WAGGA WAGGA CITY COUNCIL

WAGGA WAGGA MAJOR OVERLAND FLOW FLOOD STUDY

FINAL REPORT

AUGUST 2011

Level 2, 160 Clarence Street
Sydney, NSW, 2000

Tel: 9299 2855
Fax: 9262 6208
Email: wma@wmawater.com.au
Web: www.wmawater.com.au

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Wagga Wagga City Council</td><td>Client's Representative
Bryan Short</td></tr>
<tr><td colspan="2">Authors
Stephen Gray/Bowen Hicks</td><td>Prepared by</td></tr>
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FOREWORD

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

Under the Policy, the management of flood liable land remains the responsibility of local government. The State Government subsidises flood mitigation works to alleviate existing problems and provides specialist technical advice to assist Councils in the discharge of their floodplain management responsibilities.

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

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

The Wagga Wagga Major Overland Flow Flood Study (the Study) constitutes the first stage of the management process for the Wagga Wagga Local Government Area (LGA). WMAwater (formerly Webb McKeown and Associates) has been engaged by Wagga Wagga City Council (Council) to prepare the Study. The following report documents the work undertaken with respect to data collection, model build, community consultation, model verification and design and sensitivity runs as per WMAwater and Council's Brief.

Please note that use of the AEP notation to describe design event probability is preferred however for smaller flood events (1Y to 5Y ARI), in order to present whole number values, the ARI notation is used.

EXECUTIVE SUMMARY

The main recognised mechanism for flooding in Wagga Wagga is the Murrumbidgee River. Flooding can also be caused by local rainfall however and, as recent events have emphasised, numerous areas of Wagga Wagga, including commercial and residential areas, are liable to flooding following intense rainfall.

The Wagga Wagga Major Overland Flow Flood Study (the Study) seeks to define existing case design flood behaviour for major overland flow branches throughout the study area as defined by Map 3.1 from the Brief (and recreated herein as Figure 2).

Defining the existing flood risk is critical, particularly as this will aid future work that aims to ensure ongoing development in Wagga does not lead to exacerbation of the current level of flood risk.

Community Consultation – Observations of Historical Flooding

The main purpose of community consultation carried out during the course of the Study was to collect observations of flood behaviour that could be used to calibrate and validate the model prior to it being used in design flood estimation. Community consultation, carried out between January and February of 2010, revealed that except for very specific areas residents had very little experience of flooding. Analysis of available rainfall records (at the time) also indicated that few substantial rainfall events had occurred in recent times (previous 15 years). Although not always recorded by rainfall gauges, anecdotal information from Council indicated that thunderstorm driven intense rainfall events have occurred and caused isolated inundation in some areas such as the Glenfield Drain area. For example on February 5th 2010 a highly localised storm event occurred impacting the lower Glenfield Drain catchment, particularly from Fernleigh Road downstream to the Bunning’s site (north of the intersection of Dobney Avenue and Pearson Street). During and after this event Council officers took numerous photographs of the flooding behaviour and as such the February 5th 2010 event came to comprise the best available verification information at the time. Note that the event did not result in flood marks suitable for post flood survey.

Subsequent to the community consultation process Wagga Wagga experienced severe local flooding as a result of rainfall received between 13th and 18th October 2010. Furthermore, two additional significant events occurred during December 2010 (2nd / 3rd and 8th / 9th).

During the event of October 2010 high intensity rainfall occurred south east of Wagga in the Kyeamba and Tarcutta Creek catchments. Whilst the Wagga CBD was on the edge of the event approximately 120 mm of rainfall was recorded over a 24 hour duration at Forest Hill. Whilst not a severe event for Wagga’s CBD the 24 hour rainfall total indicated an event of a 5Y ARI and so was significant. Subsequent review of available rainfall records indicated that no higher resolution pluviograph data was available to describe the October event. This in conjunction with the lack of observed flood level data meant that the use of this event for calibration/validation was not pursued.

The two events that occurred in December of 2010 (referred to herein as the events of December 2nd and 9th) coincided with Murrumbidgee River flooding (the River peaked[1] on December 6th but was relatively high for both events, more so for the event of the 9th). Extreme local rainfalls, particularly in the case of the event of the 2nd (anecdotal observations near Lake Albert indicate a point rainfall at least of approximately 85 mm over three hours which approximates the 1% AEP event) combined with an inability for flood waters to freely drain into the River corridor led to severe local inundation. The event of December 2nd affected the CBD area severely with the Art Gallery and Library adjacent to Wollundry Lagoon experiencing over floor flooding. The event of the 9th was more prominent in the Moorong St area although areas up and downstream of Lake Albert were also flooded as were industrial areas to the east of Wagga on the Sturt Highway. Whilst few rainfall and flood observations were available for the event of December 2nd extensive rainfall and flood observations were available for the event of December 9th. As such, following submission of the Draft Report and Public Exhibition of the same, the Floodplain Management Committee (FMC) requested that WMAwater carry out a variation to the overall Study, using the December 9th event as a validation event to confirm, or at least enhance confidence in, the accuracy of the model for design flood estimates. This work has been carried out and is reported separately in Appendix D.

Model Build

The overall study area has been broken into four model domains as follows (see Figure 2):

- CITY – Glenfield Drain, Silvalite Reserve, various CBD bound flow paths;
- EAST – Marshalls and Crooked Creeks;
- LAKE ALBERT – Stringybark Creek etc; and
- NORTH – Duke’s Creek.

Internal routing in the model is carried out via the hydraulic model rather than the hydrologic model and this provides extensive amounts of detail on flooding behaviour within the study area. The 5 m grid used compliments this approach in that it allows for reasonable resolution of overland flow paths such as roads and easements.

Model Verification

Besides confirming that the model was able to replicate observed behaviour from the February 5th 2010 event the following checks have been carried out to confirm the suitability of the modelling system for use in estimating design flood behaviour:

- Comparison of WBNM flows generated as part of the study with comparable Probabilistic Rational Method (PRM) estimates;
- Comparison of model results at Glenfield Drain with results from the 2006 Glenfield Park Drainage Study (Reference 4);
- Analysis of model results at defined hotspot locations and critical comparison with expectations based on Council experience;

[1] The December 2010 flood event was an approximately 15Y ARI event at Wagga Wagga with a stage height of 9.67 m at Hampden Bridge gauge.

- Extensive internal review of results and fitness for purpose by the project manager from WMAwater; and
- Sensitivity testing.

The results of the February 5th 2010 verification and the comparison of model results with expectations of flooding behaviour, based on the accumulated Council knowledge inherent in the list of hotspots provided in the Brief, indicate that the model is performing well.

As noted above and reported upon in Appendix D, following Public Exhibition of the Draft Final report it was decided that the localised flooding which had occurred in December presented an opportunity to further check the models ability to emulate Wagga Wagga localised flood behaviour. Model performance was assessed against the December 9th event for which extensive rainfall and flood extent behaviour, at or near the peak, was available. Three of the four model domains (all except North) were run and found to emulate all observed behaviour. The validation exercise confirmed the models suitability for design flood estimation. A full report on the December 2010 validation event can be found in Appendix D.

Model Sensitivity

The 1% AEP event has been used to test model sensitivity in the City model domain. Sensitivity runs undertaken include the following:

- Change in Manning's *n* (+/- 20%);
- Climate change (increase in rainfall intensity of 7%);
- Blockage of 25% for all pipes and culverts;
- Blockage of 50% for all pipes and culverts;
- Relatively high and low rainfall loss values of (initial/ continuing) 10 mm/ 1 mm/h and 25 mm/ 3 mm/h;
- A higher tail water level at the Murrumbidgee River (5Y ARI in River); and
- Total blockage of all pipes that discharge to the Murrumbidgee River (i.e. discharge of local runoff limited to pumps only).

Results overall demonstrate that model results are relatively insensitive to parameter changes with the exception of locations in the downstream areas which are significantly affected by Murrumbidgee River water levels. Additionally, selection of hydrological losses used impact on the peak flood levels in lower areas where volume dictates peak flood levels as opposed to conveyance.

Design Flood Results

Design flood results indicate extensive inundation of private and public property in the event of a 1% AEP event. In the main however it seems that reasonable planning controls have limited the number of households/commercial operations that are likely to experience over floor flooding. Quantification of over floor flooding will have to wait for floor level survey work to be carried out however at this point it seems likely that the households/businesses most flood liable are to be found in:

- Dobney Avenue downstream of Glenfield Drain;

- Houses between Rowe Street and Bocquet Street are threatened by spill from the Lake Albert diversion;
- Stringybark Creek threatens homes at the end of Yarran Place, Hakea Place and on the Northern edge and end of Mallee Road;
- Berry Street just north of Morgan St at the southern edge of the CBD is as ever, flood prone;
- The area between Morgan Street, Thorne Street, Tompson Street and Murray Street which is just south of the Wollundry Lagoon (at its western end) appears to collect a lot of flow (note this area includes Forsyth Street);
- Spring Street off Moorong Street immediately upstream of Flowerdale Lagoon seems flood liable also;
- Homes near the corner of Urana and Macleay Streets including on Heydon Avenue are impacted by flooding, with some residences liable to over floor flooding based on anecdotal information alone; and
- Properties downstream of Brunskill Road, particularly along Sycamore Drain (and Sycamore Road) are flood liable and some of these may even be subject to floodway type flows with high velocities in larger flood events.

Note that in all cases mentioned above it is likely that the height of the floor level above the ground may mean that many houses/businesses remain free from over floor level inundation.

Hazard and Flood Risk in Wagga Wagga

Generally speaking flooding flow, where it interacts with buildings, is low hazard flow. High hazard areas tend to be limited to main channels and also retarding basins and this is mainly due to the depth criteria in the hazard calculation.

Generally the flood risk can be rated as low with one of the main forms of risk likely to be road crossings that become inundated relatively quickly following rainfall.

It’s interesting to note that if in future the 1% + 0.5 m criteria is applied to residential floor levels as it should be then in some areas of Wagga, floor levels will be substantially higher than levels used in the past. For example a number of homes in the area upstream of Plumpton Road currently are slab on ground with floor levels approximately 200 mm higher than ground levels. Future homes in such areas will need to be in the order of 0.5 m higher.

Climate Change

The impact of climate change has been assessed via a 7% increase in rainfall as per State Government guidelines (Reference 7). Generally the impact of the increase in rainfall is greater in those areas where flow is constrained, such as Glenfield Drain, with little impact on wider levels throughout the City domain. For example the climate change run produces a flood depth 0.2 m greater at the retarding basins upstream of the railway line on Glenfield Drain. Overall the climate change impact as assessed is small and relative to standard provisions for freeboard (Reference 2) is negligible.

Recommendations

Recommendations are as follows:

- Proceed to an Interim Report in order to better inform the writing of a brief for the subsequent Floodplain Risk Management Study and Plan. Such an Interim Report will provide more interpretation of design flood results and will specifically identify those areas requiring further investigation, particularly with regard to mitigation, as part of the Management Study;
- Enhance flow capacity into Lake Albert for those flows on the western side of Plumpton Road. This will ease the danger of flooding for those houses located downstream of the Plumpton Road crossing within the historical Stringybark Creek flow path. It is noteworthy that the capacity of the structure on Plumpton Road is considerably less than the capacity of the structure upstream of it on Springvale Drive;
- An improved design for the Crooked Creek diversion mechanism into Lake Albert on the eastern side such that flows in excess of the design capacity of the diverting levee are controlled and accounted for. Formalisation of the overflow mechanism and flow path downstream of the levee should protect the diversion structure as well as give better flooding outcomes to the residents downstream of the diversion;
- The possibility of failure of the Crooked Creek diversion levee during a large event should be examined to determine the consequence of such a failure. It may be that a much lower diverting structure is better suited to the location as this will divert flow to Lake Albert as well as prevent a build up of water which, if allowed to flow north without control, could put residents lives and homes at risk;
- Obtain a floor level survey for all houses and commercial buildings within the PMF flood extent (excluding all depths less than 150 mm);
- Investigate flooding behaviour in the vicinity of Brunskill and Sycamore Roads as part of more detailed stormwater work as currently a number of properties in this area are exposed to flood risk for events as small as the 10% AEP;
- Generally implement a program of drainage maintenance such that high priority systems (Glenfield Drain for example) are maintained to a high standard with regard to vegetation blockage etc;
- As part of more localised stormwater studies address issues at the corner of Urana and Macleay Streets as the currently flooding frequency is high and this is impacting on residents;
- Develop response plans for localised flooding scenarios that occur in conjunction with elevated river levels, mainly around mobile pumping operations, taking into account the fact that substantial quantities of power required may not always be available from the grid; and
- Ensure that all Development Applications are assessed using results from this study (where applicable), particularly noting the standard requirement that the residential floor level be set at the 1% AEP level plus 0.5 m. In areas close to the Murrumbidgee River levee it may be that design planning levels will be set by Riverine flooding scenarios and as such its recommended that Council compile a merged flood planning level layer to inform development applications within the LGA.

1. INTRODUCTION

Wagga Wagga (Wagga) is located at the eastern end of the Riverina weather district in NSW (see Figure 1). The principle flooding mechanism focussed on in Wagga in the past has been flooding due to the Murrumbidgee River. As studies have been carried out to address this issue Wagga Wagga City Council (Council) is now focussing on defining the flood liability within Wagga due to local flooding.

The main objectives of the Study are to:

- Define the overland flow flood behaviour within Wagga Wagga for the area defined in Figure 2 (as per defined extent from Map 3.1 of the Brief); and
- Provide a suitable basis for a subsequent Floodplain Risk Management Study and Plan that may occur as part of the State Government's flood planning process. This includes not just describing the design flood behaviour with respect to flood levels, provisional flood hazard and preliminary hydraulic categories but also providing modelling systems which will be suitable for use in examining mitigation works during the next stage of the planning process which is the Floodplain Risk Management Study (FRMS).

The study seeks to establish suitable hydrologic and hydraulic model tools, demonstrate their capacity to emulate local flood behaviour and then apply these tools to establish the existing flood risk for a range of design flood event probabilities in conjunction with a range of event durations. The range of events to be modelled ranges from the 1Y ARI to the Probable Maximum Flood (PMF) whilst durations examined range from the 15 minute to the 72 hour.

The key elements reported upon herein include:

- A summary of available data;
- Details and results regarding the community consultation work which has been undertaken;
- A description of the modelling work done (hydrologic and hydraulic) including assumptions, parameters and details of the methodology; and
- Details/results of work done to date to verify the accuracy of the models performance and the suitability of the modelling methodology used in general;
- Details/results of sensitivity testing undertaken;
- Design flood results including figures showing, for each of the four model domains, the 1% AEP flood extent with flood level (mAHD) contours as well as provisional hazard/hydraulic classification maps; and
- Profiles which describe how the peak flood water surface changes as we progress from high in the study area to downstream. Most notably the profiles give us easily utilised information on transport corridors impacted by flooding.

A glossary of flood related terms is provided in Appendix A.

2. BACKGROUND

2.1. Study Area

The Wagga Wagga Major Overland Flow Study incorporates catchments with an area of 233 km^2 and a hydraulic modelling extent of 200 km^2 both south and north of the Murrumbidgee River as shown in Figure 1 and Figure 2. The overall study area has, for modelling purposes, been broken up into four distinct areas and these are indicated on Figure 2. These four areas can briefly be described as:

- **North**. This domain covers Duke's Creek from its headwaters to its confluence with the Murrumbidgee River (Gobbagombalin Lagoon) and includes an area of 38.3 km^2. Note the area modelled does not include the suburb of North Wagga;
- **Lake Albert**. This area includes, as the name suggests, the upstream catchment of Lake Albert. As Figure 2 indicates it also includes outflow from Lake Albert as well as Stringybark Creek that runs to the south and west of Lake Albert. Flow from Stringybark Creek is diverted to Lake Albert for all events less than approximately the 10% AEP event. Crooked Creek, which runs from the south and to the east of Lake Albert is likewise diverted into Lake Albert and like Stringybark Creek only larger events will lead to significant flow continuing north at the diversion point. The total area included in the Lake Albert model domain is 66.9 km^2;
- **City**. The "City" model covers Glenfield Drain as well as the Wagga Wagga CBD and outer areas lying on the southern Murrumbidgee River floodplain. The total area in the model is 39.9 km^2; and finally
- **East**. The East model covers an area of 48.4 km^2 and most significantly includes Marshall's Creek.

Note the hydraulic model areas have been delineated with the following characteristics in mind:

- As far as possible discrete areas (from both a hydrological and hydraulic perspective) are within separate model domains so as to avoid cross flow issues;
- That model sizes should not become unreasonably large (with respect to the number of grid elements); and
- Taking into account likely inundated area (both upstream and downstream) with a view to ensuring that there will be no model edge affects on model results within the study area.

Despite these criteria it was necessary to split two hydraulically connected catchments and these are Lake Albert and East.

Council is aware of known drainage issues within the study area and compiled a list of drainage hotspots to accompany the Brief. The regularity of inundation at some of these locations is approximately annual. These areas will be discussed further in later sections but include Glenfield Drain, particularly upstream of the railway embankment as far as Fernleigh Road and downstream to the intersection of Dobney Avenue and Pearson Street.

2.2. Previous Studies

Two studies have been identified which are of relevance to the current study. These studies have been largely superseded by more recent modelling though still offer important insight into flood behaviour and problem areas. Both reports are summarised below.

Glenfield Park Drainage Study (Webb McKeown and Associates, 2006) – Reference 4

The study assessed Glenfield Drain's design capacity following various upstream residential developments. An important finding from the study was that the approximate capacity of the Glenfield Drainage system was the 20Y ARI event (5% AEP event).

The downstream limit of the study was the Sturt Highway (immediately upstream of Flowerdale Lagoon) and the catchment area incorporated into the modelling totalled 15.6 km^2. Note that following the Red Hill Road extension in 2009 the catchment of Glenfield Park has been reduced to 14.5 km^2.

The study was carried out as a one-dimensional study, with this approach being necessary because no broad acre survey data was available at the time of the study. A characteristic of such an approach is that storage and attenuation present in the natural system is likely to be underestimated and as such times to peak will likely be underestimated and peak flow magnitudes overestimated (as will water levels also be overestimated). This issue will be further discussed when the flows are compared to results from the current Study in ensuing sections. (See Section 5 for results and Section 6 for discussions).

The 2006 study confirmed the need for a series of retardation basins with the intent of the retardation strategy by Council being to reduce peak flow at Dobney Street to 23 m^3/s, which approximates the capacity of the open channel and particularly the major controlling structure upstream, i.e. the railway embankment/culvert.

Although it was Councils intent that developed flows should be reduced to 23 m^3/s for the 1% AEP event (i.e. in-bank) the study found that in reality for any event larger than the 20% AEP event flooding of the industrial area immediately downstream of the railway line was likely.

Peak flows from the 2006 study are presented in the table below and these were used in the verification sections of this report which demonstrate that the current modelling is fit for purpose.

Table 1: Peak flow results from the 2006 Glenfield Drain Study (Reference 4)

Location	**2-yr ARI (m^3/s)**		**5-yr ARI (m^3/s)**		**10% AEP (m^3/s)**	
	Existing	Developed	Existing	Developed	Existing	Developed
Red Hill Road	2.0	7.9	4.8	11.3	7.3	14.5
Dalman Parkway	2.6	8.1	5.0	11.7	7.6	15.0
Fernleigh Road	15.8*	18.7*	20.9*	25.1*	24.5*	29.8*
Main Southern Railway	10.5*	13.1*	14.0*	20.2*	18.9*	25.7*
Dobney Avenue	11.3	13.6	14.6	20.8	19.6	25.8
Sturt Highway	12.9	14.6	16.5	22.1	21.0	25.8
Location	**5% AEP (m^3/s)**		**1% AEP (m^3/s)**		**PMF (m^3/s)**	
	Existing	Developed	Existing	Developed	Existing	Developed
Red Hill Road	11.0	19.9	21.7	32.6	360	385
Dalman Parkway	11.4	20.5	22.4	33.6	390	410
Fernleigh Road	30.7*	39.0*	46.2*	61.9*	750*	780*
Main Southern Railway	26.5*	37.7*	44.6*	53.9*	400*	400*
Dobney Avenue	27.4	37.7	44.8	54.1	410	410
Sturt Highway	29.1	37.9	43.4	48.1	415	420

*Between Fernleigh Road and the Main Southern Railway line little additional catchment is picked up whilst some flow is lost as flow splits out and moves away from Glenfield Drain. Also peak flow values are attenuated by the detention ponds and general storage upstream of the Railway line. For this reason peak flow decreases between these two locations which is the reverse of what may normally be expected.

Review of Stormwater Trunk Drainage – Wagga Wagga City Centre (Boyden and Partners, 1998) – Reference 5

This report specifically focuses on the catchment in Wagga Wagga CBD in which the Council chambers are located. It continues on from a 1995 study looking at design flood levels for a proposed Woolworths on Berry Street.

The report presents flood levels and start to overtop flow levels that have been calculated/surveyed at a variety of key locations such as Wollundry Lagoon and Tony Ireland Reserve. Generally the study is at the level of detailed drainage, focussing mainly on pipes, with likely much coarser representation of overland flow paths. The report makes the following statements:

- Flooding in Berry Street occurred in 1994 and the Woolworths car park was inundated to a depth of 0.8 m;
- The catchment area, extending back south as far as Willans Hill, is 567 ha;
- Wollundry Lagoon surface area is 6 ha (at normal operating level);
- Wollundry Lagoon drains to Tony Ireland Reserve via a 1200 mm diameter pipe;
- Wollundry Lagoon breaches between the Civic Theatre and the Civic Centre at a level of 179.6 mAHD and stores ~ 170,000 m^3 of water prior to overflow;
- A gate controls the water level in the lagoon and when open the water level is controlled at 176.76 mAHD [Council advise this has been subsequently replaced with a 10 m sill

length pit at a height of 177.07 mAHD]. If closed water level is 176.99 mAHD;

- A 1500 mm diameter pipe runs under Bolton Park and into Bardo Lane (this pipe is severely under capacity). When the capacity of this pipe is exceeded water will begin to accumulate on Bolton Park and at the lowest part of the city centre, i.e. at the southern end of Berry Street. Surcharging waters will use Berry Street as an overland flow path and make their way to Tony Ireland Reserve [present study, using a more detailed approach does not support this conclusion];
- Bardo Lane pipeline drains to Tony Ireland reserve along with the 1200 mm Wollundry Lagoon pipeline as noted above. Tony Ireland Reserve has a storage capacity of 8,800 m^3 only prior to overflowing into surrounding streets at an invert level of 179.3 mAHD;
- A 1350 mm diameter pipe connects Tony Ireland Reserve to the Murrumbidgee River and it is stated that model results indicate this is a limiting factor (upgrades have occurred since this point in time);
- The report states that a number of flooding mechanisms were investigated to find out what's critical for flooding in the area. A preliminary estimate finds that the Bolton Park pipeline has a capacity of 2.6 m^3/s. However at its weakest section (immediately downstream of Bolton Park) the actual capacity is found to be 0.6 m^3/s and this value shows no sensitivity to levels at Tony Ireland Reserve (so it's a slope base limitation presumably);
- Losses used are 10mm (initial pervious) and 2.5 mm/h (continuing pervious) and 1.5 mm (initial impervious) and 0 (continuing impervious);
- Levels in Tony Ireland Reserve for 5, 20 and 100Y ARI events are respectively, 179.71, 179.80 and 179.91 mAHD [modelling undertaken as part of the present study does not find these levels to be accurate, however current modelling includes additional drainage capacity implemented in Tony Ireland Reserve];
- Pump capacity at Tony Ireland Reserve is 0.5 m^3/s; and
- Report recommendations were to:
    - 1. Increase the amount of water held at Bolton Park and
    - 2. Improve the discharge from Tony Ireland Reserve to the Murrumbidgee River [both of these suggestions have been implemented by Council and modelling undertaken for the current study suggests they have achieved the desired affect, i.e. maximised capacity in the Bardo Lane pipeline for local flows to drain into it by utilising storage at Bolton Park and ensuring low tail water levels at Tony Ireland Reserve].

3. AVAILABLE DATA

3.1. Introduction

Numerous forms of data are required in order to carry out a Flood Study. Data is required to describe the topography, land use, drainage infrastructure etc. This section discusses the data collected and presents some analysis of that data. Note that the data collection phase of the study was completed by the end of 2009, with community consultation finalised prior to February 2010. For this reason recent rainfall events are not included in collected data but more recent storms in December 2010 have been addressed in Appendix D.

3.2. Hydrological Data

Hydrological data, for the purpose of this Study, includes rainfall and stream flow data. Given that no stream gauges exist in the study area this section will be limited to a discussion of the available rainfall data.

3.2.1. Rainfall

3.2.2. Historical Rainfall Data

The main purpose of examining historical rainfall records is to find calibration data. A secondary use in this study was to develop IFD relationships for Wagga utilising local rainfall records (and records which extend well past the data utilised to develop the official IFD data for Wagga (as presented in Reference 1).

Figure 3 shows the location of rain gauges (daily and pluviometer) which are proximate to or within the Study area. The stations are also summarised in the table below and note that this table also indicates the duration of record held at each of these stations.

Table 2: Local Rainfall Stations

Pluvio	**Station Description**	**Record From**	**Record Until**	**Closed**
72150	WAGGA WAGGA BOM	Jan-45	current	
74114	SOIL CONSERVATION SERVICE RESEARCH CENTRE	1948	Jan – 2004	Feb - 2004
*RFS**	ASHMONT RFS	Feb 05 event only	-	
Daily				
74234	EUBERTA (EAST VIEW)	Jul-1972	current	
73127	WAGGA WAGGA AGRICULTURAL INSTITUTE	Nov – 1912	current	
74127	WAGGA WAGGA (GURWOOD STREET)	Jun – 2001	current	
74241	WAGGA WAGGA RSL	Aug – 1978	current	
72040	WAGGA WAGGA (BERRILLEE)	Jan-1933	Jan – 2009	

*not an official BOM gauge.

Data was collected from these stations and then analysed in order to identify any large historical events which should be targeted during the community consultation campaign. The results of the analysis were not favourable in that no recent significant events were found. A significant event would be one likely to cause widespread inundation throughout Wagga and this would be well remembered and observed by local residents.

As shown in Figure 4, the second most recent significant gauged rainfall event was the February 5th 2010 storm which was a 20Y ARI storm for the relatively short duration of 20 minutes. Other large rainfall events are recorded such as the 04/10/1974, 08/02/1974 and 11/12/1975 which were approximately 5%, 7% and 10% AEP events respectively. These rarer events occurred more than 30 years ago and older events are less likely to be recalled during the community consultation process.

Note that subsequent to the public exhibition of the Draft Final version of this document further significant events occurred in October and December of 2010. The December events are discussed in Appendix D. During the October event 118.6 mm of rainfall was recorded at Forest Hill (approximately 10 kilometres east of Wagga Wagga on the Sturt Highway) between the 13th and 17th. A burst within the event had an ARI of approximately 5Y and with substantial pre-wetting prior to the main burst resulted in substantial flooding in some areas (Crooked Creek for example).

3.2.3. Design Rainfall Data

As part of the Brief WMAWater were requested to develop IFD data for Wagga based on available rainfall records and then make a comparison of these to Reference 1 derived IFD data for Wagga. This has been carried out and the results are shown below.

In summary most of the results are quite similar however there is a 10% difference for the 2% AEP 1 h duration intensity, with the Reference 1 values giving the higher intensities. For smaller events, the design flood results achieved will be approximately the same regardless of which IFD set is used. However it is likely that the Reference 1 IFD values will give higher peak flows and levels than the custom IFD set. Given the relative lack of difference between the two and the desire to be conservative, the Reference 1 IFD values will be used.

Table 3: Comparison of IFD calculations for Wagga – Reference 1 versus updated set.

	2-year Average Recurrence Interval	
Station	Rainfall Intensity (mm/h) 1-hour duration	Rainfall Intensity (mm/h) 12-hour duration
WMAwater Calculated		
W. W. Research Centre (74114)	19.2	3.4
W. W. Airport (72150)	23.2	3.6
ARR Calculated		
W. W. Research Centre (74114)	20.9	3.9
W. W. Airport (72150)	21.1	3.9

	50 year ARI (2% Average Recurrence Interval)	
Station	Rainfall Intensity (mm/h) 1-hour duration	Rainfall Intensity (mm/h) 12-hour duration
WMAwater Calculated		
W. W. Research Centre (74114)	40.7	6.7
W. W. Airport (72150)	40.2	8.2
ARR Calculated		
W. W. Research Centre (74114)	44.8	7.2
W. W. Airport (72150)	44.9	7.2

The table below shows the IFD rainfall (as per Reference 1) for Wagga Wagga and the same is graphically displayed in Figure 6.

Table 4: Design Rainfall Depths for various events at Wagga Wagga (Reference 1)

DURATION	1 Y ARI	2 Y ARI	5 Y ARI	10% AEP	5% AEP	2% AEP	1% AEP
5 mins	54.6	72.1	97.7	114	136	166	190
6 mins	50.9	67.1	90.7	106	126	154	176
10 mins	41.4	54.5	73.3	85.3	101	123	141
20 mins	30.1	39.5	52.7	61.1	72.2	87.5	99.7
30 mins	24.3	31.8	42.2	48.8	57.6	69.6	79.1
1 hr	16.1	21.1	27.7	31.8	37.3	44.8	50.8
2hrs	10.3	13.4	17.4	19.8	23.1	27.5	31
3 hrs	7.89	10.2	13.1	14.8	17.2	20.4	22.9
6 hrs	4.94	6.33	7.98	8.96	10.3	12.1	13.5
12 hrs	3.07	3.91	4.86	5.42	6.2	7.23	8.03
24 hrs	1.88	2.39	2.94	3.27	3.73	4.33	4.8
48 hrs	1.1	1.4	1.72	1.91	2.18	2.53	2.8
72 hrs	0.781	0.99	1.22	1.35	1.53	1.78	1.97

3.2.4. Streamflow Gauged Data

No stream gauging data is available within the study area as none of the local watercourses are gauged. As such no flood frequency analysis can be carried out and no data is available for model (hydrologic or hydraulic) calibration or verification purposes. Limited spot gaugings were undertaken however for events that occurred in 1983 and 1984. The fragmented nature of this data has led to its non inclusion in the model verification process. Nevertheless for completeness the data is presented in Table 5.

3.3. Topography

ALS data utilised in the study comes from a 2007 survey carried out by Fugro Spatial Solutions on behalf of the former Department of Environment and Climate Change (DECC) as part of the Murrumbidgee Valley Wetlands Recovery project. Ownership of the data and derivative products rests with OEH with the Land and Property Management Authority (LPMA) being the custodians of the data and derivative products. Both Wagga Wagga City Council and WMAwater executed licence agreements with OEH to utilise the ALS data and ortho-rectified aerial photography for the study.

The ALS provides ground level spot heights from which a Digital Elevation Model (DEM) can be constructed. For well defined points mapped in areas of clear ground, the expected nominal point accuracies (based on a 68% confidence interval) are (vertical accuracy) +/- 0.15 m. 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 slope of steeply varying terrain. The DEM constructed using the provided data is shown in Figure 4. The ALS data points were used to create a 1 m and 2.5 m DTM grid of the study area. Ground elevations in the TUFLOW model are based on the 2.5 m grid (see Figure 7).

Please note that as part of an earlier study which was concerned with the Murrumbidgee River (Reference 8) the ALS data was assessed and found, in a comparison with survey points obtained by land based methods such as Total Station survey, to be as per specification (at least) with respect to accuracy.

As discussed further in Section 4, elevations at the locations of major controls (such as road crowns, levees etc) from the 1 m grid were implemented into the TUFLOW model break lines.

3.4. GIS Layers

A number of spatial data sets covering the study area were made available by Council. The following data sets have been utilised in this study:

- Cadastre;
- Drainage layers;
- Aerial photography based on the 2009 ALS project;
- Land Use;
- Pit/pipe system details (however spatial and attribute data are not joined[2]);
- Drainage hot spot locations;
- LEP Information; and
- Vegetation layers.

[2] Spatial data is available in ARCGIS whilst attribute data is held in Pipe Pak (a proprietary software). There are no features in either layer which would allow for a join to occur.

All GIS data has been provided in a MapInfo/ARCGIS compatible form. In some instances structural details were not available, particularly for non-Council assets, for example culvert structures servicing the railway line. A list of estimated structures is available in Section 4.4.6.

3.5. Community Consultation

A community consultation program was implemented with Council's assistance and finalised in early 2011.

A key component of the program was the community questionnaire which requested observations of flooding. By coincidence the questionnaire process was completed prior to a string of events which occurred from February through to December in 2010.

The community consultation programme involved a number of steps and these were:

- A media release advising Wagga Wagga residents that a Flood Study was to be carried out, what the goals of the Flood Study were and indicating that those with any interest/information might contact the consultant and/or Council in order to communicate information. A copy of this is presented in Appendix C;
- A questionnaire was then issued to certain residences that were, based on Council experience, likely to be impacted by drainage issues (refer to hotspots listed Page 21). A total of 100 questionnaires were hand delivered by Council to residents surrounding known drainage hotspots. Of the 100 issued 11 have been returned. Information on the returned forms has been compiled and is presented in Figure 9. Please note that the original questionnaire as well as all eleven of the returned questionnaires are presented in the attached Appendix C;
- Council has established a Floodplain Management Committee that is comprised of Council staff, SES personnel, Councillors and residents. The committee oversees the work carried out as part of the study; and
- Public Exhibition of the Draft Final report. During the period 19th November 2010 to 19th January 2011 the public were invited to review and make comment on the Draft Final version of the flood study report. 31 copies were requested by various parties and copies were also available in the Public Library for review. As a consequence five submissions were received, two of these from residents at the corner of Urana and Macleay Streets. One submission was from a resident of Sycamore Road which suffered severe flooding during the October 2010 event (as did many other residents along the same street and also along Brunskill Road), another was from a resident of Forsyth Street which experienced severe flooding during the December 2nd event (2010) and the final submission was general comment from a concerned citizen. The two submissions from the corner of Macleay and Urana Streets could be described as stormwater complaints, with the residents reportedly experiencing severe property inundation six times within 2010. The submissions have been reviewed and replied to and this work, including the original submissions, albeit with private details omitted, are presented in Appendix E.

3.6. Verification Data

3.6.1. Introduction

A key component of any Flood Study is the construction of hydrologic and hydraulic models of the study area. These models allow us to predict which areas will be flooded during events of a specific recurrence probability and to calculate a variety of other flood related metrics (such as flood hazard).

Prior to being utilised to define design flood behaviour however it is preferable to calibrate and then validate the models[3]. In order to do this specific data is required as listed below:

- Observations of flooding behaviour during the course (if not at the peak) of a flooding event. Such observations at best will include flood marks which can be surveyed to mAHD. At worst such observations may be an indicative depth, extent or flow direction/velocity; and
- High temporal resolution rainfall data which covers the time period during which the observed flood behaviour was seen to occur.

It is often the case that flood observations are collected during the community consultation phase of the Flood Study, or sometimes it is the case that a specific event was of such magnitude (or occurred so recently) that it is well remembered by Council staff and community alike. From these observations and recollections it is typically the case that specific events which are desired to be included in the suite of events to have the model calibrated and validated against, are identified.

Unfortunately the above does not appear to be the case with respect to the current Study. No specific events were recalled by long standing Council employees as being particularly suitable for use in the modelling work, analysis of rainfall records found no recent events of note and neither did the community consultation process identify many flooding observations or widespread events.

Eight events have been identified ranging from 1965 to 2010. As can be seen, beside the February 5th 2010 event, it has been approximately twenty five years since a relatively extreme rainfall event has been recorded at Wagga (refer to Figure 4). Note the location of various rainfall gauges utilised in the analysis are shown in Figure 3.

The February 5th 2010 event occurred following the completion of the community consultation process. The event occurred on a weekday and during work hours and this gave Council staff the opportunity to document the flooding impact of the event via photos. As a result approximately 50 photos which at the very least demonstrate flood behaviour are available for the February 5th 2010 event (Figure 13). A full set of photos is shown in Appendix B.

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[3] Note these terms are defined in the attached glossary.

Other miscellaneous information on observed flood behaviour is to be found in the community questionnaires that were returned to Council and these are presented in Appendix C but as noted earlier this process identified no specific events suitable for use in the calibration/validation process.

Please note that following submission of the Draft Final report the December 2010 events occurred and these were subsequently used for validation. See Appendix D for details.

3.6.2. PRM Validation of Hydrology

The Probabilistic Rational Method (PRM) is a statistically based method from Australian Rainfall and Runoff (Reference 1) used in specific areas of Australia to provide credible peak flow estimates for undeveloped catchments. Reference 1 limits its applicability to eastern NSW and to rural catchments up to a size of 250 km^2. Wagga Wagga is included in the area the method is applicable to (refer to Figure 5.1 of Reference 1 Volume 2).

Results for the comparison of hydrological model peak flow estimates with PRM estimates are presented in Section 5. The results are then discussed in Section 6.

3.6.3. Data collected by Council

Some flood survey work has been carried out in the past by Council during and following rainfall events to record flood levels and/or velocities. This work however was done on a relatively piecemeal basis and review of such records by Council officers indicates that further usage of this data is not warranted. A key limitation is that the events occurred some time ago when the extent of urban development was considerably smaller. As such the use of these older events for calibration/validation purposes is problematic. An example of the type of data Council has collected is shown below in Table 5. This information is of interest however not being attached to an event of a specific date of occurrence it is of limited use for typical calibration/validation work.

Table 5: An example of gauged data held by Council

<table>
<tr><th>Location</th><th>Size</th><th>Velocity (m/s)</th><th>Depth (m)</th><th>Total Discharge (cu m/sec)</th></tr>
<tr><td rowspan="5">Lake Albert Outlet (under Lake Albert Rd near intersection with Lakeside Dr)</td><td rowspan="5">5 cells 3 m x 0.9 m box culvert</td><td>2.4</td><td>0.15</td><td rowspan="5">3.65</td></tr>
<tr><td>2.6</td><td>0.15</td></tr>
<tr><td>2.2</td><td>0.10</td></tr>
<tr><td>1.5</td><td>0.10</td></tr>
<tr><td>1.4</td><td>0.075</td></tr>
<tr><td>Glenfield Drain Culvert (downstream Dobney Ave)</td><td></td><td>3.2</td><td>0.4</td><td>6.4</td></tr>
<tr><td rowspan="4">Crooked Creek (under Gregadoo Rd)</td><td rowspan="4">4 cell 3 m x 1.5 m box culvert</td><td>0.55</td><td rowspan="4">0.78</td><td rowspan="4">7.2</td></tr>
<tr><td>0.94</td></tr>
<tr><td>0.82</td></tr>
<tr><td>0.80</td></tr>
</table>

As noted information collected by Council for the February 5th 2010 event forms a useful data set. This is discussed further below.

3.6.4. February 5th 2010 Verification Event

The February 5th 2010 event was an event of approximately 20 mm that occurred during the early afternoon. The best available gauged representation of the storm was collected at the Rural Fire Services station in Ashmont.

Appendix B contains photos 1 to 50 for the February 5th 2010 event. The rainfall for the event (in an IFD context) is shown in Figure 4. Figure 5 shows the assumed spatial rainfall distribution of the 5th Feb 2010 rainfall event. The spatial rainfall distribution was based on anecdotal information as well as recorded daily rainfall values. As photos and not surveyed flood marks are available the February 5th 2010 event will be used to verify the models performance and not as a calibration event.

4. MODELLING METHODOLOGY

4.1. Introduction

The key purpose of this study was to define design flood behaviour for the study area as defined in Figure 2. To do this it was required that we develop detailed hydrologic and hydraulic models.

The overall modelling approach was to establish a hydrological model in conjunction with a 1D/2D hydraulic model. The hydrological model is used to generate flow hydrographs for input to the hydraulic model. The 1D/2D hydraulic model then calculates the flood levels and velocities as water moves, overland, in creeks and open channels, through culverts and underground via trunk pipes.

The hydrological model used was the Watershed Bounded Network Model (WBNM). The hydraulic model used is TUFLOW, a 1D/2D fully dynamic fixed grid based model. Both models are discussed in greater detail in the relevant ensuing sections.

The total hydraulic model area is approximately 200 km^2. Given that a 5 m grid was used, to make run times reasonable (and to make the runs possible given limitations in memory) it was necessary to split the study area into four separate model domains.

4.2. Model Domains

Figure 2 shows the four model domains the overall study area has been split into which are; City, Lake Albert, East and North. Each of the domains are discussed below.

4.2.1. Lake Albert (LA)

This model captures the entire Lake Albert watershed including Stringybark Creek, Crooked Creek, Boiling Down Creek and Cox's Creek. The model includes the flood mitigation capacity of Lake Albert and downstream flow past the confluence of Crooked Creek.

4.2.2. East

East includes Gregadoo Creek downstream past the confluence with Marshall's Creek to the Murrumbidgee River. Upstream Marshalls Creek flows are extracted from the Lake Albert model. The extraction occurs at the Vincent Road bridge. This allows all upstream flows from LA to be modelled and attenuated by the lake in a single model along with diversions works on Crooked Creek and Stringybark. In large events not all the flow of Crooked Creek converges with Marshall Creek before Vincent Road and instead tops Laurel Rd. For large events then, flow from Laurel Road is recorded from the LA model and additionally transferred to the East model as an upstream inflow along with Vincent Road flows.

Other areas included in the East model are the industrial zoned land along Copeland Street and The Sturt Highway. All flows west of the Main City Levee along Marshalls Creek are described in the City Model.

4.2.3. City

City refers to the main Wagga Wagga CBD and surrounds. The City model includes Glenfield Drain downstream to Flowerdale Lagoon as well as Wollundry Lagoon and its upstream catchment. Drainage along Silvalite Creek east of the CBD is also included. Drainage through the CBD Wagga Levee system is modelled in the City model including the potential to model the ill-consequence of blocked/backwatered levee drainage in the event a Murrumbidgee flood coincides with local overland flooding.

4.2.4. North

North refers to the Duke's Creek catchment from approximately 10 km north of Wagga to the downstream end at Gobbagombalin Lagoon. This model domain is entirely independent of the other three models since there is no interaction of overland flow. Note that the suburb of North Wagga is not modelled as it is outside of the Duke's Creek catchment.

4.3. Hydrologic Modelling

4.3.1. Introduction

Hydrological modelling was undertaken using WBNM. WBNM is a widely used hydrologic model which has been substantially tested on Australian catchments. The default runoff routing and linearity parameters are based on data from 54 catchments in Queensland, NSW, Victoria and South Australia. Parameters utilised have been found to be independent of area and are recommended for ungauged catchments (Reference 1).

4.3.2. Hydrologic Approach

In order to establish the hydrologic model, sub catchments are delineated (refer to Figure 12) and the fraction impervious assessed. WBNM utilises Reference 1 IFD design storms and applies losses to determine the excess rainfall hyetograph for a range of design storms. Excess rainfall is routed and converted to flow. This routed flow is applied to the hydraulic model at drainage lines via a GIS defined TUFLOW layer. When allocating the flow to the TUFLOW model, either pre-existing wet cells are used or the lowest cell is used and thus the flow is allocated to the drainage path within the defined hydrological sub-catchment. The small time steps at which this process occurs, and the large area the flows are applied to (at peak flow), leads to excellent replication of actual flood processes. The approach, which is typically called "Joint Modelling" (i.e. where rather than being individually run, assessed, calibrated etc the hydrological and hydraulic models work together in a modelling system) is well recognised and considered industry best practice.

Table 6: Hydrological Model Build

	# catchments	Total Area (ha)	Average Size (ha)	Average % Impervious
City	1,395	3,835	2.7	38
LA	550	9,553	17.4	21
East	68	5,900	86.8	14
North	138	3,945	28.6	8
TOTAL	**2,151**	**23,233**	**10.8**	**20 %**

To correctly model attenuating influence of control structures and basins, these features were modelled in the hydraulic model only. Sub-catchments were delineated such that flow was routed to the upstream side of these features.

4.3.3. Model Verification

As stated previously in Section 3 there is a paucity of calibration data for the study. The relatively recent occurrence of the February 5th 2010 event does however give some data to work with. The following work has been carried out in order to verify that the modelling system produces reasonable results:

- Verification of Model to February 5th 2010 event in conjunction with hydraulic model;
- Verification via comparison to PRM estimates;
- Verification in conjunction with hydraulic model to hotspots; and
- Comparison with Reference 4 results (in conjunction with hydraulic model).

Following submission of the Draft Final report the December 2010 events occurred and these were also subsequently used to confirm the models ability to replicate observed behaviour. See Appendix D for details.

4.3.4. Rainfall Losses

4.3.4.1. Observed Events

The only observed event to be modelled was the February 5th 2010 event[4]. Losses applied to the event in WBNM were small with an initial loss of 1 mm and a continuing loss of 1 mm per hour. It's noteworthy however that given that the rainfall applied to the WBNM model was informed by mapped isopleths for the event (as shown in Figure 5), which are approximate only, the losses used are relatively nominal.

[4] See Appendix D for details of the December 2010 events were also used.

4.3.4.2. Design Events

Reference 1 suggests the following losses for NSW ungauged NSW catchments (Table 7).

Table 7: Suggested losses for ungauged NSW catchments (Reference 1)

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

Neither of the categories is entirely appropriate for Wagga. Wagga is inland of the Western Slopes (Great Dividing Range) and is not arid. The conservatively adopted loss parameters are shown in Table 8.

Table 8: Adopted loss model for Wagga Wagga

	Initial Loss	Continuing Loss
Pervious	15 mm	2.5 mm/h
Impervious	1 mm	0mm/h

The most conservative value to select from the available range (based on Reference 1) would be 10 mm (for initial loss). However Reference 6 shows that the use of 10 mm for initial loss would be overly conservative and as such a value of 15 mm is used instead. Note that the previous study (Reference 4) used 25 mm as an initial loss and 2.5 mm/h as a continuing loss whilst the Reference 5 study used 10mm initial loss and a continuing loss of 2.5mm/h.

4.3.5. Pervious/Impervious Surface Mapping

The land use map presented in Figure 8 has been utilised in order to define the percentage impervious for each and every WBNM sub-catchment. Different land uses have been prescribed a specific value of imperviousness as is detailed in the table below. GIS has then been used to inform the percentage imperviousness for each of the WBNM sub-catchments and this has been done in an automated fashion which makes the process relatively efficient. Note that the land use map accounts for existing development only and does not account for proposed future development (as per the Brief).

Table 9: Land Use type and associated level of imperviousness

Land Use	Impervious
agricultural land	0%
improved pasture	0%
industrial/commercial	80%
urban	37%
native forest	0%
natural	0%
reserve/grass/dirt	0%

4.3.6. Critical Duration

Standard design flood estimation practice is for a full range of durations to be examined initially in order to find the critical durations relevant to the study area. Finding a critical duration suitable for each of the model domains was particularly critical in this Study, as model sizes and resolutions meant that each design run would take in the order of ten times modelled time (i.e. a three hour event would take approximately 30 hours to run).

The North model domain was used as it was deemed reasonably representative of the other areas in South Wagga and had the appropriate mixture of upstream and downstream areas and tributary and main stream flow paths. All durations from 15 minutes through to 72 hours were run for with the 1% AEP event.

Figure 15 shows the results for North and as can be seen a variety of events are critical over the North model domain, amongst these being the one (orange), two (red), three (green), six (blue) and nine (yellow) hour events. Also shown on Figure 15 are points P01 through to P14. Peak flood levels have been extracted for these 14 locations for each of the durations run and results are shown in Table 10. The top half of the table presents the peak flood levels whilst the bottom half presents the difference in peak flood level between the duration in question and the three hour duration event. As can be seen the three hour event produces peak flood levels which are a maximum at all locations (or at the very least equal to the maximum when values are rounded to one decimal place). As such the three hour event seems a reasonable choice for the critical duration and will be used for all design and sensitivity runs for all four of the model domains.

4.3.7. Probable Maximum Precipitation

Using the Generalised Short Duration Method (GSDM) as per the Bureau of Meteorology, 2003, the PMP has been calculated for the entire Wagga Wagga study area, including the domains North, City, East and Lake Albert. This was deemed more appropriate and reasonable than calculating a PMP for each of the domains, particularly given that the domains are a contrivance to facilitate detailed hydraulic modelling not discrete catchment areas necessarily.

WBNM was used to carry out the calculation of the PMP/PMF however the rainfall ellipsoid band which each of the approximately 2,200 sub-catchments fell in had to be ascribed manually.

4.4. Hydraulic Modelling

4.4.1. Introduction

A model of the study area was developed in TUFLOW. TUFLOW is widely used in Australia and internationally for assessing flood behaviour and hydraulic hazard. TUFLOW is a finite difference numerical model which is capable of solving the depth averaged shallow water equations in both the one and two dimensional domains. The model consists of a 2D grid defining the ground elevations within the study area with 1D branches defining sub-grid features including the pipe network, channels, culverts, open drains, creeks and the critical outflow subways.

4.4.2. Model Grid

All model runs for each of the four model domains utilised a 5 m grid resolution. Actual elevations are read from the source 2.5 m grid.

4.4.3. Break lines

A number of significant hydraulic features, which are likely to impact on the flow behaviour, exist within the catchment. Hydraulic features are particularly important due to the gently undulating/flat nature of the topography throughout much of the catchment. Breaklines were used throughout the study area in order to precisely define hydraulic controls with the best available data.

4.4.4. Boundary Conditions

Flow hydrographs from WBNM were introduced to the hydraulic model and this accounts for the flow input to the hydraulic model. The downstream boundary, for most model domains (Lake Albert excluded) is the Murrumbidgee River and this has been incorporated into the modelling as an adjustable water level which can be sloped as required. For all design runs a 2Y ARI flood level has been used for the Murrumbidgee River.

An additional sensitivity run has been carried out which utilises a higher Murrumbidgee River water level which approximates the 20% AEP (or 5Y ARI) event.

Wollundry Lagoon and Lake Albert have been set to standard operating heights as defined by the invert levels of outlets, i.e. 177.1 mAHD and 190.2 mAHD respectively. Both water bodies have been assumed full at the beginning of model runs.

Table 10: North Model Domain Peak Flood Levels for the 1% AEP event for various durations

	1% 15m	1% 30m	1% 45m	1% 60m	1% 90m	1% 2h	1% 3h	1% 4.5h	1% 6h	1% 9h	1% 12h	1% 24h	1% 48h	1% 72h
P01	195.9	196.0	196.1	196.2	196.2	196.2	196.2	196.1	196.1	196.1	196.1	196.1	196.1	196.0
P02	193.9	194.1	194.2	194.2	194.2	194.2	194.2	194.2	194.2	194.1	194.2	194.1	194.1	194.1
P03	191.7	191.8	191.8	191.8	191.8	191.8	191.8	191.8	191.8	191.8	191.8	191.8	191.8	191.7
P04	186.3	186.5	186.6	186.7	186.8	186.8	186.8	186.8	186.7	186.7	186.7	186.7	186.7	186.6
P05	179.7	180.0	180.1	180.2	180.3	180.3	180.3	180.3	180.3	180.2	180.2	180.2	180.2	180.1
P06	177.8	177.8	177.9	177.9	177.9	177.9	177.9	177.9	177.9	177.9	177.9	177.9	177.9	177.9
P07	177.1	177.4	177.6	177.6	177.7	177.7	177.7	177.7	177.7	177.7	177.7	177.7	177.7	177.7
P08	176.8	177.0	177.1	177.2	177.2	177.3	177.3	177.4	177.4	177.4	177.4	177.4	177.3	177.3
P09	176.6	176.9	177.1	177.2	177.2	177.2	177.3	177.3	177.3	177.3	177.3	177.3	177.3	177.3
P10	207.2	207.4	207.5	207.6	207.6	207.6	207.6	207.5	207.5	207.5	207.5	207.5	207.5	207.4
P11	196.8	197.2	197.4	197.5	197.6	197.6	197.6	197.6	197.6	197.5	197.5	197.5	197.5	197.3
P12	213.3	213.3	213.3	213.3	213.3	213.3	213.3	213.3	213.3	213.3	213.3	213.3	213.3	213.3
P13	196.6	196.6	196.6	196.6	196.6	196.6	196.6	196.6	196.6	196.6	196.6	196.6	196.6	196.6
P14	188.4	188.4	188.4	188.4	188.4	188.4	188.4	188.4	188.4	188.4	188.4	188.4	188.4	188.4
Difference in metres relative to 3h duration. A negative number indicates a lower value and a positive value a higher value.														
P01	-0.3	-0.2	-0.1	0.0	0.0	0.0	0.0	0.0	-0.1	-0.1	-0.1	-0.1	-0.1	-0.2
P02	-0.3	-0.2	-0.1	0.0	0.0	0.0	0.0	0.0	0.0	-0.1	-0.1	-0.1	-0.1	-0.2
P03	-0.1	-0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-0.1	-0.1	-0.1	-0.1	-0.1
P04	-0.4	-0.3	-0.1	-0.1	0.0	0.0	0.0	0.0	0.0	-0.1	-0.1	-0.1	-0.1	-0.2
P05	-0.6	-0.4	-0.2	-0.1	0.0	0.0	0.0	0.0	0.0	-0.1	-0.1	-0.1	-0.1	-0.2
P06	-0.1	-0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
P07	-0.6	-0.3	-0.2	-0.1	-0.1	0.0	0.0	0.0	0.0	-0.1	-0.1	0.0	-0.1	-0.1
P08	-0.5	-0.3	-0.2	-0.1	-0.1	0.0	0.0	0.0	0.0	0.1	0.0	0.0	0.0	0.0
P09	-0.7	-0.4	-0.2	-0.1	-0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
P10	-0.4	-0.2	0.0	0.0	0.0	0.0	0.0	0.0	-0.1	-0.1	-0.1	-0.1	-0.1	-0.2
P11	-0.8	-0.4	-0.2	-0.1	0.0	0.0	0.0	0.0	-0.1	-0.1	-0.2	-0.1	-0.2	-0.3
P12	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
P13	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
P14	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

4.4.5. Roughness

Model roughness was assigned on the basis of land type. Land type was interpreted using aerial photography of the study areas supplied by Council. Roughness values used for the various land types are shown in Table 11.

Table 11: Manning's definition for Hydraulic Model

Mannings 'n'	Land Type Description
0.06	pasture
0.04	1D cross section elements
0.07	lots
0.03	ponds and other water bodies
0.018	newly built/resurfaced road
0.022	roads
0.04	creek permanent water
0.1	vegetation
0.08	vegetated creek

Houses are not included in the roughness description. Instead, where houses interact significantly with overland flow, they have been "nulled" out of the model grid. This means that no flow can occur through a house footprint and also that areas occupied by housing are not included in any flood storage calculations. Note that where houses lie in close proximity to one another the digitised house extent has been modified to ensure that flow between houses can occur.

4.4.6. Implemented Structures

Various structures, as shown in Figure 11 were incorporated into the hydraulic model. A summary of structures included in the various model domains is provided below:

Table 12: Summary of implemented structures

Element Type	City	Lake Albert	East	North
	#	#	#	#
Pipes	373	59	37	26
Structures (culverts)	69	47	16	34
Pits	365	74	42	53
Nodes	168	105	36	68
Open Channel length	2537 m	3756 m	-	-

Below is a list of structures the dimensions of which had to be estimated as details could not be found on Council's "Documap" system. Note that estimated details are based on visual observations in most cases. Assumed structures are labelled in Figure 11.

Table 13: Assumed Structures

ID	Assumed Dimensions	Number	X (MGA-55)	Y (MGA-55)
AS_01	R: 1 (wide) by 1(height)	1	531,411	6,112,971
AS_02	R: 1 (wide) by 1(height)	1	531,499	6,113,009
AS_03	R: 1 (wide) by 1(height)	1	531,764	6,113,115
AS_04	R: 2.4 (wide) by 1(height)	1	531,926	6,113,179
AS_05	R: 1 (wide) by 1(height)	1	532,459	6,113,394
AS_06	R: 2 (wide) by 1(height)	1	532,752	6,113,510
AS_07	R: 2.5 (wide) by 1(height)	2	533,987	6,113,466
AS_08	R: 1 (wide) by 1(height)	1	534,121	6,113,467
AS_09	R: 2 (wide) by 1(height)	1	529,899	6,110,835
AS_10	R: 2 (wide) by 1(height)	1	530,531	6,110,525
AS_11	R: 2 (wide) by 1.5 (height)	1	528,929	6,112,170
AS_12	R: 2.5 (wide) by 2.5 (height)	2	528,351	6,110,864
AS_13	C: 1.2 (diameter)	1	531,501	6,114,187
AS_14	R: 3 (wide) by 0.8(height)	2	535,567	6,106,680
AS_15	Zsh: 20 (thick line)	1	530,248	6,107,598
AS_16	Zsh: 20 (thick line)	1	530,200	6,107,443
AS_17	Zsh: 20 (thick line)	1	530,169	6,107,321
AS_18	R: 2.5 (wide) by 1(height)	3	538,147	6,111,963
AS_19	R: 2 (wide) by 1 (height)	1	538,806	6,111,600
AS_20	R: 2 (wide) by 1 (height)	1	539,185	6,111,422
AS_21	R: 3 (wide) by 1.2 (height)	3	540,662	6,108,837
AS_22	R: 3 (wide) by 1.2 (height)	3	540,605	6,108,845
AS_23	R: 3 (wide) by 1.2 (height)	3	539,715	6,108,986
AS_24	R: 1.5 (wide) by 0.6 (height)	2	535,966	6,109,686
AS_25	C: 0.9 (diameter)	1	535,179	6,108,007
AS_26	R: 1.5 (wide) by 1 (height)	1	532,851	6,107,129
AS_27	R: 1.5 (wide) by 1 (height)	1	533,543	6,107,019

4.4.6.1. Pit and Pipe Representation

As per the brief, larger pipe systems (diameter greater than or equal to 1.05 m) were included in the modelling (included pipes shown on Figure 11). This amounted to the inclusion of the main arterial components of the drainage network in the hydraulic model. Standard pit connections would not adequately allow water to enter the pipe system since all the smaller diameter pipe feeder branches are not included in the model. Pit capacities are therefore exaggerated and where appropriate, upstream flows were directly applied to the pipe network. Excess flow in the pipe systems was free to surcharge from the modelled pits and into the 1D/2D overbank model. All pipe/pit details came from Council via Council's “Documap” system.

4.4.7. Cross Catchment Flow Issues

Since the hydraulic model domain for the overall study was divided into four model domains, the potential interaction of flows across model domains was an important issue. Two locations were identified as locations where cross-catchment flows would occur. These are discussed further below.

4.4.7.1. Marshalls Creek from Lake Albert model to East model

Overland flow from the Lake Albert model was recorded as it progressed to the railway embankment which was the downstream limit of the Lake Albert hydraulic model domain. Flows recorded were then used as inflows to the East model.

4.4.7.2. City Model discharge to Marshalls Creek

Certain structures defined in the City model drain through the Main City Levee into Marshall's Creek and as such the tail water level in Marshall's Creek is an important control on the ability of these structures to drain flow. Given this peak flood levels in Marshalls creek recorded from the East model were defined as tail water levels for flow discharging through the structures from the City domain.

As shown in Table 14 Marshalls Creek (recorded at Sturt Highway) is logically higher than the Murrumbidgee River and the level defined from the East model is important in characterising the ability of the City model to drain through the levee just upstream of the Sturt Hwy.

Table 14: Influence of Marshalls Creek for City model drainage

Murrumbidgee River @ Marshalls Ck	
Design (m AHD)	177.3
High River 5yr (mAHD)	180.0
Marshalls Ck @ Sturt Hwy (from East model)	
10yr (m AHD)	178.6
100yr (m AHD)	180.3

4.5. Model Verification

Model verification has been undertaken as far as is possible using the data set available. Verification efforts include the following:

- Joint verification of the hydrological and hydraulic models using the Feb 5th 2010 event photos;
- Joint verification of both models by comparing 5% AEP 3 hour results with Council identified drainage "hot spots";
- Comparison of peak flow estimates in Glenfield Drain for the 5% AEP 3 hour event with peak flow estimates from the 2006 study of the same; and
- Meetings with key experienced Council staff to review the modelling results and generally assess their reasonableness.

Results from the verification work are shown in Section 5 and the significance of the results are discussed in Section 6.

Note that subsequent to submission of the Draft Final for public exhibition significant events occurred in October and December of 2010. All events were investigated for inclusion in the verification process however representative high resolution rainfall was not identified for the October event. The December 9th event was included and this work is documented in Appendix D.

4.6. Sensitivity Runs

In the absence of extensive calibration/validation data the results of sensitivity testing become of greater significance to gaining an understanding of the likely accuracy of model results. Sensitivity runs carried out are listed below:

- Roughness altered by +/- 20% in the hydraulic model (including both in-bank and out of bank);
- Design losses altered from 15 mm/ 2.5 mm/h to a low loss scenario of 10 mm/ 1 mm/h and a high loss scenario of 25 mm/ 3 mm/h;
- Blockage of 25% and 50% for all pipes and culverts;
- Murrumbidgee River tail water level increased from the base case of 2Y ARI to the 5Y ARI (20% AEP) flood level;
- To address climate change run requirements a 7% increase to rainfall was also run. The 7% figure comes from DECCW guidelines on addressing climate change (Reference 7); and
- A run was carried out using the 2Y ARI event where it was presumed that no flow could egress to the Murrumbidgee River via pipes (but pumps were functional) and results were then compared to a base case 2Y ARI event.

Results from sensitivity runs are shown in Section 5 and discussed in Section 6.

5. RESULTS

5.1. Verification

Verification work aims to improve the level of confidence that can be had in model results. The verification work carried out in this study is as described in Section 3.6.

5.1.1. February 5th 2010

Figure 13 shows the inundation extent for the modelled February 5th 2010 event with locations of photos also shown (photo numbers are displayed). Appendix B holds photos 1-50 which should be referred to when examining the model results.

Assorted specific points are referred to in the following discussion.

Photo 48 indicates flooding at the western face of the juvenile detention centre. Within the blue framed inset this behaviour appears to be well matched.

Photo 8 shows water at the intersection of Mortimer Place and Chaston Street. This behaviour seems to be well replicated in the model.

Photos 10, 9, 10a and 13 show considerable inundation along Dobney Avenue and all seem to be well matched by the model.

Photo 38 shows water ponding at the intersection of Bye and Pearson Streets which again seems to be well matched by the model.

Photos 46, 47, 41, 42 and 43 all indicate significant flooding occurring near the traditional anabranch path in this area and the model replicates this behaviour quite well.

Photos 44 and 45 show inundation occurring in Murray Street and again the model seems to replicate this behaviour quite well.

Photo 49 indicates gutter ponding along Shaw Street and again this behaviour is very well matched by the model.

Photo 39 near Wollundry Lagoon shows water ponding on The Esplanade, mainly on the side of the street away from the lagoon. The model results replicate this quite well.

Photo 22 shows that flooding has occurred of the McDonalds along Glenfield Drain (just north of Fernleigh Rd) and the model indicates that this area has been inundated to a shallow depth and as such this is a good match.

5.1.2. December 9th 2010

Full reporting on the December 9th event can be found in Appendix D. As per the February 2009 event however no flood levels were available and instead model performance was compared to flood extents only. The available observation set was extensive and given the event was widespread three of four model domains were verified. Overall the results indicate that the model was able to replicate the observed behaviour satisfactorily and confidence is justified in design flood estimates. See Appendix D for further details.

5.1.3. PRM Check

The PRM check has been carried out for a nominal catchment with area of 5.4 km^2. The calculations and result are detailed below in Table 15.

Table 15: PRM Estimates compared to WBNM Estimates

1% AEP	**5% AEP**
WBNM	**WBNM**
Initial Loss (mm)	*Initial Loss (mm)*
10	10
Continuing Loss (mm/h)	*Continuing Loss (mm/h)*
2	2
Contributing Area (km^2)	*Contributing Area (km^2)*
5.4	5.4
Peak Flow m^3/s	***Peak Flow m^3/s***
24.2	14.5
PRM	**PRM**
runoff coefficient	*runoff coefficient*
0.43	0.31
Time of Concentration(h)	*Time of Concentration(h)*
1.4	1.4
Intensity (mm/h)	*Intensity (mm/h)*
40	30
Peak Flow m^3/s	***Peak Flow m^3/s***
25.9	14.0

5.1.4. Hotspots

Text from the Brief is recreated here in order to define the expectations for flooding behaviour at the Council nominated hotspots. Council text is recreated in italics. Please refer to Figure 14 for depictions of model results at hotspot locations. A further hotspot was added as public submissions identified that the corner of Urana and Macleay Streets is subjected to frequent inundation (six times in 2010 according to public submissions).

1. Moorong St/Edward St/Main Levee Outlet
This area is subject to a rezoning proposal. It is at the bottom end of the main city drainage system. There is existing detention storage and a pump station. Flooding has been observed to a depth of 100mm at the intersection of Spring St and Moorong St.

2. Dobney Ave/Chaston St
This area is at the confluence of 2 major catchments. One catchment (Chaston St) is totally piped. The other catchment (Glenfield Drain) is an open channel. Flows in the open channel appear to retard discharges from the Chaston St Catchment, leading to surcharging and flooding of commercial properties in Chaston St (Wagga Motors). Depth in Wagga Motors has been in the order of 150mm.

3. Glenfield Drain/Great Southern Railway Line
At this location the Glenfield Drain passes under the main Sydney to Melbourne rail line. Council investigations show that the culvert structure under the line may be undersized. Reports have been received that flows have actually built up and flowed over the rail line, however the Railway height makes this "observation" unlikely.

4. Red Hill Rd/Yentoo Dr Drainage Management
A substantial catchment has been diverted to this location. The upstream area is subject to a rezoning application. Discharge on the northern side of Red Hill road has exceeded the diversion channel and flooded downhill residences, however, this behaviour was complicated by an embankment failure.

5. Bolton Park/Bardo Lane
Bolton Park is a well used sporting venue. It has a water course through the middle. On the northern side, the park sits below the surrounding area and serves as a detention basin. Water level has been seen to build up and overflow the footpath of Morgan St.

6. Jones St/Marshalls Creek
This area is subject to extensive road flooding of up to 0.5m of Jones St on about a once in 4-5 year frequency.

7. Copland St South Proposed Industrial Rezoning
This area is proposed to be rezoned for industrial purposes. Site has been subject to local

flooding in a flood of October 1974, but depth unknown. A flood height was observed on Vincent Rd in this event – bottom of substantial brick gates, west of Crooked Creek culvert.

8. Brunskill Road
This area suffers from street flooding and threatens adjoining residential development. Depth threatens floor of eastern most house, southern side of Brunskill Rd.

9. Hakea Place
The open drain behind the cul de sac of Hakea Place overflows about once in 5 years and floods residences around the cul de sac.

10. Intersection of Urana and Macleay Streets.
Water moves down to this intersection from the Botanic Gardens and leads to inundation of Urana St as well as Heydon Ave. Flooding is regular (reported to have occurred six times in 2010 alone).

5.1.5. Glenfield Report (2006)

Please note that whilst results from the above referenced study are used herein in an attempt to verify the suitability of current study estimates it is important to clarify that current study estimates are expected to be more accurate than those derived previously. Issues with the 2006 study include the following:

- Lack of broad acre survey to inform overland flow paths;
- Reliance on surveyed cross-sections of main drainage paths only; and
- Use of a one-dimensional model which does not allow flow to overtop modelled channels unless specific facilitating schematisation is carried out.

The sum result of the methodology employed in the 2006 study means that it's expected that flow estimates, particularly for larger events, will be exaggerated. The reason for this is that flow will be constrained to drainage paths as modelled, where in reality such flow would overtop the modelled channel and spill into the overbank area.

Table 16: 5% AEP Peak Flow Comparison - Model and 2006 Glenfield Report

LOCATION	*Modelled Peak Flow (m^3/s)*	*Reference 4 Peak Flow (m^3/s)**
Dalman Parkway	13.1	11.4/20.5
Fernleigh Rd	21.0	30.7/39.0
Railway	16.5	26.5/37.7
Dobney/Pearson	13.4	27.4/37.7
Sturt Hwy	18.2	29.1/37.9

*the two numbers indicate the value firstly for "existing" conditions and secondly for "developed" conditions. The peak flow estimates for the "developed" condition are the comparable values in this case, taking into account as they do similar development extents as modelled in the current study.

As can be seen the 5% AEP peak flow estimates from the current study significantly underestimate peak flow within Glenfield Drain relative to the Reference 4 peak flows. The importance of the results is discussed at length in Section 6.

5.2. Design Flood Results

5.2.1. Introduction

Design results can be found as Figure 16 through to 39 (barring Figure 32 which identifies the location of profile and extraction points). A single A3 map is available for each of the model domains for the 1% AEP results and this plot includes peak flood level contours. For other design flood events (including the 1 and 2Y ARI events as well as the 20%, 10%, 5% and 0.5% AEP and PMF events) results for all four domains are presented on a single A3 plot of depth/extent. The spatial area over which results are available and the 2D nature of these results mean that interrogation of results via a spatial computer program such as Water Ride or via a GIS is required.

Tabulated data has been provided for reference. Tables 17-20 present detailed results for multiple significant locations, each table containing results pertinent to a given model domain. See Figure 32 for location of profiles shown in Figures 33-39 and also for point locations at which peak flood level and flow have been extracted. Please note that where applicable total flow is the sum of flow through one and two dimensional model elements. For example at Springvale Drive on Stringybark Creek some flow passes through the culvert (one dimensional model flow) and some flow (for larger events only) will pass over the road (two dimensional model flow). In this way the design capacity of various road crossings can be examined. Note that in situations where two dimensional flow is not contained in a discrete flow path (for example Plumpton Road where flow unable to be accommodated by the culvert surcharges and moves in a variety of directions) no two dimensional model flow value is reported. Results of interest have been extracted for review purposes and also to typify general flood behaviour. As such tabulated peak flood height and flow data for a variety of locations (see Figure 32 for locations) for all events modelled is shown over the page.

Table 17: North Model Domain Results

	Location	NSL	001yr			002yr			005yr			10% AEP			5% AEP			2% AEP			1% AEP			0.5% AEP			PMF		
#		Level	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D
1	Dukes Ck (#1)	195.4	195.6	NA	1.0	195.8	NA	3.1	195.9	NA	7.4	196.0	NA	10.1	196.0	NA	13.9	196.1	NA	18.6	196.2	NA	23.5	196.2	NA	28.3	196.5	NA	54.9
2	Dukes Ck (#2)	193.7	193.8	NA	1.1	193.9	NA	4.0	194.0	NA	11.2	194.0	NA	15.7	194.1	NA	22.2	194.2	NA	29.8	194.2	NA	37.2	194.3	NA	45.1	194.5	NA	86.4
3	Dukes Ck (#4)	186.0	186.2	NA	1.2	186.3	NA	4.6	186.4	NA	14.2	186.5	NA	20.7	186.6	NA	30.3	186.7	NA	42.2	186.8	NA	53.2	186.9	NA	64.7	187.2	NA	143.6
4	Dukes Ck Olympic Hwy	183.8	184.0	NA	0.8	184.1	NA	4.7	184.4	NA	14.8	184.5	NA	21.7	184.7	NA	31.8	184.8	NA	44.6	184.9	NA	56.3	185.0	NA	68.6	185.6	NA	143.1
5	Dukes Ck near Horseshoe Rd	179.4	179.5	NA	0.6	179.6	NA	5.6	179.9	NA	17.5	180.0	NA	25.5	180.1	NA	38.4	180.2	NA	54.6	180.3	NA	69.1	180.4	NA	84.2	180.9	NA	178.2
6	Dukes Creek (#7)	176.9	177.0	NA	---	177.0	NA	---	177.3	NA	---	177.5	NA	---	177.6	NA	---	177.7	NA	---	177.7	NA	---	177.8	NA	---	177.9	NA	---
7	Dukes Ck (#8)	176.7	176.8	NA	0.1	176.8	NA	1.1	177.0	NA	4.4	177.1	NA	7.1	177.2	NA	11.1	177.3	NA	15.9	177.3	NA	19.9	177.4	NA	25.3	177.8	NA	97.2
8	Dukes Ck @ Boorooma St	176.1	176.5	0.1	0.0	176.6	1.0	0.0	176.9	3.6	0.0	177.0	5.9	0.0	177.2	8.3	1.0	177.2	9.9	4.5	177.3	10.8	7.3	177.3	12.1	12.6	177.7	20.1	79.5
9	Dukes Ck just u/s RIver	171.4	176.5	NA	12.2	176.5	NA	12.2	176.5	NA	12.2	176.5	NA	12.2	176.5	NA	12.2	176.5	NA	14.7	176.5	NA	18.1	176.5	NA	25.0	176.5	NA	100.1
10	Olympic Hwy (#21)	180.3	180.8	0.3	0.0	181.0	1.5	0.0	181.3	3.6	0.0	181.5	5.0	0.0	181.8	7.6	0.0	182.1	9.1	0.0	182.4	10.8	0.0	182.7	12.3	0.0	183.3	14.8	10.2
11	Olympic Hwy (#22)	180.3	180.5	0.0	0.0	181.3	0.3	0.0	181.3	0.6	0.0	181.3	0.8	0.0	181.3	1.2	0.0	181.4	1.7	0.0	181.5	2.2	0.0	181.6	2.6	0.0	182.5	5.9	0.0
12	Un-named Ck (#10)	206.5	206.9	NA	0.4	207.2	NA	1.3	207.3	NA	3.2	207.4	NA	4.4	207.5	NA	6.3	207.5	NA	8.3	207.6	NA	10.4	207.7	NA	12.6	207.9	NA	22.5
13	Un-named Ck (#11)	195.4	195.8	NA	0.2	196.7	NA	1.4	197.1	NA	4.0	197.2	NA	5.7	197.4	NA	8.1	197.5	NA	11.4	197.6	NA	13.8	197.8	NA	16.7	198.1	NA	35.3
14	Un-named Ck at Olympic Hwy	180.5	180.6	0.1	0.0	180.8	1.0	0.0	181.2	4.8	0.0	181.4	8.3	0.0	181.5	10.7	0.0	181.8	14.8	0.0	182.0	18.8	0.0	182.4	23.4	0.0	183.4	36.8	11.2
15	Olympic Hwy (#19)	180.0	180.3	0.4	0.0	180.5	1.4	0.0	180.7	2.8	0.0	180.7	3.2	0.0	180.8	4.1	0.0	180.9	5.5	0.0	181.1	7.3	0.0	181.2	9.0	0.0	182.0	17.0	0.0
16	Olympic Hwy (#20)	180.1	180.4	0.0	0.0	180.4	0.0	0.0	180.4	0.01	0.0	180.4	0.01	0.0	180.5	0.02	0.0	180.5	0.02	0.0	180.5	0.02	0.0	180.5	0.03	0.0	181.3	1.33	0.0
17	Cnr Boorooma & Cooramin St	198.5	198.6	0.2	0.3	198.6	0.4	0.3	198.6	0.6	0.4	198.6	0.7	0.6	198.6	0.9	0.9	198.6	1.0	1.2	198.7	1.1	1.8	198.7	1.2	2.5	198.8	1.7	7.1
18	u/s Olympic Hwy (#23)	195.8	196.0	0.2	0.0	196.3	1.3	0.0	196.5	3.6	0.0	196.7	5.3	0.0	196.9	7.6	0.0	197.1	10.2	0.0	197.4	12.6	0.1	197.6	14.0	1.8	197.9	15.6	13.3
19	u/s Olympic Hwy (#3)	191.1	191.5	1.5	0.0	191.6	3.1	0.0	191.7	6.1	0.0	191.7	8.2	0.0	191.7	11.0	0.0	191.8	14.2	0.0	191.8	17.4	0.0	191.9	20.7	0.0	192.1	39.4	10.9

NA: Not applicable

NSL: Natural surface level

--- 2D flow path not well defined (able to be recorded) and hence not reported

Number of point observed on Figure 32 for relevant model domain

All levels are in mAHD

Q_1D and Q_2D results are in units of m^3/s

Q_1D indicates flow in the 1D model components. 2D indicates flow in the 2D model components. Total flow at a location can be calculated by the Q_1D and Q_2D values

Table 18: City Model Domain Results

	Location	NSL	001yr			002yr			005yr			10% AEP			5% AEP			2% AEP			1% AEP			0.5% AEP			PMF		
#		Level	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D
1	Cnr of Urana St & Mitchelmore St	206.8	207.0	0.0	1.0	207.1	0.0	0.6	207.1	0.1	1.9	207.1	0.1	2.4	207.1	0.2	3.5	207.2	0.2	4.6	207.2	0.3	5.8	207.2	0.4	7.0	207.7	2.0	33.3
2	Cnr of Colemant St & Inverary St	191.2	191.3	0.2	0.5	191.4	0.3	0.9	191.4	0.5	2.1	191.4	0.7	3.0	191.4	0.9	4.4	191.4	1.2	5.8	191.4	1.5	7.4	191.4	1.8	9.1	191.7	7.3	43.7
3	Railway line @ cnr of Inverary St & Cassidy Pde	187.5	187.5	1.1	0.0	187.5	1.0	0.1	187.5	2.1	0.2	187.5	2.9	0.4	187.5	3.9	0.8	187.5	4.9	1.4	187.5	5.7	2.2	187.5	6.4	3.4	187.8	14.7	34.9
4	Housing bound by Forsyth, Morgan & Murray St	178.6	178.8	NA	0.0	179.0	NA	0.5	179.1	NA	1.0	179.1	NA	1.3	179.2	NA	2.0	179.3	NA	3.3	179.3	NA	4.5	179.3	NA	6.0	180.1	NA	39.0
5	Concrete o/c at Forsyth St feeding Wollundry	177.6	177.6	0.6	0.0	177.7	1.1	0.0	177.8	1.7	0.0	177.9	2.2	0.0	178.1	3.2	0.0	178.3	5.2	0.1	178.5	6.7	0.3	178.6	8.3	0.5	180.0	15.2	18.5
6	Most d/s pool in Wollundry Lagoon	176.5	176.6	0.0	0.0	176.6	0.0	0.0	177.2	0.0	0.0	177.4	0.2	0.0	177.7	0.5	0.0	177.9	1.0	0.0	178.1	1.4	0.0	178.4	1.9	0.0	181.4	1.8	1.1
7	Tony Ireland Park	175.7	177.2	0.2	---	177.2	1.1	---	177.2	1.8	---	177.2	1.9	---	177.2	2.1	---	177.2	3.1	---	177.3	3.8	---	177.3	4.7	---	181.4	25.3	---
8	Cnr of Sturt Hwy & Edwina St	179.7	179.8	0.1	0.1	179.9	0.1	0.3	179.9	0.2	0.8	179.9	0.2	1.2	179.9	0.3	1.8	179.9	0.3	2.7	180.0	0.4	3.3	180.0	0.4	4.3	181.5	0.7	26.4
9	Cnr of Morgan St & Bardo Ln	178.6	178.7	0.3	0.2	178.8	0.5	0.3	178.9	1.2	0.4	179.0	1.5	0.5	179.0	1.9	0.5	179.1	2.3	0.5	179.2	2.6	0.6	179.2	3.0	0.8	181.5	3.9	75.3
10	Light Industrial Area Sth of Sturt Hwy Wst of Marshalls Ck	179.0	179.0	0.2	---	179.1	0.6	---	179.1	1.2	---	179.1	1.2	---	179.1	1.2	---	179.7	1.1	---	180.1	0.0	---	180.2	0.0	---	181.7	0.0	---
11	Mason St low point	179.4	179.5	0.1	---	179.5	0.2	---	179.5	0.5	---	179.5	0.6	---	179.6	0.8	---	179.6	0.9	---	179.7	1.0	---	179.7	1.0	---	181.6	1.1	---
12	GfD @ Dalman Pkwy	197.2	197.4	0.8	0.0	197.5	1.3	0.1	197.8	4.5	0.2	198.1	7.9	0.4	198.3	12.5	0.6	198.7	18.0	1.0	198.6	23.6	1.3	198.8	28.8	1.6	200.6	55.8	15.1
13	GfD @ Fernleigh Rd	187.5	187.8	1.5	0.0	188.0	3.1	0.0	188.4	8.4	0.0	188.6	13.6	0.0	188.9	21.0	0.0	189.2	29.3	0.2	189.4	30.7	5.1	189.5	31.8	11.4	190.5	34.1	115.0
14	GfD @ Railway Embankment	184.6	185.1	1.3	0.0	185.5	2.9	0.0	186.2	8.1	0.0	186.7	12.5	0.0	186.9	16.5	0.0	187.3	20.4	0.0	187.7	24.4	0.0	188.0	28.1	0.0	189.1	38.9	163.1
15	GfD @ Intersection of of Dobney Av and Pearson St	180.5	180.8	1.3	---	181.0	3.0	---	181.4	8.7	---	181.6	11.6	---	182.3	13.4	---	182.4	14.0	---	182.5	14.3	---	182.5	14.6	---	183.4	18.3	---
16	GfD @ Sturt Hwy	176.0	176.2	2.6	0.0	176.3	5.0	0.0	176.6	9.6	0.0	176.8	14.6	0.0	177.0	18.2	0.0	177.4	23.0	0.0	177.6	26.1	0.0	177.9	29.1	0.0	179.9	53.2	85.1
17	Flowerdale Lagoon	175.2	175.6	2.8	0.0	175.8	5.6	0.0	176.2	11.0	0.0	176.4	16.1	0.0	176.7	20.8	0.0	177.1	25.7	0.0	177.2	30.1	0.0	177.4	33.0	0.0	179.9	59.9	0.0
18	Detention basin @ cnr of Red Hill Rd & Railway	207.6	207.9	0.0	0.0	208.1	0.2	0.0	208.3	0.6	0.0	208.4	0.9	0.0	208.5	1.4	0.0	208.6	1.9	0.0	208.8	2.3	0.0	208.9	2.7	0.0	210.6	4.1	0.0
19	Silvalite @ Red Hill Rd	200.4	200.5	0.1	0.0	200.6	0.6	0.0	200.7	3.3	0.0	200.7	5.0	0.0	200.7	7.4	0.0	200.8	10.0	0.0	200.9	13.1	0.0	200.9	16.2	0.0	202.5	28.7	0.5
20	Silvalite @ Sturt Hwy	182.7	183.1	0.5	0.0	183.1	1.7	0.0	183.3	5.4	0.0	183.4	7.7	0.0	183.5	11.5	0.0	183.6	16.1	0.0	183.7	21.1	0.0	183.7	27.2	0.0	186.0	42.7	0.0

NA: Not applicable
NSL: Natural surface level
--- 2D flow path not well defined (able to be recorded) and hence not reported
Number of point observed on Figure 32 for relevant model domain
GfD Glenfield Drain
All levels are in mAHD
Q_1D and Q_2D results are in units of m^3/s
Q_1D indicates flow in the 1D model components. 2D indicates flow in the 2D model components. Total flow at a location can be calculated by the Q_1D and Q_2D values

Table 19: Lake Albert Model Domain Results

	Location	NSL	1yr ARI			2yr ARI			5yr ARI			10% AEP			5% AEP			2% AEP			1% AEP			0.5% AEP			PMF		
#		Level	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D
1	Mangoplah just South of Rowan Rd	259.0	259.0	0.1	0.0	259.2	1.3	0.0	259.4	3.5	0.0	259.5	4.9	0.0	259.6	6.9	0.0	259.8	9.2	1.3	259.9	11.5	0.0	260.0	13.9	1.1	260.7	30.4	1.2
2	u/s reach of Stringy Bark Creek	227.0	227.0	NA	0.1	227.1	NA	1.3	227.5	NA	5.8	227.6	NA	8.8	227.8	NA	13.0	228.0	NA	17.3	228.2	NA	22.0	228.3	NA	26.8	229.3	NA	60.6
3	Boiling Down Rd near Rowan Rd	216.2	216.3	0.2	0.0	216.7	2.6	0.0	217.4	9.5	0.5	217.8	14.6	0.9	218.2	21.4	1.6	218.6	26.5	4.9	218.8	29.7	11.5	218.9	31.8	18.9	219.4	37.2	90.0
4	Mangoplah just Nth of Rowan Rd	252.0	252.0	0.2	0.0	252.1	0.8	0.0	252.3	2.1	0.0	252.5	3.1	0.0	252.8	4.7	0.0	253.2	6.3	0.0	253.5	8.0	0.1	253.9	9.8	0.2	255.1	13.0	9.2
5	Lloyd Rd	220.4	220.9	0.1	0.1	221.6	0.6	0.2	221.9	0.7	2.3	222.0	0.7	4.6	222.0	0.7	7.4	222.1	0.7	10.8	222.1	0.8	13.5	222.1	0.8	17.1	222.3	0.8	41.0
6	Holbrook Rd between Indigo Dr & Featherwood Rd	227.2	227.3	0.2	0.0	227.3	0.8	0.0	227.4	4.0	0.0	227.5	6.8	0.1	227.6	8.2	3.3	227.7	8.5	8.6	227.8	8.7	13.2	227.8	8.8	17.5	228.0	9.5	52.5
7	Springvale Dr	206.2	206.5	4.2	0.0	206.5	4.5	0.0	206.6	9.3	0.0	206.9	16.7	0.0	207.2	20.6	7.7	207.4	23.3	19.8	207.5	24.9	30.8	207.6	26.3	42.9	208.0	31.3	159.7
8	Plumpton Rd	201.1	201.4	5.7	---	201.5	6.3	---	202.0	9.3	---	202.2	11.0	---	202.4	11.6	---	202.5	12.0	---	202.6	12.3	---	202.7	12.6	---	203.4	13.0	---
9	Stringybark Creek Diversion	198.9	199.4	4.7	0.0	199.5	5.7	0.0	200.4	19.6	0.0	200.9	31.6	0.0	201.2	44.3	0.0	201.5	50.8	0.0	201.6	56.4	0.0	201.8	61.7	0.0	202.7	99.7	0.0
10	Flow out of limits of Stringybark Ck diversion	200.5	200.6	NA	0.6	200.6	NA	0.7	200.6	NA	1.1	200.6	NA	1.5	200.6	NA	9.6	200.7	NA	26.4	200.7	NA	42.8	200.8	NA	61.7	201.1	NA	291.7
11	Redbank Rd	208.5	-999	0.0	0.0	209.0	3.6	0.0	209.7	10.7	0.0	210.1	14.5	0.0	210.3	18.5	0.3	210.5	20.9	1.7	210.7	22.5	3.5	210.7	23.7	5.0	211.0	26.7	14.6
12	Gregadoo Rd (West)	202.7	202.8	0.0	0.0	203.3	6.6	0.0	204.0	21.4	1.8	204.1	26.2	7.9	204.3	30.5	18.2	204.4	34.2	31.8	204.5	35.7	42.3	204.5	36.6	51.2	204.9	45.5	239.6
13	Gregadoo (East)	203.8	203.8	0.1	0.2	203.9	0.2	0.5	204.1	0.8	1.2	204.2	0.9	2.6	204.2	0.9	5.7	204.2	1.0	10.8	204.3	1.0	17.9	204.3	1.1	28.1	204.7	1.2	229.8
14	Crooked Ck diversion (Main Rd)	198.5	198.8	0.3	0.0	199.5	6.4	0.0	200.1	21.5	0.0	200.4	26.9	0.0	200.6	33.8	0.0	200.7	39.9	0.0	200.8	43.2	0.0	200.8	45.8	0.0	201.6	88.3	0.0
15	Flow out of limits of Crooked Ck div	198.6	198.6	NA	0.1	198.6	NA	0.2	198.7	NA	0.3	198.9	NA	0.7	199.2	NA	3.0	199.7	NA	10.4	199.9	NA	17.6	200.0	NA	24.8	200.6	NA	124.6
16	Un-named Ck East of #17	197.8	198.9	NA	0.7	199.0	NA	1.8	199.1	NA	4.4	199.2	NA	8.9	199.4	NA	19.4	199.5	NA	34.7	199.6	NA	51.6	199.7	NA	71.3	200.9	NA	453.1
17	Brunskill Rd	194.2	194.6	1.8	0.0	194.7	2.3	0.0	194.8	3.4	0.1	195.2	8.4	0.2	195.7	16.0	2.3	195.9	17.0	25.5	195.9	17.3	49.7	196.0	17.4	76.3	196.9	17.9	570.0
18	Vincent Rd adj STP	186.3	187.6	3.1	0.0	187.9	4.6	0.0	188.3	7.3	0.0	188.5	8.9	0.0	189.3	14.5	0.0	189.6	15.4	7.5	189.7	15.5	18.2	189.7	15.5	28.9	190.1	15.9	61.4
19	Marshalls Ck @ Vincent Rd	183.6	184.2	4.3	0.0	184.5	8.1	0.0	184.9	14.3	0.0	185.1	18.7	0.0	185.3	23.0	0.0	185.9	38.0	0.0	186.4	48.1	6.5	186.6	49.0	22.3	187.7	52.6	518.8
20	Laurel Rd	186.6	186.8	NA	0.4	186.9	NA	0.9	186.9	NA	1.2	186.9	NA	1.5	186.9	NA	2.1	187.2	NA	19.2	187.2	NA	34.0	187.3	NA	47.1	188.3	NA	446.2
21	Lake Albert Outflow	190.1	190.2	0.0	0.0	190.5	0.1	0.0	190.9	1.1	0.0	191.1	2.1	0.0	191.3	3.5	0.0	191.5	7.6	0.0	191.7	11.1	0.0	191.8	15.0	0.0	193.0	50.2	95.2
22	Marshalls Creek @ Lake Albert Rd	188.4	188.7	1.1	0.0	188.9	2.3	0.0	189.3	5.2	0.0	189.5	7.2	0.0	189.8	10.3	0.0	191.1	22.2	0.0	191.9	28.4	0.0	192.4	31.1	13.9	193.1	31.8	254.1

NA: Not applicable
NSL: Natural surface level
--- 2D flow path not well defined (able to be recorded) and hence not reported
Number of point observed on Figure 32 for relevant model domain
-999 Dry location
All levels are in mAHD
Q_1D and Q_2D results are in units of m^3/s Q_1D indicates flow in the 1D model components. 2D indicates flow in the 2D model components. Total flow at a location can be calculated by the Q_1D and Q_2D values

Table 20: East Model Domain Results

	Location	NSL	1yr ARI			2yr ARI			5yr ARI			10% AEP			5% AEP			2% AEP			1% AEP			0.5% AEP			PMF		
#		Level	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D	Level	Q_1D	Q_2D
1	Inglewood Rd (West)	193.3	193.7	0.2	0.0	193.9	1.6	0.0	194.1	4.2	0.0	194.2	5.9	0.0	194.4	8.4	0.0	194.5	11.4	0.0	194.7	14.1	0.0	194.8	17.1	0.0	196.3	39.1	82.5
2	Inglewood Rd (East)	197.4	197.9	1.2	0.0	198.0	5.5	0.0	198.3	15.3	0.0	198.5	22.4	0.0	198.8	32.7	0.0	199.0	45.7	0.0	199.4	56.8	0.6	199.6	62.4	9.5	200.6	84.3	234.9
3	Gregadoo Ck (#14)	188.2	1884	NA	1.1	188.5	NA	6.6	188.6	NA	18.7	188.6	NA	27.4	188.7	NA	39.7	188.8	NA	55.8	188.9	NA	69.6	189.0	NA	87.5	190.1	NA	485.5
4	Minor Structure at railway embankment	182.2	182.3	0.0	1.0	182.4	0.1	5.9	182.6	0.2	18.3	182.7	0.4	26.4	182.9	0.6	38.5	183.1	0.9	60.3	183.3	1.1	80.2	183.4	1.3	91.9	184.1	1.5	160.5
5	Marshalls Ck structure at railway embankment	181.6	181.9	0.0	4.1	182.0	0.3	7.2	182.3	2.1	12.7	182.4	2.8	15.3	182.5	2.9	16.4	182.9	3.4	28.5	183.2	4.1	40.7	183.3	4.4	48.7	184.2	5.3	87.5
6	Unsealed Rd u/s of Bakers Ln	181.4	-999	0.0	0.0	181.8	0.2	0.0	181.9	0.4	0.0	182.0	0.4	0.0	182.0	0.5	0.0	182.0	0.6	0.0	182.1	0.7	0.0	182.1	0.8	0.0	183.2	1.6	16.0
7	Tasman Rd	180.4	180.5	0.1	0.0	180.6	0.5	0.0	181.2	3.9	0.0	181.4	6.2	0.0	181.5	8.5	0.0	181.7	9.4	0.0	181.8	10.1	0.0	181.9	11.0	0.0	183.3	11.9	53.8
8	Confluence of Gregadoo and Marshalls Ck	179.1	179.9	NA	---	180.3	NA	---	180.8	NA	---	181.1	NA	---	181.4	NA	---	181.6	NA	---	181.7	NA	---	181.9	NA	---	183.2	NA	---
9	Copeland St structure	177.6	178.8	3.3	0.0	179.1	6.3	0.0	179.6	15.4	0.0	180.1	30.2	0.0	180.6	46.9	0.0	181.0	52.9	0.0	181.3	55.7	0.8	181.5	57.9	2.3	183.0	72.3	99.5
10	Kooringal Rd Bridge	176.7	177.9	NA	2.3	178.4	NA	6.5	178.8	NA	14.9	179.3	NA	30.3	179.8	NA	47.3	180.5	NA	76.0	180.9	NA	84.8	181.3	NA	91.4	182.7	NA	181.8
11	Sturt Hwy Bridge	176.5	177.3	NA	2.3	177.5	NA	5.7	178.0	NA	14.3	178.6	NA	30.2	179.1	NA	47.2	179.6	NA	76.1	180.3	NA	102.4	180.7	NA	124.8	181.9	NA	214.8
12	Marshall Ck just u/s of River	175.8	177.3	NA	2.3	177.3	NA	5.6	177.4	NA	14.1	177.5	NA	30.0	177.7	NA	47.2	178.2	NA	75.6	178.5	NA	101.7	178.9	NA	124.6	180.4	NA	289.9

NA: Not applicable
NSL: Natural surface level
--- 2D flow path not well defined (able to be recorded) and hence not reported
Number of point observed on Figure 32 for relevant model domain
-999 Dry location
All levels are in mAHD
Q_1D and Q_2D results are in units of m^3/s
Q_1D indicates flow in the 1D model components. 2D indicates flow in the 2D model components. Total flow at a location can be calculated by the Q_1D and Q_2D values

5.3. Sensitivity Testing

As documented in Section 4 various sensitivity runs have been undertaken.

Results can be seen for peak flood levels extracted at locations as shown in Figure 32 with values shown over the page in Table 21. Note that eight sensitivity runs have been carried out relative to the 1% AEP base case whilst a further run examining the sensitivity of flood levels to an elevated Murrumbidgee River is carried out however the 2Y ARI run is examined in this case.

Table 21: Sensitivity Results - Comparison of modelled flood levels at various locations

	1% AEP 3 hour - Peak Flood Levels (mAHD)										2Y ARI 3 hour – Peak Flood Levels (mAHD)	
#		Base	High 'n'	Low 'n'	Low Loss	High Loss	High Tail Water	20% Block	50% Block	Climate Change	5.3.1.	Base with pumps
1	Cnr of Urana St & Mitchelmore St	207.2	207.3	207.3	207.3	207.3	207.3	207.3	207.3	207.3	207.1	207.1
2	Cnr of Colemant St & Inverary St	191.4	191.5	191.5	191.5	191.5	191.5	191.5	191.5	191.5	191.4	191.4
3	Railway line @ cnr of Inverary St & Cassidy Pde	187.5	187.6	187.6	187.6	187.6	187.6	187.6	187.6	187.6	187.5	187.5
4	Housing bound by Forsyth, Morgan & Murray St	179.3	179.3	179.3	179.3	179.3	179.3	179.3	179.3	179.3	179.0	179.0
5	Concrete o/c at Forsyth St feeding Wollundry	178.5	178.4	178.5	178.6	178.3	178.8	178.5	178.6	178.5	177.7	177.7
6	Most d/s pool in Wollundry Lagoon	178.1	178.1	178.2	178.3	178.0	178.8	178.2	178.3	178.3	176.6	176.6
7	Tony Ireland Park	177.3	177.3	177.3	177.3	177.3	179.3	177.3	177.4	177.3	177.2	174.4
8	Cnr of Sturt Hwy & Edwina St	180.0	180.0	180.0	180.0	180.0	180.0	180.0	180.0	180.0	179.9	179.9
9	Cnr of Morgan St & Bardo Ln	179.2	179.1	179.2	179.2	179.1	179.3	179.2	179.2	179.2	178.8	178.8
10	Light Industrial Area Sth of Sturt Hwy Wst of Marshalls Ck	180.1	180.1	180.1	180.1	180.0	180.0	180.1	180.1	180.1	179.1	179.1
11	Mason St low point	179.7	179.8	179.8	179.8	179.8	179.8	179.8	179.8	179.8	179.5	179.3
12	GfD @ Dalman Pkwy	198.6	199.0	199.2	199.3	198.6	199.1	199.4	200.0	199.2	197.5	197.5
13	GfD @ Fernleigh Rd	189.4	189.6	189.6	189.7	189.4	189.6	189.7	189.8	189.7	188.0	188.0
14	GfD @ Railway Embankment	187.7	187.7	187.7	188.0	187.3	187.7	187.9	188.1	187.9	185.5	185.5
15	GfD @ Intersection of of Dobney Av and Pearson St	182.5	182.5	182.5	182.5	182.4	182.5	182.5	182.5	182.5	181.0	181.0
16	GfD @ Sturt Hwy	177.6	177.6	177.6	177.8	177.4	178.4	177.6	177.7	177.8	176.3	176.7
17	Flowerdale Lagoon	177.2	177.2	177.3	177.4	177.0	178.1	177.2	177.3	177.4	175.8	176.6
18	Detention basin @ cnr of Red Hill Rd & Railway	208.8	208.8	208.8	208.9	208.7	208.8	208.8	208.8	208.9	208.1	208.1
19	Silvalite @ Red Hill Rd	200.9	201.6	201.6	201.8	201.3	201.6	201.8	202.5	201.7	200.6	200.6
20	Silvalite @ Sturt Hwy	183.7	184.0	184.1	184.5	183.7	184.1	184.4	185.5	184.3	183.1	183.1

6. DISCUSSION

6.1. Verification

Lacking suitable data for a thorough calibration process (see Section 3.6 for a detailed description of the available data), the model has instead been checked extensively. This means model results have been compared to other estimates, or in some cases to expectations of what overland flow should resemble, based on two principle Council employee's impressions developed over multi-decade careers in Council (specialising in drainage).

The model results have been compared to:

- Lumped catchment peak flow estimates utilising the PRM;
- Council driven expectations compiled into a set of drainage "hot spots";
- Photographs flooding due to the February 5 2010 event (flooding focussed around the Glenfield/CBD area); and
- Photographs flooding due to the December 9th 2010 event (flooding in three of four model domains). See Appendix D for full details.

6.1.1. PRM Check

The match between the PRM estimates and modelling carried out as part of the current study in WBNM indicates that the WBNM results are reasonable. Given the approximate nature of PRM estimates this result is the best that can be expected out of the comparison and such is a positive which increases confidence in model results.

6.1.2. February 5th 2010 Event

In summary the match between the model results and photo observations is favourable. The model has adequately replicated the recorded behaviour and this gives some confidence in the City model which is a significant achievement given the size and importance of the City model domain.

6.1.3. December 9th 2010 Event

In summary the match between the model results and photo observations is good. The model has generally replicated the recorded behaviour and this gives some confidence in at least three of the four model domains (all but North Wagga for which no observations were available). For full details please refer to Appendix D.

6.1.4. Council Hotspots

Hotspots have generally been well replicated. From hotspots one through to nine model results indicate general inundation as per the description found within Council's Brief (and recreated herein for ease of comparison). A tenth hotspot was added on the basis of two (of a total of five) public submissions. The hotspot is located at the intersection of Macleay and Urana Streets and it is of note that the model results do also reflect the observed behaviour described by the two respondents.

Note also that a public submission which identified flooding in Sycamore Road as an issue is also reflected in model results, albeit for design flooding.

6.1.5. Comparison to Reference 4 Findings

Given that the 2006 study (Reference 4) was undertaken using different data sets and overall methodology compared to the current Study it is not expected that model results will be equivalent. The differences between the two studies need to be understood in order to put the results into context and for this reason they are listed below:

- Reference 4 used a catchment area of approximately 16.5 km^2 whilst the modelling reported upon herein used a Glenfield Drain catchment area of 14.5 km^2 (i.e. incorporating the Red Hill Road diversion);
- Reference 4 used surveyed cross-sections, routed flow via the hydrological model in many cases and modelled one drainage path, Glenfield Drain, using a one-dimensional model; and
- Losses used in Reference 4 are an initial loss of 25 mm and a continuing loss of 3 mm/h whilst the current modelling is using 15/2.5.

Overall there are factors then that are likely to lead to the Reference 4 results being larger than the current study's results (one-dimensional model, hydrological routing, one main drainage path, larger catchment area) although the opposite is also true due to implementation of higher losses.

As can be seen from the results however it seems that the Reference 4 study consistently produces results which exceed the results from the current study with respect to peak flow magnitude at the very least. At Dalman Parkway Reference 4 produces a peak flow 55% larger than the current study (13.1 versus 20.5 m^3/s). Further downstream, particularly as flow is seen to bifurcate, the discrepancy increases. At Fernleigh Road Reference 4 peak flow exceeds the current study by 85%. At the railway the difference is 130% whilst at the Dobney Avenue/Pearson St intersection the difference is 180%.

A key difference between the two approaches which contributes to this large discrepancy is that the 2D model continually distributes water from Glenfield Drain to other flow paths/attenuation areas but in the one-dimensional model (Reference 4) the flow is constrained to Glenfield Drain.

Overall the comparison confirms the current modelling only in that the Reference 4 methodology was rather conservative and as such it's logical that current model estimates are less than those in Reference 4. Certainly the current modelling is achieving much better estimates of available storage and resolving, at relatively high levels of detail thanks to the 5 m grid size and input ALS data, alternative flow paths. As such the current estimates are certainly better design flow estimates than those provided by Reference 4 and model results from the current study should supersede findings of Reference 4.

6.2. Sensitivity Testing

Sensitivity results shown below in Table 21 indicate that:

- Changing overbank roughness by 20% produces very little change at all;
- The low and high loss scenarios also produce some impacts and indicate some sensitivity to the selection of loss rates. Note that this is to be expected given that the 1% AEP (3 hour) rainfall depth is approximately 69 mm and the low loss scenario removes 13 mm of this (~ 20%), the standard run removes 22.5 mm of this (~ 30%) and the high loss scenario removes 34 mm of this (~50%);
- Model results are most sensitive to the change in tail water level however the areas affected are all at the extreme downstream end of catchments. Note results for Glenfield Drain upstream of Sturt Highway and Silvalite Creek upstream of the Sturt Highway;
- Blockage impacts on results to a small degree and this is expected given that most structures at road crossings for example are not sized to pass the 1% AEP flow but instead a significantly smaller event, usually the 1 in 10 year or the 1 in 5 year flows. As such a lot of flow overtops or bypasses the structures and as such they are not as controlling of peak flood level as they might otherwise be;
- Climate change will produce negligible changes to design flood levels (a maximum of 0.2 m) relative to standard freeboard of 0.5 m; and
- The inability of local flow to move into the Murrumbidgee River will only impact on the most downstream areas of flooding and existing pumps will do little to mitigate the issue.

All results in Table 21 increase confidence in the accuracy/robustness of the results. The results do indicate however that selecting an appropriate Murrumbidgee River level for design runs will be important for locations lower within the study area and also that for some areas which are volume sensitive the choice of losses used is important.

6.3. Design Flood Results

Design flood results indicate extensive inundation of private and public property in the event of a 1% AEP event. In the main however it seems that reasonable planning controls have limited the number of households/commercial operations that are likely to experience over floor flooding. Quantification of over floor flooding will have to wait for floor level survey work to be carried out however at this point it seems likely that the households/businesses most flood liable are to be found in:

- Dobney Avenue downstream of Glenfield Drain;
- Houses between Rowe Street and Bocquet Street are threatened by failure and spill from the Lake Albert diversion;
- Stringybark Creek threatens homes at the end of Yarran Place, Hakea Place and on the Northern edge and end of Mallee Road;
- Berry Street just north of Morgan St at the southern edge of the CBD is as ever, flood

prone;

- The area between Morgan Street, Thorne Street, Tompson Street and Murray Street which is just south of the Wollundry Lagoon (at its western end) appears to collect a lot of flow (note this area includes Forsyth Street);
- Spring Street and Kincaid Street off Moorong Street immediately upstream of Flowerdale Lagoon seems flood liable also;
- Homes near the corner of Urana and Macleay Streets including on Heydon Avenue are impacted by flooding, with some residences liable to over floor flooding based on anecdotal information alone; and
- Properties in the vicinity of and downstream of Brunskill Road, particularly along Sycamore Drain (and Sycamore Road) are flood liable and some of these may even be subject to floodway type flows with high velocities in larger flood events.

Given the very large study domain and the wealth of available flood data available as a result of this study it is recommended, particularly prior to the drafting of a brief for the Management Study, that an Interim Report be put together. The Interim Report would extract a variety of flow/flood level information for known hotspots and also create overall priority lists for areas to be targeted based on a property flood tagging exercise.

6.3.1. Hazard and Flood Risk In Wagga Wagga

Generally speaking flooding flow, where it interacts with buildings, is low hazard flow. High hazard areas tend to be limited to main channels and also retarding basins and this is due to the depth criteria in the hazard calculation.

Generally the flood risk can be rated as low with one of the main forms of risk likely to be road crossings that become inundated relatively quickly following rainfall.

6.3.2. Climate Change

The impact of climate change has been assessed via a 7% increase in rainfall as per government guidelines (Reference 7). Generally the impact of the increase in rainfall is greater in those areas where flow is constrained, such as Glenfield Drain, with little impact on wider levels throughout the City domain. For example the climate change run produces a flood depth 0.2 m greater at the retarding basins upstream of the railway line on Glenfield Drain. Overall the climate change impact as assessed is small and relative to standard provisions for freeboard (Reference 2) it is negligible.

7. CONCLUSIONS

The models representing major overland flow paths in Wagga Wagga have been set up utilising best practice methodologies and have been shown to be performing well via a series of checks or verifications, utilising, as far as is available, observed data. Further sensitivity testing indicates that model predictions are relatively robust within a standard range of possible parameters.

Design flood results indicate that whilst a variety of private and public land is impacted by flooding in the 1% AEP event few buildings are liable to over floor flooding and few residents are likely to be impacted by high hazard flood flows.

7.1. Recommendations

Recommendations are as follows:

- Proceed to an Interim Report in order to better inform the writing of a brief for the subsequent Floodplain Risk Management Study and Plan. Such an Interim Report will provide more interpretation of design flood results and will specifically identify those areas requiring further investigation, particularly with regard to mitigation, as part of the Management Study;
- Enhance flow capacity into Lake Albert for those flows on the western side of Plumpton Road (Stringybark West Diversion). This will ease the danger of flooding for those houses located downstream of the Plumpton Road crossing within the historical Stringybark Creek flow path. It is noteworthy that the capacity of the structure on Plumpton Road is considerably less than the capacity of the structure upstream of it on Springvale Drive;
- An improved design for the Crooked Creek diversion mechanism into Lake Albert on the eastern side such that flows in excess of the design capacity of the diverting levee are controlled and accounted for. Formalisation of the overflow mechanism and flow path downstream of the levee should protect the diversion structure as well as give better flooding outcomes to the residents downstream of the diversion;
- The possibility of failure of the Crooked Creek diversion levee during a large event should be examined to determine the consequence of such a failure. It may be that a much lower diverting structure is better suited to the location as this will divert flow to Lake Albert as well as prevent a build up of water which, if allowed to flow north without control, could put residents lives and homes at risk;
- Obtain a floor level survey for all flood liable houses and commercial buildings;
- Investigate flooding behaviour in the vicinity of Brunskill and Sycamore Roads as part of more detailed stormwater work as currently a number of properties in this area are exposed to flood risk for events as small as the 10% AEP;
- Generally implement a program of drainage maintenance such that high priority systems (Glenfield Drain for example) are maintained to a high standard with regard to vegetation blockage etc;

- As part of more localised stormwater studies address issues at the corner of Urana and Macleay Streets as the currently flooding frequency is high and this is impacting on residents;
- Develop response plans for localised flooding scenarios that occur in conjunction with elevated river levels, mainly around mobile pumping operations, taking into account the fact that substantial quantities of power required may not always be available from the grid; and
- Design flood data from current study be used as basis for setting flood planning levels in the interim period pending completion of the Flood Risk Management Plan.

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