6.9. Estimated Model Accuracy

The Reference 1 study provides an estimate of the order of accuracy for design flood levels of ±0.5 m. This accuracy has been improved upon for the current study by utilisation of additional calibration events and data, as well as modern engineering techniques. Sensitivity analysis results indicate that the order of accuracy of design peak flood levels for the current study is ±0.25 m. This should be taken into account when determining freeboard within the Study Area.

6.10. Impacts of proposed levee Upgrades

Impacts associated with upgrading the Main City and North Wagga levees have been assessed. Typically where works are carried out no flood impact (typically defined as anything less than or equal to 0.01 m) on adjoining properties is the goal.

Results indicate impacts of up to 0.15 m do occur. Widespread impacts are present on the northern floodplain (related to Main City levee and North Wagga levee for 5% AEP runs).

The following scenarios have been considered when assessing impacts associated with the Main City and North Wagga levee alignments:

1. 1% AEP event - Main City Levee raised relative to current conditions;
2. 5% AEP event – North Wagga Levee raised relative to current conditions;
3. 1% AEP event – Main City Levee and North Wagga Levee implemented;

Further details are contained in the following sections.

6.10.1. 1% AEP event - Main City Levee Impact – (1)

Figure 25 displays impacts associated with the Main City levee being raised for the 1% AEP event. Chief impacts are as follows:

- Impacts of up to 0.03 m as far upstream as Gumly Gumly and also in the downstream due to lost floodplain storage;
- Peak impacts are approximately 0.15 m in the region south of North Wagga, few properties are situated in this region; and
- Entire area north of Hammond Avenue (in upstream) and northern floodplain from Eunony Road to Gobbagombalin Bridge has impacts of up to 0.1 m – again few properties within impacted area despite its size.

Impacts in the region of the Wagga Main City Levee are generally less than 0.15 m. The majority of the impacted region is sparsely populated and the relatively small impacts in relation to the freeboard (0.5 m) will minimise any additional over floor flooding. In contrast to impacts there are substantial benefits associated with the levees being upgraded (see for example grey areas behind Main City Levee which indicate areas that are no longer flooded). Overall implementation of the new levee alignment will largely improve flood mitigation in the Wagga region.

6.10.2. 5% AEP event – North Wagga Levee Impact – (2)

Figure 26 displays the impacts associated with North Wagga levee being raised. Note impacts shown are for the 5% AEP event. As can be seen impacts are far less substantial than for the Main City Levee upgrade (in context of 1% AEP event). Maximum impact is up to 0.1 m (for a relatively small area) to east of North Wagga.

6.10.3. 1% AEP event – Main City and North Wagga Levee Impact – (3)

The 1% AEP results for when both levees are implemented do not vary significantly to when only the Main City Levee is implemented. For a full explanation of results refer to Section 6.10.1.

7. ACKNOWLEDGEMENTS

WMAwater would like to acknowledge the assistance of Wagga Wagga City Council staff in carrying out this work as well as The Office of Environment and Heritage. WMAwater would also like to acknowledge the OEH and SES for managing and financing the work in References 4 and 5 which was vital for model calibration/validation.

Wagga City 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

FIGURE 1
STUDY AREA

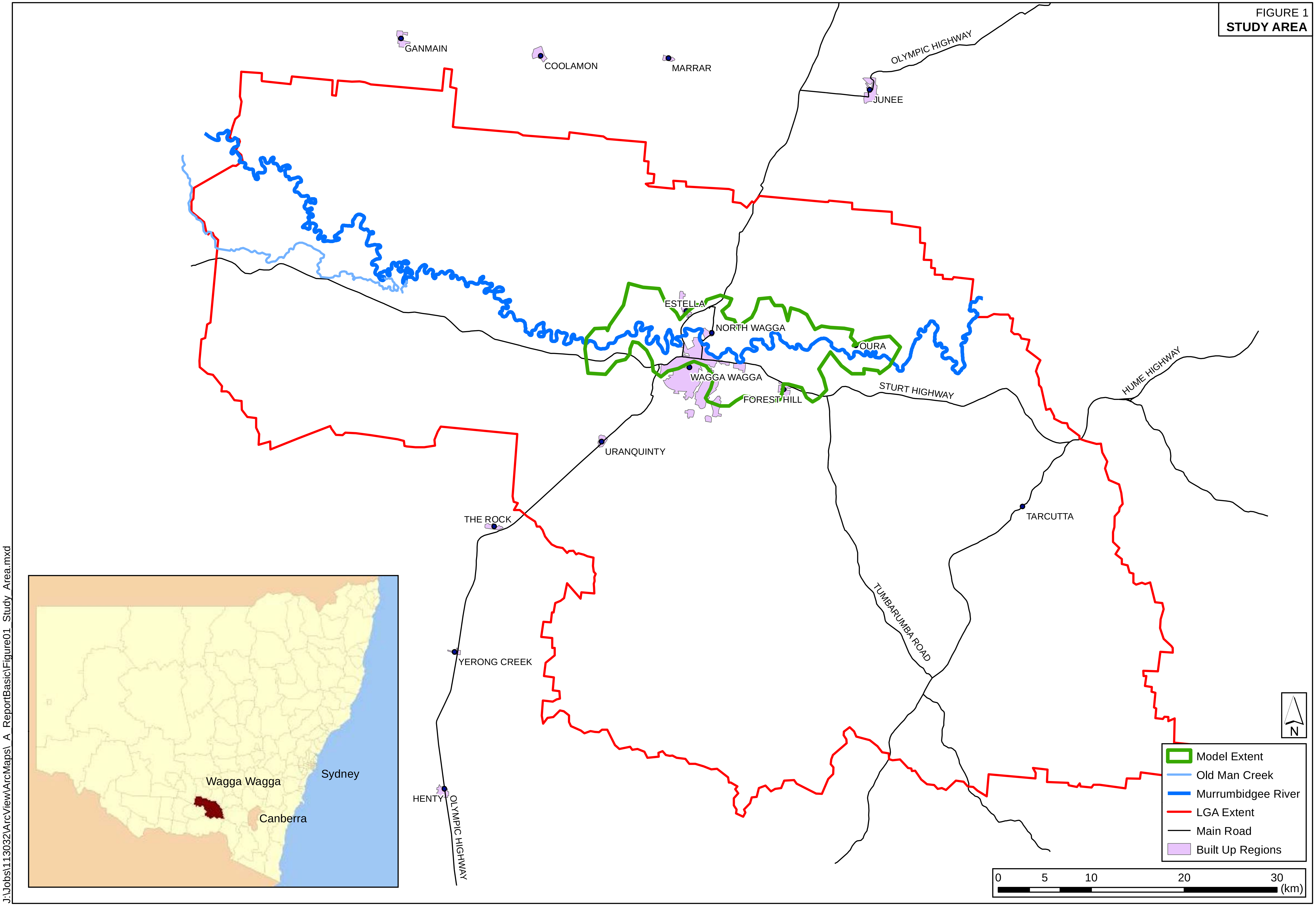

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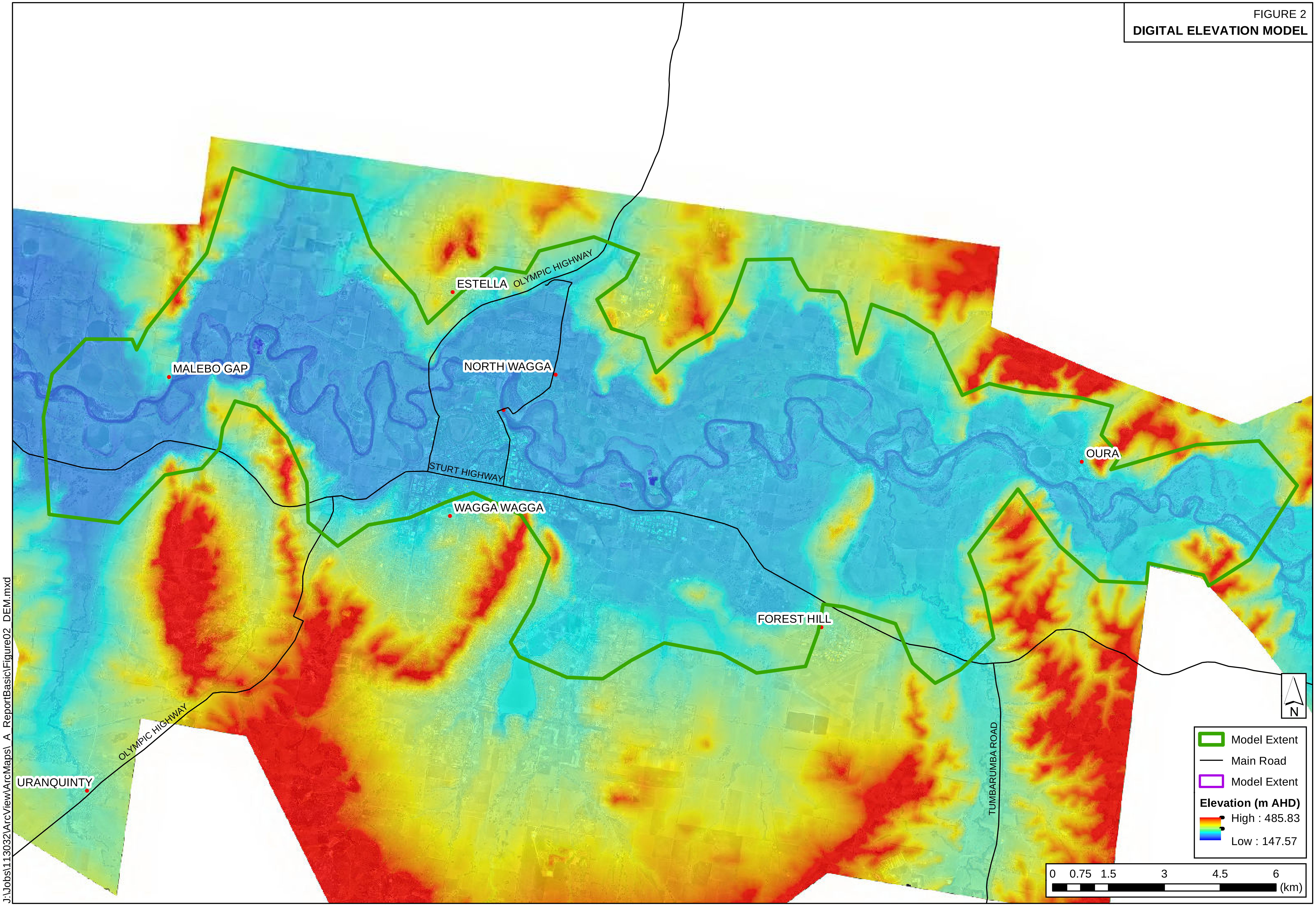
FIGURE 2
DIGITAL ELEVATION MODEL



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